APPENDIX B
GEOTECHNICAL

APPENDIX B
Geotechnical Report

List of Figures

List of Tables

This page intentionally left blank.

1.0 DESCRIPTION OF EXISTING PROJECT

The Dallas Floodway Project is a levee system located on the Elm Fork, West Fork and Trinity River in Dallas, Texas. The Dallas Floodway project includes 22.6 miles of levees: 11.7 miles on the northeast levee (usually referred to as the East levee) and 10.9 miles along the southwest levee (generally referred to as the West levee). The East levee protects the Stemmons Corridor (a major transportation route through the City), and parts of Downtown Dallas and the Central Business District from flooding on the Trinity River, while the West levee protects a large portion of West Dallas from the Trinity.

These levees were originally constructed by the City of Dallas in the 1930s in response to extreme flooding along the Trinity River in 1908 (USACE 1968). Originally constructed with 2.5H:1V side slopes, a maximum height of 35 feet and a crown width of 6 feet (USACE 1952), the levee system was 'strengthened' by USACE in the late 1950's by flattening the side slopes and increasing the crest width to 16 feet. Additionally, improvements to the interior drainage system were also made at that time (USACE 1968). For more information on the existing Dallas Floodway Levee System, refer to Appendix D, Civil and Structural Assessment.

2.0 PURPOSE AND SCOPE

The Dallas Floodway Project was originally authorized to provide flood protection to a level of standard project flood (SPF) + 4 feet; however, major urban development and land-use changes in the area since the project was completed by USACE in 1959 have increased the risk of the levee system design capacity to be exceeded. The purpose of this appendix is to define the geotechnical conditions of the levee system in the study area as it relates to flood risk and to evaluate the geotechnical aspects of the City of Dallas, Texas Balanced Vision Plan (BVP) and Interior Drainage Plans (IDP) in accordance with Section 5141 of the Water Resources Development Act (WRDA) of 2007 (WRDA Project). The WRDA Project evaluation includes evaluating the existing performance of the levee system and interior drainage system, as well as numerous ecosystem and recreation projects within the Floodway. This appendix documents the feasibility level geotechnical design and analysis of the existing and modified levee embankment sections, environmental restoration measures that will be implemented, and preliminary foundation recommendations for miscellaneous new structures associated with the modified project (i.e. gate closures for bridges, etc.). In addition to documenting the geotechnical evaluation results of the WRDA Project, it also documents the evaluation results of the Comprehensive Analysis as defined in Section 1.7 of the main feasibility report. This analysis ensures proposed local projects (e.g. Trinity Parkway) meet Corps engineering and safety standards, are compatible with the proposed WRDA Project features, and would not have significant adverse effects on the functioning of the existing Dallas Floodway Levee System. It also determines that components of the BVP and IDP are technically sound and environmentally acceptable.

3.0 EXISTING CONDITIONS: GENERAL GEOLOGY AND PHYSIOGRAPHY

3.1 TOPOGRAPHY AND PHYSIOGRAPHY

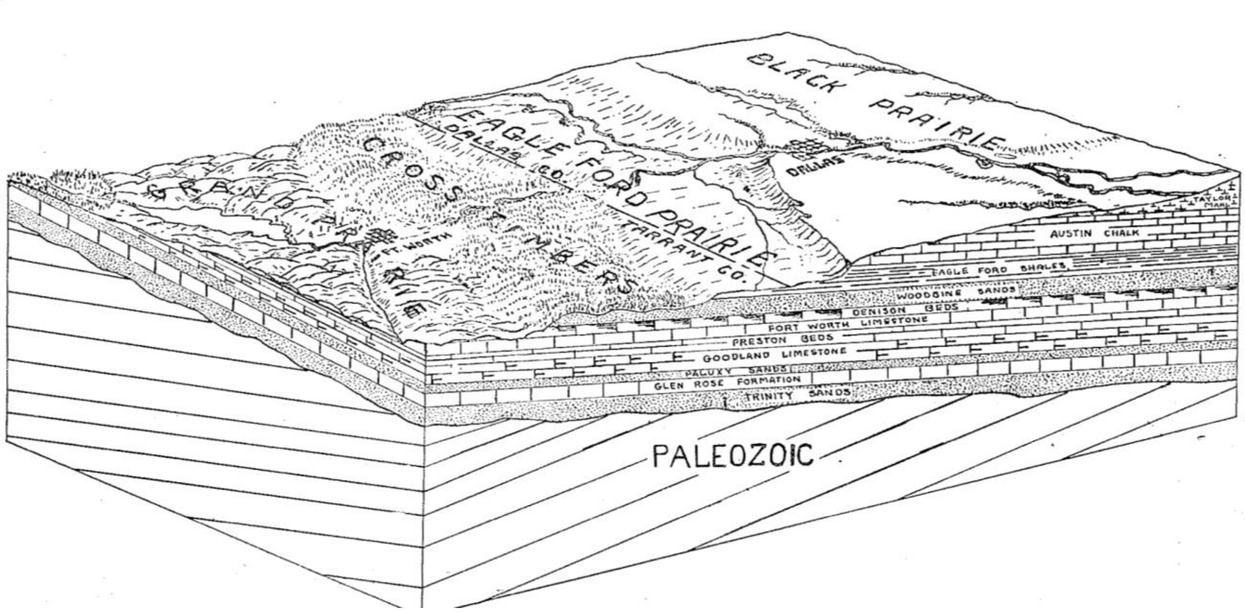
The project area of the Upper Trinity Feasibility Study lies entirely within Dallas County, an area of low topographic relief. Located within the western part of the Gulf Coastal Plain near the northwestern limit of the East Texas Embayment, Dallas County is situated on the Black Prairie Belt, on the outcropping rocks of the Eagle Ford, Austin and Taylor Formations, (Figure 3-1 to Figure 3-3) which are three broad bands of Cretaceous rocks that are exposed on the surface within the county. These beds form the northern portion of the Gulf Coastal Plain, and strike nearly north and south with a gentle eastward dip. Within the project area, the Eagle Ford formation is overlain by alluvial flood plain deposits from the Trinity River, consisting of silt, sand and gravel. Additionally, residual soils derived from the Eagle Ford and Taylor Formations are present as high plasticity clays (Dallas Geological Society 1965).

Figure 3-2 shows the Trinity River levee system, and the limits of the major mapped stratigraphic units: undifferentiated Quaternary floodplain alluvium (Qal) and terraces (Qt), Upper Cretaceous Eagle Ford, (Kef), and Austin Chalk (Kau) Formations. The type location for the Eagle Ford Shale is in the floodplain of the West Fork Trinity River at the abandoned rail-road station of Eagle Ford (center of graphic). Note the location of Arcadia Park at the contact between the Eagle Ford (Kef) and Austin Chalk (Kau) and the subdivision of the Eagle Ford Formation into stratigraphic units in Figure 3-2.

3.2 GEOLOGIC HISTORY

The oldest rocks in Texas are a direct response to the Ouachita Orogeny that occurred during the Paleozoic Era. The first orogenic movement occurred during the late Mississippian Period and into the early Pennsylvanian Period. Four thrust sheets completed the orogeny in the early Permian. As a result of this mountain building event, two foreland basins formed that created the Gulf embayment. These basins surround Dallas County; the Fort Worth Basin is to the west while the East Texas Basin sits to the east of Dallas. The first recorded deposition dates back to the early Cretaceous Period starting with the Hosston (also known as Travis Peak, or the Basal Lower Trinity) and followed by the Trinity, Fredericksburg, and Washita Groups. These groups were deposited on land as the surrounding uplifted mountains eroded. The lower and upper Cretaceous deposits are similarly composed of sediments deposited on a subsiding and rising sea floor, their strike and dip of the rocks are in the same direction, and each is characterized by an excess amount of lime or calcium carbonate (Hill 1889).

There is a distinct contact between the upper Cretaceous stratigraphy and the lower Cretaceous stratigraphy, as well as a change in the species found in each series (Figure 3-1 to Figure 3-4). The upper Cretaceous sediments, unlike the lower series, were deposited in an oceanic subsidence environment. Each lithologic group is a variation representative of local changes in the same subsiding environment. These sediments are grouped into the Woodbine, Eagle Ford, Austin Chalk, and Taylor Marls.



Source: Schuler 1918.

Figure 3-1. Block Diagram of the Stratigraphy Beneath the Dallas Metropolitan Area

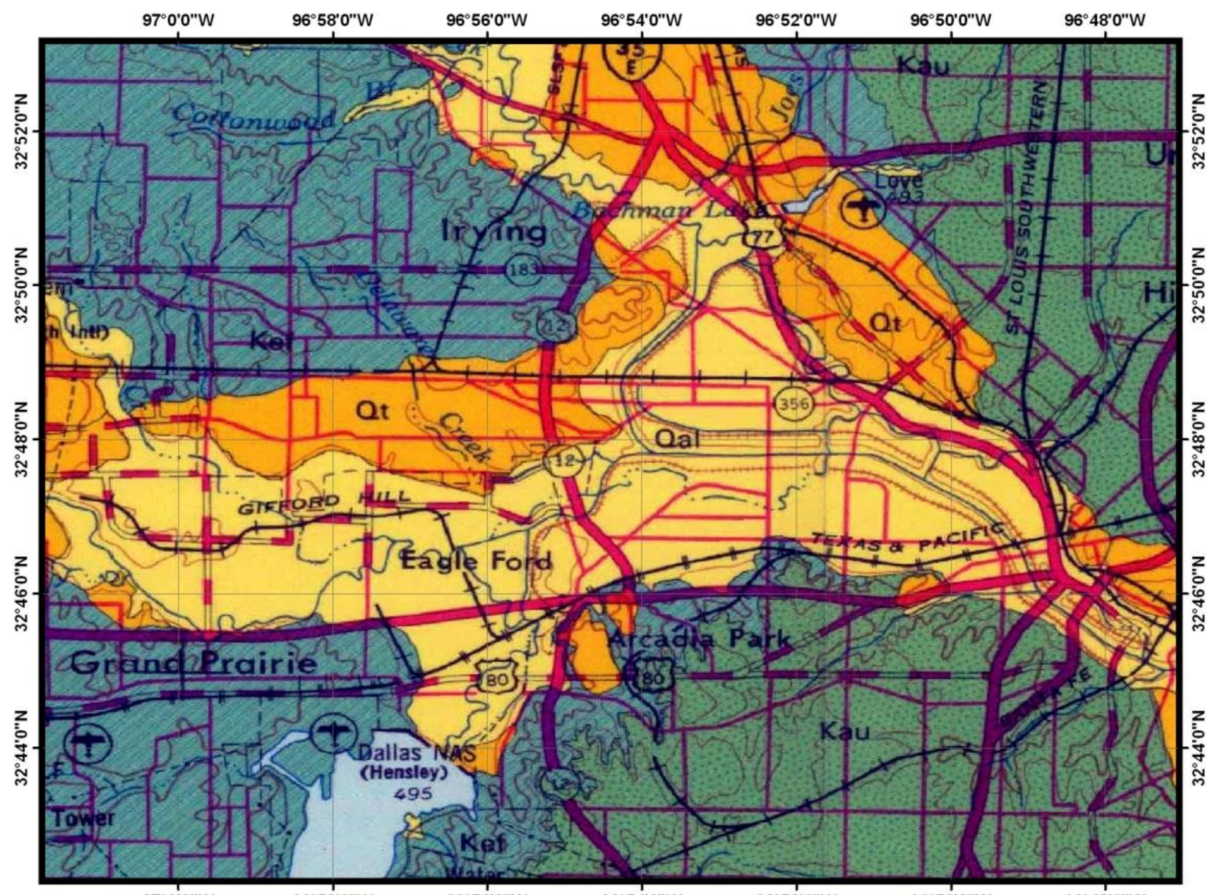


Figure 3-2. Portion of the Bureau of Economic Geology's (1972) Dallas Geologic Map

3.3 Depositional Environments of the Outcropping Units

The Woodbine Formation consists of sandstone with interbedded and interfingering clays and shales (Hendricks 1972). The two members found in Dallas County are Dexter and Lewisville. The Dexter Member is sand and silty clays that were deposited in brackish water or deltaic environment with short marine intervals of deposition as sea level rose or subsidence occurred. The Lewisville Member contains carbonaceous clays indicative of marsh deposition during a regression. Some of the sands found in this member are believed to be deposited by stream channels or distributary channels (Hendricks 1972).

The Eagle Ford Formation sits unconformably on the Woodbine Formation within this study area. The rocks of this formation were deposited in an epicontinental sea in anoxic conditions (Charvat 1985). In the study area, the Eagle Ford is subdivided into the Tarrant sandy clay and limestone, the Britton clay, which is the member that the Trinity River is incised in, and the Arcadia Park Shale. Volcanic ash from the surrounding regions settled into the shallow water to form smectite-rich bentonite layers, which indicates the depositional environment had active ions and a high SiO_2 content (Charvat 1985).

The Austin Chalk and Eagle Ford contact is also an unconformity. The chalk was deposited as a deep sea deposit and consists of Foraminifera evenly distributed throughout the entire formation (Moreman). Originally, the Austin Chalk was deposited as a chalky limestone with layers of soft blue marl. These marl layers are formed mostly by the casts of Foraminiferia. The Taylor Marl Formation has been weathered away as early streams flowed after the sea level regression. This formation was deposited in a shallow water environment.

3.4 Eagle Ford Formation and the Trinity River Floodplain

The geology of the Dallas County area has been mapped and described in detail by various subject matter experts in a report by the Dallas Geological Society (1965). This report is the most comprehensive study of its kind since Shuler (1918) first described the geology of Dallas County (see Figure 3-1). Eubank (1965) identifies the major physiographic features of the Dallas area in relationship to the underlying stratigraphy. Foster (1965) examines the regional structure and character of the stratigraphy in the subsurface while Norton (1965) has compiled a comprehensive listing of outcrop exposures that have been used to define the present day surface geology in the greater Dallas metropolitan area.

Mapping by Norton (1965) in the Dallas County area was incorporated into the 1:250,000 scale Dallas Sheet, Geologic Atlas of Texas, (Bureau of Geology 1972) (see Figure 3-2). The Eagle Ford and Austin Chalk Formations' contact is located in the Arcadia Park vicinity. The Eagle Ford is named after the locality where it outcrops as clay soil. Eagle Ford Formation stratigraphic subdivisions are shown in Figure 3-3. The floodplains of the Elm Fork and Trinity River systems and the levee system protecting the City of Dallas crosses Upper Cretaceous (approximately 95 to 85 million years old) sediments assigned to the Eagle Ford and Austin Chalk (Hendrix 1972, Bureau of Economic Geology 1972). Residual soils derived from weathering of shales within the Eagle Ford are responsible for the Grand Prairie area west of Dallas (Figure 3-2 and Figure 3-3). Soil mapping by the United States Department of Agriculture (USDA 1980) indicates these Prairie areas have high shrink-swell clay soils. General engineering properties of these units for the Tricities area in Dallas and Tarrant Counties are described by Hendrix (1972).

The overlying Quaternary (last 2 million years) alluvial geology of the Trinity River has been described in detail by Ralston (1965) and by Slaughter et al (1962). Slaughter used faunal occurrences from the mined gravel deposits from the floodplain and nearby terraces (i.e., higher level floodplain surfaces). He noted the occurrence of terrace alluvial gravels deposits on the Cretaceous bedrock surface throughout the

Dallas metro study area. The evolution of the Trinity River drainage system and its tributaries during the Quaternary has created the present day incised topography in the Dallas metro area, which is shown by the block diagram in Figure 3-1 and Figure 3-2. The Austin Chalk has created a distinct topographic ridge or cuesta which forms a bluff along the east edge of Village Creek, and at Acadia Park there is a type section for a mappable formation in the upper Eagle Ford.

3.5 Slope Stability and Geology

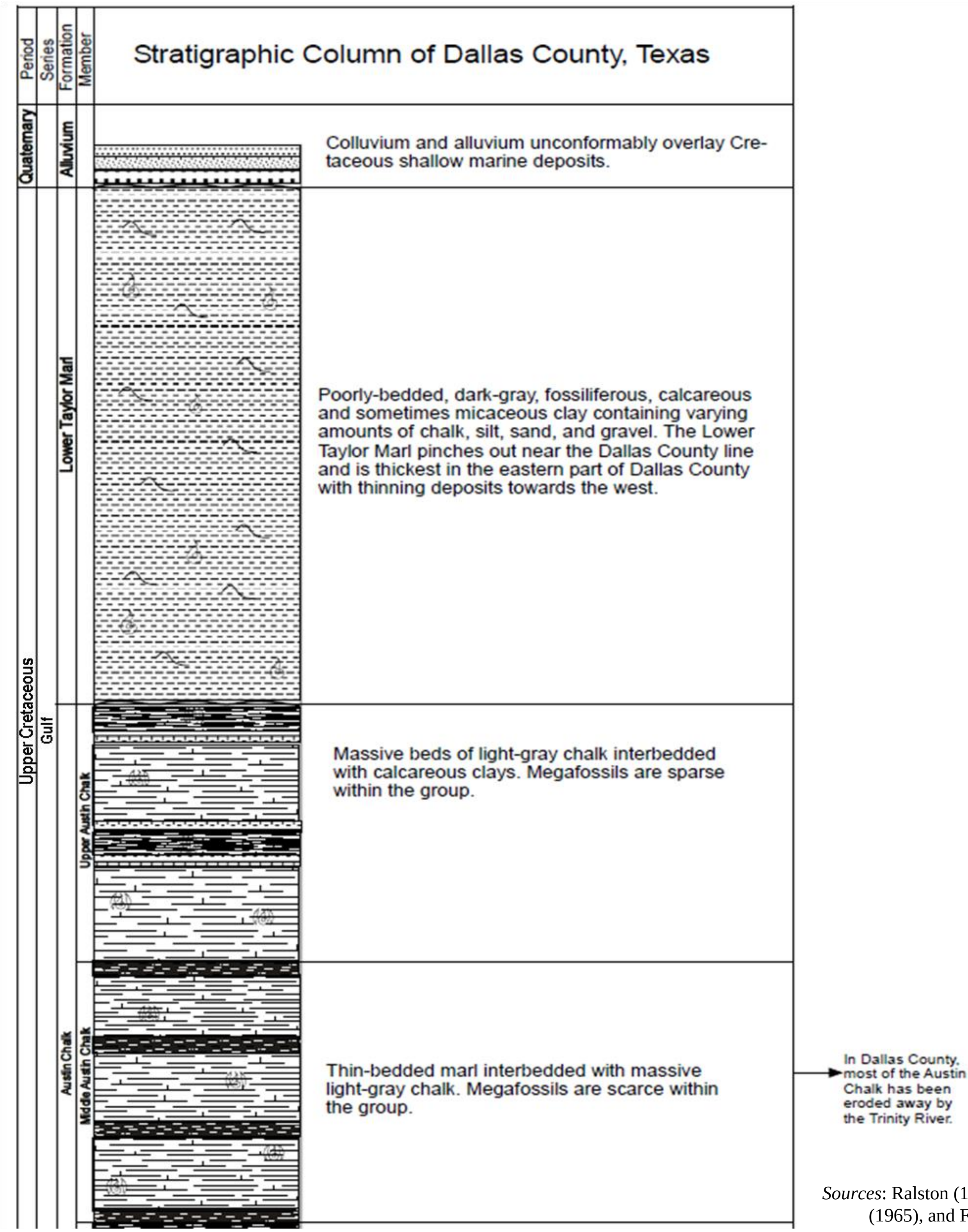
A summary review of the identified outcrops by Foster (1965) was conducted to determine which foundation units of the Eagle Ford may negatively impact the levee system. The occurrence of shallow slides in the levee profile has been reported for sections which are orientated along a north to south axis in the Federal floodway and associated with the Britton Formation. This orientation and geographic position would indicate a possible relationship to the foundation geology (i.e., dark shales of Norton's units 7, 8, and 9), steeper levee slopes on the flood side, or the sources of construction material used to build the levee. A significant portion of the slides occurred on both the East and West levees in areas where the levees are on an east/west axis. Slides in these areas appear to be related to steeper slopes and higher PI soils. Further to the east, levees built upon the Kamp Ranch Limestone, Acadia Park, and Austin Chalk Formations do not have the known stability problems with shallow slides, and/or the high shrink-swell clays which are characteristic of the levee system further to the west.

3.6 Seismicity

Dallas County lies between the East Texas Basin on the east, and the Fort Worth Basin on the west, an area of low seismicity. The Mexia Fault Zone is further east in Kaufman County; however, the Balcones Fault Zone approaches the southern edge of Dallas County. The underlying rock strata dip gently southeastward in a homocline, little effected by minor flexures and small faults (Dallas Geological Society 1965).

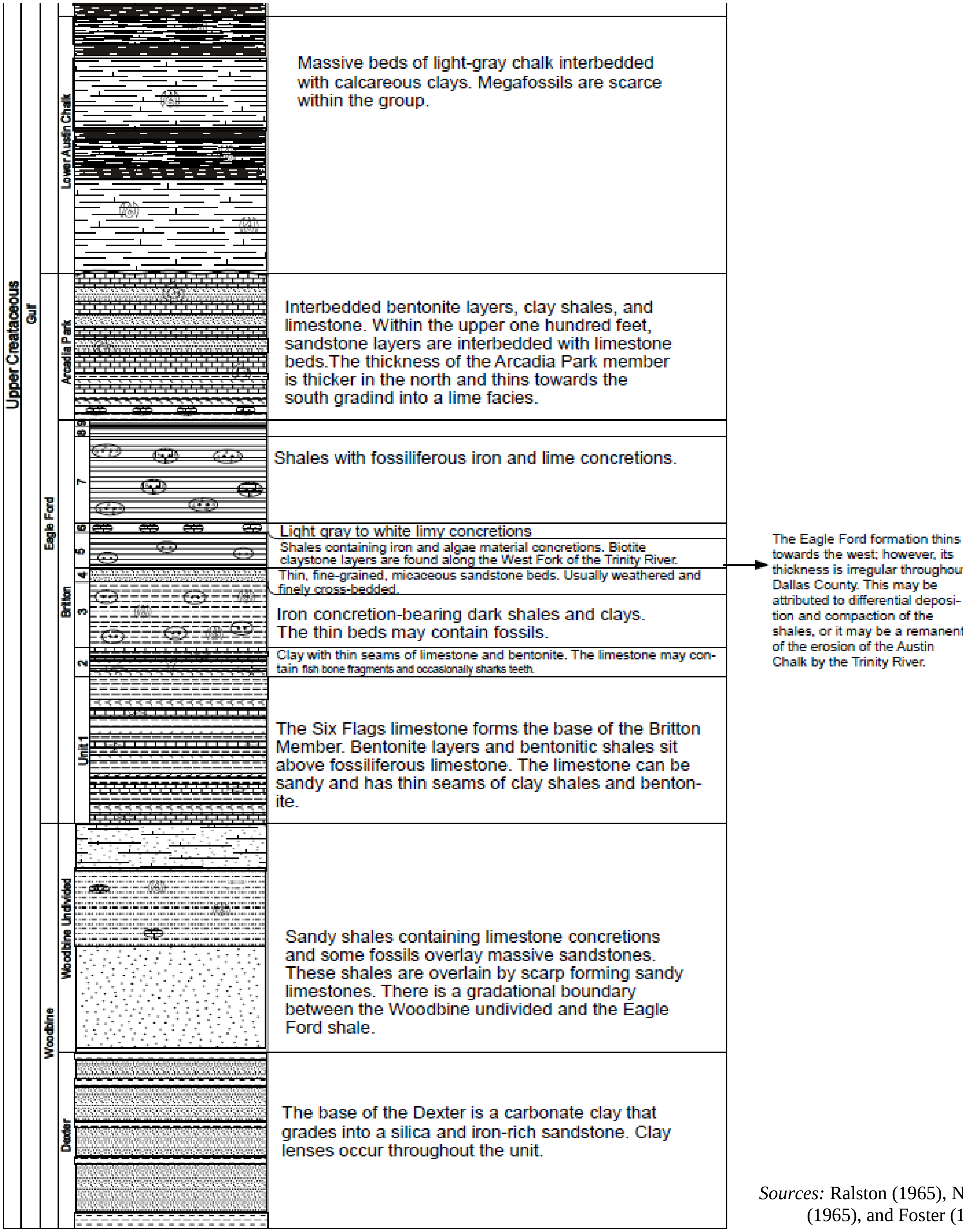
3.7 Economic Geology

Due to rich soils, farming and ranching played a significant role in the development of Dallas County. However, since the 1920s, lands once used for agrarian pursuits have been converted to urban and industrial use (http://www.dallaschc.org/history.html). The geology of the area has also been exploited for purposes of brick making, Portland cement manufacture, and crushed limestone production; however, of greatest impact to the Dallas Floodway project is the sand and gravel deposits that have been mined from the Trinity River floodplain for more than 100 years. Within the limits of the Dallas Floodway project, gravel deposits up to 15 feet thick have been mined out or covered over by more lucrative forms of development (Dallas Geological Society 1965).



Sources: Ralston (1965), Norton (1965), and Foster (1965).

Figure 3-3. Generalized Stratigraphic Column for the Upper Cretaceous Rocks Underlying Dallas, Texas



Sources: Ralston (1965), Norton (1965), and Foster (1965).

Figure 3-4. Generalized Stratigraphic Column (continued from Figure 3-3) for the Upper Cretaceous Rocks Underlying Dallas, Texas

4.0 CLIMATE

The climate in Dallas, Texas resembles a continental subtropical climate and is characterized by a wide annual temperature and precipitation range. The summer period is hot with temperatures exceeding 95˚F during the day. Summers are also characterized by lower humidity and precipitation. The bulk of the precipitation during summer occurs during active thunderstorm periods. Winters are relatively mild with an average temperature around 45˚F and low precipitation amounts. The annual temperature ranges from the low 30s during the winter to well above 100˚F during the summer. Annual precipitation ranges from less than 20 inches to more than 50 inches. Most of the yearly rainfall occurs during the transitions seasons of spring and fall. Annual relative humidity is higher during the winter and relatively low during the summer. Figure 4-1 shows the Long Term Monthly Means of the temperature (blue) and precipitation (green) measured within 1/10 inch at the Dallas Love Field (ICAO:KDAL) from 1960 to 2010, and relative humidity (purple) from 1999 to 2010. Also shown is the total number of landslides by month since 1974.

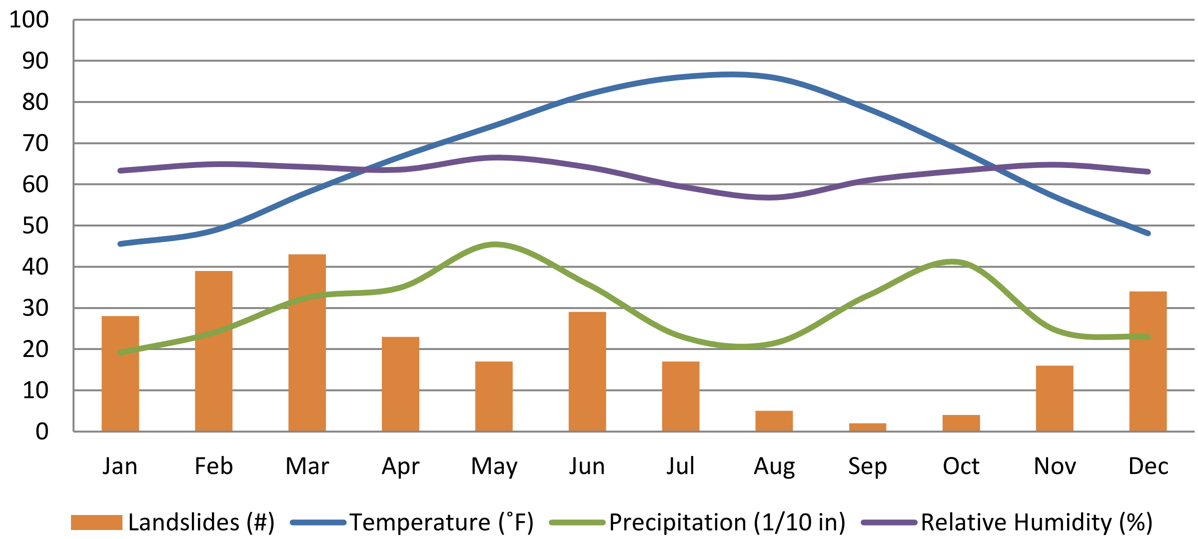


Figure 4-1. Climate and Landslide Summary from Dallas Love Field in Dallas, Texas

4.1 CLIMATE AND LANDSLIDES

Records maintained by the City of Dallas show that landslides on the Trinity River Levee System occur all year long with the bulk occurring during the winter months. It has been observed that most of the landslides occur at the river side of the levee system with the exception of a small number that occur at the landside of the system. Landslides during the summer months are isolated events and are associated with a significant increase in rainfall or a high flood stage; they typically occur within 3 to 5 days of a heavy rainfall and/or flood stage. Landslides during the late fall and early winter months are typically preceded by a significant rainfall event, in the order of 3 to 5 inches, approximately 14 days prior to the landslide occurring. As the winter season progresses, the total rainfall prior to a landslide occurring are typically lower and occur closer to the onset of the landslide, usually 7 to 10 days. During spring the total amount of rainfall in the 5 to 7 days preceding a landslide is low, approximately 0.5 inches. The decrease in time between the occurrence of rainfall and the onset of a landslide as the seasons' progress is a result of a swollen ground due to the water infiltration during the fall months.

5.0 SUMMARY OF PERIODIC INSPECTION #9

A periodic inspection (PI) of the Dallas Floodway project was performed on 3-5 December 2007. This inspection was the 9th PI for the East and West Levee systems. The inspection was conducted using procedures utilized during all past PIs of the project (i.e., 'legacy' type inspections), and did not incorporate the Levee Inspection Checklist distributed in June 2007. When the report documenting the inspection and findings was being written, it was determined that failure to use the new inspection checklist was inappropriate. Therefore, information from the legacy inspection was transferred to the new inspection template. During this transfer, it became apparent that the more subjective ratings from previous inspections of the Floodway would be replaced by ratings determined in accordance with the very specific language and rating criteria described on the checklist. As a result, significant deficiencies were documented that resulted in unacceptable ratings for each of the systems in the Floodway, and for the Dallas Floodway project overall. Items that generated unacceptable ratings include:

- Insufficient crest height rendering the East and West Levees incapable of successfully accommodating the Standard Project Flood without overtopping
- Significant encroachments and penetrations that impact the integrity and performance of the levees, as well as inhibit access for O&M, surveillance, and flood-fighting purposes
- Damaged gate closures
- Unstable structures
- Severe desiccation cracking of the levees
- Erosion
- Vegetation
- Siltation
- Channel instability

In addition to numerous unacceptable ratings, it was determined that the Dallas Floodway does not meet current USACE design criteria regarding relevant factors of safety for embankment stability and seepage gradients.

It is noted that the results of the inspection identify negative impacts during base flood (100-year event) conditions which would jeopardize performance of project features to reliably function as authorized. This is a significant concern that may have a substantial negative impact on FEMA flood mapping of the areas outside the levees and the residents and businesses protected by those levees. The City of Dallas is currently designing fixes intended to address deficiencies related to the 100-year event.

According to the Inspection Report Template, the East Levee and West Levee systems had one or more items rated as unacceptable. Since there is a significant number of deficiencies that would prevent the systems from performing as intended, the overall rating for the Dallas Floodway project is unacceptable (see Exhibit 2).

Exhibit 1 is a list of the deficiency items that require USACE-SWF (Southwest Division-Fort Worth District) to address during development of the comprehensive plan.

6.0 CURRENT SUBSURFACE INVESTIGATIONS

In order to obtain subsurface information necessary to determine the existing conditions of the site soil and rock samples were collected for geotechnical testing and characterization of the study area. Additionally, a review of available geotechnical data obtained by USACE during previous investigations of the site, as well as subsurface data obtained by others in support of the design of bridges, trails and other structures within the Floodway, was performed. A summary of these investigations are described below.

6.1 North Texas Tollway Authority (NTTA), February 2009

In February 2009, Fugro was awarded a contract by NTTA to perform geotechnical investigations for approximately 8 miles of the East Levee. This investigation was conducted for the Trinity Parkway. 215 borings were drilled and 147 cone penetrometer tests (CPT) were performed.

Drill rigs models CME 55 and CME 75 were used for the investigation. In order to prevent damage to the levee and levee foundations, the drilling was performed in accordance with procedures cited in ER 1110-1-1807, Procedures for Drilling in Earth Embankments. Eight-inch hollow stem augers were used for advancing the boreholes. Continuous samples (1- to 2-foot centers) were obtained in the levee embankment and upper 30 feet of levee foundation materials and 5-foot centers thereafter. Standard Penetration Tests were performed during the investigation in order to determine the relative density of the granular materials within the floodplain. Additionally, Shelby tube, split spoon, and 2 inch diameter rock core samples were collected, sealed in airtight containers, and taken to the laboratory.

6.2 City of Dallas, June 2009

In June 2009, HNTB was contracted by the City of Dallas to perform geotechnical investigations for the Dallas Floodway System Levee Remediation Plan. 540 borings were drilled and 415 CPTs were performed.

CME 55 and CME 75 drill rigs were used for the investigation. In order to prevent damage to the levee and levee foundations, the drilling was performed in accordance with procedures cited in ER 1110-1-1807, Procedures for Drilling in Earth Embankments. Eight-inch hollow stem augers were used for advancing the boreholes. Continuous samples (1- to 2-foot centers) were obtained in the levee embankment and upper 30 feet of levee foundation materials and 5-foot centers thereafter. Standard Penetration Tests were performed during the investigation in order to determine the relative density of the granular materials within the floodplain. Additionally, Shelby tube, split spoon, and 2 inch diameter rock core samples were collected, sealed in airtight containers, and taken to the laboratory.

6.3 Previous USACE Investigations

Additional geotechnical data and reports evaluated as part of the assessment of subsurface conditions for this project include:

1) USACE, September 1952, Definite Project Report, Dallas Floodway. The original levee system constructed by the City of Dallas was completed in 1932; however, during the April 1942 flood, it became apparent that insufficient interior drainage and poor levee construction practices (lack of proper compaction and moisture control for levee materials, and incomplete cutoff of seepage through granular layers) had produced an inadequate flood protection system that was in danger of failing under flood loads of the same or greater magnitude as the April 1942 event. As a result, the City solicited the help of USACE in improving the project (Ajemian et al 2003). USACE was granted authority to participate in the project under the Rivers and Harbors Act of 2 March 1945 (RHA 1945), and the Rivers and Harbors Act of 17 May 1950 (RHA 1950). RHA 1945 authorized the 'strengthening' of levees previously built by the City of Dallas and clearing of the floodway on the Elm and West Forks. RHA 1950 provided for increasing the existing pump and sump capacities and construction of pressure sewers, diversions, and gravity outlets to facilitate interior drainage.

For the design of 'strengthening measures' for the levees, as well as design of structures to provide additional pumping capacity to accommodate interior drainage, a total of 32 auger and Dennison barrel borings were advanced at locations across the floodway. No rock samples were reported from this investigation; however, the report documents that rock was encountered at an elevation of 349.7 feet.

2) USACE, September 1953, Seepage Investigations of West Levee. This study was performed to evaluate the need for underseepage cutoff measures on the West Levee between Stations 134+90 and 196+40. Subsurface investigations undertaken during the original design of the project in the 1920s indicated the presence of sand and gravel lenses in this area; however, during the actual construction of the levee, the area was excavated down to the water table. At that point, a dragline was used to remove the wet sands and gravels all the way down to the top of the underlying shale. The dragline operator then scarified the surface of the shale by moving the bucket back and forth, forming a slurry of shale, gravel and sand. The 1953 study concluded that this established a successful cutoff since thirteen borings taken at that time showed gravel in only 3 of the holes, and sand in 6. The effectiveness of this cutoff was characterized as being 'at least 50% effective.' Additional test pits excavated as part of this study were considered to support the recommendation that no further cutoff was required for the levee under the current configuration of the area.

3) USACE, June 1968, Review of Levee Design, Dallas Floodway. In the 8 years following completion of the levee strengthening in 1959, 23 shallow slides occurred along the alignments of both the East and West levees. This prompted a review of the construction history and design data, along with limited analysis of the reconstructed levee to determine if additional rehabilitation was required.

4) USACE, November-December 2004, Upper Trinity Feasibility Project. In November 2004, Giles Engineering was awarded a contract to perform geotechnical investigations for the Upper Trinity Feasibility project. Inspection services were provided by the Vicksburg District of USACE. Originally, 14 borings were scheduled for drilling; however, heavy rainfall throughout the drilling period resulted in high river levels that prevented some borings from being drilled.

Giles started the investigation with a truck mounted CME 45 drill; however, when that drill proved incapable of drilling to the required depths, a CME 55 and CME 75 were brought onsite to complete the investigation. In order to prevent damage to the levee and levee foundations, the drilling was performed in accordance with procedures cited in ER 1110-1-1807, Procedures for Drilling in Earth Embankments. Eight-inch hollow stem augers were used for advancing the boreholes. Samples were generally obtained at 5-foot intervals within the levee embankment and at 10-foot intervals within the foundation. Standard Penetration Tests were performed during the investigation in order to determine the relative density of the granular materials within the floodplain. Additionally, Shelby tube, split spoon, and 4-inch diameter rock core samples were collected, sealed in airtight containers, and taken to the laboratory of TEAM Consultants, Inc. in Arlington, Texas for testing.

