Preliminary Technical Memorandum

Improvement Concepts for Salt Flats Levee

Salt Flats Levee System, Phase 2, Task Order 4

City Project Number: **E12070**
HDR Project Number: **10026049**

City Funding Source: **Type A Corporation, Salt Flats Levee Dedicated Sales Tax**

PRELIMINARY

THIS DOCUMENT IS RELEASED FOR THE PURPOSE OF INTERIM REVIEW AND IS NOT INTENDED TO BE USED FOR CONSTRUCTION, BIDDING OR PERMIT PURPOSES.

ENGINEER: Rolland G. Boehm, PE

REGISTRATION NO.: TX 101598

DATE: January 27, 2017

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

APPENDICES

EXECUTIVE SUMMARY

The City retained HDR Engineering (HDR) to further evaluate priority vulnerabilities previously identified along the Salt Flats Levee, and advance repair and/or modifications concepts as appropriate, including developing a planning-level cost estimate. Much of the effort focused on an existing 130-foot long concrete crib wall adjacent to West Broadway Street and a 350-foot segment of earthen levee south of Brewster Street. Other ancillary items include replacement of flap gates, minor repair/improvements of closure structures, and the removal of several trees on or near the earthen levee.

Based on this assessment, the existing crib wall is recommended to be replaced with an inverted concrete T-Wall. The base slab of the T-Wall would be founded on existing embankment material and the stem portion would structurally tie into the adjacent concrete storage bunker to the south and concrete wingwall to the north. As conceptualized, the wall would extend approximately 6.5 feet above the supporting base slab to coincide with a top elevation of +13 feet (NAVD88). Compacted cohesive material would be placed along the protected side to enhance stability of the T-Wall during a significant flood.

Geotechnical stability analyses were performed on the earthen levee within the study area. The soil data were obtained from previous geotechnical investigations supplemented with a recently-completed effort conducted by the City as part of this project. The results from the analyses indicate the levee is less vulnerable to a stability failure than was previously suggested by a cursory-level assessment that was based on more limited geotechnical and hydraulic information. Therefore, no recommendations for improving the earthen levee (within the study area) are being made at this time. This recommendation should be re-evaluated if improvements are made to the interior (downtown) drainage system and pump stations that would result in a greater head differential at the levee during floods.

A total of nine flap gates are currently attached to the discharge end of storm drains that penetrate the levee and empty into the Salt Flats Channel. The purpose of these gates is to prevent reverse flow from the channel, in particular during a high water condition within the channel. Eight of the gates are in marginal condition and should be replaced as part of routine maintenance. It is recommended these eight flap gates be replaced with duckbill-type check valves, which are generally considered more reliable.

Four closure structures exist at rail crossings through the levee. Recommended improvements to the closure structures include 1) repair stop log cover plates, 2) repair concrete support at closure structure near E. Port Avenue, and 3) fabricate and install upstream neoprene plugs at railroad crossings associated with two of the closure structures. In addition, several trees were noted along the levee that could impact the structural integrity of the earthen levee. The trees and their root mass should be removed and the resulting cavity filled with compacted clay.

The overall Opinion of Probable Project Costs (OPPC) associated with the items described above is $1,097,000. This cost should continue to be refined as the improvement concepts progress through final design.

A new vulnerability was discovered during completion of this Project. The existing concrete headwall south of Brewster Street (which supports two pipe penetrations) has a stabilizing effect on the levee. Therefore the internal and external stability of the headwall is important to the overall stability of the levee. The internal and external stability of the headwall should be evaluated for up to a 100-year flood condition. In addition, there are a number of known and possibly unknown pipe penetrations through the levee, including pipelines that have likely been abandoned. These penetrations should be investigated in more detail to determine if they could create a destabilizing effect on the levee, especially during a prolonged flood event. An allowance for analysis (but not construction) of these additional items is included in the OPPC listed above.

1. INTRODUCTION

1.1 AUTHORIZATION

The work outlined in this study was authorized by representatives of the City of Corpus Christi, Texas (City). The work was completed in accordance with HDR Engineering, Inc. (HDR) Task Order No. 4 and authorized by Mr. Jeff Edmonds, PE on May 3, 2016, with Mr. Daniel Deng, P.E., serving as the City's Project Manager. This work is part of the overall assessment for the Salt Flats Levee System, City Project Number E12070.