6.4 Other Investigations

The City of Dallas provided copies of previous subsurface investigations it had performed within the Floodway. This information was reviewed for this feasibility study and serves to supplement the most recent City of Dallas investigations and previous USACE investigations. Brief summaries of those investigations are provided below.

1) Rone Engineers, Inc., Volume 1, Environmental Investigation, 120-Inch Interceptor, July 1995. Environmental samples were obtained from 23 borings drilled between the AT&SF Railroad Bridge and the Central Wastewater Treatment Plant. Information on the boring logs appears to indicate that soil samples were classified in the field by an experienced geologist. Groundwater levels encountered during drilling are recorded on the logs. The occasional presence of fill materials including glass, sand, brick and wood is also noted throughout the investigation area.

Environmental testing confirmed the presence of relatively low-level soil and groundwater contamination, including metals, total petroleum hydrocarbons, and semi-volatiles. No volatiles were encountered. As part of this investigation, four monitoring wells were installed to evaluate groundwater contamination. The current status of these wells is unknown.

2) Maxim Technologies, Inc., Trinity River Floodplain Modification, September 1995. The stated primary purpose of this investigation was to evaluate the Trinity River Floodplain as a borrow source for use as *'levee material or … as capping materials for …. dredge spoils"*. Additionally, sampling was performed in the Trinity River channel to evaluate river sediments for Priority Pollutant Metals analysis.

Utilizing continuous flight augers and rock coring bits, ten borings were advanced on the north and south sides of the Trinity River between the AT&SF and Corinth Street Bridges. Each boring was advanced to a depth of 20 feet using a truck mounted rotary rig. Six piezometers were installed to evaluate fluctuations of the groundwater. Shelby tube samples were obtained from each borehole, with NX size cores obtained when rock was encountered in 5 of the 10 borings. Collected soil samples were analyzed for Atterberg limits, moisture content, dry unit weight, moisture-density relationships (Standard Proctor), gradations and permeability.

The results of this investigation showed that fill material, including rubble, had been placed on the north side of the river channel. A 6-foot thick layer of gravel was also encountered on that side of the river. Rock was not encountered on the north side of the river within the 20-foot depth drilled.

On the south side of the river, limestone rock was encountered in all 5 borings. The depth to rock varied from 8 to 14 feet. Additionally, sand and gravel layers from 2 to 4.5 feet thick were encountered on top of the limestone primary.

3) Terra-Mar, Trinity River Floodplain Modification, October 1999. In support of the City's plans to construct lakes within the Trinity River flood plain, Terra-Mar performed a geotechnical investigation that included evaluation of the suitability of material types encountered for levee and road fills, underseepage concerns, and dewatering requirements during construction. Thirteen borings were drilled to depths of 15 feet using marsh buggy and truck-mounted drill rigs. Both Shelby tube and split-spoon samples were obtained for lab testing, which included Atterberg limits, moisture contents, unconfined compression strength, moisture-density relationships (Standard Proctor), sieve analyses and permeability. Testing for the presence of environmental contamination was also performed. Results from that testing confirmed concentrations of chromium, lead and mercury above regulated groundwater protection limits; but the presence of pesticides, herbicides, volatiles, and semi-volatiles above detection limits was not indicated. Even though the metals concentrations were above allowable groundwater limits, Terra-Mar stated that:

"…the floodplain soils do not appear to have hazardous levels of contaminants that would preclude the use of these soils for construction of on-site berms and roadway embankments."

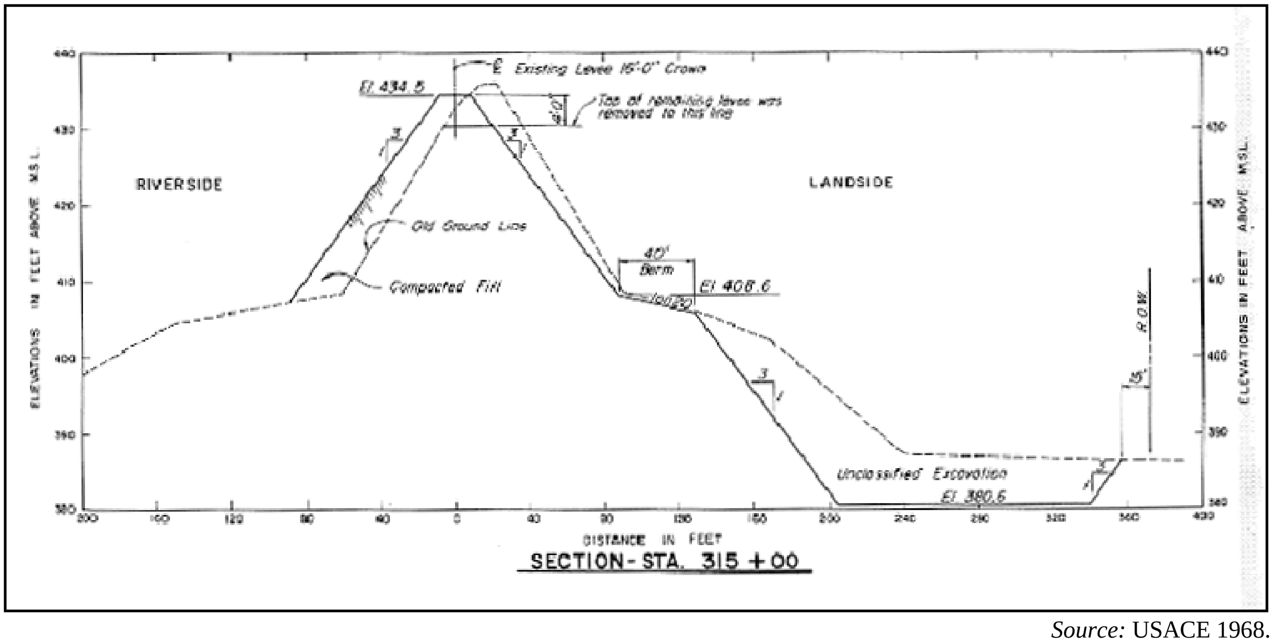
The Terra-Mar study concluded that seepage control measures, including a slurry wall or clay cutoffs, would be required to prevent water losses for the proposed lake(s). In addition to the field investigation performed as part of this study, Terra-Mar summarized previous studies of the project area including those borings obtained for bridge design purposes.

4) Archaeological Testing for the Trinity River Parkway, Interim Report dated 11 April 2006. In spring 2006, AR Consultants performed a trenching investigation to evaluate cultural resources within the floodway. Of particular concern relative to the flood protection system was the discovery of numerous landfill areas. The Interim Report notes that such deposits were frequently observed near the downtown Dallas area, closer to the East levee than to the channel. The landfill materials were described as ranging from limestone rubble to construction and demolition debris.

7.0 EXISTING LEVEE SYSTEM

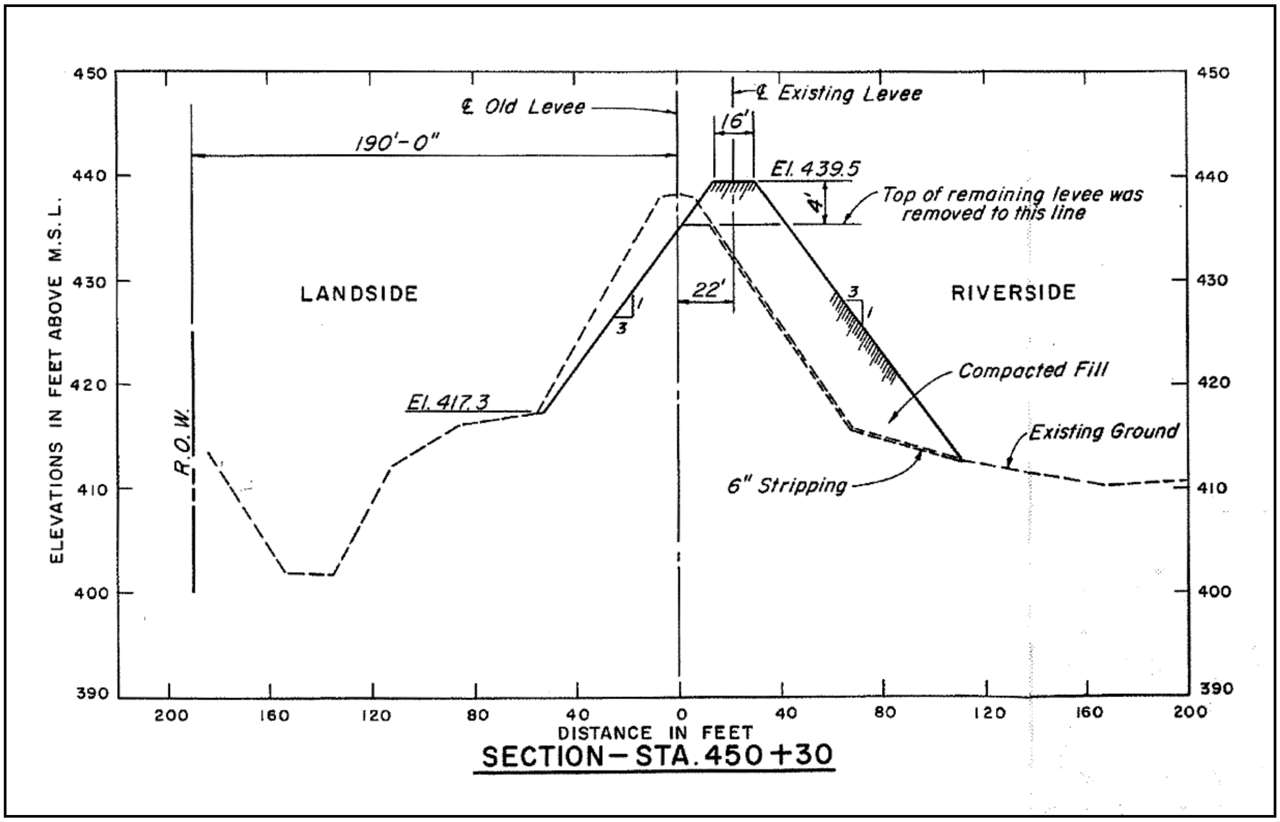
7.1 CONSTRUCTION

As constructed by the City and County of Dallas in the 1930s and 1940s, the levees comprising the Dallas Floodway project had side slopes of 2.5H:1V, a crest width of six feet, and a maximum height of 35 feet. USACE constructed modifications performed in the 1950s included flattening the existing 2.5H:1V slopes to 3H:1V or flatter (up to 4H:1V) and widening the crest from six to 16 feet. As shown in Figures 7-1 and 7-2, most of the additional fill material added to the levee slopes was added on the riverside of the levees. Prior to placement of additional material, the levee was stripped to a depth of six inches and the top four feet of the embankment removed to facilitate construction of the wider crest.



Source: USACE 1968.

Figure 7-1. Typical Cross-Section of East Levee Modifications



Source: USACE 1968.

Figure 7-2. Typical Cross-Section of West Levee Modifications

7.2 LEVEE EMBANKMENTS

Evaluation of subsurface data indicates that the existing levees consist of well-compacted, high plasticity clays. The data indicates that both the East and West embankments are very consistent with respect to material distribution. Nearly half of the samples obtained from the embankment had moisture contents less than the plastic limit, a condition indicative of desiccation. These drier soils were generally found in the upper 25 feet of embankment, although some were found at depths close to the base of the embankment, i.e. approximately up to 36 feet. It should be noted that these samples were obtained in Fall 2004, one of the wettest years on record. The current 2-year drought began several months after that investigation was completed, so samples obtained at this time would likely show moisture contents lower than those recorded for the 2004 samples.

7.3 FLOODS OF RECORD

In May of 1908, Dallas experienced its highest flood in recorded history: a flood stage of 52.6 feet. As a result of that flood, the levees were constructed to provide flood protection for the City. Since the Dallas levees were modified by USACE in the 1950s, the Flood of Record for the Dallas Floodway occurred on 3 May 1990 (Figure 7-3) when a discharge of 82,300 cfs was recorded. Although flood levels were high, the maximum flood height for this event, elevation 415.1 (gauge height of 47.1 feet), was more than 14 feet below the design crest elevation. The Flood of Record has an estimated probability of annual occurrence of 0.022, roughly equivalent to a '45-year storm'.

Figure 7-3. Photo of Modern Flood of Record for the Project, 3 May 1990

7.4 HISTORICAL PERFORMANCE

To date, both the East and West levees have performed well at the flood levels that have occurred since the levees were modified by USACE. However, when evaluating the existing flood protection system, it is important to consider the levels of previous floods that the levees have retained. All other conditions being equal, levees that have performed well at higher levels will usually perform well at floods of equivalent or lesser height; however, good performance at higher levels is not guaranteed. This is because higher floods exert higher forces on the levee. In addition to stability related concerns generated by these greater forces, underseepage is of greater consequence during higher floods, since the higher water pressures may find previously unidentified weaknesses in the embankment and foundation.

Figure 7-4 was taken on 15 June 1989 during a flood event that peaked at Elevation 409.6 on the Commerce Street gauge with a discharge of 43,000 cfs. A flood occurring a month earlier on 17 May 1989 peaked at Elevation 411.3 – a flood having an estimated frequency corresponding to a 15-year event. Although the water is high on the levee in this photo, it is still 20-feet below the design flood elevation of 429.4 (as measured at the Commerce Street gauge). Duration of flood events is also of significance when evaluating levee performance; however, stage hydrographs that show the retreat of floodwaters in hours or days are not indicative of the porewater pressures that exist within an embankment that has accommodated back-to-back floods or significant steady precipitation. After the back-to-back flood events in 1989, three slides required repair. Five slides requiring repair developed before and after the Pool of Record event in 1990. Two of the slides were discovered in January and the other three slides were discovered in July.

Figure 7-4. 15 June 1989 Flood Event

While the overall performance of the levees has been good, hundreds of shallow infinite slope failures requiring repair have developed on the highly plastic clay embankments. Repairs generally consist of removal of the soil materials within the slide area, mixing of those materials with lime, placement and compaction of the amended fill, followed by revegetation. Records maintained by the Dallas Flood Control District (DFCD) show that 283 such slides have been repaired since 1966. Each repair is numbered and tracked by levee station, the date the repair was started, and the date the repair was completed. Additionally, each slide is described and its location relative to the levee geometry documented. DFCD personnel report that the slides occur randomly throughout the floodway and that neither levee appears to be more prone to developing these shallow slides. Under normal conditions, these slides are considered a recurring maintenance issue; however, under design flood conditions, these slides could become critical.

8.0 DESCRIPTION OF LEVEE REACHES

8.1 Reach Criteria

The levee system along the Trinity River was separated into nine reaches using geotechnical and geological criteria in order to characterize its existing condition. This was achieved by grouping areas of common conditions into reaches. Geotechnical criteria used to define the reaches are defined in Table 8-1.

Table 8-1. Geotechnical Criteria Used to Define Reaches

Levee Material	*Foundation Material*	*Levee Geometry*	*Misc. Concerns*
High Plasticity Clays (CH)	Depth to basal sands and gravel	Slope angles	Bridge piers
Low Plasticity Clays (CL)	Are basal sands and gravels semi-continuous/continuous?	Crest elevation	Buildings encroaching on levee
Other	Top stratum thickness in drainage ditch	Drainage ditch Locations and depth	Lack of data (boring or CPT)
Historic Landslide events	Historic Landslide events	Depression locations on river side of levee	

Geological cross-sections were generated using the existing borings in order to characterize existing geological conditions. A summary of geological conditions found to be unique to each reach is found in Table 8-2. A more thorough discussion follows each geotechnical description for each levee.

Table 8-2. Summary of Geological Conditions for Each Reach

Reach	*Foundation Conditions*
1	Austin Chalk formation occurrence Terrace deposits, continuous sand and gravel deposits
2	Continuous basal sand and gravel deposit
3	Characterized by a relatively unweathered Eagle Ford with a basal sand layer, and a weathered Eagle Ford shale with point bar and alluvium deposits on top
4	Continuous basal sand and gravel deposit.
5	Highly weathered, irregularly surfaced shale, and a semi-continuous sand and gravel layer
6	Terrace deposits, semi-continuous sand and gravel deposits
7	-Semi-continuous basal sand and gravel
8	Irregular, highly-weathered shale, and a semi-continuous basal sand and gravel layer
9	Semi-continuous sand and gravel deposit, with a very wide alluvium deposit on top

8.2 Levee Soil

Levee soil plays a major role in the overall stability of the levee. The strength of the high plasticity clays has decreased over time due the effects of weathering caused by alternating cycles of wetting and drying. Areas where the levee is predominately constructed with high plasticity clays also coincide with areas where landslides have occurred in the past and areas where landslides are more than likely to occur in the future.

8.3 Foundation Soil

Foundation soils were used to discern cross-sections where there would be a problem with seepage from areas where seepage would not be problematic. The most critical situation for sections with seepage was characterized by presence of granular material that had a minimal amount of fines (percent by weight passing the #200 sieve). This type of material classifies as either a poorly graded or well graded sand or gravel (SP, SW, GP or GW). Pervious soil becomes an issue when they form a continuous layer underneath the levee due to their potential to carry a high amount of water flow when subjected to differential head. For sections with a drainage ditch on the protected side of the levee, this issue is compounded because the presence of the ditch effectively decreases the blanket thickness (seepage resistance). The illustration presented on Figure 8-1 depicts this condition.

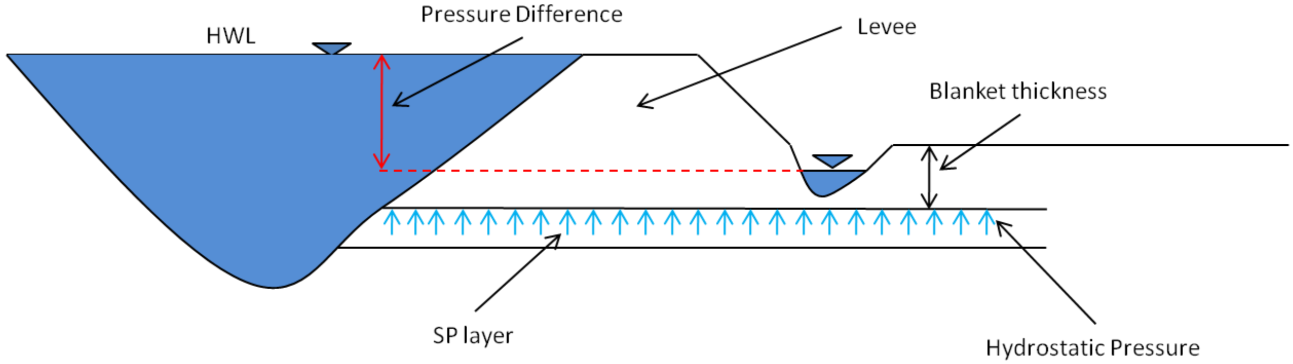


Figure 8-1. Clean Granular Material Underlying Levee with Drainage Ditch on the Protected Side of the Levee

8.4 Levee Geometry

Historic landslides occur at levee locations composed of high plasticity clays (CH) and steep levee slopes. The slopes along the levee system are similar on both the landside and riverside slopes, and vary at different stations. Areas with steeper slopes, on either the river side or landside, often coincide with areas where there are slope stability issues. These stability issues are further compounded when the levee is constructed of high plasticity clays (CH).

Additionally, the location of a drainage ditch along the protected side of the levee as well as its proximity to the toe of the levee can cause slope stability and seepage issues during a high water event.

8.5 Miscellaneous Conditions

Other conditions that were taken into account were the amount of data currently available, the location of neighboring structures and the location of bridge piers.

8.6 Levee Reaches

Figure 8-2 depicts the levee reaches along the Trinity River. There are 9 reaches total, with 5 along the east side levee and 4 along the west side levee. The historic landslide data referred to in the following commentary mainly pertains to the river side of the levee. The majority of the slides for Reaches 1 and 4 occurred on the landside, while the majority of slides for Reach 5 occurred on the riverside. There was little or no historical slide activity reported for Reaches 2 and 3.

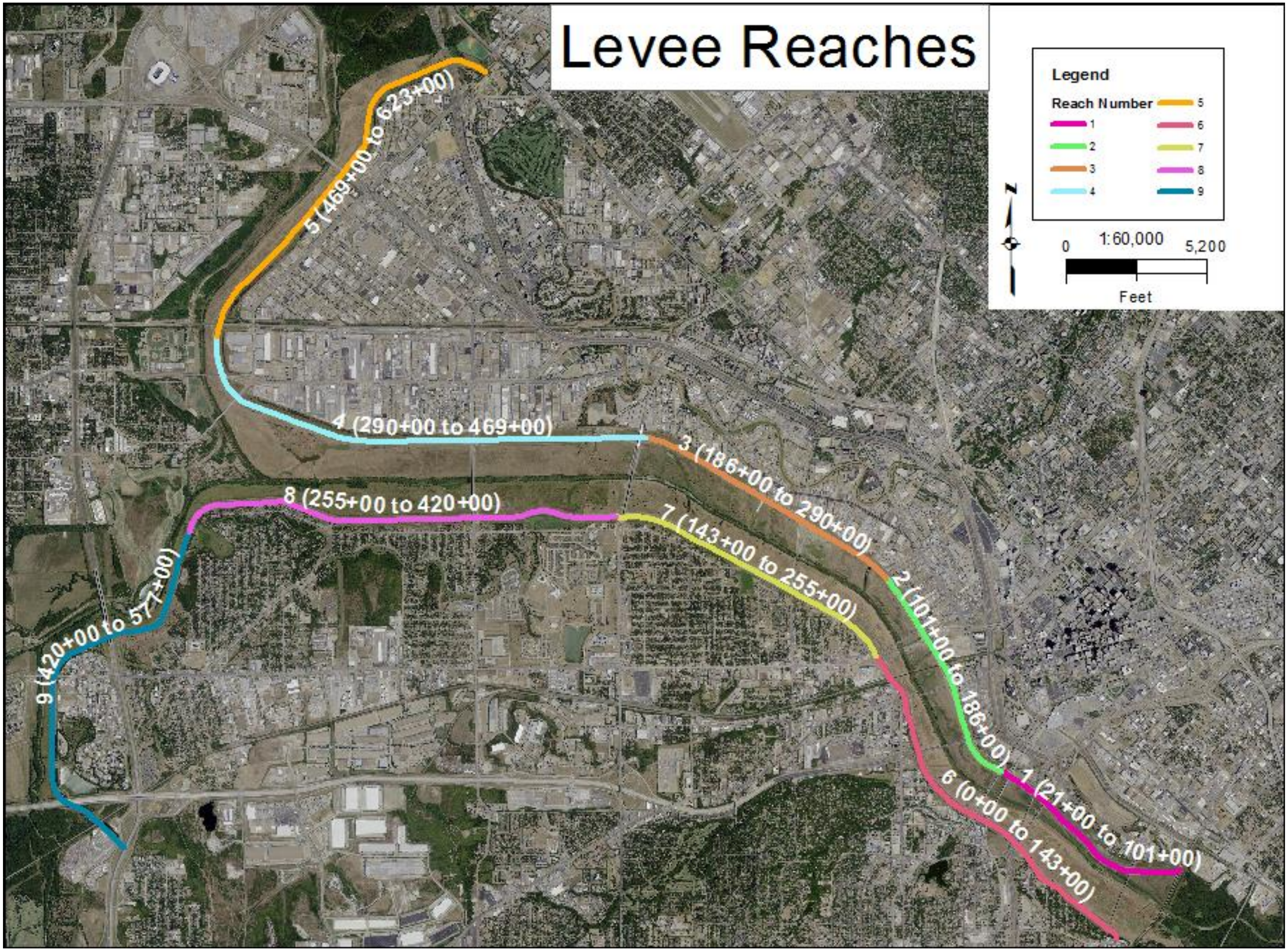


Figure 8-2. Trinity River Levee Reaches

8.7 East Levee

The East Levee contains Reaches 1 through 5. A brief description of each reach follows:

- Each of the five reaches along the east levee contain areas where the basal sands are relatively shallow (10 to 30 feet) and continuous under the levee.
- All the reaches along the east side levees contain a drainage ditch on the protected side of the levee except for Reach 2 and part of Reach 3.
- Reach 1 levees are mainly constructed of high plasticity clays (CH). The presence of CH and steeper slopes (between 3H:1V and 4H:1V) is made apparent by the amount of historic landslides which is shown in Figure 8-3.
- Reach 2 levees are mainly composed of CH with a few pockets of low plasticity clays (CL). Figure 8-3 shows that there are no landslides recorded along this reach. The reason for this may be that the slopes along this reach are shallower on average 4H:1V.
- Reach 3 levees are composed of two subreaches composed of CL and CH, respectively. There is also a lack of historic landslide data along this reach, which is more than likely due to shallower levee slopes (on average 3.5H:1V on riverside) and the levee material.
- Reach 4 levees are composed of CH with a large amount of historic landslide data. The slopes are on average between 3H:1V and 3.5H:1V. Seepage is a concern along this reach due to the depth and continuity of the basal sand.
- Reach 5 levees are composed of CH. This reach contains a large amount of historic landslide data. The slopes along this reach average between 3H:1V and 3.5H:1V on the riverside of the levee.

Typical features that describe each reach along the East Levee are summarized in Table 8-3.

Table 8-3. Features Defining Each Reach for the East Levee

Reach Feature	*Reach*				
	1	*2*	*3*	*4*	*5*
Typical Landside Slopes:					
CL Offset:20-60 feet	3.8H:1V	3.8H:1V	3.1H:1V	3H:1V	3.3H:1V
CL Offset: 60-100 feet	3.8H:1V	3.8H:1V	3.75H:1V	4.0H:1V	3.75H:1V
Typical Riverside Slopes:					
CL Offset:20-60 feet	3.75H:1V	3.75H:1V	3.5H:1V	3.5H:1V	3.2H:1V
CL Offset: 60-100 feet	3.75H:1V	3.75H:1V	3.5H:1V	3.75H:1V	3.75H:1V
Embankment Material	CH	CH with pockets of CL	50% CH 50% CL	CH	CH
Depth to top of Shale @CL (feet)	91.6	88.2	62.6	66.1	81.4
Sand Depth from Surface (feet)	53	58	35	46	58
Average thickness of basal sand @ CL(feet)	6.4	8.3	5.5	6.5	5.1
Are Basal Sands Semi-Continuous/Continuous across levee?	Yes	Yes	Yes	Yes	Yes
Number of historical slides:					
On Riverside slope	7	0	0	9	75
On Landside slope	14	1	0	20	1

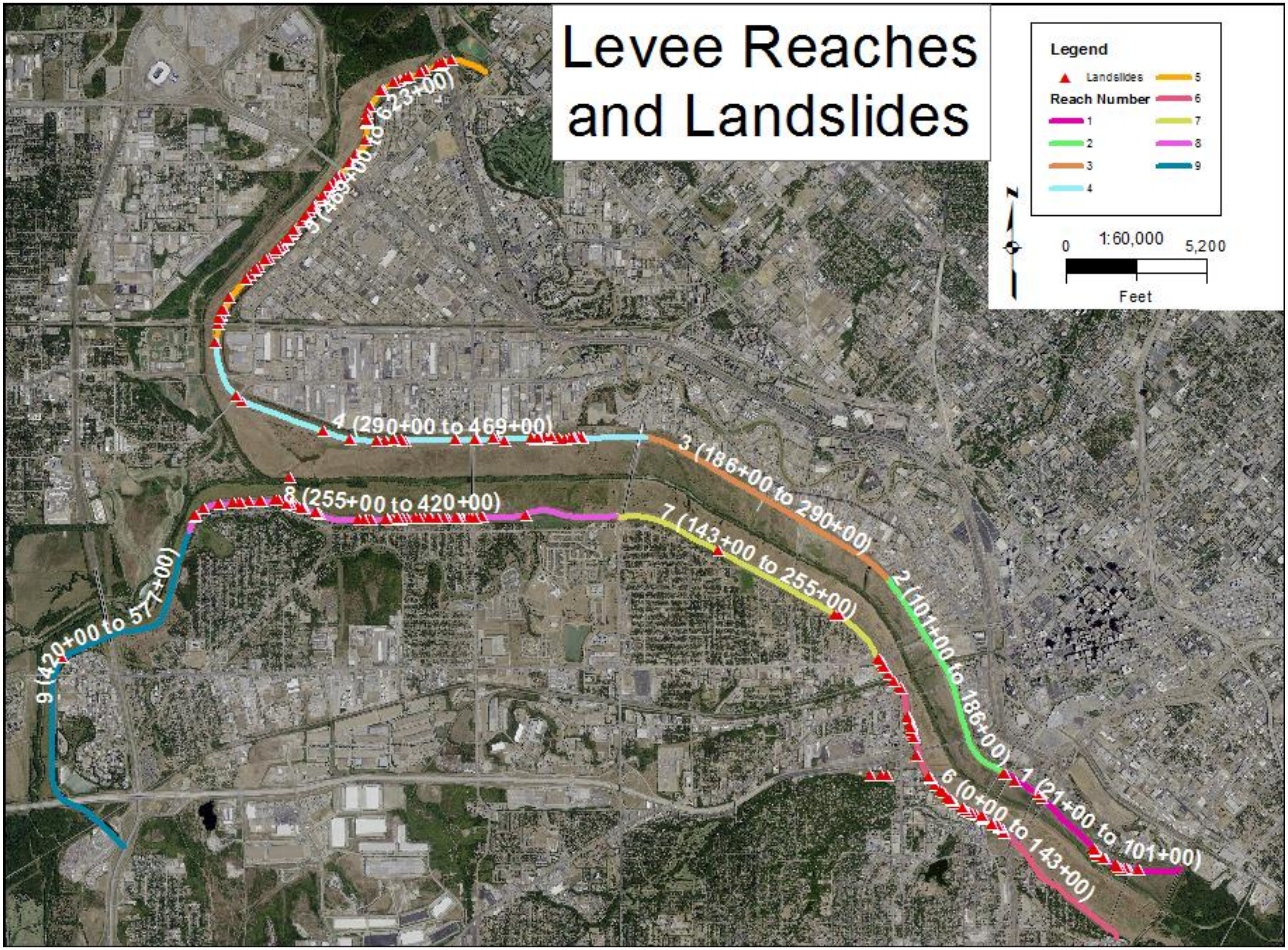


Figure 8-3. Levee Reaches and the Location of Historic Landslides (red triangles)

8.8 SUBSURFACE CONDITIONS FOR THE EAST LEVEE

Foundation conditions at each reach support the reach delineation by the Engineer Research and Development Center (ERDC) using geotechnical criteria. A general description of each reach and a geological cross-section follows. Color legend depicting the lithology is shown in Figure 8-4.

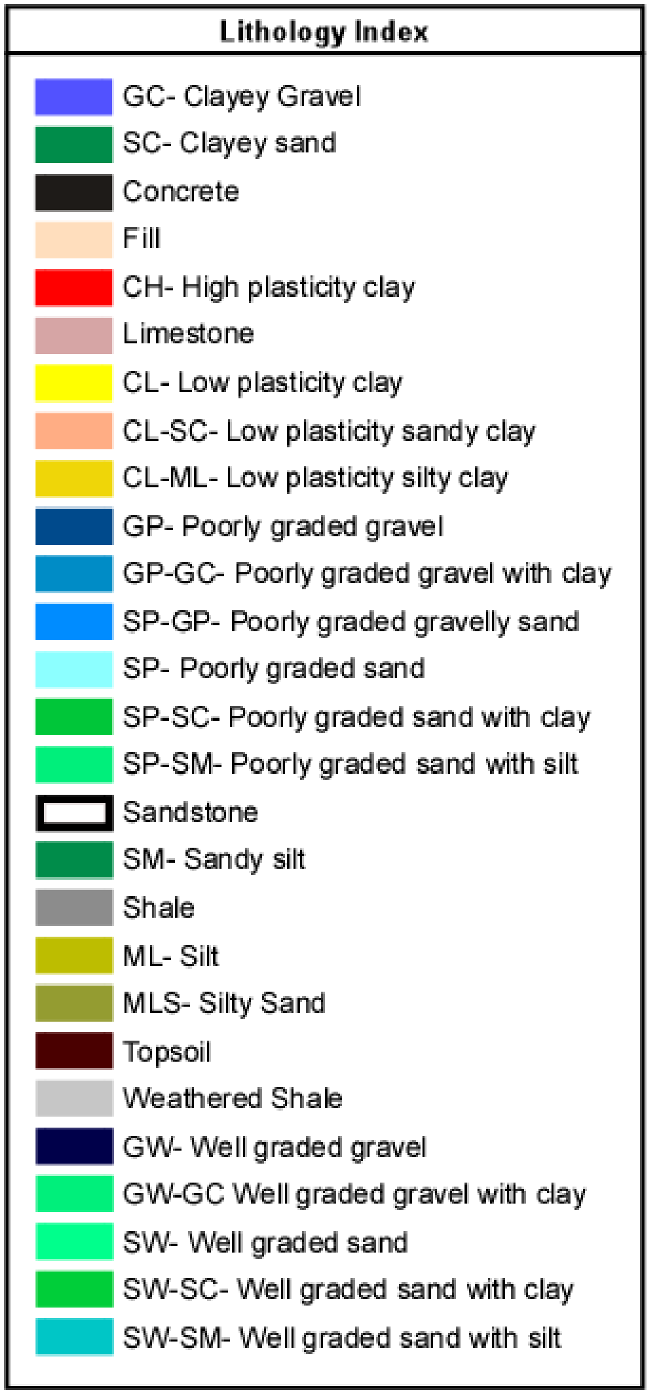


Figure 8-4. Color Legend to be Used with the Lithologic Cross-Section.

8.8.1 Reach 1

The foundation for Reach 1 comprises a unique geologic setting relative to other reaches of the levee system. The levee in this area partially sits on the Austin Chalk Formation, and a mixture of the basal sands and gravels, and the Eagle Ford (Figure 8-5). Quaternary sediments in this area had been identified as terrace and backswamp deposits. Levees in this location are composed mostly of high plasticity clays.

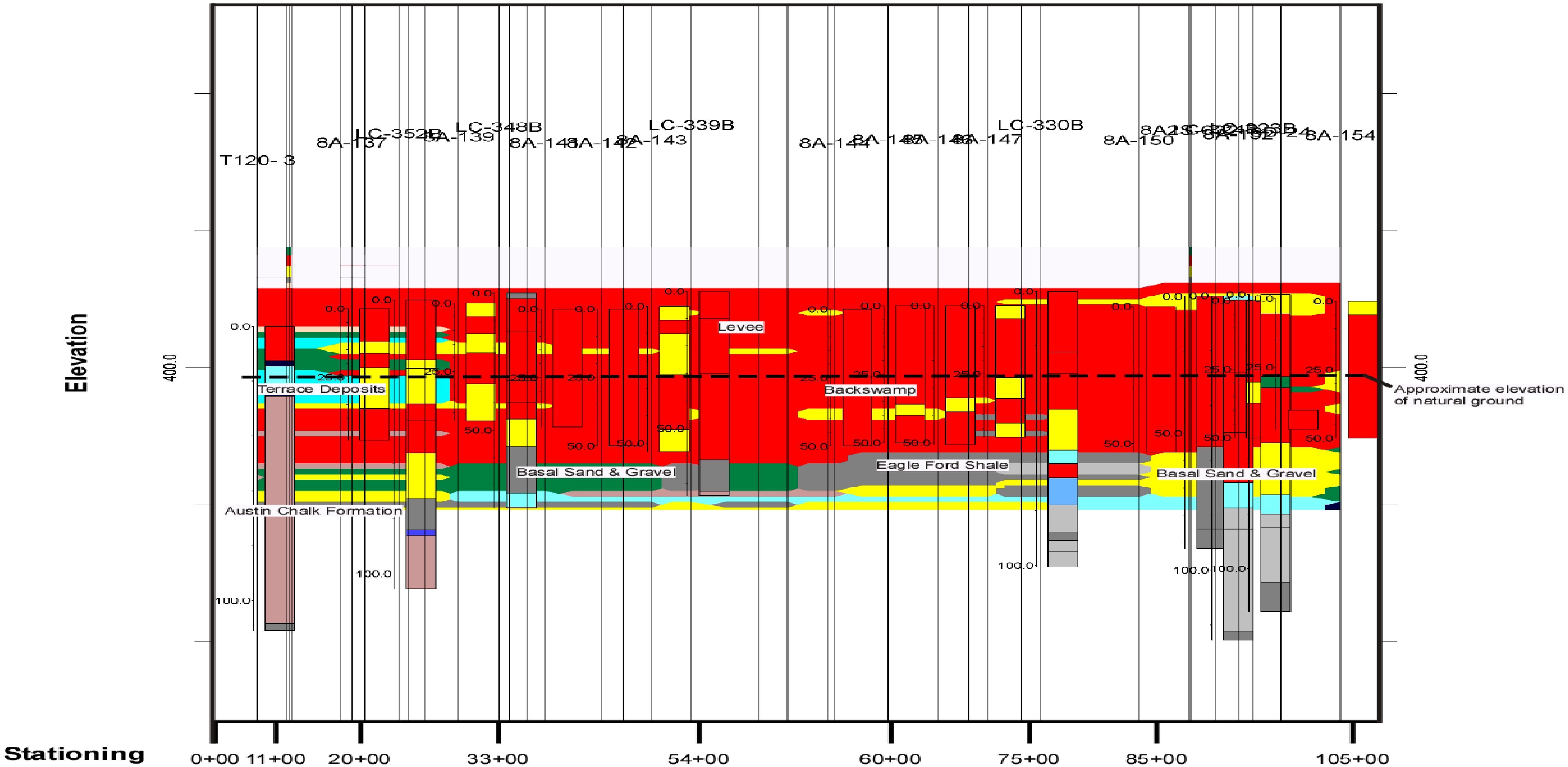


Figure 8-5. Reach 1 Geologic Cross-Section

8.8.2 Reach 2

The levee foundation in Reach 2 is mostly composed of high plasticity clays. Quaternary deposits in this location are undifferentiated alluvium and backswamp deposits, which overlie almost continuous basal sand and gravel layer on top of the Eagle Ford. The Austin Chalk has not been identified within this reach using the existing boring data (Figure 8-6).

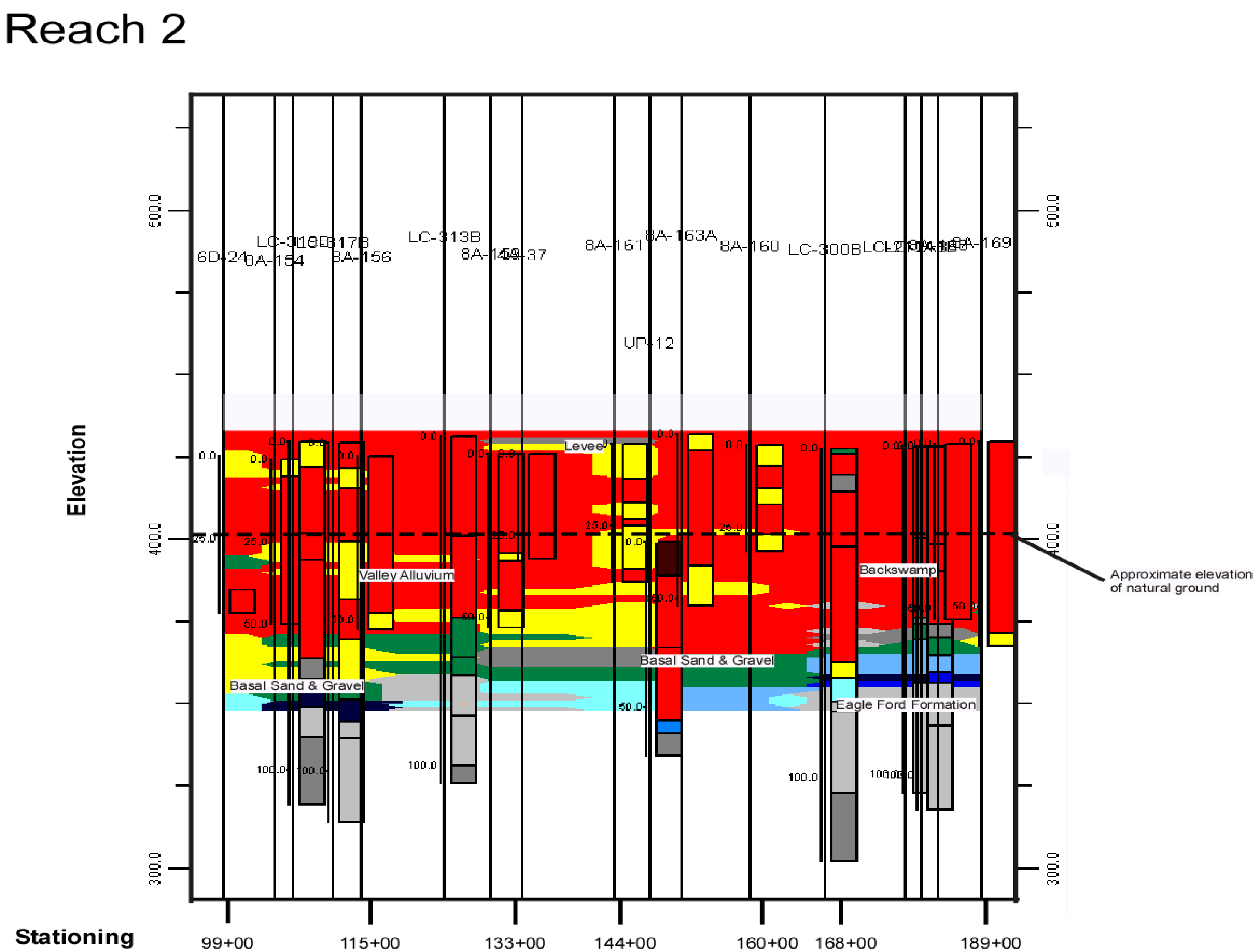


Figure 8-6. Reach 2 Geologic Cross-Section

8.8.3 Reach 3

The foundation of Reach 3 is composed mostly of point-bar and backswamp deposits. The backswamp deposits near stationing 181+00 sits on top of basal sand and gravel, whereas point-bar deposits overly a relative rise of the Eagle Ford Shale. According to boring data, this location can be subdivided into a relatively unweathered zone of the Eagle Ford, overlain by basal sand and backswamp deposits, and weathered Eagle Ford overlain by point-bar and finer sediment deposits (Figure 8-7).

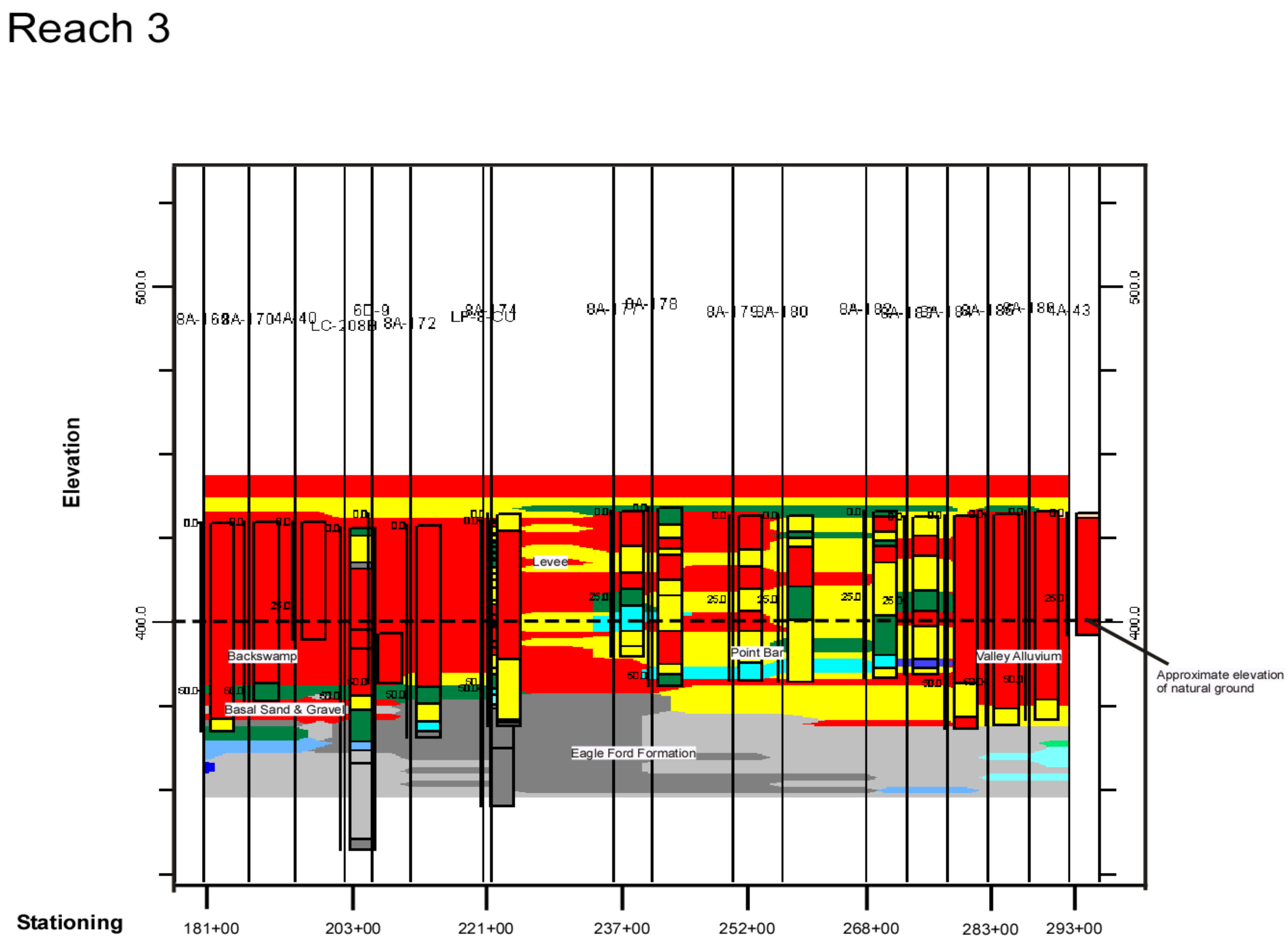


Figure 8-7. Reach 3 Geologic Cross-Section

8.8.4 Reach 4

At Reach 4, the Eagle Ford in this area occurs in a topographic low (Figure 8-8). Overlain by an irregular shale surface, there is a continuous basal sand and gravel deposit, followed by undifferentiated alluvium deposits. This reach is characterized by a fairly thick sand and gravel layer (approximately 5-15 feet). Levees in this location are composed of sand lenses as well as CL soils. Levee composition is mostly high plasticity clay.

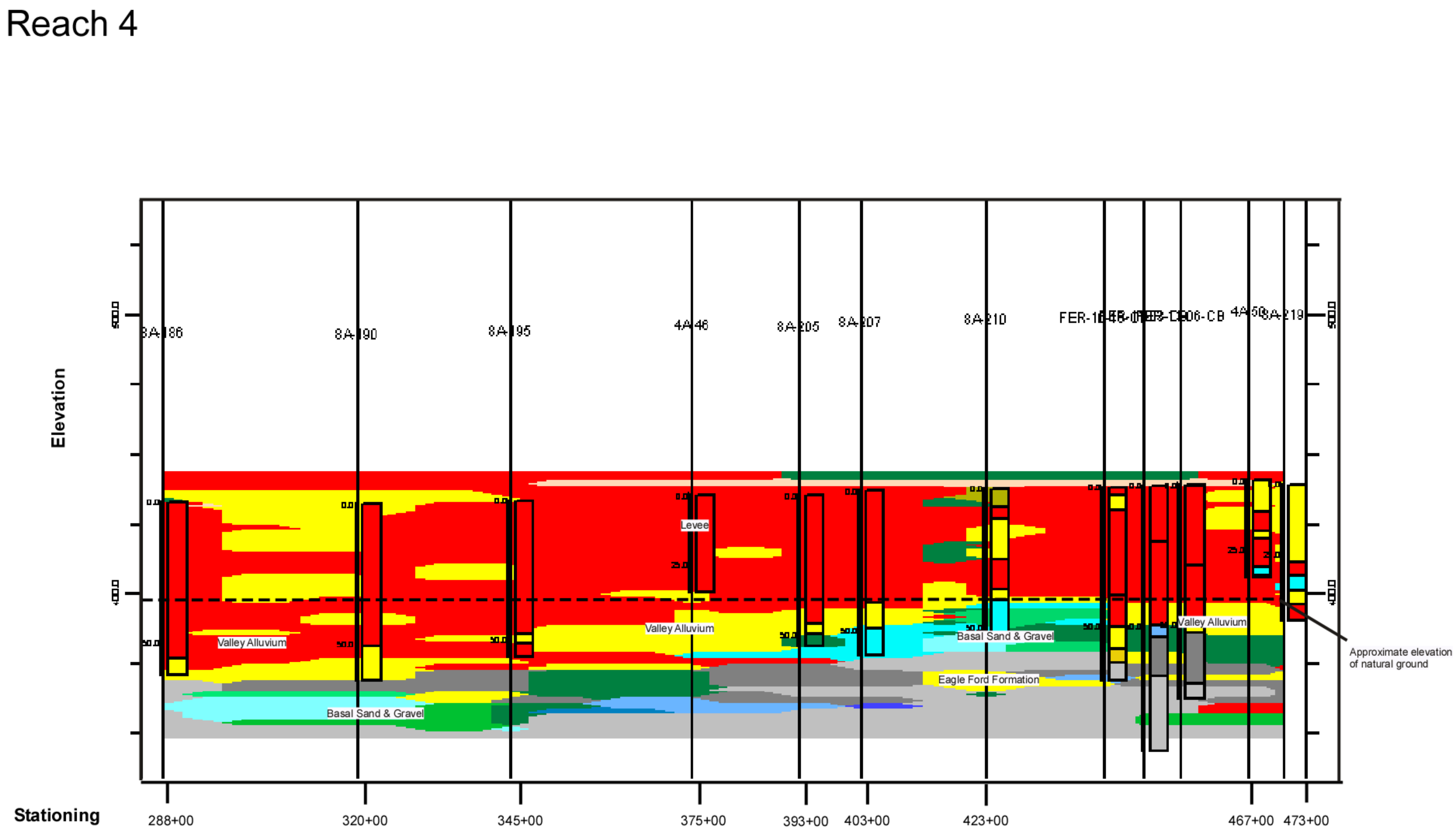


Figure 8-8. Reach 4 Geologic Cross-Section

8.8.5 Reach 5

In Reach 5, irregularly shaped and highly weathered Eagle Ford shale forms the bedrock (Figure 8-9). Lows in the shale represent incised channels that are evidence for old paths of the Trinity River. A continuous basal sand and gravel layer sits on top of the weathered shale. Backswamp and point-bar deposits are sitting on top of the basal sand and gravel. Levees in this are composed mostly of high plasticity clay.

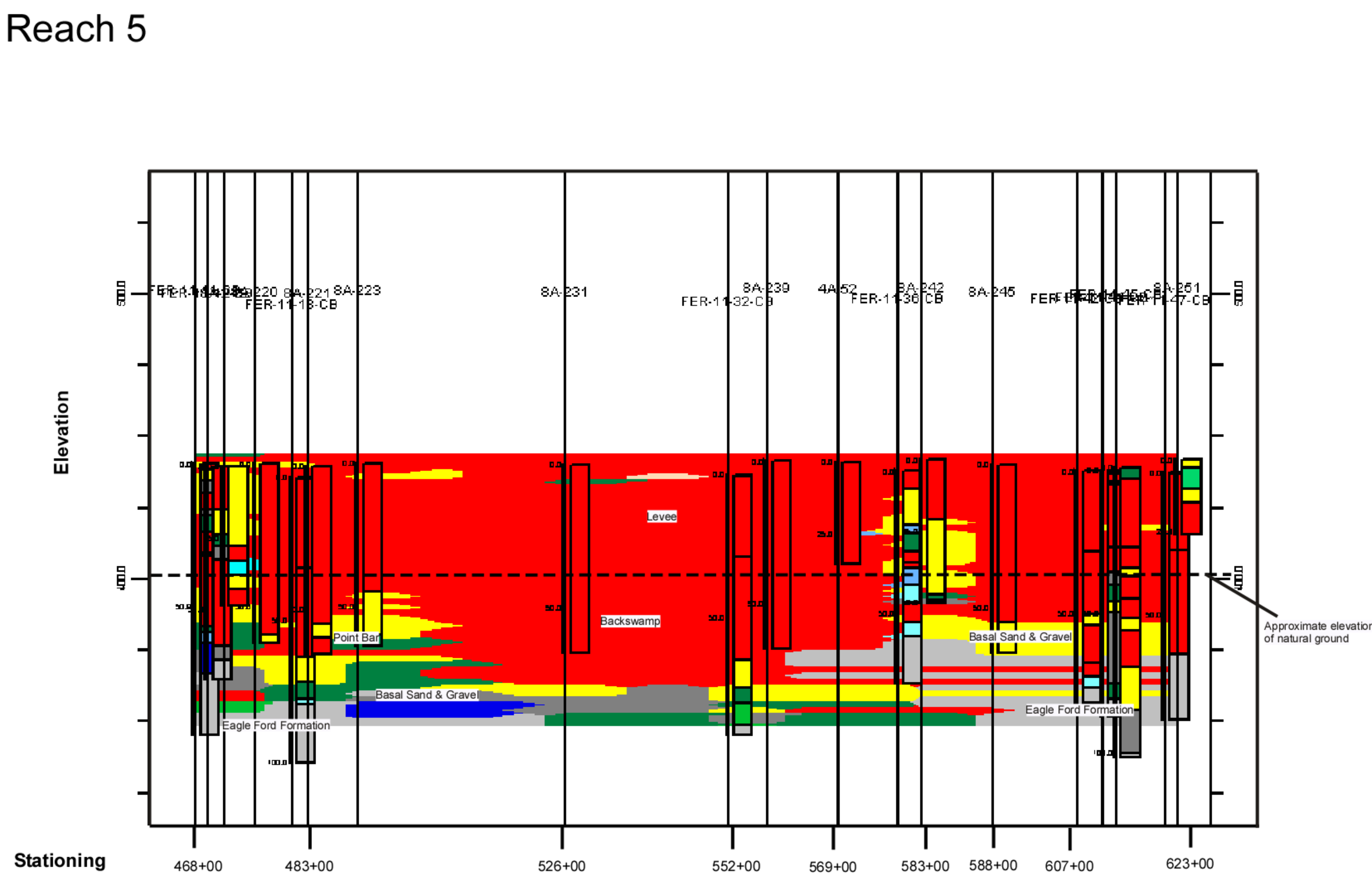


Figure 8-9. Reach 5 Geologic Cross-Section

This page intentionally left blank.

8.9 WEST LEVEE

The West Levee consists of Reaches 6 through 9. The levee drainage ditches run along the protected side of the levee in each of the four reaches. A brief description of each reach follows:

- Reach 6 contains levees composed of 50% high plasticity clays (CH) and 50% low plasticity clays (CL). This reach contains a large amount of historic landslide data. Most of the historic slides occurred on the riverside slope. Seepage is less of an issue along this reach, due to the decreased presence of the basal sands and gravels.
- Reach 7 contains levees which are mainly composed of CL and foundation seepage may be a potential issue. There are very few historic landslide events in this area.
- Reach 8 contains a large amount of historic landslide data likely due to the levee being mainly composed of CH with side slopes on average of about 3H:1V (for 20-60 ft. CL offset). There are also potential seepage issues along this reach, due to basal sands that are semi-continuous underneath the levee.
- Reach 9 levees are composed of both CL and CH. The levees along Reach 9 contain areas where seepage may be a concern due to the presence of basal sands and gravels. This reach has very little historic landslide data.

Typical features that describe each reach along the West Levee are summarized in Table 8-4.

Table 8-4. Features Defining Each Reach for West Levee

Reach Feature	*Reach*			
	6	*7*	*8*	*9*
Typical Landside Slopes:				
CL Offset:20-60 feet	3H:1V	3H:1V	3H:1V	3.25H:1V
CL Offset: 60-100 feet	3.5H:1V	3.75H:1V	3.5H:1V	4.5H:1V
Typical Riverside Slopes:				
CL Offset:20-60 feet	4H:1V	3.1H:1V	3H:1V	3.25H:1V
CL Offset: 60-100 feet	4H:1V	4H:1V	3.75:1V	3.75H:1V
Embankment Material	50% CH 50% CL	CL	CH	CH with pockets of CL
Depth to top of Shale @CL (feet)	74.9	41.1	72.0	62.1
Sand Depth from Surface (feet)	42	21	54	44
Average thickness of basal sand (feet)	6.7	4.5	6.9	6.2
Are Basal Sands Semi-Continuous/Continuous across levee?	Yes	No	Yes	No
Number of historical slides				
On Riverside slope	23	0	67	0
On Landside slope	2	1	3	1

This page intentionally left blank.

8.10 Subsurface Conditions for the West Levee

8.10.1 Reach 6

Reach 6 has a topographic low on the Eagle Ford shale with a semi-continuous sand and gravel deposit overlying the shale. Terrace and undifferentiated alluvium deposits overly the basal sand and gravel. The Austin Chalk Formation is observed in this area. Hence, the Formation is much thinner at this location, relative to Reach 1. Levees within this area are composed of lean clay material as well as high plasticity clays (Figure 8-10).

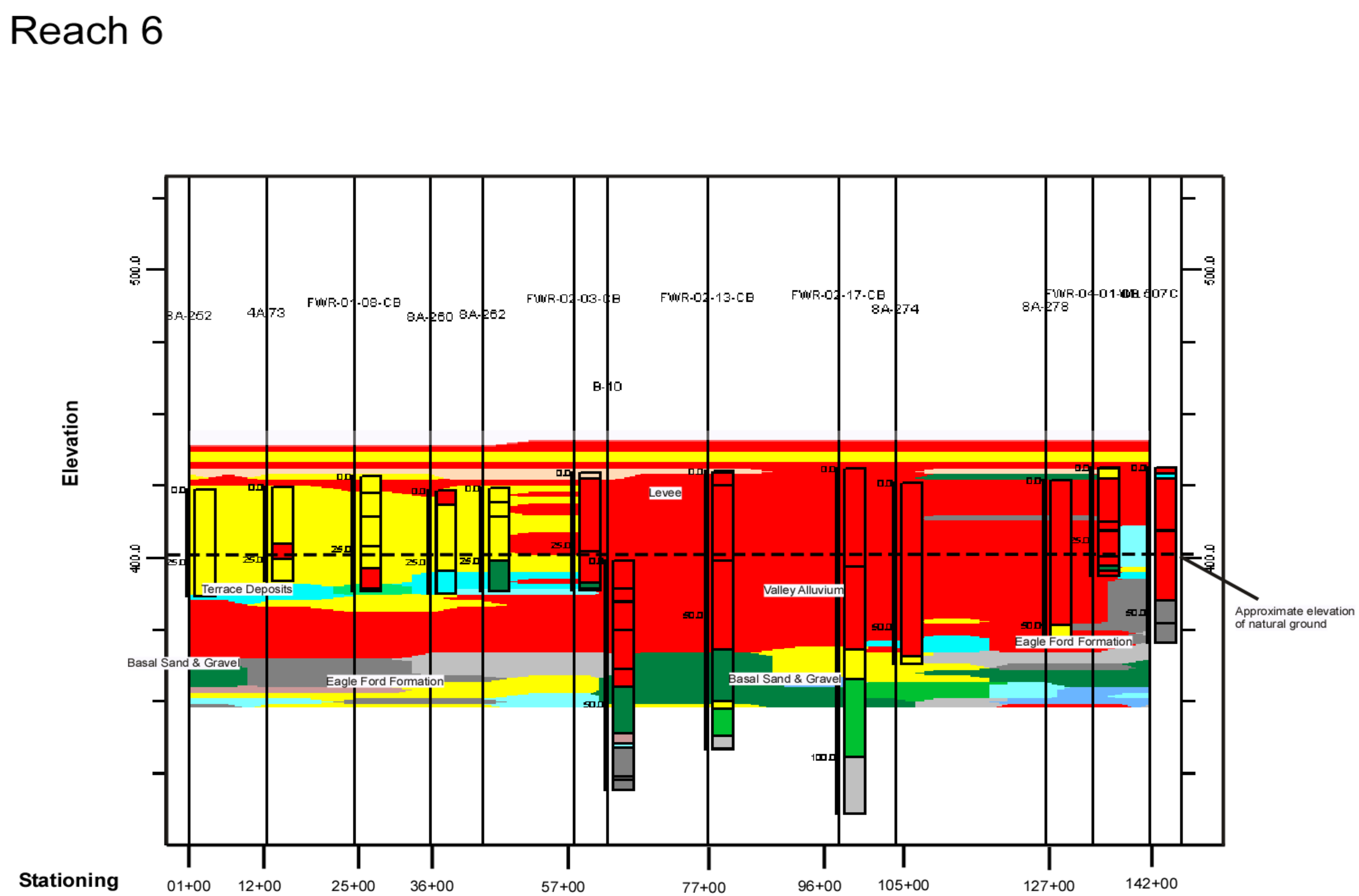


Figure 8-10. Reach 6 Geologic Cross-Section

8.10.2 Reach 7

Fairly thick and highly weathered Eagle Ford shale forms the bedrock for Reach 7 (Figure 8-11). Non-continuous basal sand and gravel deposits overlie the bedrock, followed by abandoned channel and point-bar deposits and backswamp. Levees within this location are mostly composed of lean clay.

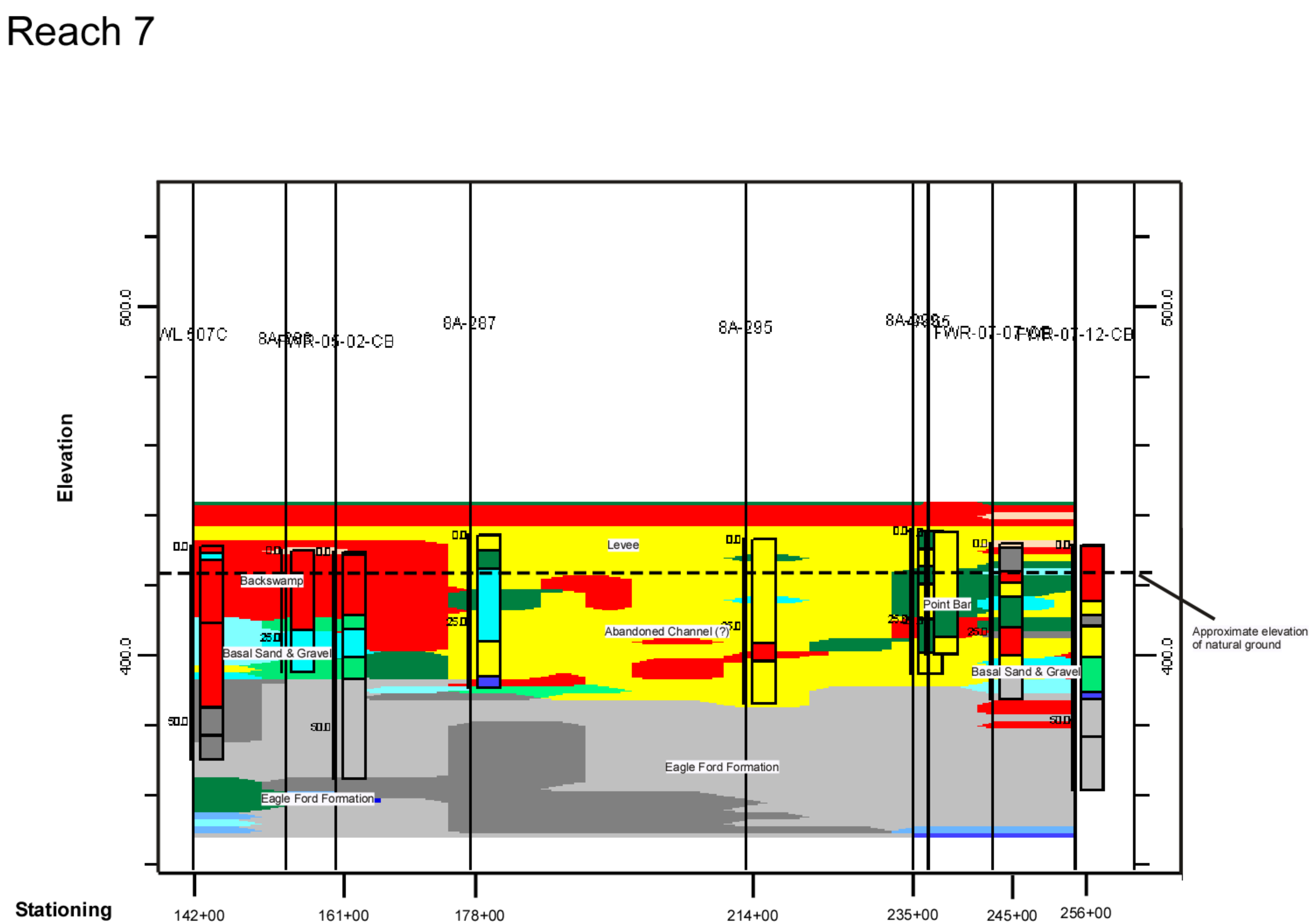


Figure 8-11. Reach 7 Geologic Cross-Section

8.10.3 Reach 8

A semi-continuous basal sand and gravel overlies irregular Eagle Ford shale at this location (Figure 1-12). Backswamp and undifferentiated alluvium deposits overlie the basal sand and gravel. Levees in the area are mostly composed of high-plasticity clay, with minor sand and lean clay lenses (Figure 8-12).

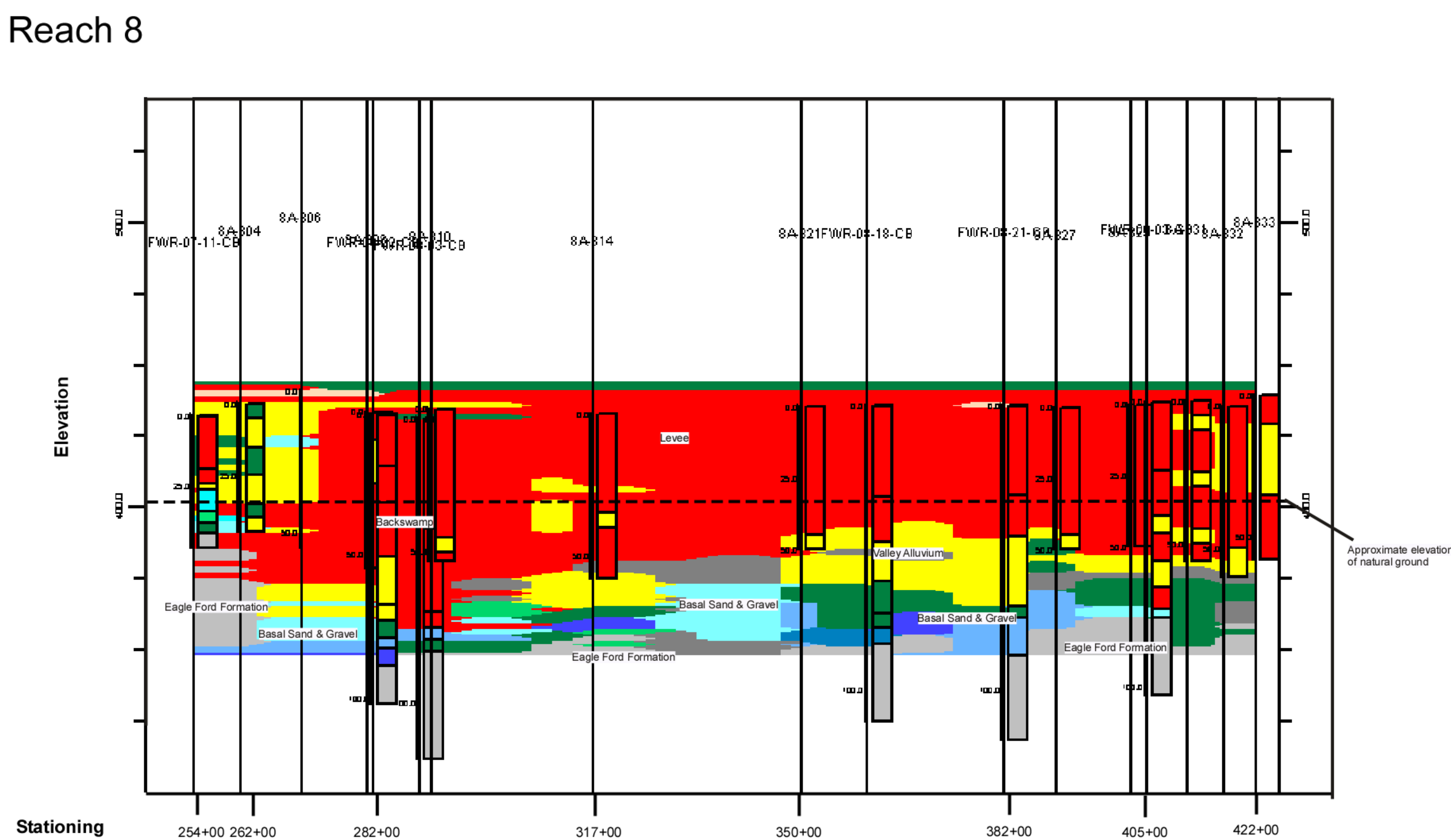


Figure 8-12. Reach 8 Geologic Cross-Section

8.10.4 Reach 9

A basal sand and gravel confined by incised and highly weathered Eagle Ford shale occurs at this location (Figure 8-13). Backswamp and alluvium deposits had been interpreted to occur on top of the basal sand. Levees at this location are highly heterogeneous, with high plasticity clays, lean clays, and sand lenses.

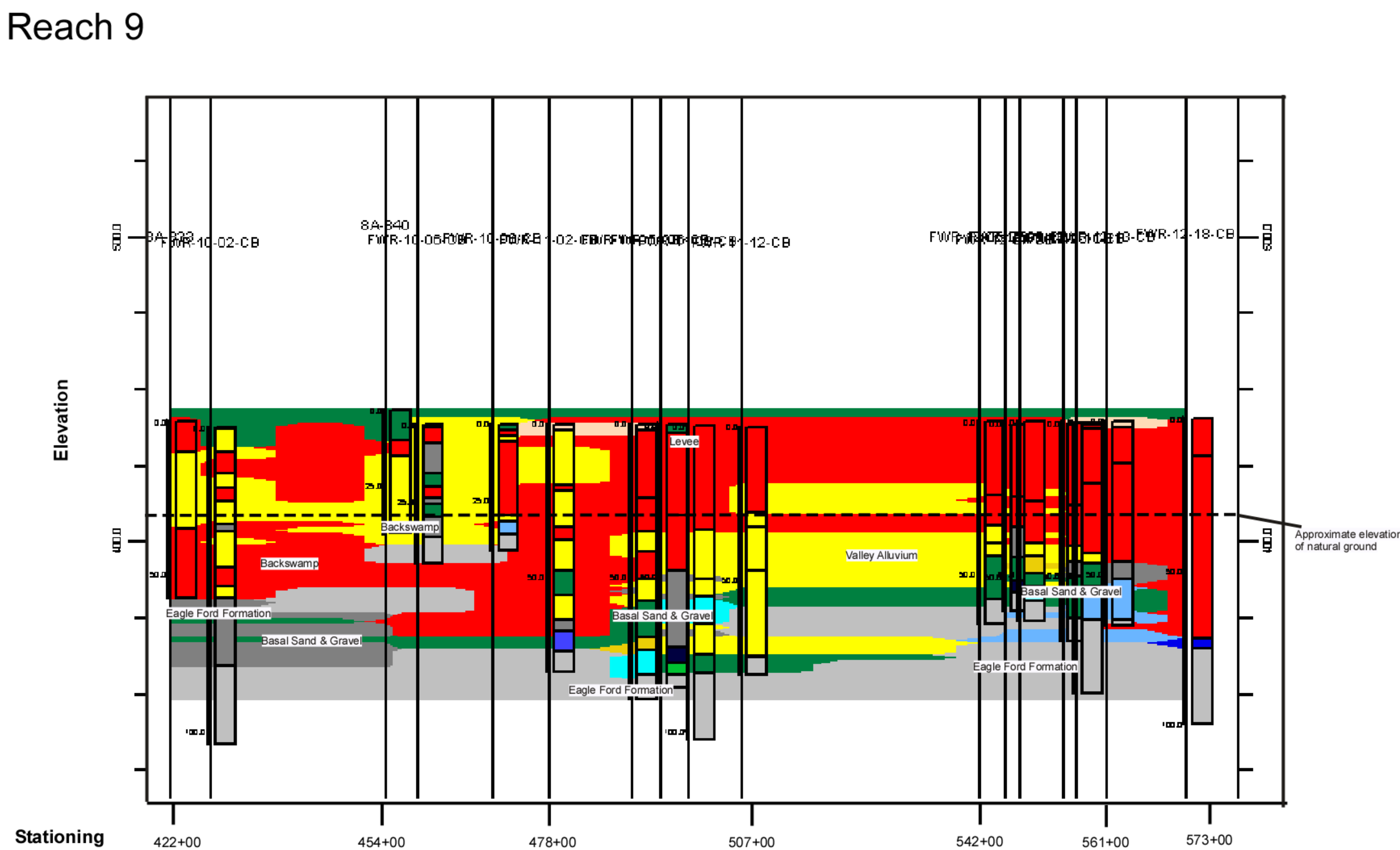


Figure 8-13. Reach 9 Geologic Cross-Section

9.0 ANALYSIS OF SUBSURFACE CONDITIONS

9.1 Near Surface Sands

Analysis of boring data shows significant quantities of sand within the subsurface of the project area including under the levee footprint. Table 9-1 displays the proportion of borings that contain sand ('*with sand regardless of classification*') as compared to the total number of borings collected ('*total*') within 180 ft. of the levee center line. Around the east levee, nearly 50% of the borings within Reach 1 contain significant sand facies (i.e., sands lenses of at least 1 ft thickness) and approximately 75% of borings within Reaches 2, 3, and 4 contain significant quantities of sand. The borings within the west levee system indicate a similar frequency of sand, ranging from 56% in reach 8 to 88% in Reach 7.