1.2 PURPOSE & SCOPE

The Salt Flats Levee (SFL) was constructed in 1956 and comprises the northwestern-most component of the Corpus Christi downtown flood protection system (Figure 1). For the most part, the levee is immediately adjacent to the Salt Flats Channel, and ties into high natural ground at its south end (Station 0+00) and the Port Authority wharves system at its north end (Station 35+68). The Salt Flats Channel is lined with concrete and discharges into the Harbor Ship Channel; this channel serves as a significant drainage feature for the downtown area. The vast majority of the approximate 3600-foot long levee consists of an earth embankment, though other important features include four temporary closure structures, storm water drainage pipe penetrations, and a 130-foot long concrete crib wall.

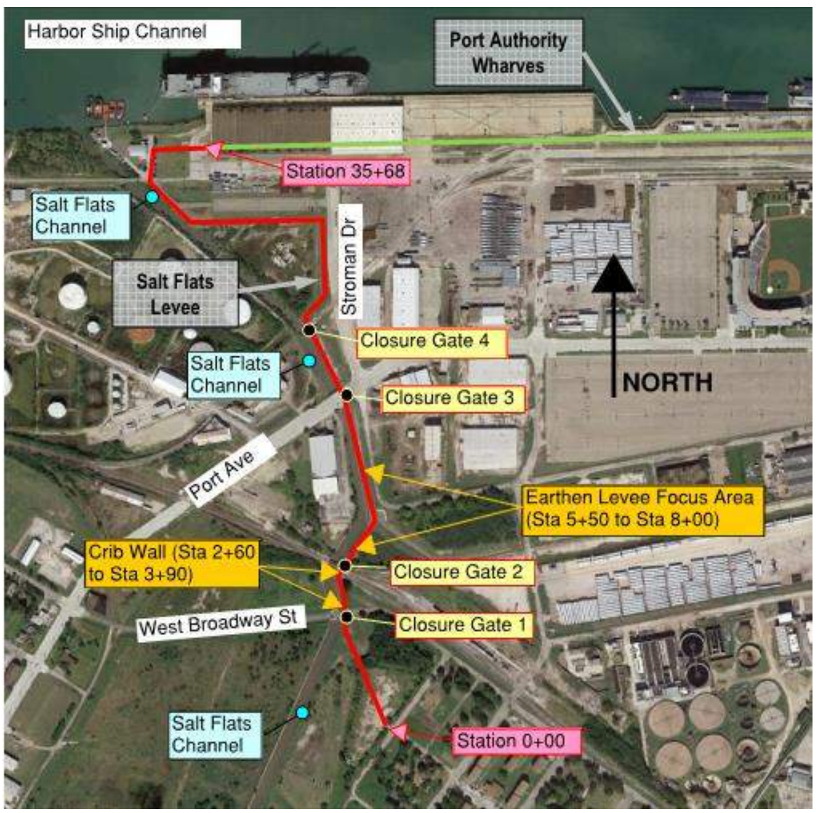


***Figure 1** – Salt Flats Levee Alignment*

The purposes of Task Order No. 4 are to further evaluate priority vulnerabilities along the SFL system that were previously identified by HDR (2016a) and the City; advance repair and/or modification concepts; and develop associated planning-level cost estimates. It should be noted that, while certain other vulnerabilities have been previously identified for the entire downtown flood protection system, the focus of the present memorandum is the Salt Flats Levee, and only the potential vulnerabilities which have been identified to be the highest priority are discussed. These vulnerabilities consist of:

1) Potential embankment and foundation instability associated with seepage along a 250-foot long segment (Station 5+50 to 8+00) of the earthen levee, as shown on Figure 1.
2) Potential inadequate structural integrity of a crib wall, which exists along a 130-foot segment of the levee (Station 2+60 to 3+90).
3) Potential lack of "water tightness" at two closure structures (located at railroad crossings).
4) Needed replacement of existing flap gates located at the discharge end of several storm water drainage pipe penetrations.
5) The presence of trees that are rooted in (or directly adjacent to) the levee, which present a potential threat to the levee integrity.

Specific work items associated with this Task Order include:

- Develop a subsurface data collection plan for execution by the City's geotechnical consultant.
- Provide coordination and testing requirements during the subsurface investigation.
- Obtain survey data along the levee that included the 250-foot segment between Stations 5+50 and 8+00, and develop four representative surface profiles.
- Develop four representative cross sections of the earthen levee based on the surface profiles and soil data collected during the subsurface investigation.
- Conduct seepage analyses on the four cross sections of earthen levee and perform a preliminary assessment of its stability during a modeled flooding event.
- Develop recommendations and design concepts for mitigating further seepage issues within or adjacent to the 250-foot long segment of the earthen levee (if required).
- Further evaluate the structural integrity and stability of the existing crib wall relative to a peak flooding event.
- Develop a design concept for replacing the crib wall, and perform a preliminary assessment of its stability with regards to foundation support and seepage.
- Develop design concepts for installing seepage plugs at the two railroad closure structures.
- Develop a conceptual plan for tree removal.

The above scope of work was developed based on previously-completed assessments for the SFL including Urban Engineering (2012), HDR (2015), and HDR (2016a).

The City's primary focus for the current evaluation is functional improvements to the SFL portion of the downtown flood protection system. These improvements are not being pursued specifically for certification and accreditation of the levee under the Federal Emergency Management Agency's (FEMA's) National Flood Insurance Program, although FEMA's criteria for levee design[1] have been generally followed for the identified scope of work. Additional evaluations and investigations will be required for levee certification[2]. The focus of the current scope of work is to address very specific vulnerabilities with the levee which were previously prioritized based on functional risk and cost factors.

Note that a portion of the downtown area has been designated on FEMA's preliminary Flood Insurance Rate Maps (FIRMs) as a seclusion zone. The preliminary maps are anticipated to be adopted as the

[1] FEMA's criteria for levee design are described in the Code of Federal Regulations, Title 44, Section 65.10 (44CFR65.10).
[2] Note that the SFL does not meet FEMA's freeboard (i.e., height) requirements, but could still be accredited as a "freeboard deficient" levee if it meets all other certification requirements.

"effective" FIRMs in 2017. The "seclusion zone" designation allowed FEMA to release the FIRMs for other areas where the mapping has been finalized while discussions with the City on the certification and accreditation process for the SFL continue.

Although the analyses and conceptual-level engineering summarized herein have been performed in accordance with standards of practice, this technical memorandum has not been prepared with the intent of serving as a stand-alone document to pursue levee certification. Levee certification would require a more in-depth study of the entire system, including verification that all components and reaches of the levee were actually constructed in accordance with the design plans, as well as validation that all other FEMA certification requirements are satisfied. Given the conceptual nature of this assessment, certain assumptions have been made, which are discussed later in this report, which will need to be verified or otherwise confirmed during detailed design, especially if the intent is to support any future levee certification efforts.

2. HISTORICAL INFORMATION

HDR previously reviewed historical information regarding the levee, which was detailed in the Phase 2A Final Report (HDR 2015) for this project. The following is an abbreviated summary of these findings:

- The SFL was originally designed and constructed in 1956 by the Nueces County Engineering Department. Major components of the system included:
    - Compacted Earth Embankment
    - Concrete Crib Wall Supported on Compacted Fill
    - Nine Discharge Pipe Penetrations with Flap Gates
    - Four Removal Stop Log Gate Closures with Supporting Concrete Abutments and Storage Bunkers
- In 1965 the levee was extended roughly 1,500 feet by the Nueces County Navigation District during construction of new the dock facilities at the Port.
- The levee is located east and adjacent to a drainage ditch, which today is part of the Salt Flats Channel. The ditch was widened and reshaped in approximately 1971 by the City of Corpus Christi – Public Works Department. Also during this period the channel was lined with concrete starting approximately 1,200 feet south of West Broadway Street to approximately 500 feet north of Port Avenue.
- In 1999, Shiner Moseley and Associates, Inc. (now HDR) designed modifications to all four closure devices for the Salt Flats Levee to replace the original creosote timber stop logs with an aluminum stop long system.
- In 2007, Maverick Engineering, Inc. designed infrastructure improvements to the interior drainage system within an area adjacent to the east side of the SFL. The improvements included the installation of a new 36-inch HDPE pipe through the earthen levee at approximate Station 7+00, just south of Brewster Street. The new pipe and an adjacent existing reinforced concrete pipe were incorporated into a concrete headwall structure at the toe of the levee. A small concrete lined basin area was developed at the headwall to optimize the hydraulics and reduce erosion during high storm water flows.
- More recently, the City retained Urban Engineering to design various improvements to the channel including deepening the channel along a 700-foot stretch that coincides with the 250-foot earthen levee section that is currently being evaluated for potential seepage issues.