Table 9-1. Frequency of Sand Ground with Geologic Borings

Reach	*Reach Extent*	*Frequency*
	Station #, (feet [approx])	*with sand (total), (n)*
1 East Levee	1+028 – 10+113	52 (105)
2 East Levee	10+113 – 18+607	65 (91)
3 East Levee	18+607 – 29+017	58 (75)
4 East Levee	29+017 – 46+914	97 (133)
5 East Levee	46+914 – 62+446	66 (110)
6 West Levee	0+00 – 14+292	126 (164)
7 West Levee	14+292 – 25+820	74 (84)
8 West Levee	25 +820 – 42+000	51 (91)
9 West Levee	42+000 – 57+700	73 (96)

Table 9-2 illustrates the mean depths of the sand facies within geologic borings near (+/- 30 feet) the levee center lines and the mean thickness of the sand facies. The depth of the sand facies within Reaches 3, 4, 6, and 7 have relatively shallow mean depths and may routinely approach or enter the levee substrate, which typically composes the top 30 feet of the boring material in near-center line locations. The mean facie thickness for each reach does not vary significantly (typically displaying more variance within a reach than between reaches), ranging from 4.5 feet in Reach 5 to 7.6 feet in Reach 2. Borings may contain more than one sand facie.

Table 9-2. Sand Depth From Surface and Sand Facie Thickness

Reach	*Sand Depth from Surface (feet)*	*Facie Thickness (feet)*
	mean (standard deviation)	*mean (standard deviation)*
1	53 (27)	5.8 (3.9)
2	58 (24)	7.6 (5.3)
3	35 (21)	4.7 (3.2)
4	46 (20)	6.6 (3.9)
5	58 (30)	4.5 (2.4)
6	42 (25)	6.3 (4.9)
7	21 (13)	5.7 (3.6)
8	54 (26)	6.9 (4.8)
9	44 (23)	6.0 (3.5)

Figure 9-1 illustrates the distribution of sand along the longitudinal center line of the east and west levees. As depicted in Figure 9-1, the shifting location of the channel is illustrated by the gray lines. Dashed lines represent locations were the channel shifts position due to meandering and correspond with fluvially deposited sand lenses. The gray arrows represent a sudden shift in position due to channel avulsion or meander cutoff. Clay plugs may form in abandoned channels that fill with fine overbank sediments. These clay plugs are relatively impervious and may block or redirect seepage into more permeable material. Figures 9-2 and 9-3 display the same information as Figure 9-1, separated by reach and at a higher resolution. The majority of the sand identified within the borings is located within a relatively continuous basal layer resting on top of the bedrock surface; however, in a few notable locations, additional sand facies are identified elevated above the basal layer. Along the east levee, an area approximately spanning station values 30+000 to 31+500 in Reach 4 contains a number of borings with sand approaching the topographical surface.

Figure 9-5 shows the spatial coverage of each plate in relation to the full project area. Figures 9-6 to 9-16 illustrate the horizontal (areal) distribution of borings with sand. Figure 9-12 covers the critical area along the east levee with shallow sands. The distribution of sand found near the topographical surface extends from the landward (i.e., the protected) side of the levee toe through the levee substrate to the contemporary floodplain. If the sand observed in the borings is continuous, the relatively high hydraulic conductivity of the sand may promote groundwater flow (seepage) at this location.

Varying quantities of sand extend above the basal layer along the west levee between station values 17 +500 and 27 +000 in Reach 7 and 8. This area is covered in Figures 9-10, 9-11, and 9-12. The extent of the shallow sand is observed through the landward and river-side toes of the levee. There are few borings extending into the river floodplain making it difficult to estimate the riverward extension of this sand. The southwest extension of the west levee in Reach 9, approximately spanning station values 43 +000 to 47 +700 contains a significant quantity of borings with shallow subsurface sands. This area has no borings outside of the levee footprint; however, sand is located under both the landward and river-side toes making it likely that a continuous sand body underlies the levee at this location.

The locations of shallow sand in the project area represent areas of possible concern regarding levee performance. Typically, this sand has been deposited by fluvial processes in active and former river channels. Large, low-gradient rivers with meandering planforms (such as the Trinity River in its recent history, i.e., the Holocene epoch) regularly shift their course throughout their active floodplain (see Figure 9-4 for an illustration of this process). This shifting of position is caused by multiple processes including meander evolution (i.e., eroding the outer channel bank of a meander, while depositing transported sediment along the inner bank of a meander), meander cutoffs, and channel avulsion. These processes may create large (ranging in size from one channel width, approximately 200 feet, to greater than 1,000 feet wide), continuous sand deposits interwoven into the floodplain substrate. Because the presence of sand in the soil matrix increases it's permeability to groundwater flow, the near-surface sand deposits located nearby areas in contact with river water can quickly become saturated and serve as seepage pathways extending away from the river channel. This is of particular concern where a shallow sand lens laterally transects a levee, creating a seepage pathway from an area exposed to river water to the near surface substrate on the protected side of the levee.

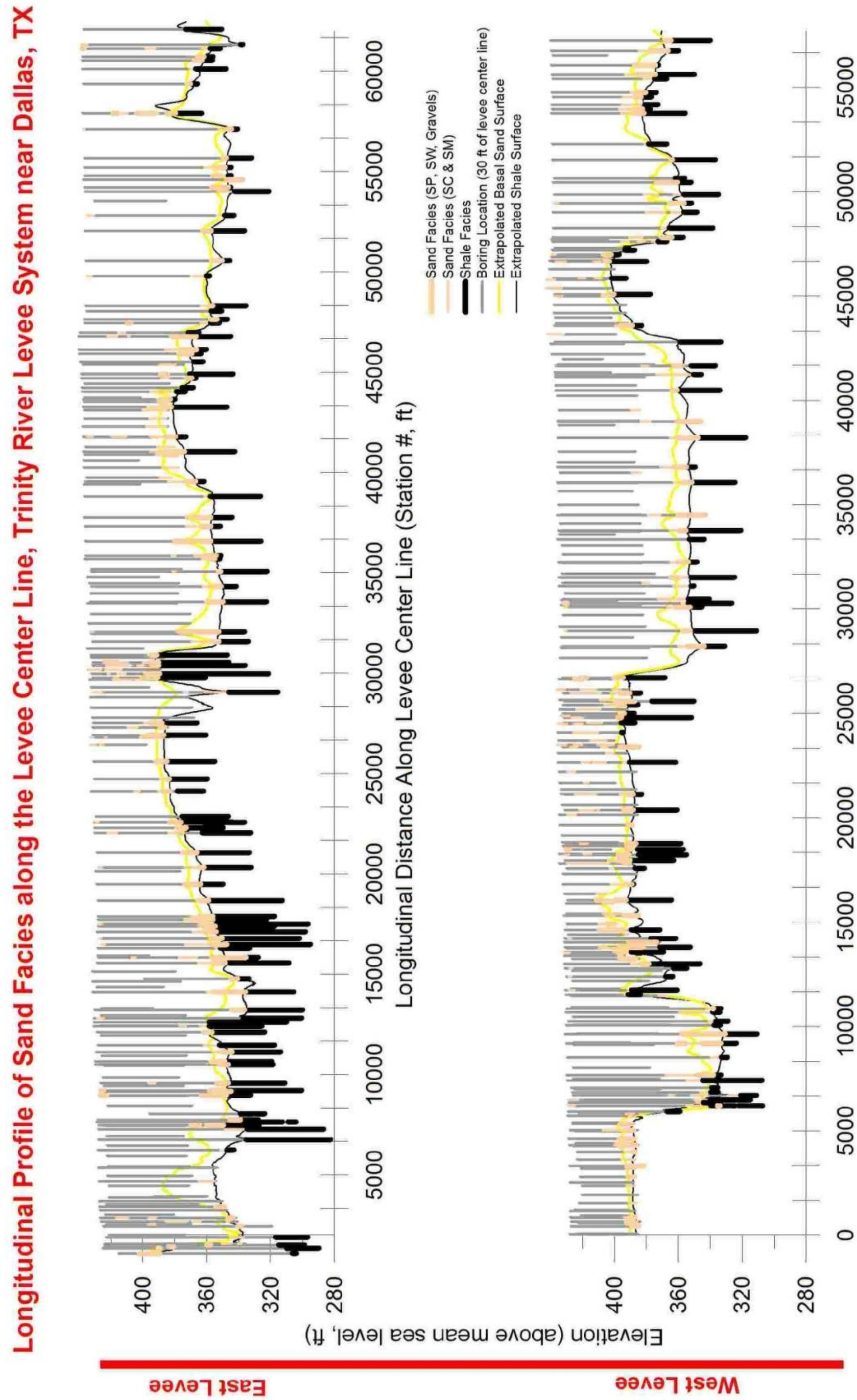


Figure 9-1. Distribution of Sand Facies Along the Levee Center Lines

The density of borings in the project area is typically too sparse to reliably interpret or extrapolate the extent of the identified sand deposits throughout the floodplain. However, clusters of borings in close proximity to each other with varying amounts of sand indicated in the boring extending through a levee section, indicate areas with an increased seepage risk. Table 9-3 identifies three such clusters where borings indicate the presence of shallow sand in both levee toes (i.e., the river-side toe and the protected side toe). Figures 9-6 through 9-16 illustrate the location and depth of the shallowest sand identified by borings in the project area. Any indication of the presence of shallow sand resents some degree of increased seepage risk. Sand in the contemporary floodplain (i.e., on the river side of the levee system) could serve as an entrance for seepage and sand along the protected-side of the levee toe serving as a possible seepage exit. Sand located at depths less than 4 feet near the protected-side levee toe are high risk areas for seepage exit (and are prone to sand boils) if they were to come into contact with existing seepage pathways.

Table 9-3. Areas with Significant Shallow Sand

Levee	*Location (Station #, feet)*	*Identified near-surface* Sand Facies (from Borings)*	*Approximately minimum depth at Levee Toe (feet)*	
		(n)	*Landward*	*River-side*
East	30 000 to 31 500	20	9	7
West	17 500 and 27 000	70	4	7
West	43 000 to 47 700	22	9	7

*Less than 10 feet below original (pre-levee) topographical surface.

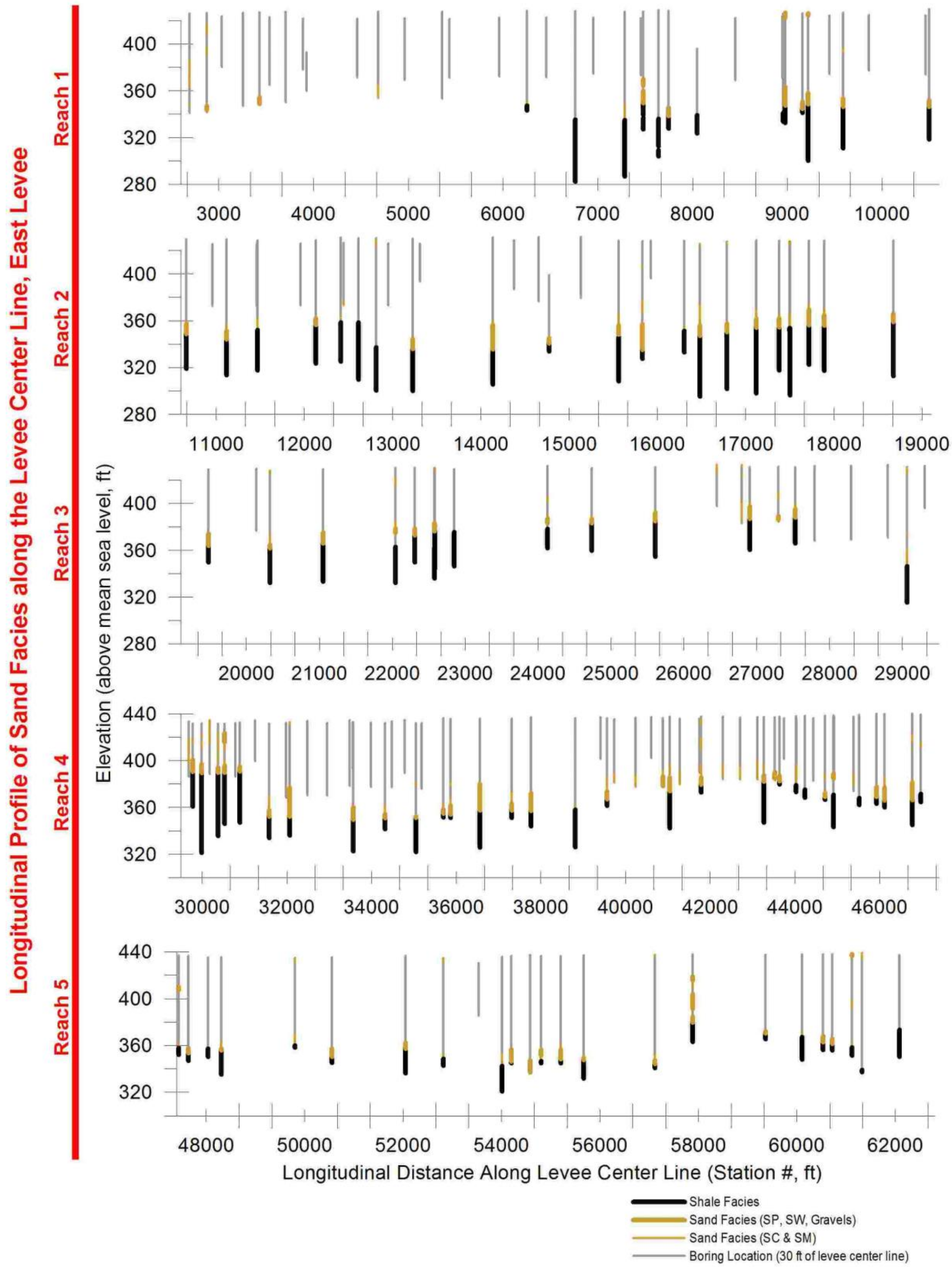


Figure 9-2. Distribution of Sand Facies Along the East Levee Center Line by Reach

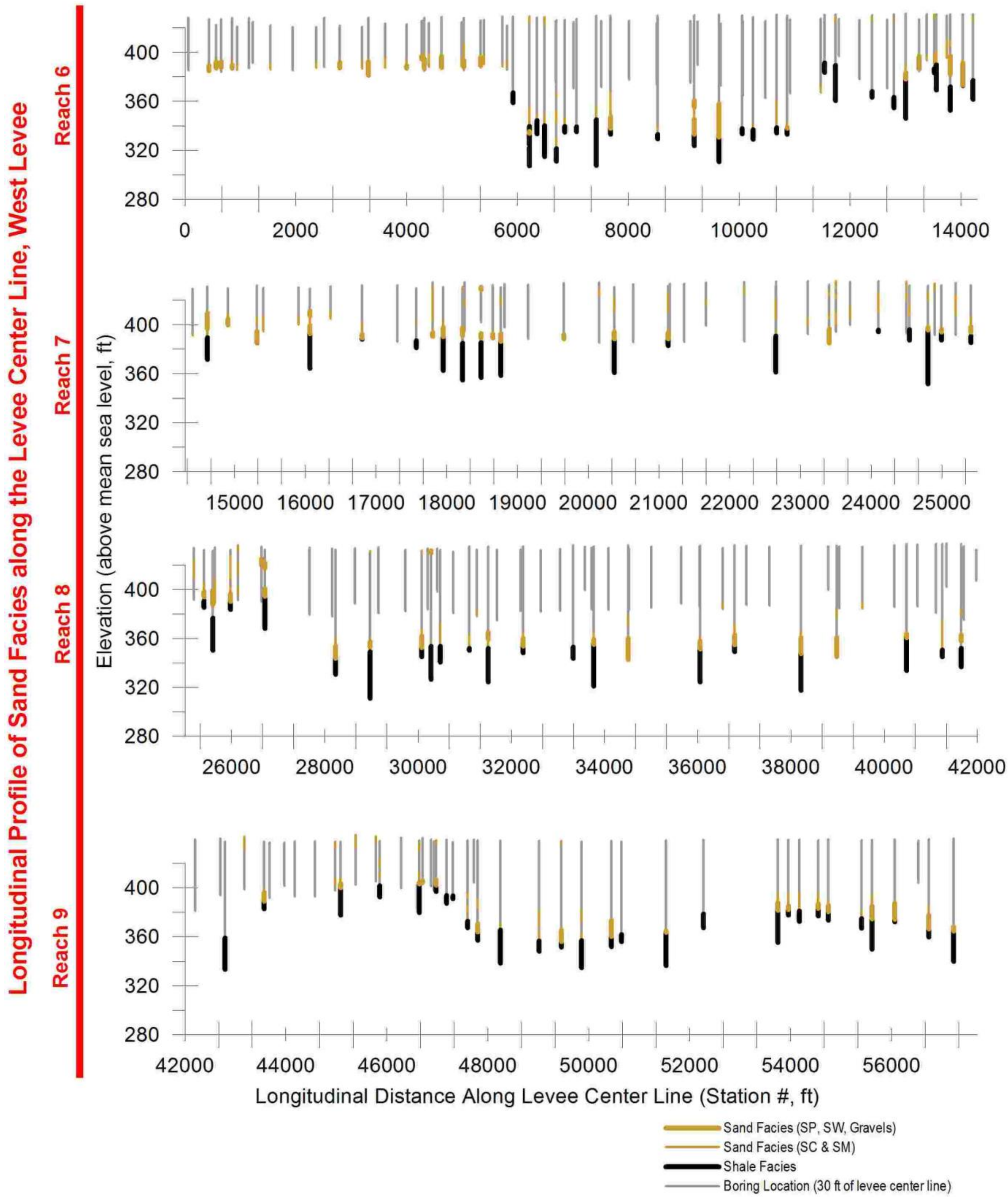


Figure 9-3. Distribution of Sand Facies Along the West Levee Center Line by Reach

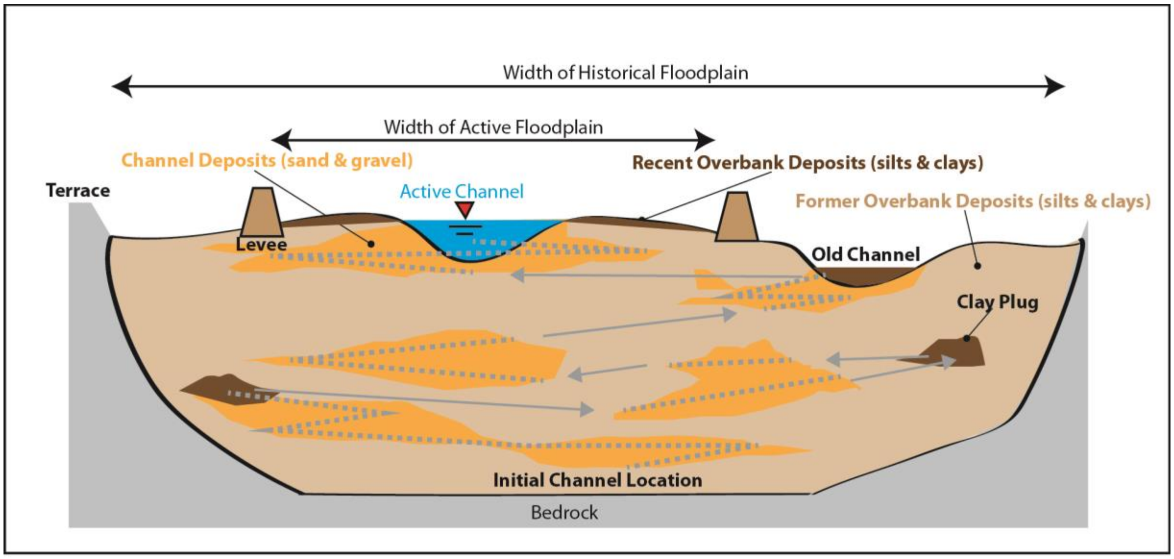


Figure 9-4. Diagram of an Idealized Cross-Section of an Aggrading Alluvial Basin

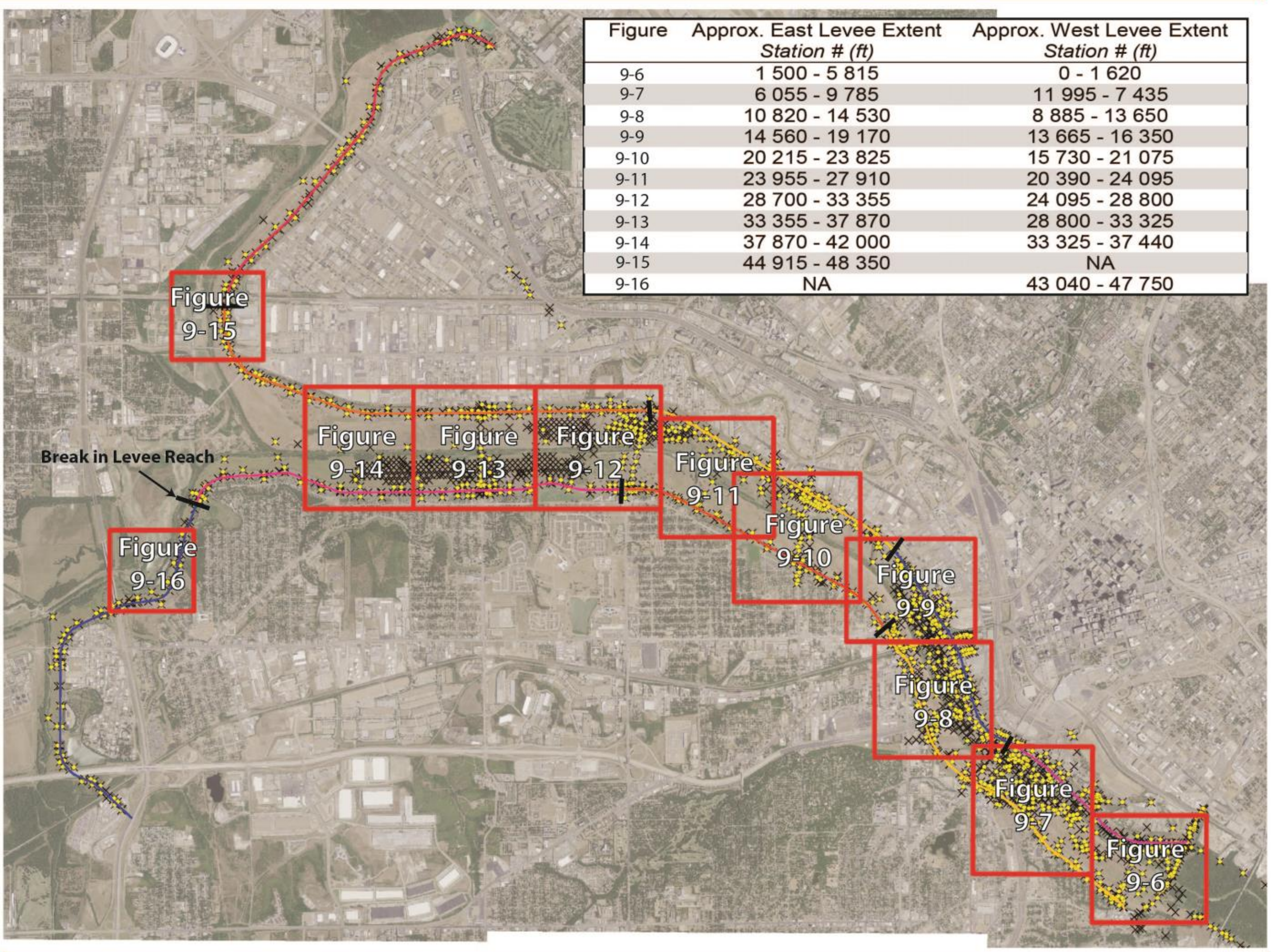


Figure	Approx. East Levee Extent *Station # (ft)*	Approx. West Levee Extent *Station # (ft)*
9-6	1 500 - 5 815	0 - 1 620
9-7	6 055 - 9 785	11 995 - 7 435
9-8	10 820 - 14 530	8 885 - 13 650
9-9	14 560 - 19 170	13 665 - 16 350
9-10	20 215 - 23 825	15 730 - 21 075
9-11	23 955 - 27 910	20 390 - 24 095
9-12	28 700 - 33 355	24 095 - 28 800
9-13	33 355 - 37 870	28 800 - 33 325
9-14	37 870 - 42 000	33 325 - 37 440
9-15	44 915 - 48 350	NA
9-16	NA	43 040 - 47 750

Figure 9-5. Relative Location of Figures 9-5 through 9-15 within the Project Area

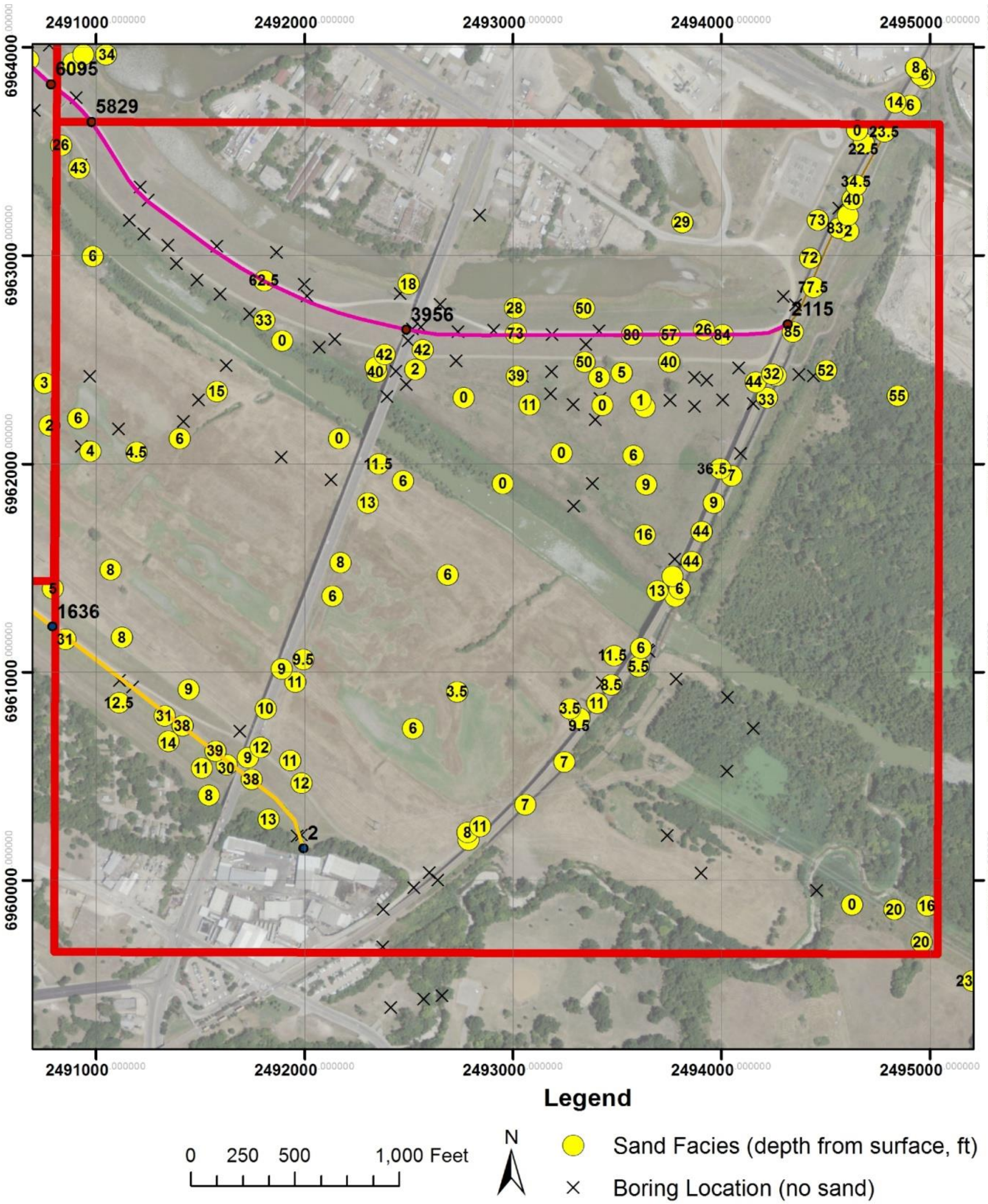


Figure 9-6. Location and Depth of the Shallowest Sand

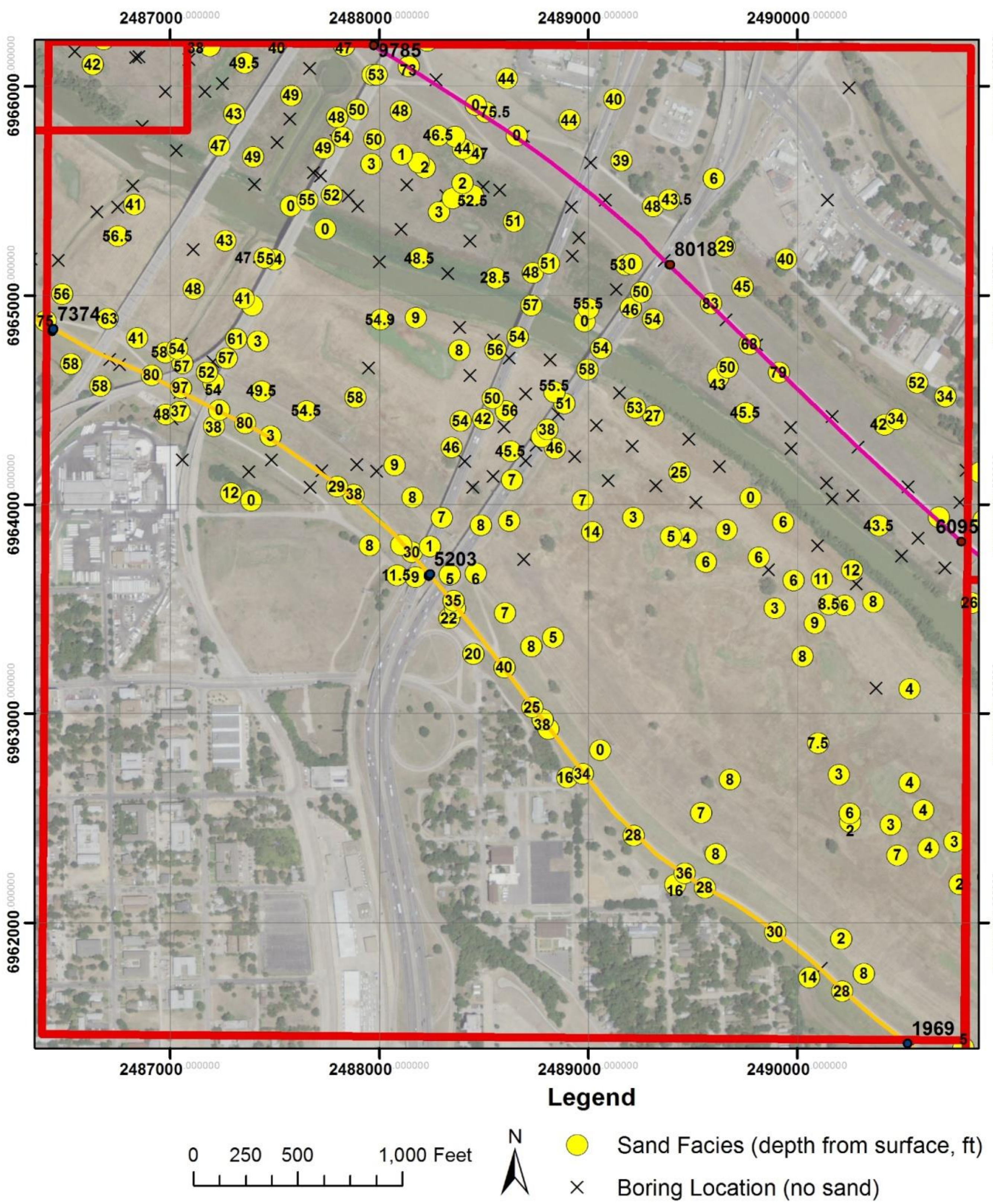


Figure 9-7. Location and Depth of the Shallowest Sand

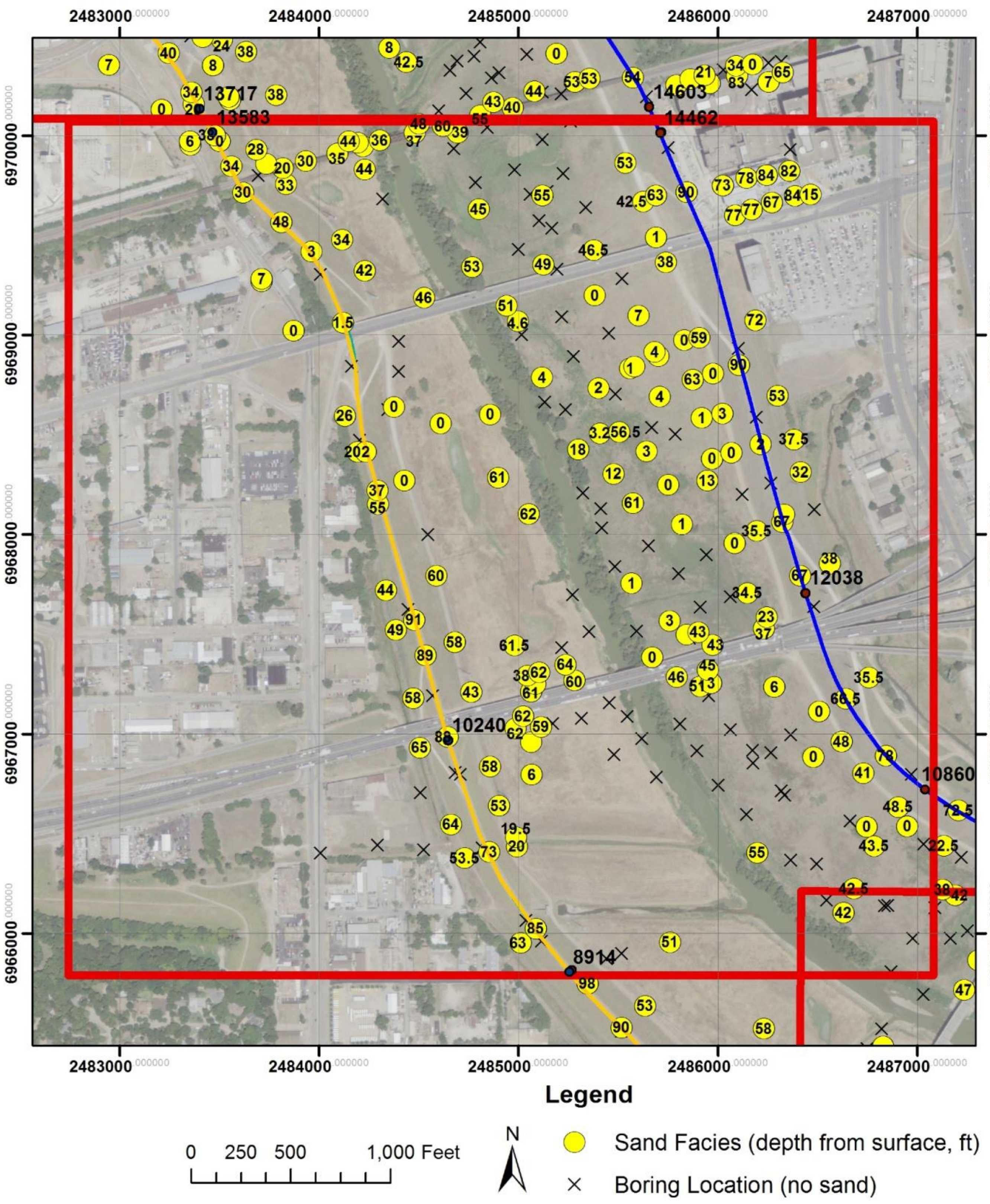


Figure 9-8. Location and Depth of the Shallowest Sand

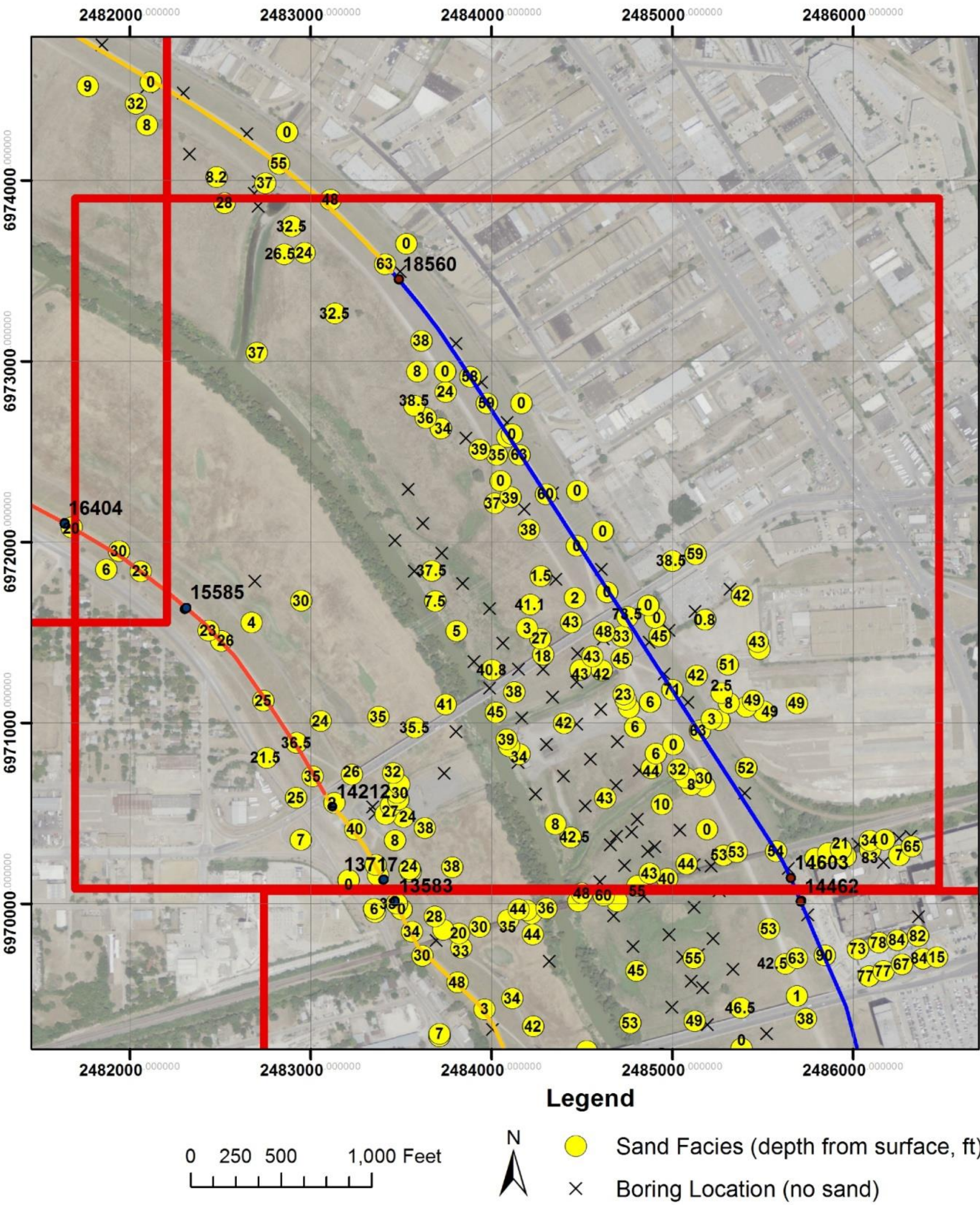


Figure 9-9. Location and Depth of the Shallowest Sand

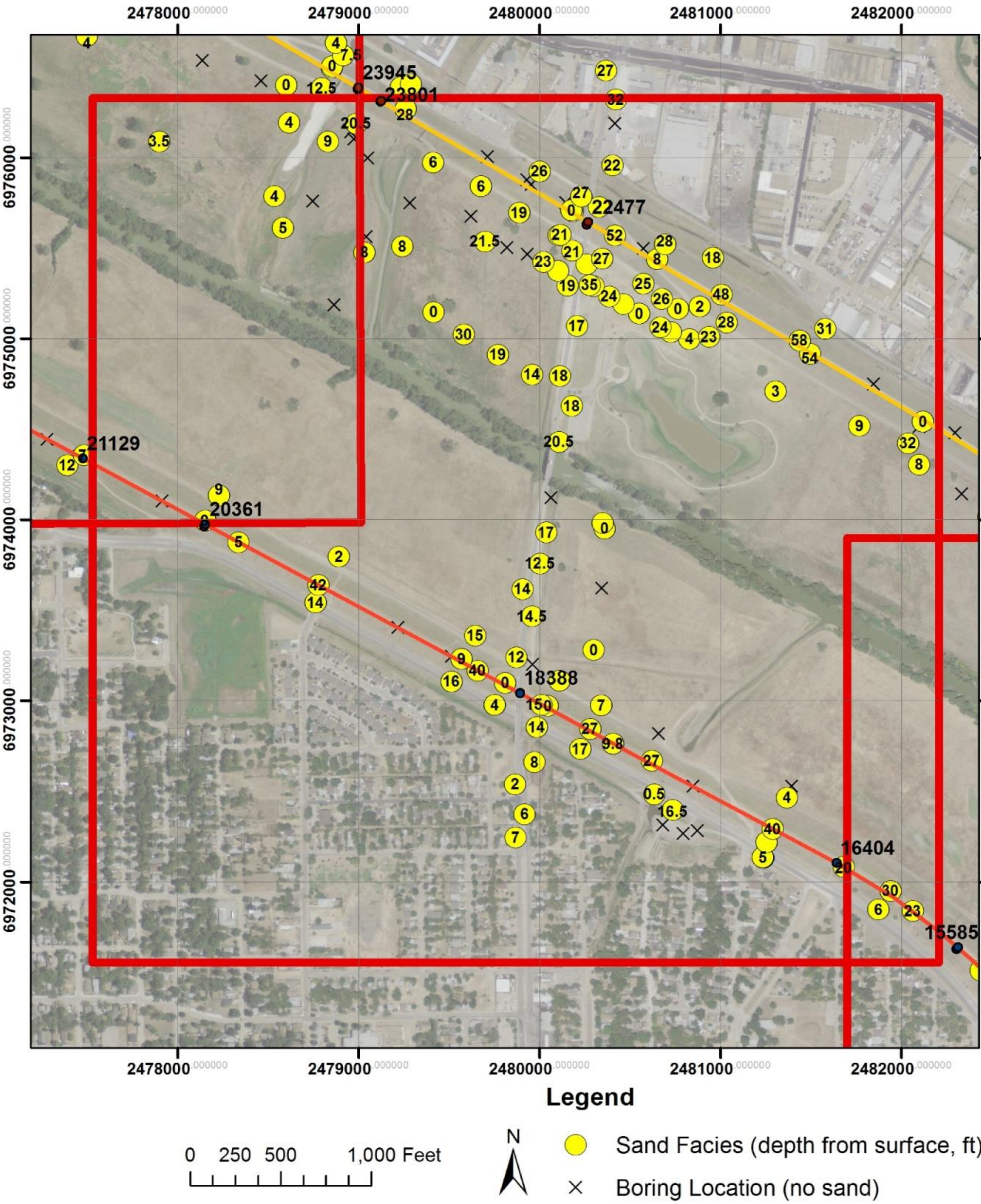


Figure 9-10. Location and Depth of the Shallowest Sand

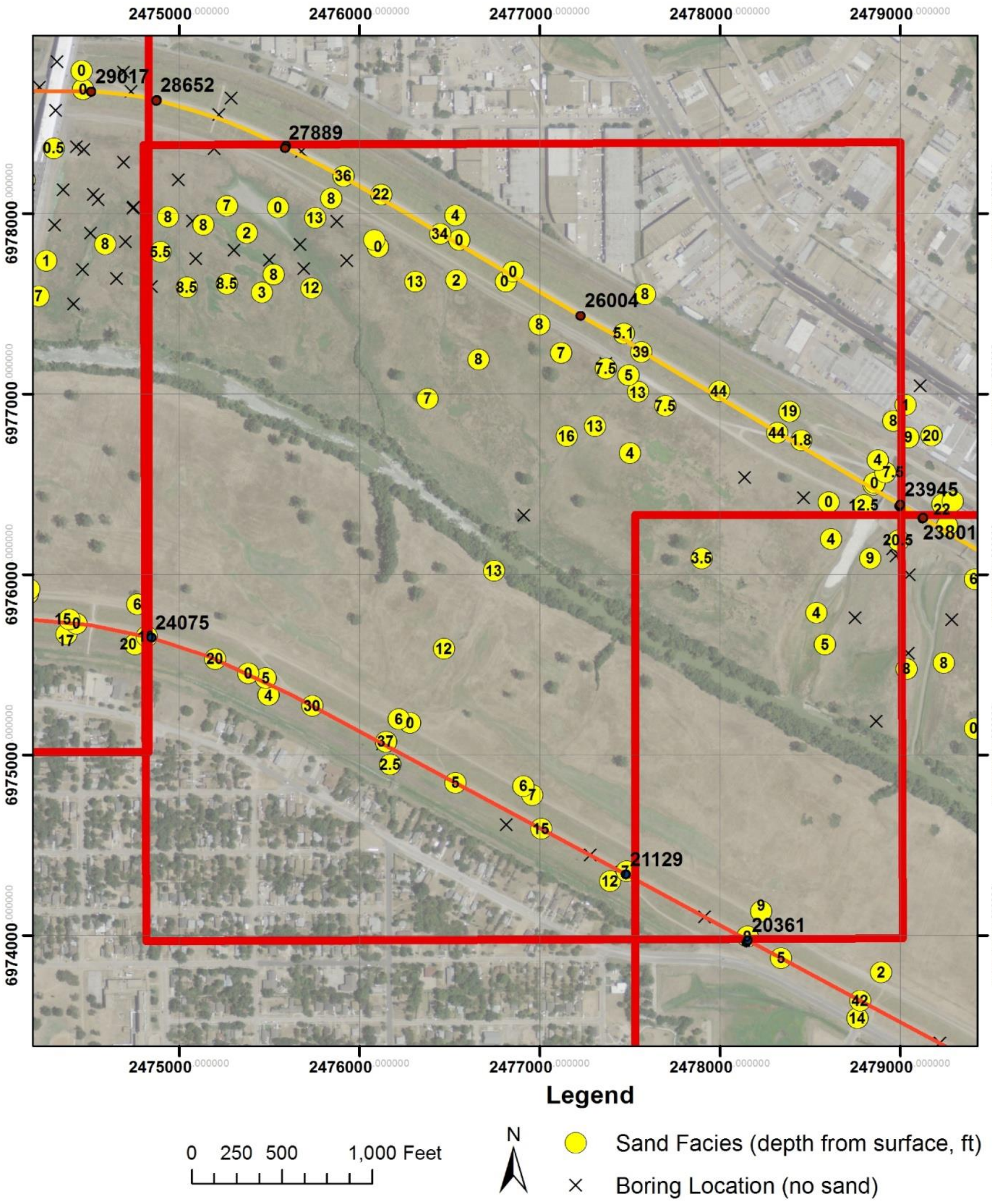


Figure 9-11. Location and Depth of the Shallowest Sand

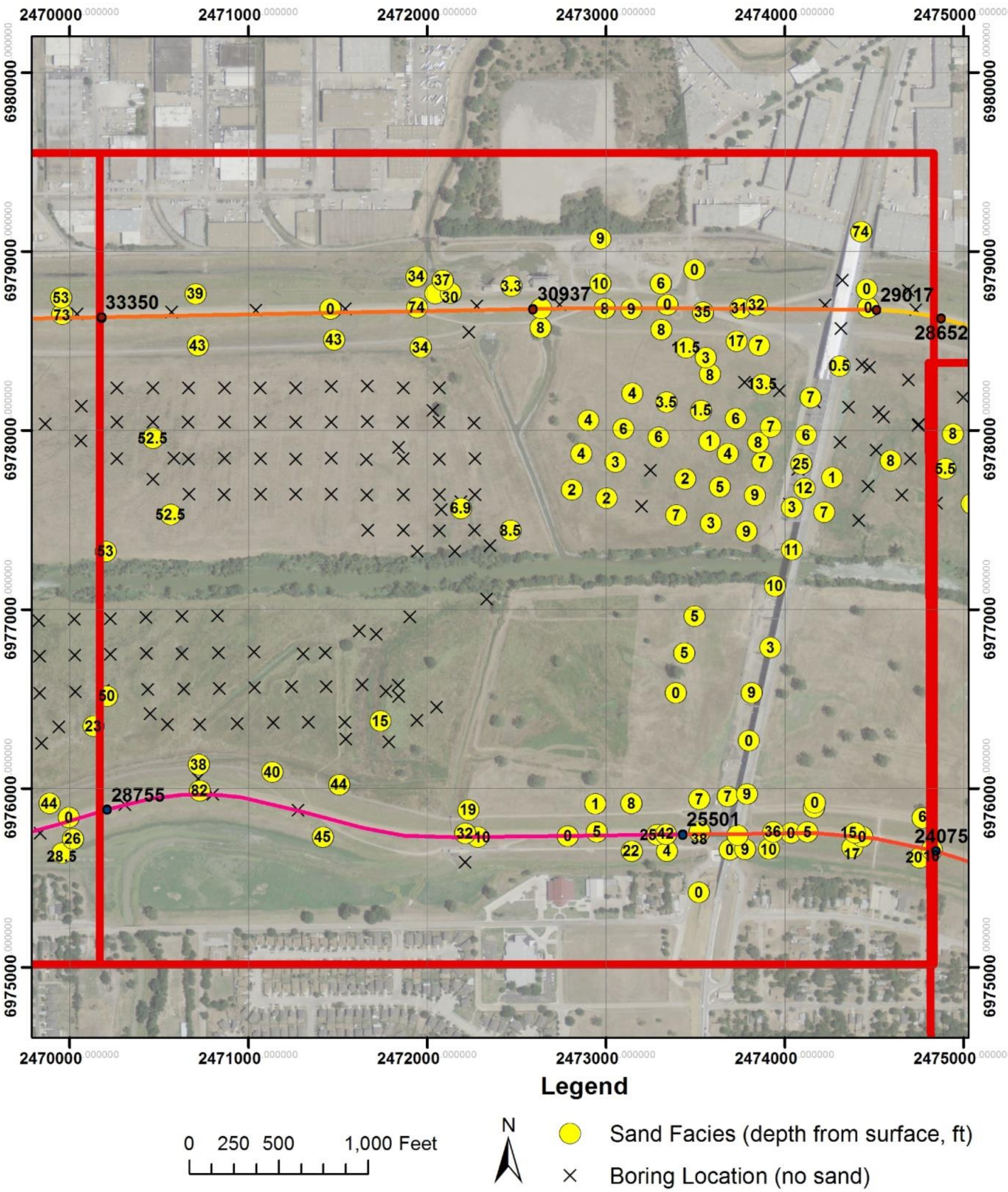


Figure 9-12. Location and Depth of the Shallowest Sand

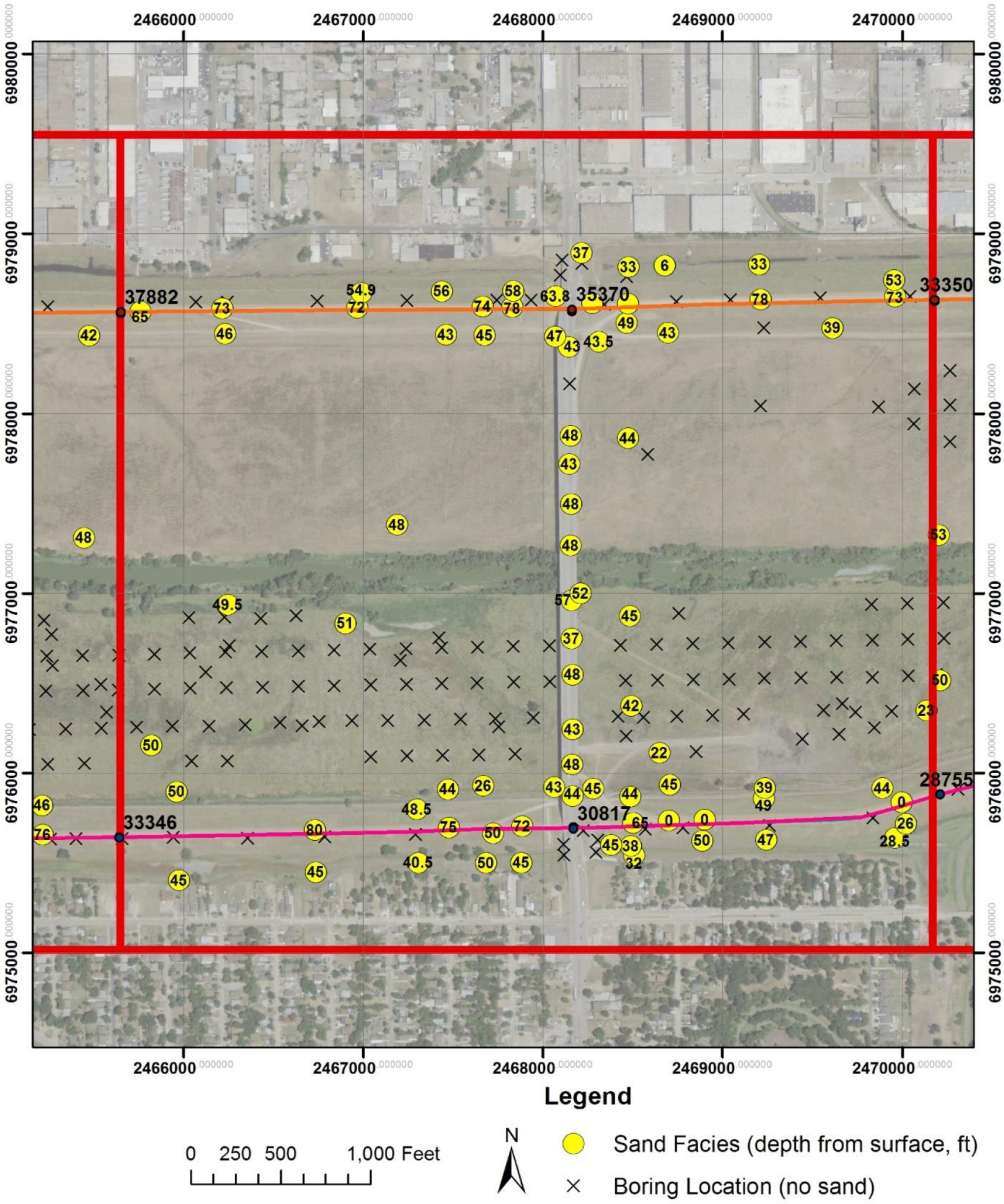


Figure 9-13. Location and Depth of the Shallowest Sand

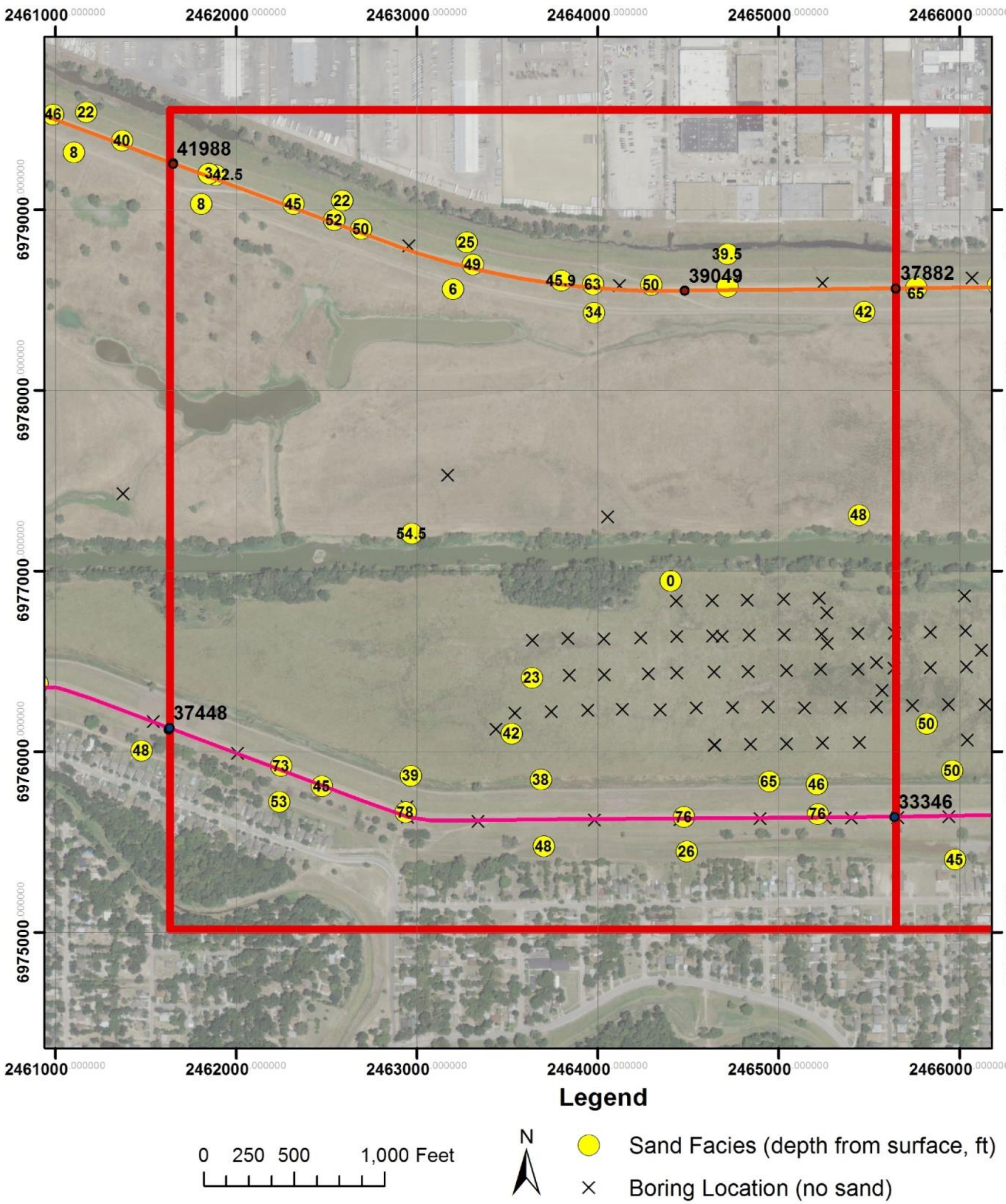


Figure 9-14. Location and Depth of the Shallowest Sand

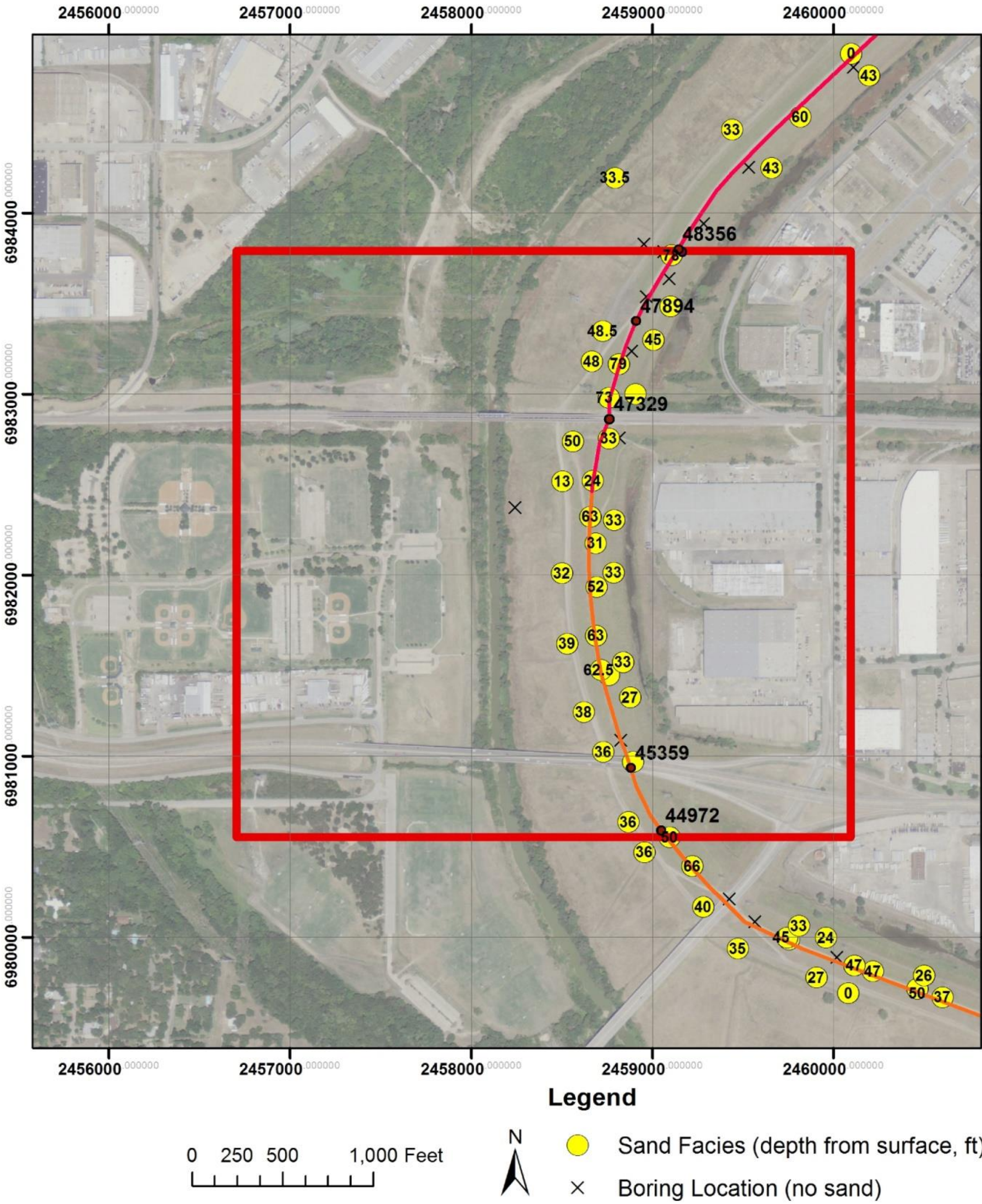


Figure 9-15. Location and Depth of the Shallowest Sand

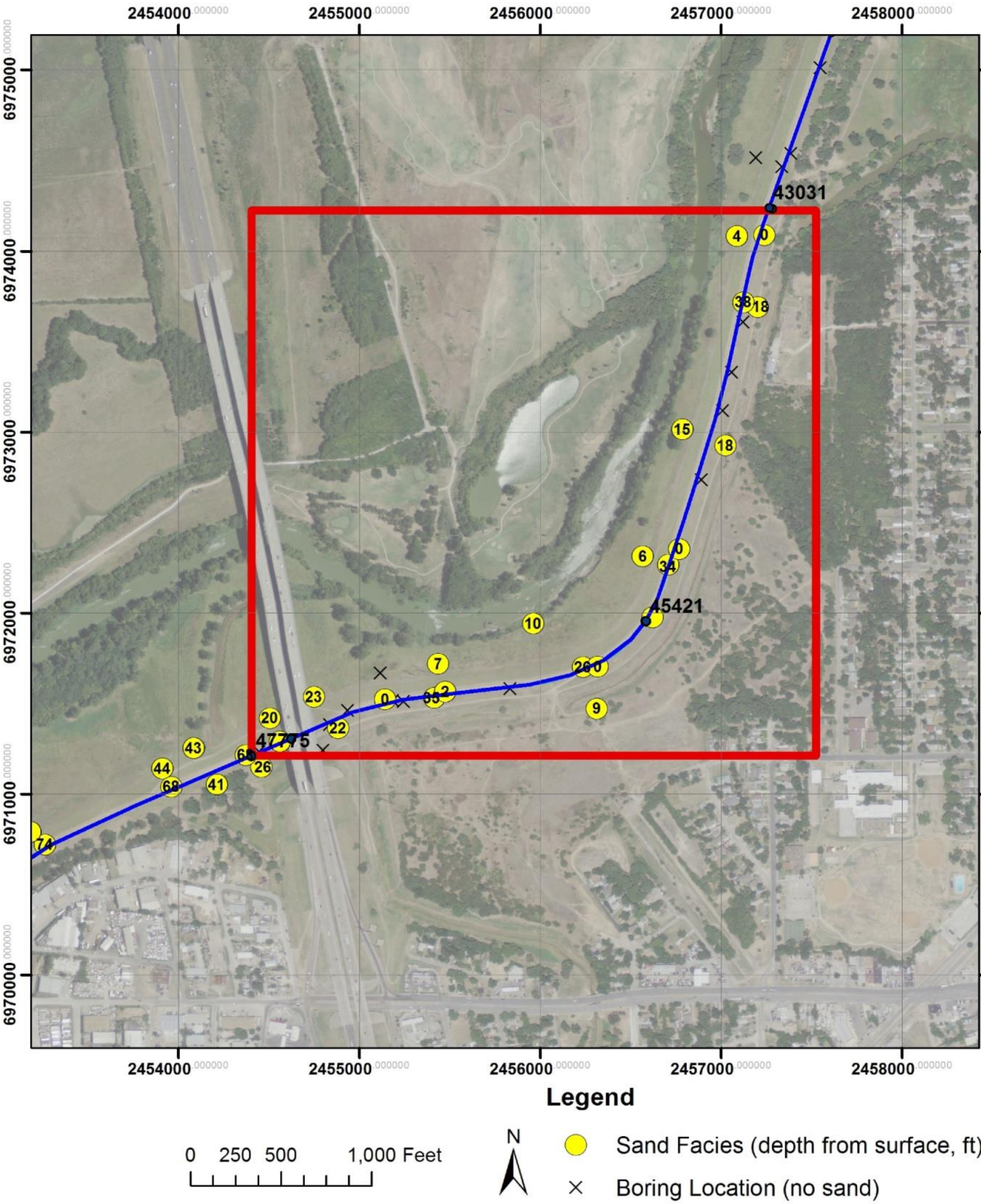


Figure 9-16. Location and Depth of the Shallowest Sand

9.2 Bedrock and Basal Sands and Gravel Units

Logs from 1,543 borings were used to interpolate two-dimensional (2-D) and three-dimensional (3-D) surfaces representing the uppermost portion of two significant geologic features in the Trinity River alluvial plain: (1) semi-continuous basal sands and gravels overlying bedrock; and (2) shale and limestone bedrock units. All borings that reached bedrock and had accurate x-y locations and lithologies in the gINT database provided to the ERDC team by HNTB were used in surface interpolations. Of the 1,543 borings used, 763 showed evidence for the existence of a basal sand and gravel unit. Boring logs were sampled for elevations of the top surface of basal sand/gravel and bedrock units. All geologic units were identified using the "graphic" of the unit contained in the gINT database (Table 9-4). The boring data was sorted and classified, and the shallowest shale, weathered shale, limestone, or sandstone units in each boring was used to interpolate the bedrock surface. The shallowest elevations of coarse-grained, porous soils immediately overlying the bedrock were used to interpolate the basal sand and gravel surface. The basal sand/gravel and bedrock surfaces were developed to assess the engineering impacts of the bedrock surface and overlying coarse grained materials on the levee system. Analysis was targeted on areas where slope stability and seepage are of concern.

Table 9-4. List of "Graphic" Identifiers Used to Extract Discreet Bedrock and Basal Unit Elevations from the gINT Database Provided to the ERDC Team by HNTB

Unit	*Graphic*
Bedrock	ACE_LIMESTONE ACE_SANDSTONE ACE_SHALE ACE_ WEA SHALE SH SHC L
Basal Sands and Gravels	GP GP-GC GS GW GW-GC GWS SC-SM SM SP SP-GP SP-SC SP-SM SW SW-SC SW-SM

A 3-D surface analysis was conducted using three continuous raster digital elevation models (DEMs) with a pixel resolution of 5 feet: (1) ground surface; (2) top of basal sand and gravel; and (3) top of bedrock. Elevation profiles for use in subsurface and geotechnical analysis were extracted from each surface along multiple cross-sections, levee centerlines, and levee reaches (see Figures 9-22 and 9-23). Figure 9-17 illustrates the 3-D surfaces in oblique view looking upstream from the southeastern edge of the study area at a vertical exaggeration of 100x. As shown on Figure 9-18, the semi-transparent grayscale surface is the ground surface DEM. Red to blue shaded relief surface is the top of basal sands and gravels. Red and orange colors are high elevation, blues and greens are low. The top of rock DEM is out of view beneath the basal sand and gravel. Red lines indicate levee centerlines. Note the elevation variation of the basal sands and gravel.

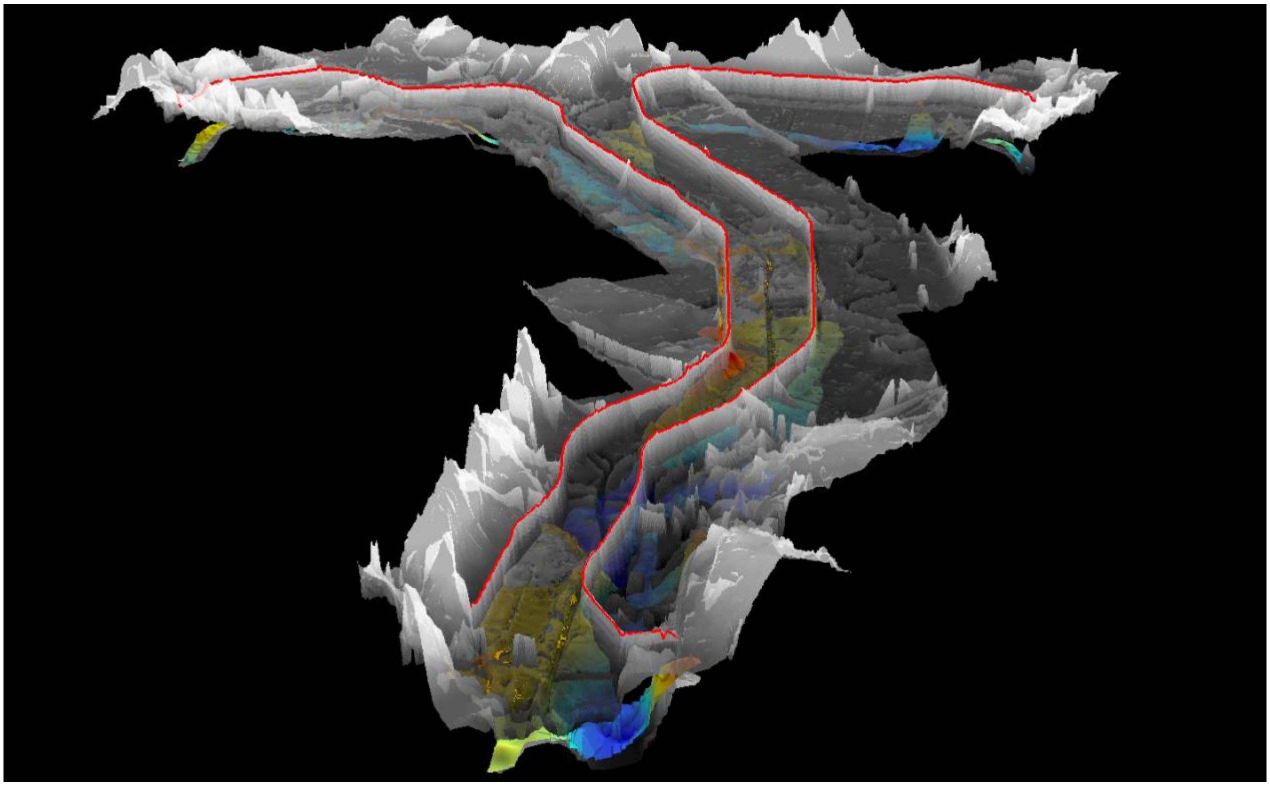


Figure 9-17. Oblique 3-D View of the DEM Surfaces Looking Upstream From the Southeastern Margin of the Study Area

Much of the Dallas – Fort Worth metropolitan area is situated in an alluvial plain near the confluence of the Elm and West forks of the Trinity River. The subsurface in this region is comprised of three major geologic units. The first and shallowest unit, the alluvial blanket (i.e., topstratum), consists of interfingered fluvial clays, silts, sands, and gravels that have variable thicknesses and extents. Clays and silts were deposited in back swamp areas of the Trinity River floodplain before the river course was altered and constrained within the Dallas Floodway. The majority of soils in the study area are comprised of fine-grained (i.e., clays and silts). Sands and gravels are less pervasive, and were deposited in historic river channels and/or alluvial outwashes juxtaposed to terraced uplands.

The second major geologic unit in the study area is a semi-continuous basal sand and gravel (Figure 9-18a). This unit is generally porous, loosely cemented (if at all), and variably graded. It lies immediately beneath a fine-grained top-stratum or a course grained transitional material such as a SC and was deposited in a high-energy fluvial system. Because this unit is both porous and semi-continuous, it may serve as a seepage pathway under high hydraulic head. Thus, its morphology and spatial distribution relative to the Dallas Floodway levee system is of engineering significance.

The third unit is comprised of shale (Eagle Ford formation) and limestone (Austin chalk formation) bedrock that underlies the basal sand and gravel (Figure 9-18b). As shown on Figure 9-18a, the red lines are top of levee centerlines. Black/blue areas are low elevation; white/red areas are high. Vertical exaggeration is 100x. Note the distinct high and low zones corresponding to paleo-channel incision within the floodplain. Zones with high bedrock and basal sand and gravel elevations have high seepage potential. The shales are often highly weathered and can resemble and exhibit engineering properties similar to clays. Limestone units are soft to hard with shaley concretions. All bedrock units dip slightly to the east. The stratigraphic zones of the Eagle Ford formation have varying compositions, and likewise have weathered at different rates. These stratigraphic zones often exhibit dissimilar engineering properties because of their variable composition and levels of weathering. Because these stratigraphic zones outcrop at different locations throughout the floodplain, the morphology and spatial distribution of bedrock relative to the Dallas Floodway levee system is of engineering significance.