Further details regarding the configuration and elevations of the various components of the levee system and drainage channel are discussed in subsequent sections of this report.

3. ADDITIONAL SITE OBSERVATIONS

HDR staff conducted additional site observations during July and August of 2016. The focus of these observations was the crib wall, the 250-foot segment of the earthen levee, and the closure structures. The primary purpose was to confirm previous observations, collect dimensions, and to better visualize possible concepts for repairs and/or modifications.

4. TOPOGRAPHIC SURVEY

HDR retained Urban Engineering to gather topographic survey data along a portion of the earthen levee from Stations 5+50 through 9+00. The survey data included the exposed side slopes of the embankment, as well as the crest and adjacent toe areas. Urban provided a certified topographic survey map, which is provided in Appendix A.

The primary purpose of the survey data was to develop four representative surface profiles, which were then used in developing four cross sections of the levee for geotechnical stability analyses by HDR.

5. GEOTECHNICAL INVESTIGATION

Readily available and pertinent geotechnical/subsurface data were gathered and evaluated, which included previous soil borings and the 1956 design drawings of the Salt Flats Levee. The findings from this initial effort were used to develop a scope of work for collecting additional subsurface data, with the focus being on the crib wall and the area between Stations 5+50 and 9+00.

Utilizing the scope of work recommended by HDR, the City retained Rock Engineering and Testing Laboratory, Inc. (RELT) to collect additional subsurface data. In accordance with the work plan, the data collection effort included seven borings (along the Salt Flats Levee), as well as field and laboratory testing on select soil samples. HDR staff provided continual support during the investigation, including coordination, sampling protocol, requirements (standards) for laboratory testing, and field observation.

Results of the sampling and testing are documented in a geotechnical data report prepared by RETL (2016). The results and findings are discussed in more detailed in subsequent sections of this report.

6. WATER SURFACE ELEVATIONS

6.1 PEAK FLOOD ELEVATION

FEMA's Flood Insurance Rate Map (FIRM) delineations for the downtown area reflect flooding associated with a storm having a 1% annual chance of occurring (i.e., a "100-year" flood). The most current (preliminary) FIRMs indicate a base flood elevation (BFE) along the Salt Flats Channel ranging from +9.0 feet to +11.0 feet (NAVD 88), with the BFE at the crib wall (Station 2+60 to 3+90) being approximately +9.8 feet. Within the earthen levee study area (Station 5+50 to 8+00) the BFE is approximately +9.7 feet.

HDR (2016b) recently completed updated hydraulic modeling for the seclusion zone using an improved approach than previously applied by FEMA. These results indicated a maximum water surface elevation of +7.6 feet (NGVD88) in the channel between Stations 2+00 and 8+00 for the 100 year flood. The associated hydrograph for this area indicates two peaks, with the first (higher) peak associated with storm surge and the second (lower) peak associated with rainfall (Figure 2). The model indicates the peak water elevation would be of relatively short duration, exceeding an elevation of +7 feet for less than 6 hours.