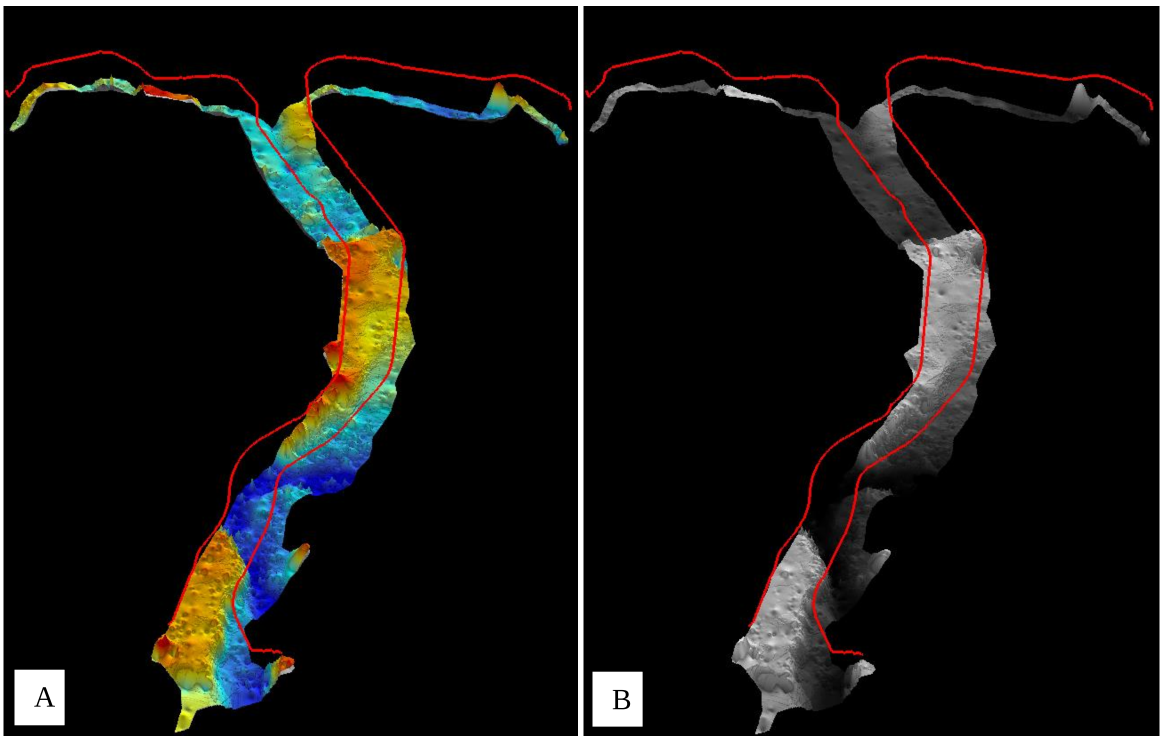


Figure 9-18. Oblique 3-D views of the (a) basal sand and gravel surfaces, and (b) bedrock looking upstream from the southeast.

The DEMs shown in Figure 9-18 are interpreted raster grids of the top surfaces of bedrock (Figure 9-18b) and basal sand/gravel (Figure 9-18a) underlying the Dallas Floodway. On Figure 9-19, the light colors are high elevation and dark colors are low. Note the deeply incised paleo-channels and high zones. Discreet elevations of the top surface of each unit were extracted from boring data in the gINT database. Elevations between the discreet data points were interpolated in ArcGIS® using tools in the 3-D Analyst extension. An Inverse Distance Weighted (IDW) function with a power of 2 and a search radius of 10 points was used to produce continuous-value raster images. Each pixel in the rasters has a value that is the average elevation for the corresponding 5 foot x 5 foot square real-earth area. The interpolation was clipped to exclude data points that fell outside of the study area. This was done to improve accuracy and legitimacy of interpreted features.

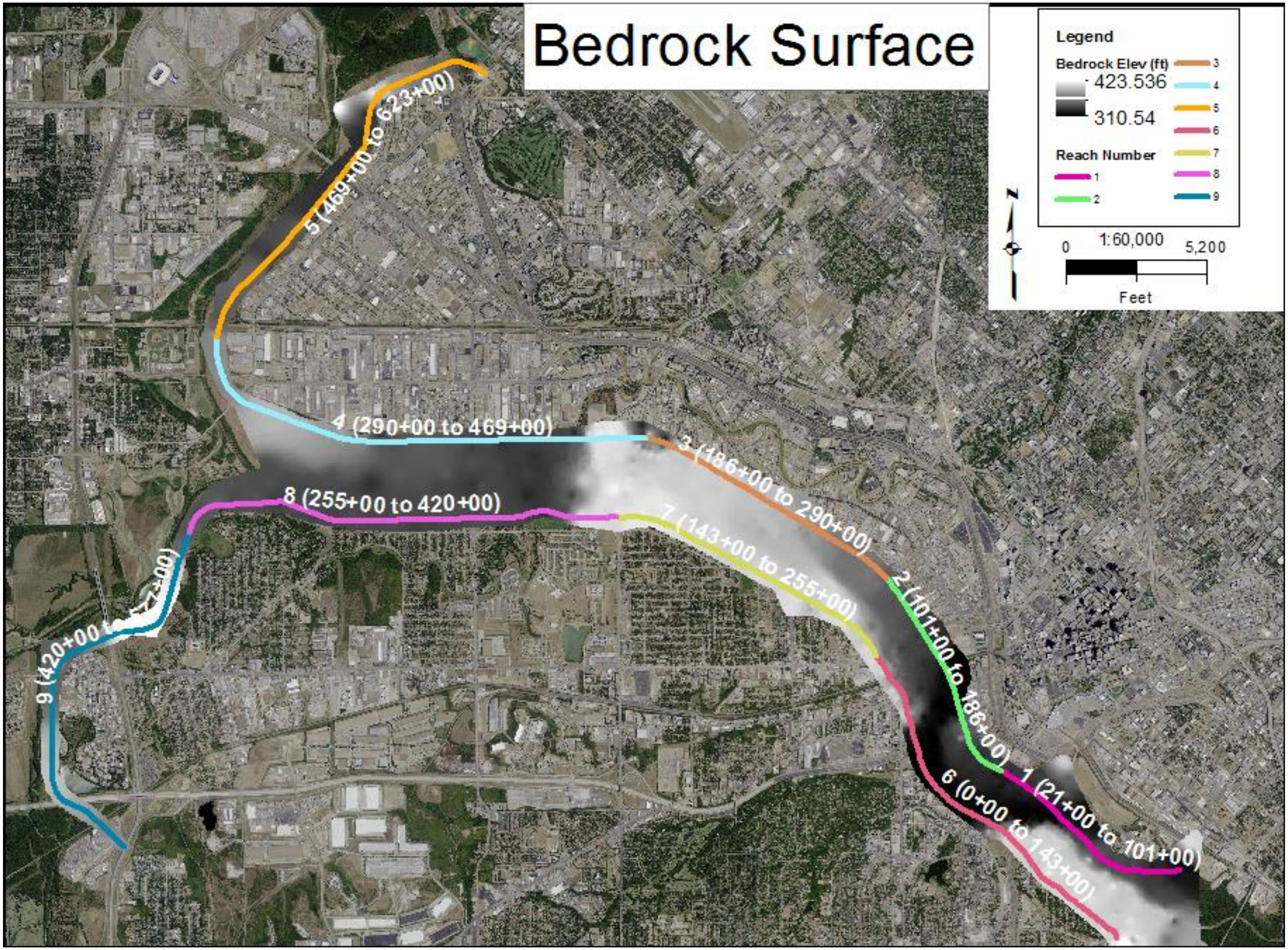


Figure 9-19. Orthographic Map Showing Interpreted Bedrock Surface and Levee Reaches

Bedrock descriptions included in the gINT database were inconsistent and did not permit accurate delineation of solid rock from weathered bedrock units. Thus, for the sake of continuity and timeliness, no attempt was made to interpret the top of competent rock separately from that of the weathered shale. The identifiers used to distinguish these units are included in Table 9-4. This approach was considered reasonable because the uppermost portions of the Eagle Ford shale are expected to have been highly weathered; the top of the bedrock surface is known to be an erosional unconformity.

A series of very wide (>1000 feet), deeply-incised paleo-channels exist in the bedrock beneath the study area (see Figure 9-19). The depth of the paleo-channels increases in the downstream direction. Upstream incisions are wider and less distinctive, indicating prolonged channel migration over wider areas. Downstream incisions indicate that the river system has long been constrained to a narrow physiographic corridor by more resistive rock units, topographic trends, or both. Relatively resistive limestone (Austin Chalk) and shale (Eagle Ford members) in the terraces and uplands immediately adjacent to the floodplain have limited the extent of channel migration.

Levee sections with high shale elevations are listed in Table 9-5. The sections of levee identified in Table 9-5 do not show evidence for significant paleo-channel incision. Mean bedrock elevation throughout the study area within the Trinity River Floodway is 366.6 feet above sea level (asl). Minimum bedrock elevation is 310.5 feet asl. Maximum bedrock elevation is 423.5 feet asl.

Basal sand and gravel elevations tend to follow the same topographic pattern as the top surface of the bedrock (Figures 9-19 and 9-20). The basal sand and gravel DEM shown in Figures 9-18a and 9-20 is an interpreted representation of the top surface of the basal sand and gravel unit underlying the Dallas Floodway. On Figure 9-19, the reds are high elevation, blues are low. Note the similarity between basal sand/gravel morphology and bedrock morphology shown on Figure 9-19. Less than half of the borings used in the interpretation of the bedrock and basal sand and gravel surfaces indicated the presence of a coarse basal soil. To aid interpretation and avoid artifacts of the interpolation process (e.g., bedrock elevations appearing higher than basal soils), elevations of the bedrock surface were substituted in these locations. SC (i.e., sand with clayey sand) soils in many locations were found immediately above bedrock and/or coarse basal units, but were not included in the interpretations of the basal sand and gravel. They were not included because they have lower porosity and permeability – which are the driving factors in seepage analysis – than coarser units with smaller clay fractions.

Table 9-5. List of Levee Sections with Shallow Bedrock

Levee	*Station – Begin*	*Station – End*	*Reach(s)*
West	0+00	65+00	6
West	113+00	273+00	6, 7, 8
West	430+00	476+00	9
West	519+00	565+00	9
East	10+28.08	16+00	1
East	212+00	312+00	3, 4
East	390+00	471+00	4, 5
East	571+00	610+00	5
East	618+00	623+00	5

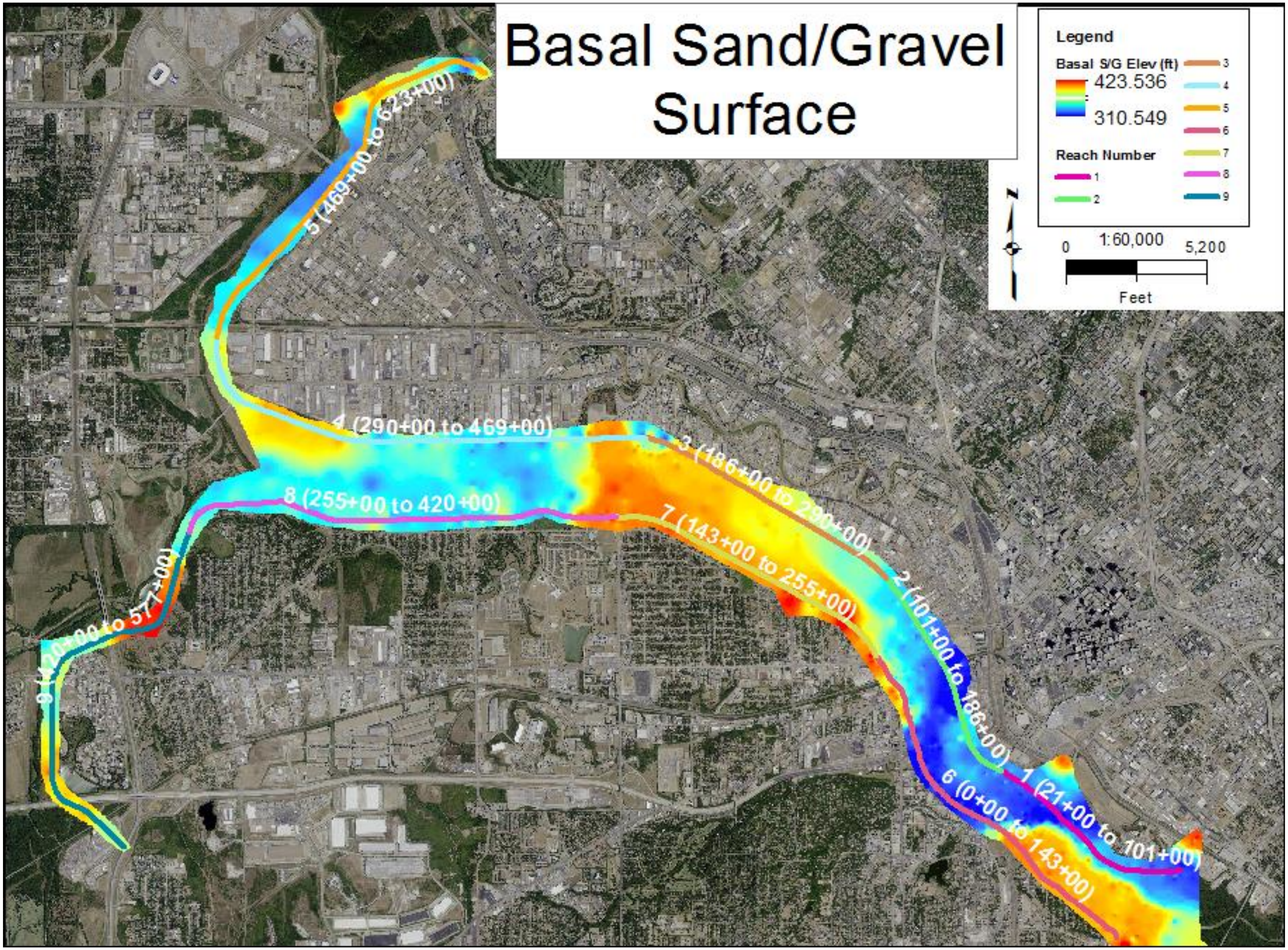


Figure 9-20. Orthographic Map Showing the Interpreted Top Surface of the Basal Sand And Gravel Unit and Levee Reaches

Basal sands and gravels in the study area are generally thicker, coarser, and more laterally continuous that other shallower sands and gravels found in the subsurface. Basal sand and gravel elevations follow the same elevation trends as the bedrock surface. The thickest deposits of basal sands and gravels are coincident with the locations of incised paleo-channels (Figures 9-21 and 9-22). On Figure 9-20, note the thickness of basal units is greatest within incised paleo-channels . Also note the zones where basal sands and gravels are both shallow and thick. Thick deposits of sand and gravel are also found in areas adjacent to terraces and uplands. These deposits are reworked fluvial outwash. In several locations basal sand and gravel units are very shallow. Levee sections with shallow basal sands and gravels are listed in Table 9-6. Due to the potential for seepage during high head, regions of shallow basal sands and gravels are significant engineering considerations. These zones are important to engineering evaluation of the levee system due to their potential as seepage pathways during periods of flooding. Where coarse basal soils are both thick and shallow, the engineering significance of their morphology and spatial distribution is greatest (Figures 9-21 and 9-22).

Table 9-6. List of Levee Sections with Shallow Basal Sand and Gravel

Levee	*Station – Begin*	*Station – End*	*Reach(s)*
West	0+00	59+00	6
West	116+00	271+00	6, 7, 8
West	429+00	477+00	9
West	526+00	567+00	9
East	10+28.08	16+00	1
East	36+00	55+00	1
East	67+00	75+00	1
East	220+00	313+00	3, 4
East	398+00	472+00	4, 5
East	576+00	607+00	5

The elevation of the bedrock and basal sands and gravels are important for planning future geologic and geotechnical work. Basal sands and gravels may pose seepage and/or stability concerns, which must be addressed in levee remediation and construction. Shallow sands of any type must be considered in geotechnical analysis, but major concern should be focused on continuous or semi-continuous coarse-grained soils like those found in the basal sand and gravel units beneath the Dallas Floodway. Coarse-grained soils present at or near the ground surface are significant risks for seepage. If near-surface sands and gravels are present, their continuity and dimensions across levee foundations need to be fully evaluated, such that appropriate measures can be taken to mitigate any associated threats to the integrity of the levee system. In the Dallas Floodway, there are areas where coarse basal materials are interpreted as shallow (< 10 feet; Figure 9-22) continuous, and thick (Figure 9-21). These areas are especially important engineering considerations.

On Figure 9-21, red and deep orange areas have very shallow basal sands and gravels (0-5 feet) and are critical investigation zones that pose seepage risks. Progressively cooler colors indicate greater depths from the ground surface to the top of the coarse basal soils. Note the intersection of these zones with water bodies, stream channels, and other topographic depressions. The spatial relationships of permeable and porous materials in the subsurface are of critical importance to geotechnical analysis and rehabilitation planning.

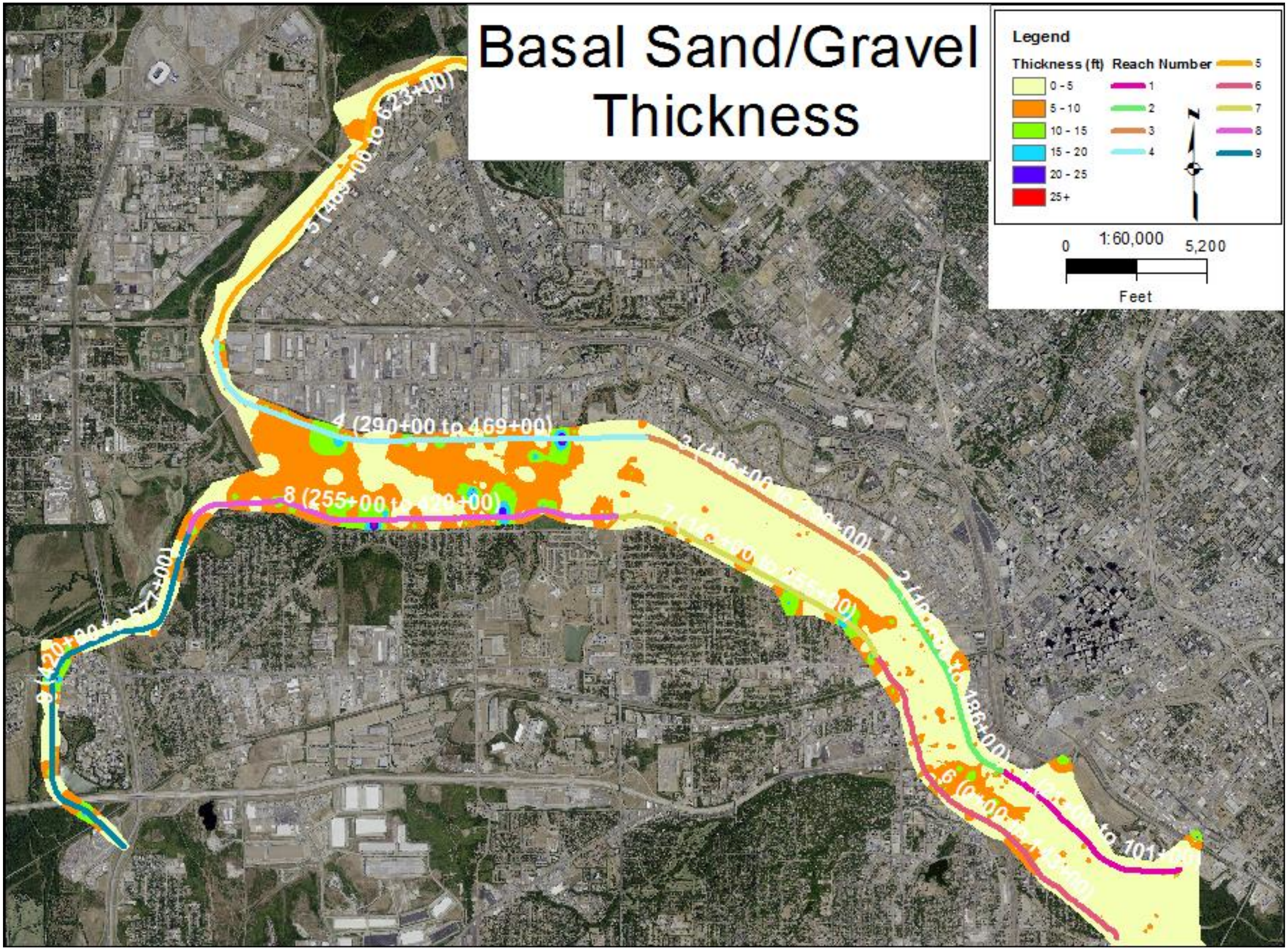


Figure 9-21. Map of Base Sand and Gravel Thickness

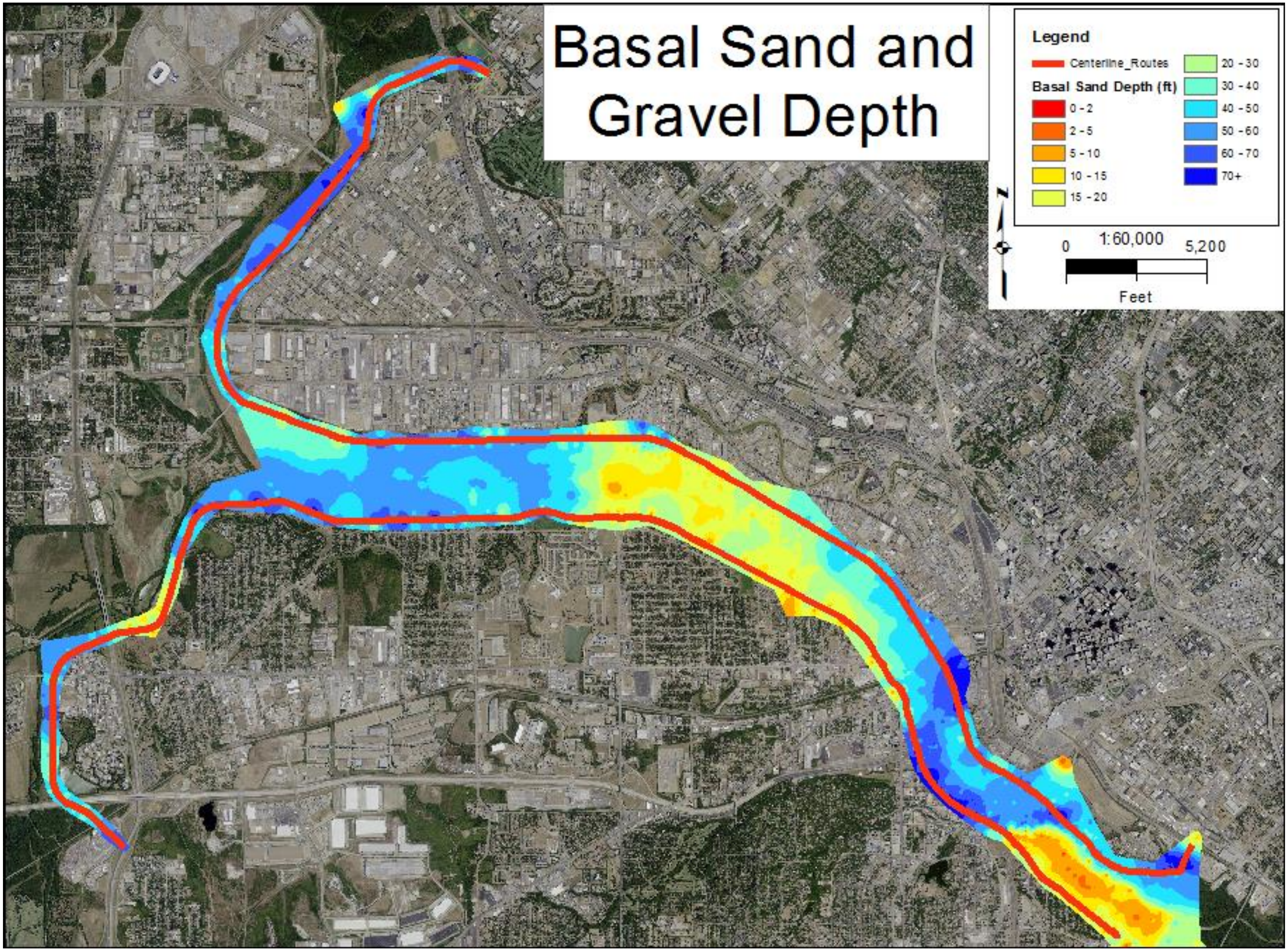


Figure 9-22. Map of Basal Sand and Gravel Depths

9.3 Eagle Ford Formation

As shown by the geologic map in Figure 3-2, much of the levee system protecting the Dallas metropolitan area is built upon the Eagle Ford Formation and is situated stratigraphically between the Woodbine and Austin Chalk Formations. The Cretaceous stratigraphy in the Dallas area dips to the east and southeast, between 50 to 100 feet per mile, as shown by Schuler's block diagram (Figure 3.1; 1918). Mapped Upper Cretaceous outcrops and marine fossils within the Woodbine, Eagle Ford, and Austin Chalk Formations in the West Fork and Elm Fork of the Trinity River floodplains and surrounding tributary valleys in Figure 3-2 are the basis for subdividing the Eagle Ford Formation further in into distinct mapable units in Figure 3-3 (Norton 1965). The presence of Bentonite and flaggy limestone beds in the Eagle Ford has been used to differentiate the Eagle Ford into three major subdivisions which in ascending (i.e., oldest to youngest) order are the Britton, Kamp Ranch, and the upper Arcadia Park Formations.

The lower Britton Formation has been further subdivided by Norton (1965) into 9 stratigraphic units in Figure 3-3 with descriptions of these units tied to known outcrop exposures in the Dallas metropolitan area. Much of the western extent of the floodway is located in Norton's units 7, 8, and 9, which are mainly the dark grey shales, as compared to the upper Eagle Ford calcareous rich strata of the Kamp Ranch and Arcadia Park Formations. Exposures of the Kamp Ranch Limestone and Arcadia Park Formations are described in the west facing bluffs along the east side of Village Creek, at Arcadia Park (type section), and further to the east in the Trinity floodway, at the Inwood-Hampton Viaduct (Norton's exposure 78). This limestone is described by Norton as being "…the most useful key-bed in the entire Eagle Ford section (Norton 1965, p77)." The occurrence of this resistant bench rock is likely responsible for the topographic high observed in deeper levee borings immediately east of the Hampton pump plant at the North levee (see Figures 9-18 and 9-19; note occurrence of shallow sands and abrupt elevation change of basal units). The old West Fork Trinity river channel abruptly turns north at this buried topographic bedrock high (see Figures 9-17a and 9-18), before the floodway was built, and its confluence with the main channel of the Elm Fork was diverted to the present location.

9.4 Liquid Limits and Normalized CPT Strengths

Cross section of liquid limit (LL) can be used for slope stability evaluation and correlates to historic slides. The Dallas levee system is 12 miles long on each side and LL ranges vary from extremely low (i.e. silts) to the upper 90s. High LL are concentrated in specific zones, but without LL cross sections it is impossible to visualize or establish these zones. If LL were truly random then an average could be used.

Available data was used to generate two cross sections of LL in terms of elevation and levee station. The final data file used to generate the LL cross section was simply composed of elevation, station, and LL. The data file required numerous steps to combine four separate database files. Surfer software (by means of advanced options) was used to force lateral cross section generation. Red zones represent high LL and blue represents low LL values.

Figures 9-23 and 9-24 show the cross sections of LL for both levees. The high LL values are concentrated in zones inside the foundation and also inside the levee. In general, the levee zones having high LL also had foundations of high LL, which infers that the borrow material for the levees was taken from borrow pit between the river and the levee.

Figures 9-23 and 9-24 also show cross sections of normalized strength that were estimated based on a USACE ERDC procedure using CPT data. This procedure uses non-linear CPT data normalization and also uses both CPT measurements to predict the normalized soil strength. Normalized strength is defined as the strength divided by the vertical effective stress; for sands this strength is a drained strength, whereas for clays the strength is an undrained strength. Low LL clays (i.e. less than 50) generally have a normalized strength of about 0.31, whereas increasing clay over consolidation will cause an increasing normalized strength (from 0.31 to 2). Low relative density sands have normalized strengths as low as 0.5 and will increase to 1.0 for high relative density. Desiccated clay above the water table will have a much higher normalized strength (i.e. greater than 0.31) compared to the historical lowest water table, generally blue in the cross sections. These cross sections of CPT predicted normalized strength show numerous normally consolidated clay pockets (i.e. normalized strength from 0.30 to 0.4). These normally consolidated pockets generally extend over the width of several CPT soundings.

Figures 9-23 and 9-24 show the strong history of landslides on the river side of the East and West levees. These cross sections of landslides are actually in terms of the year of the landslide on the vertical axis. The most obvious observation is that there are numerous levee segments having a strong history of landslides, while other levee segments have no landslides. Landslide segments with a strong landslide history do not show any decrease of landslide occurrence with time; all undrained strength based landslides have a decreased intensity with time after construction. Only in the last 7 years have the landslide size been quantified by City of Dallas in terms of landslide width and length. Many of the landslides in the last 40 years have occurred on the river side of the levee after heavy rainfall and increasing river stage level.

There appears to be a good correlation between levee segments having high landslide occurrence and foundations having high LL and steeper levee slopes. LL is a good index of drained softened FSS strength, whereas undrained strength is a reflection of the in situ strength. The slope angle is a reflection of embankment driving force and strength is the embankment resisting component; consequently, the occurrence of landslides is the resultant of driving forces versus resistance. Although the landslides have a strong correlation to LL, slope angle, and pore pressure, another factor can be the presence of stiff clay (and clay shale) softening.

This page intentionally left blank.

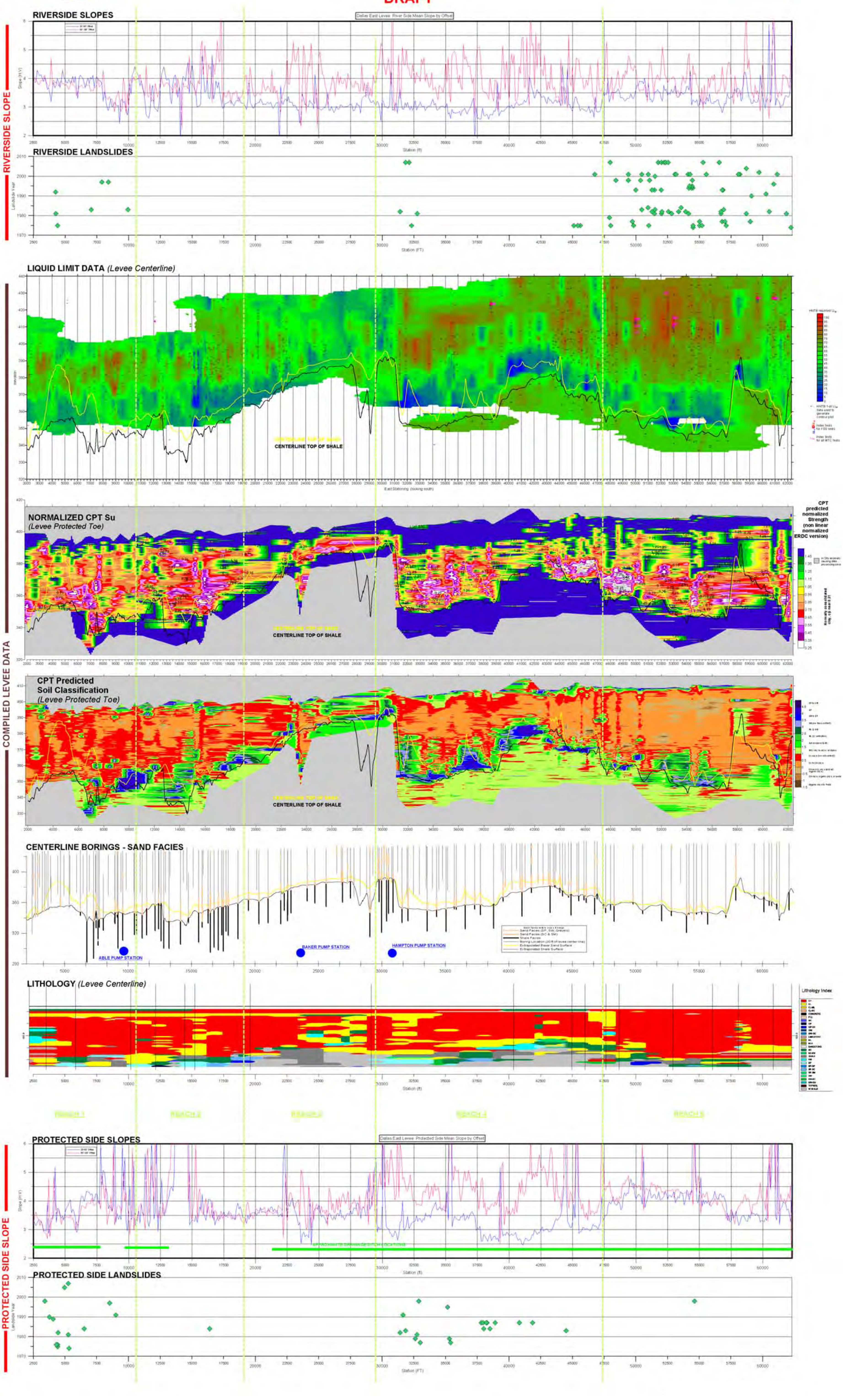
FIGURE 9-23

DALLAS TRINITY RIVER: EAST LEVEE DATA

PREPARED BY GEOTECHNICAL AND STRUCTURES LABORATORY,
GEOTECHNICAL AND GEOSCIENCE BRANCH

MAY 27, 2011

DRAFT



This page intentionally left blank.

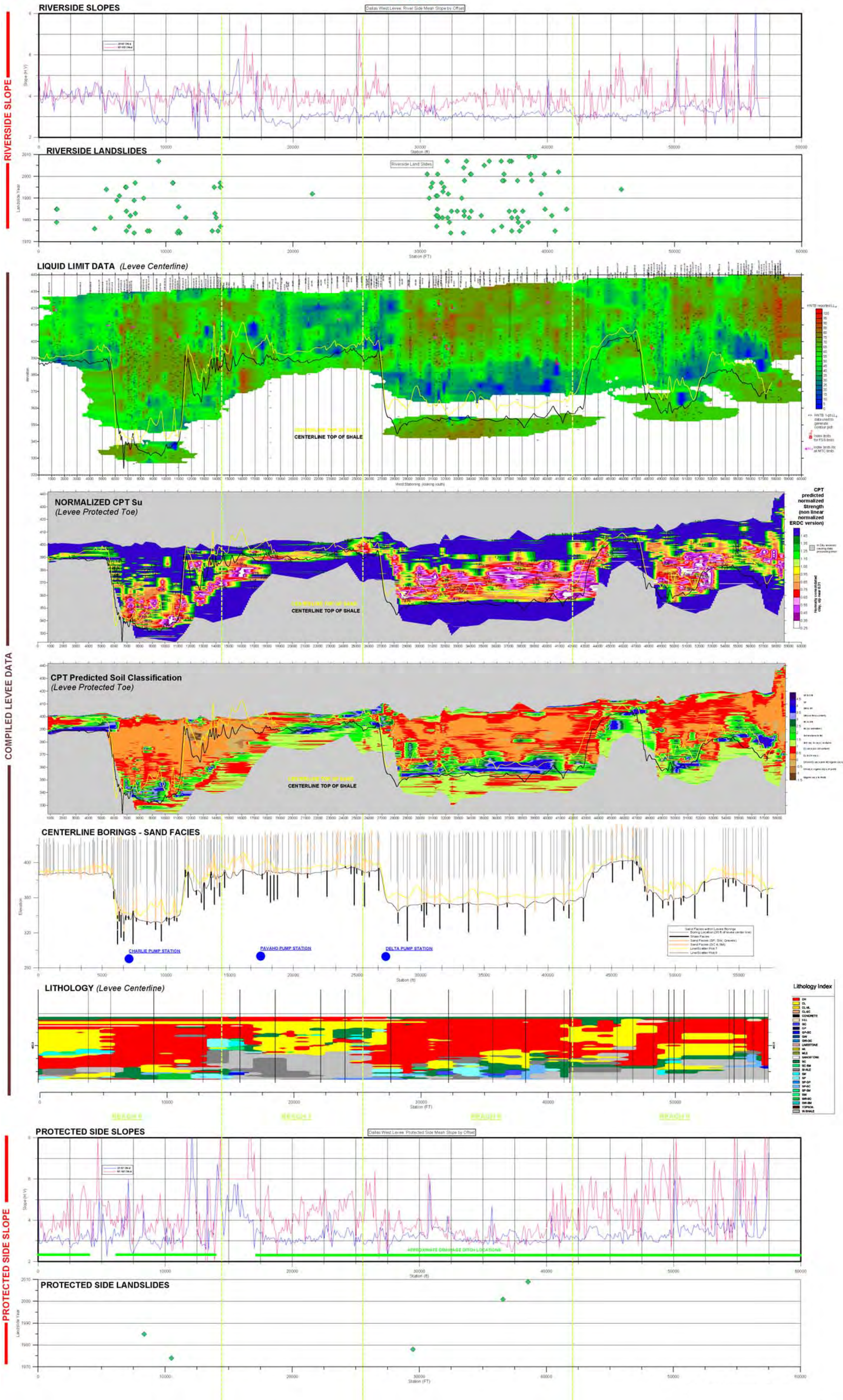
Figure 9-24

DALLAS TRINITY RIVER: WEST LEVEE DATA

PREPARED BY GEOTECHNICAL AND STRUCTURES LABORATORY, GEOTECHNICAL AND GEOSCIENCE BRANCH

MAY 27, 2011

DRAFT



This page intentionally left blank.

10.0 STABILITY AND SEEPAGE ANALYSIS

A risk assessment of the Dallas Floodway was performed by the Risk Management Center (RMC) to assess the baseline conditions for the Dallas Floodway. Extensive seepage and stability analysis was performed as part of this effort. These analyses and the Base Condition Risk Assessment (BCRA) results were adopted for this report.

Transient seepage analysis was used for the seepage and stability analyses performed for the BCRA. Routine USACE practice in the past has been to use steady state seepage analysis. The BCRA team agreed that transient seepage analysis was appropriate due to the relatively low permeability of the levee soils and the relatively short duration of flood loading. The hydrology and hydraulics (H&H) analysis indicated the levee would be loaded at 75% for three to five days and 50% for one to two weeks. Use of the transient analysis in lieu of the steady state analysis for the Dallas Floodway feasibility study was endorsed by the USACE Engineering and Construction Community of Practice (Geotechnical sub-community). Therefore, all of the analysis presented in the BCRA and adopted by this report will rely on transient seepage analysis.

10.1 Cross Section Selection

A total of eight cross-sections were selected for seepage and stability analysis from the east and west levee reaches of the Floodway. They were selected to be representative of the most critical conditions on the levee system using engineering judgment. Reasonable amounts of uncertainty were factored into the analysis for parameters that displayed varying results during field and laboratory testing. Any gaps in data were typically bridged with reasonably conservative assumptions. The stationing corresponding to the cross sections selected is presented below.

East Levee Alignment

- 74+00
- 220+00
- 311+00
- 410+00

West Levee Alignment

- 10+00
- 188+00
- 250+00
- 335+00

The factors considered for cross sections, how the cross sections were constructed and the analysis of these sections is discussed in Appendix A of the BCRA report. The BCRA report is also included as an appendix to the feasibility report (Appendix C).

10.2 Seepage Analysis

Seepage and stability analyses were performed on various sections of the Dallas Floodway Levee System in support of the risk assessment. These analyses were carried out before and during the assessment as a tool for use by the risk cadre to provide a greater understanding of how the performance of the levees will be affected by varying flood loads, varying material permeability and strength, and various deficiencies. The results provided reference points for an informed discussion by the entire risk analysis group during the elicitation process. All analyses were carried out using GeoStudio 2007, Version 7.17.

Seepage analyses were carried out on each cross section to provide an estimate of seepage through the levee section, gradients, and an estimate of pore water pressures for subsequent stability analyses. Each cross section has a suite of analyses developed for it that use three different sets of permeability estimates for each soil in each model and use two different historical storms scaled to three different heights to calculate 18 different seepage regimes. Following the calculation of each set of pore water pressures, a stability analysis is carried out to see how different hydrologic conditions affect the performance of the Dallas Floodway Levee System.

The levees are made up of either low or high plasticity clayey materials (or a mixture of both). Both of these materials have a relatively low permeability in comparison to coarser grained materials. Hydrologic records of the levee system indicate the Trinity River typically stays within its primary banks near the river thalweg the majority of the year and water is only against the levees during flood events. Therefore, it's prudent to assume that flood waters will not have enough time to fully penetrate the levees and their foundations and subsequently develop steady state conditions during a Standard Project Flood (SPF) event or during a modified historical event that has a relatively long duration. Consequently, transient seepage analyses were performed for all sections instead of steady state seepage analyses. The transient analyses showed that the piezometric grade did not have an opportunity to stabilize to a steady state type of surface and failed to penetrate the more impervious areas of the levees and foundations. Seepage parameters and boundary conditions are discussed in detail in Appendix B of the BCRA report. The district Project Delivery Team (PDT) concurs with the BCRA recommendation of using transient seepage analysis.

Because desiccation cracking begins at the ground surface, the BCRA considered the increased permeability in the upper portion of the embankment and foundation due to desiccation cracking in their analysis. The RMC found in the analysis, "the duration of loading is likely not sufficient to saturate the levee system enough to cause effective strengths to reduce far enough to lead to global failure. The gradients induced in the basal sand layer are likely not sufficient to cause internal erosion to progress beneath the levees." The district PDT concurs with the BCRA findings that desiccation cracking is not likely to lead to levee failure.

10.3 Stability Analysis

A stability analyses were run using the results of every seepage analyses as a parent analysis in SLOPE/W. The stability analyses provide the metric that describes how robust the levee system is under the changing seepage conditions. All stability analyses carried out for this investigation used the optimization feature in SLOPE/W to determine the most critical failure surface. Stability analyses were performed using the step in the seepage analysis that corresponds to the peak flood stage of the flood event. Some additional stability analyses were also done on time steps beyond the peak time to account for the possibility that later stages could produce more critical pore pressures.

As discussed in the preceding Seepage Analysis section, it is anticipated that steady state conditions will not have an opportunity to develop due to the brief nature of flood events and low permeabilities of the fine-grained soils in the levees and foundations of the Dallas Floodway Levee System. Therefore, drained shear strength parameters were used for the stability analyses.

The stability parameters, analyses, and results are discussed in detail in Appendix C of the BCRA report. The following tables (Tables 10-1 through 10-3) summarize the results of the stability analyses. The stability results indicate the most critical sections were Station 220+00 East and Station 10+00 West. Permeability of the levee and foundation was the single most important factor in the stability analyses.

The lowest factors of safety were generally achieved using the highest permeabilities for the levee and foundation soils.

The event tree for global slope instability includes a node for the probability of factor of safety less than 1.0. The RMC team decided that a factor of safety less than 1.0 as calculated by normal two-dimensional slope stability analyses would be a reasonable representation for a failure condition. The software (SLOPE/W) used for the analyses has the capability to calculate the probability of factor of safety less than 1.0 using a Monte-Carlo approach if distributions are input for shear strength.

Table 10-1. Seepage And Stability Analysis Results For Sections Under The Load Of The 1990 Hydrograph Scaled To The Elevation Of The Levee Crest

Case	Parameter	74+00 East	220+00 East	311+00 East	410+00 East	10+00 West	250+00 West	335+00 West	188+00 West
Best k, Best Str	FoS	**2.05**	**1.75**	**2.75**	**2.25**	**1.22**	**2.42**	**1.61**	**2.50**
Low k, Prob Str	Mean FoS	**2.19**	**1.84**	**2.87**	**2.06**	**2.71**	**2.71**	**2.04**	**2.38**
	P(failure)	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
	Min FoS	1.65	1.43	1.89	1.6853	2	2.03	1.47	1.7707
Best k, Prob Str	Mean FoS	**2.18**	**1.77**	**2.63**	**2.30**	**1.38**	**2.38**	**1.7**	**2.38**
	P(failure)	0.00%	0.00%	0.00%	0.00%	13.00%	0.00%	0.00%	0.00%
	Min FoS	1.58	1.31	1.84	1.6524	0.95	1.98	1.21	1.7707
High k, Prob Str	Mean FoS	**1.91**	**1.24**	**2.35**	**1.88**	**1.89**	**2.24**	**1.56**	**2.15**
	P(failure)	0.00%	1.67%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
	Min FoS	1.48	0.82	1.58	1.528	1.41	1.87	1.12	1.6145

Table 10-2. Seepage And Stability Analysis Results For Sections Under The Load Of The 1990 Hydrograph Scaled To 75% Of The Elevation Of The Levee Crest

Case	Parameter	74+00 E	220+00 E	311+00 E	410+00 E	10+00 W	250+00 W	335+00 W	188+00 W
Best k, Best Str	FoS	**2.22**	**1.84**		**2.35**	**1.46**	**2.43**		**2.50**
Low k, Prob Str	Mean FoS		**2.05**		**2.09**	**2.71**	**2.71**		**2.38**
	P(failure)		0.00%		0.00%	0.00%	0.00%		
	Min FoS		1.71		1.7163	2	2.03		1.7707
Best k, Prob Str	Mean FoS		**1.89**		**2.30**	**1.60**	**2.42**		**2.38**
	P(failure)		0.00%			0	0.00%		
	Min FoS		1.56		1.6568	1.15	2.02		1.7707
High k, Prob Str	Mean FoS		**1.56**		**1.95**	**2.05**			**2.31**
	P(failure)		0.00%		0.00%	0.00%			0.00%
	Min FoS		1.15		1.6023	1.54			1.7683

Table 10-3. Seepage And Stability Analysis Results For Sections Under The Load Of The 1990 Hydrograph Scaled To 50% Of The Elevation Of The Levee Crest

Case	Parameter	74+00 E	220+00 E	311+00 E	410+00 E	10+00 W	250+00 W	335+00 W	188+00 W
Best k, Best Str	FoS	**2.11**	**1.95**		**2.36**	**1.76**	**2.49**		**2.50**
Low k, Prob Str	Mean FoS		**2.08**		**2.30**	**2.71**	**2.71**		**2.38**
	P(failure)		0.00%		0.00%	0.00%	0.00%		0.00%
	Min FoS		1.71		1.6568	2	2.03		1.7707
Best k, Prob Str	Mean FoS		**2.01**		**2.30**	**1.86**	**2.5**		**2.38**
	P(failure)					0.00%	0.00%		
	Min FoS		1.56		1.6568	1.38	2.06		1.7707
High k, Prob Str	Mean FoS		**1.82**		**2.01**	**2.09**			**2.38**
	P(failure)		0.00%		0.00%	0.00%			0.00%
	Min FoS		1.15		1.6508	1.7			1.7707

10.4 Seepage and Stability Results and Conclusions

The probability of levee failure resulting from progressive slope failures was also considered on the East and West levees. The risk analysis indicates the probability of either a global slope failure or a progressive slope failure occurring and leading to a subsequent levee breach is low. The critical sections for a seepage failure mode initiated by internal erosion of the sand foundation were determined to be Station 311+00 East Levee and 250+00 West Levee. The results of the seepage analysis indicate that the horizontal seepage gradient at Station 311+00 East Levee and Station 250+00 West Levee does not exceed the critical gradient for the foundation sands and internal erosion is not likely to occur. However, the risk analysis for this failure mode indicates the combination of consequences, probability of failure, and uncertainty justify further study of the potential for an internal erosion failure mode. Therefore, this failure mode was evaluated further in the FRM analysis described in sections 11.0 and 12.0.

The critical sections for seepage failure mode initiated by heave of the surface clay and subsequent internal erosion of the foundation sand were determined to be Station 220+00 East Levee and 335+00 West Levee. The results of the seepage analysis indicate that the factor of safety for heave is below 1 for Station 220+00 East Levee and just above 1 for Station 335+00 West Levee. The seepage analysis also indicates the horizontal seepage gradient does not exceed the critical gradient for the foundation sands; therefore internal erosion of the foundation sands is not likely to occur. However, the risk analysis for this failure mode indicates the combination of consequences, probability of failure, and uncertainty justify further study of the potential for a heave failure mode.

The critical sections for a global slope stability failure resulting in failure of the levee were determined to be Station 220+00 East Levee and 10+00 West Levee. The factors of safety were greater than 1 for almost all cases. The probability of slope failure for Station 220+00 East Levee for the 1990 hydrograph scaled to the top of the levee was less than 2% and zero for the other loading cases. The probability of slope failure for Station 10+00 West Levee for the 1990 hydrograph scaled to the top of the levee was 13% and zero for the other loading failure is below the tolerable risk guidelines for dams currently being used by USACE even when considering uncertainty. Analysis conducted to date indicate that the risk of a stability failure of the levee is acceptable. However, localized slope failures will continue to occur and must be repaired by the City of Dallas as they occur.

11.0 GEOTECHNICAL PARAMETERS FOR FLOOD RISK MANAGEMENT ALTERNATIVES

Geotechnical parameters were developed to facilitate the various alternatives necessary to support the feasibility level design for net benefits and benefit/cost ratio calculations. The following structural alternatives were evaluated for FRM development to address the potential failure modes identified in the BCRA and the internal erosion and heave described in the previous section 10.4:

a. Overtopping (limited to levee raise), PFM2

b. Overtopping and breach (Armoring, limited to ACB only), PFM2

c. Internal Erosion (limited to cutoff wall), PFM7 and 8

d. Instrumentation (monitoring/non-structural)

11.1 OVERTOPPING (LEVEE RAISE)

Levee raises would consist of a 3:1 slope extending from the edge of the crest to the riverside as needed. (Reference Sheet C-501 in Appendix D *Civil and Structural Design* for a typical template). The existing levee has typical side slopes of 3H:1V; however, the slopes do vary due to degradation of the levee across all reaches. There was consideration given to add fine-grained material to the flood side surface of the levee to decrease the embankment slope to a maximum of 4H:1V. This measure was considered to address desiccation cracks that are known to cause slides.

The existing levee crest has an emergency access road running the length of the levee. This gravel access road will be demolished and removed prior to the raising of the levee, as the existing gravel road is not compatible as levee material. The gravel road material is unsuitable for levee use and will be replaced with suitable levee material. The raised levee will include a new 8 inch thick crushed limestone access road with a geotextile liner. The new crushed limestone road will meet all TxDOT geotechnical paving requirements. “Effective levee height” does not include the 8” crushed limestone road. Bridges would be affected by levee raises. Analyses have been performed to obtain a cost estimate for development of FRM components. Additional engineering analyses should be considered in future studies in Planning, Engineering and Design Phase (PED), (e.g. type and placement of material around bridge members to reduce risk of settlement).

11.2 OVERTOPPING WITH BREACH (ARMORING)

Levee armoring would consist of articulated concrete block (ACBs) starting 10 feet below the riverside crest of the levee and extending to 50 feet past the toe of the landward side of the levee. (Reference Figure D-26 in Appendix D *Civil and Structural Design* for the levee armoring template ACB details). The dimension of the area of armoring was based on an estimate of the extent at which there would be dissipation of velocity and reduced potential for damage caused by overtopping. Armoring on the levee will be along the existing contours of the levee only. For the purposes of this study, it is assumed armoring and levee raises are considered two separate items and evaluated independently and not in combination with each other.

11.3 INTERNAL EROSION (CUT-OFF WALLS)

To assess seepage mitigation solutions and placement of those techniques, the borings completed along the reaches of both levees were analyzed. The locations selected for placement of cut-off walls have one or more of the characteristics listed below. The criteria were selected based on engineering judgment and is subject to further refinement at design level.

- Greater than 35% sand in the matrix
- Sand layers with thickness greater than one foot in the levee or in the foundation. Some locations have multiple sand layers
- Sand layers are continuous through the levee profile and layers daylight at or near both the riverside and landside toes

Locations chosen have the above characteristics; see Table 11-1 for locations. The soil borings on either side of these sections did not meet the above specifications; in some cases the extent of the potential cutoff wall was extended in order to encompass a reasonable economic length. For purposes of this study, a minimum length of 900 feet of cutoff wall was established as a technically viable economic option. Soil characteristics on either side of the chosen areas, however, are marginally outside the chosen criterion and consequently are considered a potential concern for internal erosion. Further detailed analysis of these areas would be warranted during design. The total length of the cut-off wall proposed is 4.69 miles. The City has installed cut-off walls (for purposes of the FEMA 100-year accreditation) from 3+00 to 29+00 for a total length of 0.49 miles on the West Levee and from station 285+00 to 442+00 for a total length of 2.97 miles along the East Levee. The extents of cut-off walls are estimated in addition to the cut-off walls installed by the City.

Table 11-1. Extent of Cutoff Walls, East and West Levees

East Levee and Elm Fork Cut Off Walls*		West Levee and West Fork Cut Off Wall*	
Begin Sta.	*End Sta.*	*Begin Sta.*	*End Sta.*
459+00	468+00	0+00	3+00
531+00	551+00	29+00	67+50
585+50	611+04	117+50	135+50
		160+00	195+50
		329+50	346+00
		390+00	409+00
		435+50	444+00
		450+50	480+70
		530+00	547+00

Note: *All Stationing in Corps Levee Stationing.

11.4 LEVEE FRAGILITY

Geotechnical parameters along with a description of how and when the under seepage process would occur (i.e. river stages and durations that initiated the process, likely locations and elevations of the material being removed, and a description of the likely time progression from initial saturation to levee failure) were developed to support the HEC-RAS computer modeling for HEC-FDA. It should be noted that the only seepage related failure addressed is internal erosion or piping. "Blowout", which usually refers to a heave related failure mode, was one of the failure modes examined in the BCRA. The heave failure mode was not considered probable and not addressed here. The Dallas Floodway Risk Reduction Analysis (Appendix C, Part II) discusses the alternative analysis relative to the HEC-FIA modeling effort.

The BCRA team estimated probabilities of failure for river stages equal to 50%, 75%, and 100% height of levee. The results are as follows:

- The probability of failure for a river stage up to 50% of the levee height is 3.04e-7 or an annual probability of failure of 4.56e-9.
- The probability of failure for a river stage up to 75% of the levee height is 1.46e-4 or an annual probability of failure of 9.03e-7.
- The probability of failure for a river stage up to 100% of the levee height is 1.42e-3 or an annual probability of failure of 1.35e-6. This means there is a 1 in 700 chance that the levee will fail due to internal erosion for floods that reach the top of the levee.

When an internal erosion breach starts to occur it will be a very large sand boil (several feet in diameter) and the flows will be on the order of just a few cubic feet per second (cfs). From Table 31 on page D-39 of the BCRA: It takes 26 hours for the breach to go from the point where intervention has failed to full breach for river stages up to 50% of the levee height, and six hours for river stages of 75% or more of the levee height. The width of the breach formation is estimated to be 150 feet. The invert of the breach at East Levee Station 311+00 is elevation 405 and West Levee Station 250+00 is elevation 400. Based on the above, the following fragility curve information was prepared for use in the HEC-FDA analyses:

Station 311+00 East Levee		
Height of Levee	*River Stage*	*Probability of Failure*
0%	418.8	0
50%	419.8	3.0e-7
75%	426.6	1.5e-4
100%	432.2	1.4e-3

Station 250+00 West Levee		
Height of Levee	*River Stage*	*Probability of Failure*
0%	418.3	0
50%	419.3	3.0e-7
75%	426.2	1.5e-4
100%	431.8	1.4e-3

The estimated fragility curves developed over the course of this study were tested within the HEC-FDA modeling package and considered in the projection of equivalent expected annual inundation damages (EADs) used to economically assess the existing and alternative levee heights, as per establishment of the National Economic Development (NED) Plan. In practice; however, due to an HEC-FDA modeling software limitation, it was not possible to simultaneously apply both "fragility curve" and "interior versus exterior flood stage" methodologies. The latter methodology was used in the overtopping with a subsequent breach analysis because it was deemed to much better represent flood inundation risks behind the levees, since it takes into account all of the carefully-configured Unsteady HEC-RAS modeling capabilities associated with potential breach formation and timing. The fragility curve presented here was used for the internal erosion HEC-FDA runs.

11.5 INSTRUMENTATION

The City of Dallas has installed piezometers and inclinometers along the cut-off walls they constructed to monitor the effect of cut-off walls installation on the levee system. Additional piezometers and inclinometers should be installed in critical areas as part of any structural measure or by itself to continue to monitor the levee system in the future.

12.0 FLOOD RISK MANAGEMENT ALTERNATIVES

Alternatives were developed based on their ability to alter the frequency and inundation impacts for levee overtopping flood events or to prevent internal erosion. Levee raises, levee armoring, and cutoff walls are three separate alternatives to solve different failure modes and are evaluated independently first, and then in combination with each other. A historic railroad bridge is located at the downstream end of the Dallas Floodway called the AT&SF Railroad Bridge. The modification of the abandoned AT&SF Railroad

Bridge has been identified as a flood risk management measure due to its impact to the SPF water surface profile, its location at the downstream end of the Dallas Floodway, and the fact that the bridge is no longer needed for rail traffic. Hydraulic analysis has shown that the bridge causes a rise in the Standard Project Flood (SPF) water surface profile due to its numerous closely spaced piers, low deck height, and large earth embankments within the Floodway. The AT&SF bridge modification is considered a first added increment for the levee raise, levee armoring, and cutoff walls alternatives.

12.1 LEVEE RAISES

The levee raise alternatives were evaluated by determining the levee raise height and length of levee modification required at every location along the existing levee crest that the selected flood event exceeds the levee crest; see Appendix D *Civil and Structural Design*, Figures D-1 through D-6. The levee is raised to the height of the selected flood event water surface profile. This flood risk management alternative reduces the frequency of levee overtopping. Borrow pit locations were selected based on an evaluation of the available boring data to determine the acceptability and depths of suitable soil. Only material classified as SC, CL, and CH by USCS classification were considered acceptable for borrow. The same performance was assumed for the 3:1 and 4:1 side slopes. While it is true that a flatter side slope would produce very slightly lower velocities along the back sides of the levees, those velocities would still be far in excess of those capable of initiating surface erosion and subsequent initiation of erosion at the downstream crest. Similarly, the flatter side slope is associated with a broader levee template, which would require a slightly longer time period to erode downward. This was viewed as a minor change in erosion rate and was essentially insignificant.

12.2 ARMORING

The levee armoring alternatives were evaluated by determining the length of levee armoring required at every location along the existing levee crest that the selected flood event exceeds the levee crest. The levee is armored only along the length of levee required as indicated by the profile plot comparison for each selected flood event; see Appendix D *Civil and Structural Design*, Figures D-1 through D-6. A small increment of levee armoring is extended above the flood event water surface profile to ensure against flanking of the armoring for the selected flood event. These alternatives do not prevent flooding in the protected area but reduces the severity (flooding depth) and alters the arrival time of flooding in the levee protected area for the selected flood event by preventing levee breaching following the overtopping. This alternative is expected to also delay the onset of levee breaching for flood events that exceed the capacity of the selected measure.

12.3 CUT-OFF WALLS

The City of Dallas has constructed cut-off walls along portions of the Dallas Floodway levee alignments in support of 100-year flood FEMA accreditation of the levee system. The cut-off walls are designed to cut off any continuous sand layers that may penetrate below the levees. Continuous sand layers were identified in the BCRA as potential pathways for seepage to penetrate the foundation soils beneath the levees, potentially leading to a decrease in the stability of the levees, an increased potential for internal erosion under the levees, and an increase in protected side seepage. Based on plans and specifications provided by the City of Dallas, the cut-off walls are located within the floodway, a minimum of 25 feet in front of (away from) the flood side toe of the levees. The cut-off walls are approximately three feet wide and extends from the ground surface through the alluvial soils and penetrates at least 5 feet into the Eagle Ford Shale and one to three feet into the Austin Chalk bedrock layer. The construction specifications indicate that the cut-off walls would have permeability no greater than 2.5e-4 feet/hour.

The PDT designed three seepage mitigation alternatives to further stabilize the levee system:

1. A weighted clay seepage berm placed on the protected side of the levee would provide a filter on the downstream side of the levee that would pass seepage but inhibit any material from eroding from the surface of the levee or from the downstream ditch. The clay seepage berm is not intended to pass seepage but to lengthen the seepage path as stated. This alternative was analyzed in the seepage models with and without the City of Dallas cutoff wall.
2. A soil-bentonite cutoff wall placed on the flood side of the levee. This would essentially be the same as the City of Dallas cutoff wall. It is designed to cut off the flow of seepage through the continuous sand layers, decreasing exit gradients and downstream seepage. Thickness, penetration, and permeability characteristics will be similar to that used for the City of Dallas cutoff wall design. The 25 foot distance relative to the levee toe location of the cutoff wall allows the use of soil bentonite slurry rather than a more costly bentonite cement cutoff wall.
3. A 3 foot thick flood side clay cap on the ground surface over the cutoff wall. This alternative would only be used in areas where there is sand exposed on the flood side surface of the floodway. The proposed clay cap would provide a horizontal impermeable blanket at the ground surface to inhibit floodwaters from penetrating the floodway surface in front of the levee, bypassing the cutoff wall, and reaching a subsurface basal sand unit.

Paragraph 11.0 of this appendix presents the criteria used in determining seepage areas and seepage mitigation techniques. The cutoff wall with the clay cap was the seepage mitigation measure carried forward for NED and life safety evaluation.

12.4 AT&SF Bridge Modification

The AT&SF Bridge Modification plan is for removal of portions of the bridge and includes: 1) removing approximately 1,100 feet of wood trestle bridge on the left bank side of the Floodway from the new Santa Fe Trestle Trail bridge to the left bridge abutment at the East Levee, 2) removing a 660 foot concrete railroad bridge segment on the right bank side, and 3) removing two embankments on the right bank side of the Floodway. The plan is to cut-off the bridge piles just below ground level. During PED, bridge construction drawings and site specific geotechnical data will be reviewed to determine appropriate method to address any resulting seepage issues if the bridge piles have to be removed.

12.5 FRM Formulation Results

An economic analysis was conducted on the structural measures listed above. The NED Plan was determined to be the 277,000 cfs levee raise with 3H:1V side slopes including the AT&SF Bridge modification. The cutoff walls provided significant overall reduction in annualized loss of life analyzed independently for the internal erosion failure mode; however, based on further evaluation, the cut-off walls with levee overtopping did not reduce total risk and they were not economically feasible. The 277,000 cfs levee raise and AT&SF Bridge modification provides greatest net economic benefit and reduces life safety risks when compared to the other alternatives. Appendix C, Part II provides more discussion of the risk analysis results. The main feasibility report and Appendix E provide more discussion on the plan formulation and economic analysis results.

13.0 WRDA PROJECT ALTERNATIVES

The BVP is a project developed by the City of Dallas to utilize the area between the East and West Levee in the Dallas Floodway Levee System for the purposes of recreation and environmental restoration, while maintaining the primary purpose of flood control. The proposed plan consists of three lakes, the river relocation plans, wetlands, recreation fields, and various other hardscape and landscape features. More information on the Trinity River Relocation Project, the three lakes, and other features such as wetlands, utility relocations, recreation fields, etc. are provided in Appendix D – Civil.

The Water Resources and Development Act (WRDA) Plan requires the construction and implementation of its comprising features to be technically sound and environmentally acceptable. Environmentally acceptable is not part of the scope of this appendix. This section defines the geotechnical criteria of technically sound for this study. This definition is used throughout the report as the basis for the evaluation of the various features of the WRDA Plan.

Technically sound criteria, for geotechnical purposes, includes compliance with USACE criteria as provided in the USACE Engineer Regulations (ER), Engineer Manuals (EM), and Engineer Technical Letters (ETL). Also included are "*Risk Assessment* Trinity River Corridor Dallas Floodway near Dallas, Texas", 7 September 2012, "*Risk Assessment* of Proposed Remediation Methods, Trinity River Corridor Dallas Floodway", 2 November 2012, "*Study of the Impact on Risk* of the Proposed Balanced Vision Plan and Trinity Parkway, Trinity River Corridor Dallas Floodway", 26 June 2013, Fort Worth District Pamphlet (SWFP) 1150-2-1, and "*Preliminary Design Information, Guidelines, and Criteria, Geotechnical Design – City of Dallas Levees*", dated June 6, 2012 by HNTB. The memorandum was developed by the USACE and the City's contractor, HNTB.

Feasibility level seepage and stability analyses were performed in critical areas using Geostudio software by both USACE SWF and the City's Consultant, HNTB. Critical areas determined by the criteria are presented in Section 11, Geotechnical Parameters for FRM alternatives. Analyses were in compliance with USACE guidance in ERs, EMs, ETLs, as well as results of the BCRA and other risk assessments (RAs) performed. In addition, the memorandum titled, "Preliminary Design Information, Guidelines, and Criteria, Geotechnical Design – City of Dallas Levees", dated June 6, 2012 by HNTB was used as well. This memorandum was developed with the coordination and review of USACE Fort Worth District, and SWD DSPC Project Lead Engineer and Lead Geotechnical Engineer. With the adoption of the BCRA as criterion, the use of unsteady flow in both seepage and stability analyses resulted, in most cases, an increase in safety factors which met or exceeded USACE requirements for the critical cross-sections analyzed. With respect to the lakes and river meanders, both transient and steady-state analyses were performed. Critical cross-sections and locations were developed based on subsurface conditions from available geotechnical information (borings, CPT logs) across the entire levee system, topography, and location of the sump; specifically, the location of sand layers and potential for daylighting within a sump area, or heave of a thin clay layer overlying sand. In areas where the geotechnical information was limited, the PDT made interpretations between points of known subsurface conditions and in some cases, used material properties estimated from typical values for similar soils, and extended mitigation techniques to assure appropriate costs were included. The PDT recognized that the critical cross-sections may not necessarily reflect actual conditions across the entire levee system and that many BVP features are conceptual and the project will evolve as it moves to the PED phase. The PDT concluded that the critical cross-sections analyzed met criteria for both "existing" and "with project features", and are in agreement with the RAs performed. During the PED phase, site-specific geotechnical data and cross section will be developed as necessary to verify the feasibility level design and that deterministic criteria is met.

13.1 Balanced Vision Plan

The BVP consists of three lakes, the river relocation, wetlands, recreation fields, and various other hardscape and landscape features. Preliminary design and descriptions of the BVP features are provided in Appendix D – Civil and Structural Design. Lakes and river relocation are the main concern for the geotechnical evaluation.

In the event the review identified a technically sound criteria were not met, a risk based decision was made whether further feasibility level design was required or whether the design could be considered technically sound and the deficiency could remedied in future design phases.

13.1.1 River Relocation

The existing Trinity River channel will be relocated and reconfigured to a more meandering geometry within the floodway to improve the riverine ecosystem and provide room for construction of the proposed lakes, wetlands, and park features. The proposed river meanders are sinuous and move closer to the levees in multiple locations. The RMC performed a risk assessment to study the impact of the Proposed Balanced Vision Plan and Trinity Parkway. The purpose was to assess changes in risk to the existing Dallas Floodway Project by implementing the BVP and proposed Trinity Parkway. Consideration was given during the analysis for the BVP features to not increase risk from base condition as criteria and because there are currently no policy on tolerable risk guidelines for levees. BVP features are intended for recreation and environmental restoration purposes rather than risk reduction measures. The results of assessment of the River Meanders are summarized below. Appendix C, Part III presents the detailed information on the risk assessment for the BVP Lakes and River Relocation features.

Seepage pathways are shortened by the river relocation and the risk for heave (PFM 8) increases in the following locations: 1) West Levee, Station 3+00 to 29+00; 2) East Levee, Station 285+00 to 442+00; and 3) East Levee, Continental Avenue to Station 285+00. The city has completed construction of cut-off walls as a part of their 100 year certification effort in these locations except the section on the East Levee from Continental Avenue to Station 285+00. The existing cut-off walls the city has constructed on the East Levee at Station 285+00 will be extended downstream to approximately Continental Avenue (approx. Station 170+00) to mitigate for the increase in risk due to the river meander.

13.1.2 Lakes

There is some concern as to how close the three proposed lakes are to the levees. Encroachment to the toe of the levee by these features that have significant pore pressure during flood conditions could cause increased risk to the levees, such as seepage issues. In determination of the potential issues that could arise from seepage, it is estimated that a 150 foot buffer from the proposed levee toe should be sufficient to reduce the seepage failure mechanism. A clay liner 18-30 inches thick will be applied to the bottom of the lakes to help prevent seepage. Clay liner will only be provided in areas where basal sands and gravels are either exposed along the excavated side slopes or on the lake bottom. Detail clay liner design will be developed during the PED. Lakes will be evaluated in PED using steady state seepage analysis to ensure no additional cut-off walls are needed.

The lakes will be separated from the Trinity River by earthen berms to ensure proper separation from a hydraulic and geotechnical standpoint. Additional geotechnical analysis is necessary during PED to confirm the final design is adequate.

The proposed levee improvements (4:1 slopes) would change the levee toe. This design pushes out the existing levee toe towards the interior of the floodway, as described in Section 2.4 of Appendix D. This toe was offset 150 feet towards the interior of the levee and any overlap with proposed top of bank of river and lake features was identified. There are no significant issues that arose from this evaluation; however, there are some concerns regarding the depth to which the lakes are being excavated. The normal pool elevation will be 405 feet above mean sea level with a maximum depth of 15 feet. Further seepage analysis may need to be completed at this location to determine appropriate offset distances for the depth of this lake. At this stage of design, there is no requirement for cut-off walls. Additional geotechnical analysis at the lakes is necessary during PED to confirm there is no requirement for cut-off walls. If the footprint of the lakes changes, the cut-off wall option would have to be re-evaluated at that juncture.

In the existing plans and details of the BVP, the final depths of the proposed West Dallas Lake, Natural Lake, and Urban Lake are on the order of 10 to 15 feet. Based on available subsurface data, excavations for these lakes do not advance deep enough to penetrate the surficial clay layers that provide an aquatard between the basal sand lenses that typically overlie bedrock in the area of the Dallas Floodway and any free-surface floodwaters that move into the area. Except in the area of Oxbow Lake, clay thicknesses are typically maintained to a minimum of 10 feet. As it is currently designed, excavations for Oxbow Lake will penetrate through the clay cover and underlying basal sand layers and advance into the shale bedrock. This would provide a window through the clay aquatard for floodwaters to penetrate into the basal sands and potentially increase the piezometric pressure to a critical point under the levees and under the land-side toe of the levees. However, the City of Dallas has placed a soil-bentonite cutoff wall from Station 3+00 to 29+00 along the river-side levee toe of the west levee alignment. Due to the existence of the cutoff wall along the west alignment and a relatively thick landside clay blanket and high land-side ground surface on the east alignment, excavation for Oxbow Lake is not expected to impact the stability of the levees in this area. The RMC determined that placement of the proposed lakes detailed in the BVP will not impact the ability of the Dallas Floodway Project to reduce the risk of flooding and the PDT agrees with the RMC findings. Because the lakes will be in excess of 150 feet from the riverside toe of the levees, construction of the lakes will not increase the risk to the levees. Deterministic criteria will be confirmed during PED phase when designing BVP project features.

13.1.3 Hardscape and Landscape Features

Hardscape features of the BVP include access roads, paths, parking structures, promenades, hard-court recreation features, and amphitheaters. Currently, most of these features are not a geotechnical concern unless otherwise indicated. Primary vehicle paths and maintenance roads are used to access all features within the Dallas Floodway and are part of the overall flood-fighting and maintenance effort. These features would need to be designed to assure proper maintenance and access can be maintained for required flood-fighting and maintenance vehicles. An equestrian trail is located along the West Levee and pedestrian paths and bridges allow access between features and across the realigned Trinity River. The bridges are shown as crossing the Trinity River in several places. A foundation geotechnical report was not provided with the drawings. The foundations, as shown on the plans, do not appear to conform to USACE requirements for bridge foundations. A geotechnical report should be submitted along with the drawings for USACE review. Landscape features include the recreation fields, treed areas, wetlands and other parklands. The landscape and wetland features do not impact the existing levees. Additional geotechnical analysis is necessary during PED to confirm that the final design is technically sound.

13.1.4 Bridge Pier Modifications

The proposed River Relocation and Trinity Lakes portion of the BVP will have an impact on the existing bridge piers located within the limits of the relocated channel and lakes. Preliminary design and descriptions of the bridge pier modifications are provided in Appendix D – Civil and Structural Design Additional geotechnical analysis of the bridge pier modification is necessary during PED to confirm that the final design is technically sound.

13.1.5 Utility Adjustments and Relocations

There are major pressure storm sewers, various water lines and other utilities that are affected by the BVP. Four pressure storm sewers (Belleview, Dallas Branch, Woodall Rodgers, and Turtle Creek) pass underneath the East Levee and discharge into the Trinity River. With the realignment of the Trinity River and the construction of three lakes as part of the BVP, these storm sewers need to be rerouted and extended to accommodate their new outfall location. In addition, the BVP requires the relocation and modification of existing water mains crossing the Trinity River to accommodate the proposed lakes and Trinity River realignment. Further discussion on this feature of the Dallas Floodway occurs in Section 14.1.9. Levee crossings should preferably occur by going over a levee. Crossings beneath a levee will require more rigorous geotechnical analysis and approval of the construction methodology. All crossings over a levee shall have a minimum of two feet of cover. Thrust blocking systems are prohibited on the levee template so thrust restraint via mechanical means may be necessary. Crossings beneath a levee shall be by means of borings, tunneling, or horizontal directional drilling. Chapter 8 of EM 1110-2-1913 presents the criteria for these methods.

The "*Utility Adjustments and Relocations Design Report, Trinity Lakes Project*", dated September 2008 identified relocation of four underground water mains, removal of five miscellaneous pipelines, and relocation of 13 underground and/or aerial franchise utilities. The water main relocations will be designed and constructed by the city in advance of most other project improvements. The gas, electric, telecommunications, fiber optics, and jet fuel franchise utility relocations will be designed and constructed in advance of other improvements by each respective franchise utility company. Abandonment of utilities is the responsibility of the utility franchise and/or City of Dallas. Method of abandonment must be reviewed and approved by USACE. All proposed utility, pressure sewer and water mains crossing a levee will need to be coordinated with USACE during final design.

13.2 Interior Drainage Plan

The city's IDP contains improvements to existing and construction of new pumping stations (including the Able, Baker, Charlie, Delta, Hampton, Trinity Portland, and Pavaho pump stations), to restore sump capacity to provide protection against the 1% ACE (100-year event) from interior flooding. These features are defined in the reports prepared by the City of Dallas for the East Levee (Phase I) and the West Levee (Phase II). Technically sound criteria are the same as BVP and are provided in section 13.0 of this report.

13.2.1 East Levee IDP

The East Levee IDP includes three pump stations: Able, Baker, and Hampton. The pump stations are at various stages of design. Refer to Section 2.6 of Appendix D for more information on design and project description of the IDP features. At the time this report was prepared, the Section 408 submittal for the Able pump station was under review. Design of the Baker 3 pump station and sump modification is complete and the pump station is currently under construction. Hampton 3 pump station was reviewed at a 35% design level for technical soundness and appropriate review comments have been provided. Improvements to Nobles Branch sump do not have a geotechnical impact on the levee.

13.2.2 West Levee IDP

The West Levee IDP includes three pump stations on the West Levee: Charlie, Pavaho, and Delta, and one new pump station on the West Fork Levee (Trinity Portland). The West Levee pump stations are at various stages of design. Refer to Section 2.6 of Appendix D for more information on design and project description of the West Levee IDP features. The Pavaho pump station design was submitted as a separate Sect 408 project and is in the final stages of construction. The New Charlie Pump Station, New Trinity Portland Pump Station, and Rehabilitation of Delta Pump Station 35% designs have been reviewed by USACE for technical soundness and appropriate review comments have been provided.

13.2.3 BVP and IDP Review Findings

The BVP and IDP feasibility level design is technically sound and it is expected that any geotechnical issues with the current design can be remedied in PED and future design submittals. The IDP is in various stages of design and construction. Based on the analysis performed by HNTB there is only a slight increase in the exit gradient and seepage between the existing and "with BVP" conditions. Total head increased a little at the landside between existing and "with BVP" conditions. There was no difference in the slope stability factor of safety between the existing and "with BVP" conditions. The factor of safety is greater than 1.4. The seepage and stability analyses will need to be updated in future design to include the use of unsteady flow in lieu of the steady state analyses (if appropriate). Deterministic criteria will be confirmed during PED phase when designing BVP and IDP project features.

14.0 COMPREHENSIVE ANALYSIS

The comprehensive analysis phase of the current study looks at the Dallas Floodway Levee System from a system wide approach. It takes into account all projects going into the Levee System and evaluates them on a project by project basis and how they interact on an overall level. The purpose of the study is to determine potential conflicts in the integration of the multiple local features (Section 408 projects) and WRDA projects. The interaction is evaluated based on constructability, functionality, and risk. Conflicts will be resolved in further design phases and are identified in this stage of the study only for discussion purposes on the feasibility of the design. The alterations/modifications evaluated in the Comprehensive Analysis include the Trinity Parkway, Trinity River Standing Wave, the Santa Fe Trestle Trail, the Pavaho Wetlands, the Dallas Horseshoe Project, the Sylvan Avenue Bridge, Jefferson Bridge, Dallas Water Utilities (DWU) Waterlines, Continental Bridge, and the East Bank/West Bank Interceptor Line. Multiple projects (excluding the Trinity Parkway) listed here have received initial "approval" under Section 408. This section provides the results of the BVP/IDP with and without the Trinity Parkway first, followed by a status of evaluation for the other Section 408 projects proposed in the study area.

Review criteria for all Section 408 projects includes ERs, EMs, ETLs, "*Risk Assessment* Trinity River Corridor Dallas Floodway near Dallas, Texas", 7 September 2012, SWFP 1150-2-1, and "*Preliminary Design Information, Guidelines, and Criteria, Geotechnical Design – City of Dallas Levees*", dated June 6, 2012 by HNTB. With the adoption of the BCRA as criterion, the use of unsteady (transient) flow in both seepage and stability analyses also applies, if appropriate.

14.1 BVP WITH TRINITY PARKWAY

A Draft Geotechnical Engineering Report accompanied a Section 408 submission for the Trinity Parkway. The laboratory data in the report could not be considered acceptable as presented. A quality assurance and quality control assessment by the city's contractor may resolve the majority of the defects found in the laboratory data. The geotechnical report analyses, conclusions, and recommendations were not reviewed because of the questionable laboratory testing and reporting. A supplemental Geotechnical Report has been provided to address the outstanding issues with the data quality. A revised geotechnical report will be provided with the 65 percent design for the Section 408 submission at that stage. The revised geotechnical report will be reviewed and commented on by USACE.

Another Geotechnical Engineering Report "*Borrow Soil Suitability and Shrinkage Factor, Trinity Parkway, Dallas, Texas*", dated September 25, 2009, by Terracon Consultants, Inc. was prepared for the NTTA to investigate borrow sites suitable for levee construction and the roadway embankment. The report also presented guidance on the shrinkage factor used for volume estimates. The report did not provide specific fill quantities and material types required for the project construction, nor identify available volumes of the material types identified in the report or the expected utilization rates. The soil excavated from the lakes will be used for levee construction and the roadway embankment. It is important to determine the volume of suitable fill material available within the borrow sites. A Memorandum dated May 3, 2011 from Halff Associates clarifies the borrow sites suitable for levee construction and roadway embankment. Refer to Appendix D for further details on borrow sites and estimated quantities for BVP and Trinity Parkway construction.

A risk assessment was conducted on the placement of the Trinity Parkway roadway embankment along the East Levee. For the overtopping and subsequent breach failure mode, the placement may prevent the levee from fully breaching during an overtopping event on the East Levee (where the Trinity Parkway embankment is placed). Should PFM 2 occur in the enlarged embankment section from the Trinity Parkway, time required to completely erode would be much longer because the section is wider. This would reduce the depth of flooding and increase warning time for evacuation behind the East Levee. The risk from overtopping is not changed from the base condition for the West Levee, because the frequency of overtopping is not increased with implementation of those project features. For the heave failure mode, the risk assessment concluded that the risk is increased along the East Levee because the river meanders move the river closer to the levee thereby shortening the seepage pathways. The city has constructed cut-off walls in the locations of concern except the section on the East Levee from Continental Avenue to Station 285+00. The city's cut-off walls will be extended in this section to mitigate for the increase in risk due to the river meanders. Implementation of the river meanders and the Trinity Parkway, along with the extension of the cutoff wall on the East Levee alignment from 285+00 to 170+00 (Continental Avenue), reduces overall risk on the East Levee alignment compared to the base condition. Implementation of the river meanders and Trinity Parkway will not change the overall risk of PFM 8 on the West levee alignment from the base condition. On the East Levee, with the addition of the Trinity Parkway, risk for heave is further reduced because the section width on the East Levee from Continental Avenue to Station 285+00 is increased by the Trinity Parkway embankment.

14.2 BVP WITHOUT TRINITY PARKWAY

The evaluation results of the BVP "without" Trinity Parkway condition in the floodway is essentially the same as the "with" Trinity Parkway from a geotechnical standpoint. This is because the plans include placing some material along the East Levee in the Floodway for disposal purposes or for construction of the BVP features.

14.3 OTHER SECTION 408 PROJECTS

Other Section 408 projects evaluated for potential conflict and integration with the WRDA project include the Pavaho Wetlands, Trinity River Standing Wave, the Santa Fe Trestle Trail, the Dallas Horseshoe Project, the Sylvan Avenue Bridge, Jefferson Bridge, DWU Waterlines, Continental Bridge, and the East Bank/West Bank Interceptor Line. The following Section 408 projects were reviewed for Section 408 approval and appropriate review comments have been provided. The Section 408 approval takes into account the existing conditions of the floodway, whereas the review in the Comprehensive Analysis takes into account the integration with the future BVP and IDP projects.

- Pavaho Wetlands
- Trinity River Standing Wave
- Santa Fe Trestle Trail
- Horseshoe
- Sylvan Bridge
- Jefferson Bridge
- Continental Pedestrian Bridge
- DWU Waterlines
- East Bank/West Bank Interceptor

The Pavaho Wetland consists of three wetland habitat cells created on the riverside of the West Levee. The Standing Wave is located adjacent to Moore Park and is located downstream of the Corinth Street Viaduct, adjacent to the DART Rail Bridge and Santa Fe Railroad Trestle. The Standing Wave project includes an in-steam standing wave for recreation use. The Santa Fe Trestle Trail is a hike and bike trail providing access to Moore Park south of Downtown Dallas. The Pavaho Wetlands, Standing Wave and Santa Fe Trestle Trail have been approved under Section 408 and are now constructed.

The Dallas Horseshoe, Sylvan Bridge, Jefferson Bridge, and Continental Pedestrian Bridge are transportation projects that intersect the Dallas Floodway Levee System. The Dallas Horseshoe Project will reconstruct the existing IH 30 and IH 35E bridges across the Dallas Floodway. The Sylvan Bridge replaces the existing Sylvan Bridge approaches and low water crossing over the Trinity River with a single bridge structure that will span the Dallas Floodway. The Jefferson Bridge proposal would replace the existing Jefferson Street Bridge and provide a direct connection to and from IH-35. The existing Continental Avenue Bridge would be converted from vehicle to pedestrian and bicycle use. The Dallas Horseshoe, Sylvan Bridge and Continental Pedestrian Bridge have been approved as minor, low impact 408 projects. At this time, plans and reports have not been provided for the Jefferson Bridge project.

Some utility projects are proposed to cross the Dallas Floodway. The Dallas Water Utilities plans to implement projects that will involve a force main replacement, a 60-inch re-use waterline at the Central Wastewater Treatment Plant, and four utility crossings of the Trinity River:

- Crossing at Inwood (36-inch)
- Crossing at Corinth (upgrade from 24 to 48-inch)
- Crossing at Mockingbird (48-inch)
- Crossing at Houston (24-inch)

The proposed installation calls for open-cut and/or auguring techniques. These techniques do not comply with EM 1110-2-1913. The project is on hold pending resolution of review comments.

There is also the East Bank/West Bank interceptor line project that consists of two tunnels that cross the Dallas Floodway. The first tunnel was submitted and approved under 408 and is under construction. The second tunnel has been submitted for USACE review. Approval of the second tunnel is contingent upon completion of remediation efforts by the city on the first tunnel. A 650-foot section of the tunnel under the East Levee was filled with low strength grout through holes drilled from the ground surface into the tunnel. Re-mining of the grout and placing of liner plate and grout is in progress.

14.4 Comprehensive Analysis Review Findings

During comprehensive analysis all BVP and IDP features were evaluated for technical soundness. The projects are in various stages, from design submittal to construction completion. Because of the variances in design stage it is difficult to evaluate how each feature relates to the remainder of the Floodway. Compliance can be obtained by applying USACE criteria in future design phases for those projects that are not in full compliance with the USACE criteria for construction in the floodway. Geotechnical design criteria were deliberated at the time several of the projects were under design. Seepage and stability analyses will need to be updated in future design to include the use of unsteady flow in lieu of the steady state analyses (if appropriate). Deterministic criteria will also be confirmed during future design of project features.

15.0 TENTATIVELY SELECTED PLAN – OVERALL PROJECT

All BVP and IDP features have been determined to be technically sound at the feasibility level of design development and it has been determined that with some modifications during detailed design, they would all function on a comprehensive system-wide level. The features implemented under WRDA 2007 will be a subset of the city's overall BVP/IDP plans.

The WRDA Project - Tentatively Selected Plan (TSP) includes the FRM - TSP (277K levee raise with AT&SF Bridge modifications), the IDP Phase I (Able, Hampton, and Baker), the proposed river relocations, and the Corinth Wetlands. All these features combined are referred to as the Overall Project TSP. Desiccation cracking in the levee system was not considered to be high risk based on the BCRA results. The desiccation cracking and the number of slope failures has led to increased operation and maintenance cost. This feature will be pursued as a betterment at 100% non-federal cost.

16.0 RECOMMENDED PLAN

The Overall Project TSP is the Recommended Plan for Dallas Floodway. It assumes the WRDA project is implemented with the preferred Trinity Parkway Alternative 3C (East Levee alignment).

16.1 Construction Phasing

Construction phasing will be incorporated into the various elements of the BVP and WRDA project during the detailed design phase. See Appendix D – Civil for a general description of recommended construction sequencing.

16.2 Operation, Maintenance, Repair, Replacement and Rehabilitation

The non-Federal sponsor is responsible for the Operations, Maintenance, Repair, Replacement, and Rehabilitation (OMRR&R) of the complete project. The district will update the existing Dallas Floodway Operation and Maintenance Plan dated May 1960 upon successful completion of the project. A comprehensive operation and maintenance manual will need to be created for the entire Floodway. Maintenance will be required throughout project construction.

The City of Dallas currently maintains the levees and fixes slides. With a flattening of the levee side slope, it is expected that this need will decrease; however, it is an important operation and maintenance function. The berms separating lakes and the Trinity River will need to be periodically inspected for erosion or other flaws. Critical elements of the project would require OMRR&R and would need to be fixed immediately as they directly impact the functionality of the Dallas Floodway Levee System. Refer to Appendix D for a description of these critical elements of the project.

16.3 Total Risk of Recommended Plan

The risk assessment performed has determined that total risk for the levee system with the Recommended Plan in place has resulted in a reduction in probability of the failure mode from base condition for the East and West Levees. The combined baseline risk of PFM 2, 7 and 8 for the East and West Levee for the Recommended Plan is located above the recommended risk guideline, because the risk is dominated by the higher risk of PFM 2.

Probability and consequences are reduced from baseline conditions with the Recommended Plan on the East Levee. There's a reduction in probability and a slight increase in consequences for the West Levee. The reduction in probability of overtopping offsets the slight increase in consequences and reduces the overall risk for the West Levee raises. With the Trinity Parkway, probability and consequences of the combined risk is reduced further for the East Levee, but not below the recommended risk guideline. The risk assessment identified no affect on the failure modes on the West Levee as a result of the Trinity Parkway.

16.4 Future Studies

Due to the large number of proposed BVP and IDP projects it is recommended that future studies be conducted as designs of the various features continue to progress. The recommended future studies include potential for internal erosion and heave failure modes at critical sections along East and West Levee, type and placement of material around bridge members to reduce risk of settlement, extent of cut-off walls to prevent internal erosion failure, minimum distance between the lakes and the Trinity River, seepage issues if the AT&SF bridge piles have to be removed, site specific geotechnical data and cross-sections for BVP and IDP features to ensure that deterministic criteria is met and clay liner design for the lakes to ensure no additional cut-off walls are needed.

17.0 PERIODIC INSPECTION NO. 9 CLOSEOUT

The intent of this section is to assess the 21 Periodic Inspection (PI) No. 9 items deferred to feasibility in the context of the risk assessment and plan formulation results. All remaining PI No. 9 inspection items are individually addressed and a case is made whether: (1) the items should be cleared from the list with no further action, (2) a change in rating in future inspections was warranted, (3) it contributed to a Probable Failure Mode (PFM) and should be carried forward for potential inclusion in plan formulation for corrective action, or 4) it remain with the City of Dallas as OMRR&R.

The PI No. 9 inspection checklist provides ratings for flood damage reduction systems as a whole, as segments within the system, or as individual features (items). An unacceptable rating (U) is given to an item if one or more serious deficiencies exist that need to be corrected. A minimally acceptable rating (M) is given if one or more minor deficiencies exist. Unacceptable items which would prevent the levee system from performing as intended need to be corrected within a two-year period. Other items are noted while on the inspection and are given an observed (Obs) rating, which is not part of the inspection checklist and no immediate action is required, but exception or permit might be determined necessary.

The PI No. 9 remaining 21 items were contributing factors considered in the risk assessment. Some of these items were given U or M ratings and determined to have tolerable risk; however, if these items remain unaddressed, some level of risk remains, and the item(s) will remain unacceptable in future inspections. The system as a whole could get an unacceptable rating and become inactive in USACE Rehabilitation and Inspection Program under P.L. 84-99, unless the rating can be changed or the problem is corrected.

To address this issue, an assessment of whether the items could be documented by exception or permit, changed to a different rating on the inspection checklist, or in need of future evaluation in this study or by the sponsor as summarized below. Table 16-1 presents the conclusions of the assessment.

The following PI No. 9 items (rated U) are addressed by PFM #1 and/or #10: 005 Bridge crossings with piers in embankment; 133 Walton Walker Bridge pier in the levee; 006 Electric Power tower 20+00 to 81+00 East Levee; 126 Power line tower 18+90 East Levee; 127 Power line poles 171+40 East Levee; 128 Power poles 267+95 West Levee; 129 Pole 500+00 West Levee; 230 Electric power tower, 237+50, 320+, 364+, 612+; and 231 Power poles 515+70. Based on the results of the failure modes analysis, utility penetrations and bridge penetrations in the embankment were not seen as credible failure modes. Concerns raised by lack of information about design, construction, and maintenance of these encroachments raised questions that needed to be addressed. The District subsequently has reviewed existing boring data and prior 408 approvals (if they existed) for the encroachments. Based on the review, ratings will be revised to M or A during the next annual inspection.

PI No. 9 item 46 and 152 (rated M) noted that drainage ditch adjacent to the land side levee toe and in the sump areas would obscure seepage that develops during a flood event. Evidence of internal erosion could be masked by the inability to see the ditch areas. This was identified as an issue in failure mode (PFM #7), which was carried through the risk assessment. The results of the risk assessment showed that this failure mode, while borderline on the tolerable risk guidelines, warranted further analysis and was carried forward into the feasibility study. Based on analysis, the District will keep the rating as M or change to an A during the next annual inspection.

PI No. 9 item 008 (rated U) is noted as an encroachment by the construction of a jail on the East Levee. This is not specifically addressed by any of the PFMs. The sponsor has subsequently provided as-builts and existing geotechnical information on the jail construction. Based on analysis of the new data, the District will revise the jail rating to M or A during the next annual inspection.

PI No. 9 item 077 (rated U) noted the AT&SF Railroad Bridge is an obstruction to flow in the channel. Obstruction of flow leads to higher water surfaces in the floodway which could contribute to overtopping of the levee in the event of high flows. This failure mode was carried forward for risk assessment. Overtopping of the levee was determined to have unacceptable risks in PFM #2. Removal or modification of the bridge will be evaluated in this study. The district has evaluated the AT&SF bridge and the bridge modification is part of the Recommended Plan. The AT&SF bridge will stay a U in accordance with the policy in future inspections, but will be classified as a non-system determining U. The U rating cannot be changed until the items are fixed.

PI No. 9 item 34 and 145 (rated U) were noted levee height deficiencies of the East and West Levees based on the 2003 crest survey and the 1950s design elevation. A decrease in levee height could lead to overtopping and breach. This deficiency should be corrected at a minimum by raising or otherwise restoring the levee to the design grade. USACE will evaluate whether or not this could be considered reconstruction during the feasibility study. The district has evaluated the levee raises and the levee raises are part of the Recommended Plan. The levee height deficiency will stay a U in accordance with the policy in future inspections, but will be classified as a non-system determining U. The U rating cannot be changed until the items are fixed.

PI No. 9 item 038 and 148 (rated U) noted pervasive desiccation cracking on the East and West Levee. The extent, length, and depth of these cracks place the levees in an unacceptable category. The levee material is prone to desiccation and cracking. The cracking has led to numerous slides in the levee which if left unattended could lead to a breach of the levee. Repair of the slides is a continuing O&M issue. The concern of the District is this condition will only worsen over time. Correction of the cracking is possible by lime treatment, covering the slopes with soils that are not subject to desiccation, or flattening the slopes which reduces the intensity and depth of the slides. Desiccation cracking was considered under PFM #5 and 13. These failure modes were not considered to be credible. This PI No. 9 item was not considered to be high risk based on the risk assessment. Flood risk management measures considered during the Feasibility Study may reduce some of these issues. The city plans to implement 4H:1V side slopes at 100% non-federal cost. The 4H:1V side slope help reduce the number and frequency of slides, thus becoming less of an O&M issue for the city. The desiccation cracking will stay a U in accordance with the policy in future inspections, but will be classified as a non-system determining U. The U rating cannot be changed until the items are fixed.

Detailed information should be presented to the District to allow evaluation of observed items and determine whether allowance or permit is necessary. The following PI No. 9 items were rated “Obs” during the inspection:

085 Sewer line at East Levee station 503+50

137 Sewer line at West Levee station 503+50

086 Sludge Lagoon in the floodplain new East Levee station 621+00

087 Sylvan Avenue Lake Development in the floodplain

Table 16-1 presents a summary of the assessment of each PI No. 9 item described in this section. As indicated in Table 16-1, the PI No. 9 items that require additional evaluation for risk reduction include the AT&SF Railroad Bridge flow obstruction, the levee height deficiencies and the cracking caused by desiccation. In addition, internal erosion and heave were evaluated further in this study.

Table 16-1. Assessment of Periodic Inspection Report No. 9 Items Considered in the BCRA

PI No. 9 Item(Rating)	*Exception/Permit & Change Rating*	*Further Evaluation for Risk Reduction*
005 (U) Bridge Encroachment	X	
133 (U) Bridge Pier in Levee	X	
006 (U) Electric Tower Encroachment	X	
126 (U) Power Line Encroachment	X	
127 (U) Power Line Encroachment	X	
128 (U) Power Pole Encroachment	X	
129 (U) Power Pole Encroachment	X	

PI No. 9 Item(Rating)	*Exception/Permit & Change Rating*	*Further Evaluation for Risk Reduction*
230 (U) Electric Tower Encroachment	X	
231 (U) Power Poles Encroachment	X	
046 (M) Drainage Ditch Obscure Observation	X	
152 (M) Drainage Ditch Obscure Observation	X	
008 (U) Jail	X	
077 (U) AT&SF		X
034 (U) Levee Height		X
145 (U) Levee Height		X
038(U) Desiccation Cracking		X
148 (U) Desiccation Cracking		X
085 (Obs) Sewer Line	X	
137 (Obs) Sewer Line	X	
086 (Obs) Sludge Lagoon	X	
087(Obs) Sylvan Avenue Lake	X	

There were 21 items deferred to the feasibility study. The PI No. 9 rated Item 3, Encroachments, ID 234 (Levee crest excavated at the Houston Street Bridge [Houston Street Viaduct] crossing) as a deficiency to be addressed in the MDCP. It has been determined now to be appropriate to address the item in the feasibility study.

The City has included the Houston Street Viaduct as a location where sandbags are required during flood events in their Emergency Action Plan. During flood operations and potential evacuations, Houston Street Viaduct would not be accessible as an evacuation route. It will be important to block both sides of the bridge via traffic barriers and proper traffic control/notification devices during flood stages approaching the 277K cfs flood event. The City should conduct exercises annually on procedures to sandbag the bridge.

18.0 REFERENCES

Ajemian, G., Furlong, J., McPherson, T. 2003. History of the Dallas Floodway. September.

AR Consultants. 2006. Interim Report, Archaeological Testing for the Trinity Parkway. 11 April.

Bureau of Economic Geology. 1967. Geologic Atlas of Texas, Sherman Sheet, 1:250,000 scale. Bureau of Economic Geology. University of Texas, Austin, Texas.

Bureau of Economic Geology. 1972. Geologic Atlas of Texas, Dallas Sheet, 1:250,000 scale. Bureau of Economic Geology. University of Texas, Austin, Texas.

Bureau of Economic Geology. 2010. Eagle Ford Bibliography. Bureau of Economic Geology. University of Texas, Austin, Texas.

Cedergren, H. R. 1989. Seepage, Drainage, and Flownets, 3rd Edition. Published by John Wiley & Sons

Charvat, W. A. 1985. The Nature and Origin of the Bentonite Rich Eagle Ford Rocks, Central Texas: M.S. Thesis, Baylor University, 136.

Dallas Geological Society. 1965. The Geology of Dallas County. Dallas Geological Society, p. 211.

Dunlop, J. C. 1965. Engineering Geology of Dallas County. The Geology of Dallas County, Dallas Geological Society, p. 206 – 211.

Eubank, L. A. 1965. Physiography of Dallas County. The Geology of Dallas County, Dallas Geological Society, p. 14 – 39.

Foster, P. W. 1965. Subsurface Geology of Dallas County. The Geology of Dallas County, Dallas Geological Society, p. 126 – 189.

Hendrix, L. 1972. Geology of Midcities Area, Tarrant, Dallas, and Denton Counties, Texas. Scale 1:62.500, Geologic Quadrangle Map No. 42, Bureau of Economic Geology. University of Texas, Austin, Texas.

Hill, R.T. 1889. A Preliminary Annotated Check List of the Createaceous Invertebrate Fossils of Texas, Accompanied by a Short Description of the Lithology and Stratigraphy of the System," United States Geological Survey, p 6-31.

Hunt, R.E. 1986. Geotechnical Engineering Techniques and Practices, published by McGraw-Hill.

Moreman, W. L. 1925. Micrology of the Woodbine, Eagle Ford, and Austin Chalk. University of Texas Bulletin. Texas University, Texas.. 2544: 74-78.

McKenzie, M. 2009. Comparison of Three Contemporaneous Decapod Communities From the Upper Cenomanian Briton Formation (Late Createaceous) of North Texas, unpublished paper, 48 p., email communication with the author. 12 April 2011.

Norton, G. H. 1965. Surface Geology of Dallas County. The Geology of Dallas County, Dallas Geological Society, p. 40 – 125.

Ralston, W. 1965. Regional Geologic Setting for Dallas County. The Geology of Dallas County, Dallas Geological Society, p. 11 – 13.

Sampson, H. H. Jr. 1972. Geology of Midcities Area, Tarrant, Dallas, and Denton Counties, Texas, Report to Accompany Geologic Quadrangle Map No. 42, Bureau of Economic Geology, University of Texas, Austin, Texas.

Shuler, E. W. 1918. Geology of Dallas County. University of Texas Bulletin No. 1818, University of Texas, Austin, Texas.

Slaughter, B. H., Crook, W. W., Harris, R. K., Allen, D. C., and Seifert, M. 1962. The Hill Shuler Local Faunas of the Upper Trinity River, Dallas and Denton Counties, Texas, Report of Investigations No. 48, Bureau of Economic Geology, University of Texas, Austin, Texas, p. 75.

The Dallas Geological Society. 1965. The Geology of Dallas County. December.

USACE, Fort Worth. 1952. Definite Project Report, Dallas Floodway. September.

USACE, Fort Worth. 1953. Seepage Investigations of West Levee, Dallas Floodway. September

USACE, Fort Worth. 1968. Review of Levee Design, Dallas Floodway. June.

U.S. Department of Agriculture. 1980. Soil Survey of Dallas County, Texas, U.S. Department of Agriculture, Washington, DC.

33 CFR 208.10. Maintenance & Operation of Local Protection Works.

EXHIBIT 1

SWF Master MDCP List

System Study

ID	*Item #*	*Rated Item*	*Overall Rating*	*Levee/ Structure*	*Template Section*	*Remark Rating*	*Location/ Remarks/ Recommendations*	*5141*	*DFE*	*Explanation*
005	3.	Encroachments	**U**	Floodway	Embankment	**U**	Bridges crossing over the floodway, many with piers in the levee crest or slopes.	☐	☐	Permitted encroachments include: MHH (Phase 1), Hampton Rd, TRE, DART, Westmoreland, Turtle Ck Intake Bridge. All others are considered Unauthorized Encroachments as no documentation of District Engineer review or authorization has been located. See Definitions for Deficiency Spreadsheet.
006	3.	Encroachments	**U**	East Levee	Embankment	**U**	Sta. 20+00 to 81+00 have electric power towers on the landside levee lower slopes between DART & I-35 bridges.	☐	☐	Unauthorized Encroachment. No documentation of District Engineer review or authorization.
008	3.	Encroachments	**U**	East Levee	Embankment	**U**	Sta. 147+40 an unauthorized encroachment for construction of a jail annex that includes a basement adjacent to the landside levee toe that damaged the levee and removed material from the foundation adjacent to the levee toe.	☐	☐	Unauthorized Encroachment. Investigation and analysis needed to determine appropriate remedial action. City to submit engineering documentation for jail construction.
034	7.	Settlement	**U**	East Levee	Embankment	**U**	Crest survey in 2003 indicates levee crest was below the authorized original design elevation.	☐	☐	Owner should maintain levee crest elevation to authorized project design elevation. Appendix H of the PI #9 Report is being amended to reflect original project levee crest design elevations. Study to determine justifiable levee raise. O&M levee raise is 100% non-Federal cost.
038	9.	Cracking	**U**	East Levee	Embankment	**U**	Cracking due to desiccation is a known climatic condition. Extensive cracking occurs seasonally and over extended dry periods. The usual condition of the levees was to be riddled with desiccation cracks. Some cracks were measured in September of 2008 and found to be up to 4 feet in depth.	☐	☐	Material and slopes will be re-evaluated as a part of the Feasibility Study.
046	15.	Seepage	**M**	East Levee	Embankment	**M**	Sta. 473+90 - Drainage ditch adjacent to the landside levee toe and in the sump areas was the standard project condition. This would obscure seepage that develops during a flood event.	☐	☐	Project component should be taken into consideration during evaluation of design alternatives.
077	1.	Vegetation and Obstructions	**U**	East Levee	Flood Damage Reduction Channel	**U**	The abandoned old Santa Fe Railroad Bridge obstructs flow and catches debris.	☐	☐	
085	3.	Encroachments	**M**	East Levee	Flood Damage Reduction Channel	**Obs**	West Levee Sta. 503+50 had new sewer manhole in the floodplain.	☐	☐	Project component should be taken into consideration during evaluation of design alternatives.

EXHIBIT 1

SWF Master MDCP List

System Study										
ID	*Item #*	*Rated Item*	*Overall Rating*	*Levee/ Structure*	*Template Section*	*Remark Rating*	*Location/ Remarks/ Recommendations*	*5141*	*DFE*	*Explanation*
086	3.	Encroachments	**M**	East Levee	Flood Damage Reduction Channel	**Obs**	Sludge lagoon in flood plain near East levee Sta. 621+00.	☐	☐	Project component should be taken into consideration during evaluation of design alternatives.
087	3.	Encroachments	**M**	East Levee	Flood Damage Reduction Channel	**Obs**	Sylvan Ave Lake Development in flood plain.	☐	☐	Project component should be taken into consideration during evaluation of design alternatives.
126	3.	Encroachments	**U**	West Levee	Embankment	**U**	Sta. 18+90 had power line tower on landside levee toe.	☐	☐	Unauthorized Encroachments. City to provide permit request with documentation.
127	3.	Encroachments	**U**	West Levee	Embankment	**U**	Power line poles on landside levee slopes and toe, and cabled pole vehicle barriers on levee slopes upstream of Sta. 171+40.	☐	☐	Unauthorized Encroachments. City to provide permit request with documentation. Vehicle Barrier will be covered under MDCP.
128	3.	Encroachments	**U**	West Levee	Embankment	**U**	Power poles on landside levee toe downstream of Sta. 267+95.	☐	☐	Unauthorized Encroachments. City to provide permit request with documentation.
129	3.	Encroachments	**U**	West Levee	Embankment	**U**	Sta. 500+00 had pole on riverside levee slope.	☐	☐	Unauthorized Encroachments. City to provide permit request with documentation.
133	3.	Encroachments	**U**	West Levee	Embankment	**U**	Walton Walker Bridge Sta. 475+65 had buried pier support in levee.	☐	☐	Unauthorized Encroachment. No documentation of District Engineer review or authorization.
137	3.	Encroachments	**U**	West Levee	Embankment	**Obs**	Sta. 503+50 had a sewer line repair (cross section at the line crossing larger than typical levee section) (Observed).	☐	☐	Project component should be taken into consideration during evaluation of design alternatives.
145	7.	Settlement	**U**	West Levee	Embankment	**U**	Levee crown was below design elevations at 54+14(-1.17), 70+00(-0.66), 160+00 to 315+00 (max. of -2.61), 340+00 (-0.33), and 380+00 to 570+00 (max. of -1.21) per 2003 survey.	☐	☐	Owner should maintain levee crest elevation to authorized project design elevation. Appendix H of the PI #9 Report is being amended to reflect original project levee crest design elevations. Study to determine justifiable levee raise. O&M levee raise is 100% non-Federal cost.
148	9.	Cracking	**U**	West Levee	Embankment	**U**	Cracking due to desiccation is a known climatic condition. Extensive cracking occurs seasonally and over extended dry periods. The usual condition of the levees was to be riddled with desiccation cracks. Some cracks were measured in September of 2008 and found to be up to 4 feet in depth.	☐	☐	Material and slopes will be re-evaluated as a part of the Feasibility Study.
152	15.	Seepage	**M**	West Levee	Embankment	**M**	Drainage ditch adjacent to the landside levee toe and in the sump areas was the standard project condition. This would obscure seepage that develops during a flood event.	☐	☐	Project component should be taken into consideration during evaluation of design alternatives.

EXHIBIT 1

SWF Master MDCP List

System Study										
ID	*Item #*	*Rated Item*	*Overall Rating*	*Levee/ Structure*	*Template Section*	*Remark Rating*	*Location/ Remarks/ Recommendations*	*5141*	*DFE*	*Explanation*
230	3.	Encroachments	**U**	East Levee	Embankment	**U**	Sta. 237+50, 320+, 364+, and 612+ had electric power towers/poles on the landside toe.	☐	☐	Apparent Permitted Encroachment. A DP&L Power line was approved 24 JUL 1981. Verification needed.
231	3.	Encroachments	**U**	East Levee	Embankment	**U**	Sta. 515+70 had electric power poles on the riverside levee toe.	☐	☐	Unauthorized Encroachments. City to provide permit request with documentation.
21								21	0	20

EXHIBIT 2

DALLAS FLOODWAY
WEST FORK AND ELM FORK, TRINITY RIVER, TEXAS
PERIODIC INSPECTION NO. 9
Final Ratings for Levee Embankment Systems, June 2007

Levee System	*Appox Length (ft)*	*Unwanted Vegetation*	*Sod Cover*	*Encroach-ments*	*Closure Structure*	*Slope Stability*	*Erosion*	*Settlement*	*Depression/ Rutting*	*Cracking*	*Animal Control*	*Culverts/ Pipes*	*RipRap Revetment*	*Other Revetment*	*Well/ Drainage*	*Seepage*	*System*
East Levee System		U	M	U	U	M	U	U	M	U	A	M	M	A	N/A	M	
West Levee System		U	M	U	N/A	M	M	U	M	U	A	N/A	M	N/A	N/A	M	

USACE Inspection Rated Items	**Description of Rated Items**
1. Unwanted Vegetation Growth	Unwanted vegetation includes overgrown grass and weeds that limit or prohibit proper inspection. This also includes woody growth along the system that may negatively impact the integrity of the system. Establishment of a 15 foot Vegetation Free Zone (VFZ) is required as defined in ETL 1110-2-571.
2. Sod Cover	Grass or sod cover is one of the most effective and economical means of protecting flood control levees and drainage swales against erosion caused by rain runoff, channel flows, and wave wash. Failure to properly maintain the grass cover can result in unnecessary erosion and possible embankment failure.
3. Encroachments	Encroachments include obstructions or inappropriate activities being conducted within the system's ROW and easement. Lack of appropriate easement to minimize impacts of adjacent activities on performance of the system, will also be considered. Encroachments shall reviewed by USACE in accordance with 33 USC § 408 and 33 CFR § 208.10 to determine the effect on the system.
4. Closure Structures (Stop Log, ECS)	Closure structures should be in proper condition with all required materials and equipment readily available. Installation instructions should be available and trial closures shall be conducted per the requirements of the O&M Manual. Records should be provided for the inspection.
5. Slope Stability	The stability of the levee embankment is critical with respect to the systems integrity during a flood event. Steep levee slopes are difficult to maintain and are susceptible to sloughs and slides.
6. Erosion/ Bank Caving	Erosion of Levee Embankments, Interior Drainage Features, Structures, and Channels should be monitored. Revetments and other improvements shall be made as necessary.
7. Settlement	The settlement of the system should be measured using a topographic crest survey, with datum per the requirements of EC 1110-2-6065.
8. Depressions/ Rutting	Ruts and depressions allow water to pond on the levee embankment, which can lead to seepage and stability problems for the system.
9. Cracking	Cracking due to desiccation and differential settlement should be kept minimal with no vertical movement.
10. Animal Control	Burrows created by animals (and insects) can lead to rapid levee failures during floods. For this reason, an active abatement program needs to be implemented to remove these rodents (and pests).
11. Culverts/ Discharge Pipes	All pipes and culverts within the levee template shall be inspected on a periodic basis to establish the condition of the utility. Reports of these inspections shall be made available to USACE for review.
12. Riprap Revetments/Bank Protection	Riprap revetments should be in proper condition with minimal displacement, degradation, or unwanted vegetation.
13. Revetments other than Riprap	Other revetments, such as blankets and blocks, should be in proper condition with minimal displacement, degradation, or unwanted vegetation.
14. Underseepage Relief Wells/ Toe Drainage Systems	Relief wells and toe drains are used to relieve hydrostatic pressures in the foundation of a levee, caused by fluctuation in the water table or seepage under a levee or flood control structure during a flood. Maintenance of these features should be conducted per the requirements of the O&M Manual and records should be provided for the inspection.
15. Seepage	Seepage problems are critical with respect to the system's integrity during a flood event. Continuously saturated soils (not caused by ponded water or poor drainage) are an indication of seepage areas of concern.