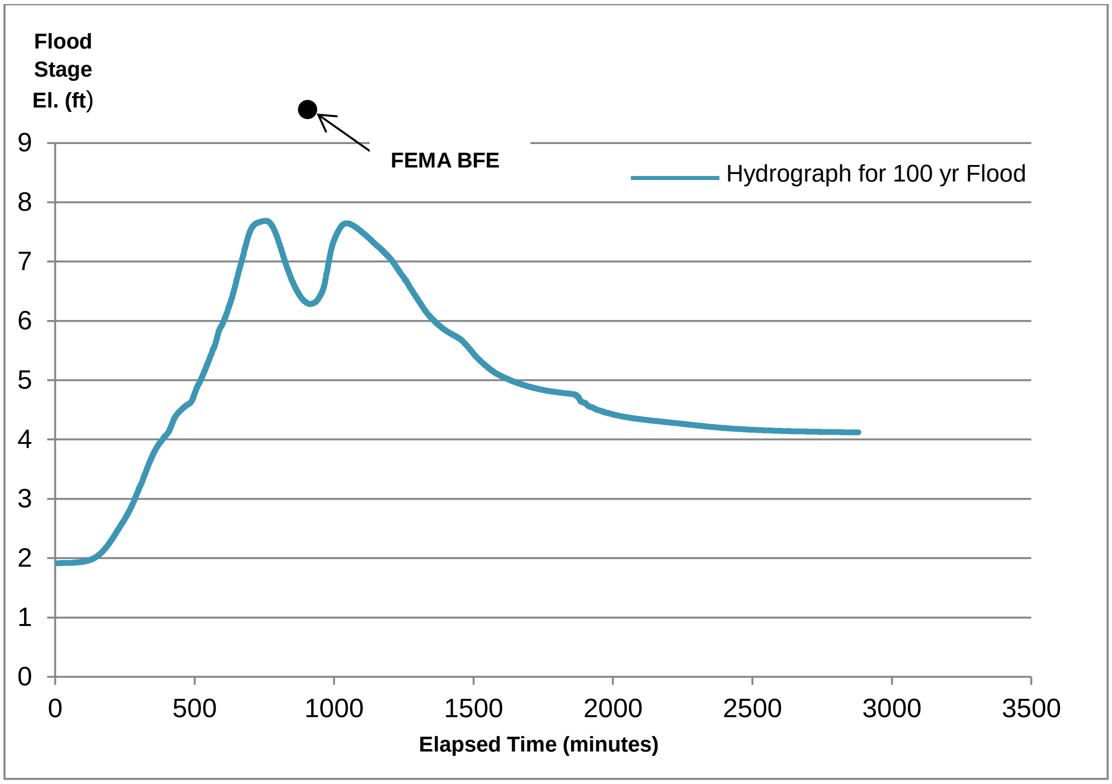


***Figure 2** – Modeled Hydrograph in Salt Flats Channel for 100-Year Flood (after HDR 2016b).*

While the results from HDR's hydraulic modeling are considered more representative of the actual 100 year flood conditions in the Salt Flats Channel, FEMA's BFE's were used for this conceptual assessment because they are more conservative (i.e., they represent higher water levels), and FEMA has not yet reviewed and accepted HDR's model.

6.2 Prolonged Flood Elevation

Various seepage analyses, as presented herein, assume the retained flood waters will seep through the earth embankment materials, as well as underneath the levee. Based on available soil information, seepage is not likely to develop into a steady-state condition until at least 24 hours of time has elapsed, which is after the flood has started to recede from its peak.

Using the shape of HDR's modeled hydrograph, it was estimated that FEMA's base flood elevation (within the Salt Flats Channel) would remain near the peak elevation of +9.7 feet for approximately 4 to 6 hours, and above +6.5 feet for approximately 24 hours (refer to Figure 2). For the purposes of this study, a prolonged flood elevation of +8 feet is used in steady-state seepage analyses.

6.3 Tail Water Surface Elevation

HDR's hydraulic modeling indicated a certain amount of interior flooding within the seclusion zone. Near levee Station 7+00, the predicted tail water flood elevation is +5.75 feet for the 100-year flood, which is fairly consistent with FEMA's interior BFE at this location.

Based on storm water discharge constraints, the tail water elevation within the seclusion zone would persist until the flood water in the Salt Flats Channel recedes below elevation +5.75 feet. At that point, the two water surfaces would nearly coincide until the flood completely recedes.

6.4 Initial Water Surface Elevation

Based on HDR's modeled hydrograph, the initial water surface elevation in the Salt Flats Channel is approximately +1.92 feet prior to the surge that would be associated with a 100 year flood. For the purposes of this assessment, flood stage is considered above this elevation, and the soils adjacent to the channel are considered saturated below this elevation.

7. Existing Earthen Levee

7.1 Study Area

The Phase 2A report (HDR, 2015) identified seepage as a potential risk to the stability of the earthen levee between Stations 5+50 and 8+00. In accordance with the present scope of work, HDR has further assessed the stability of the levee within this area for the 100 year flood. The primary purpose of this effort was to either confirm the need for seepage to be addressed, or to reduce the length of levee that may need improvement, and if appropriate, develop design concepts for such improvements. Given the purpose of this assessment, the study area was extended 100 feet north to Station 9+00 (Figure 3), with the intent of better confirming the northern limits that may require improvement.

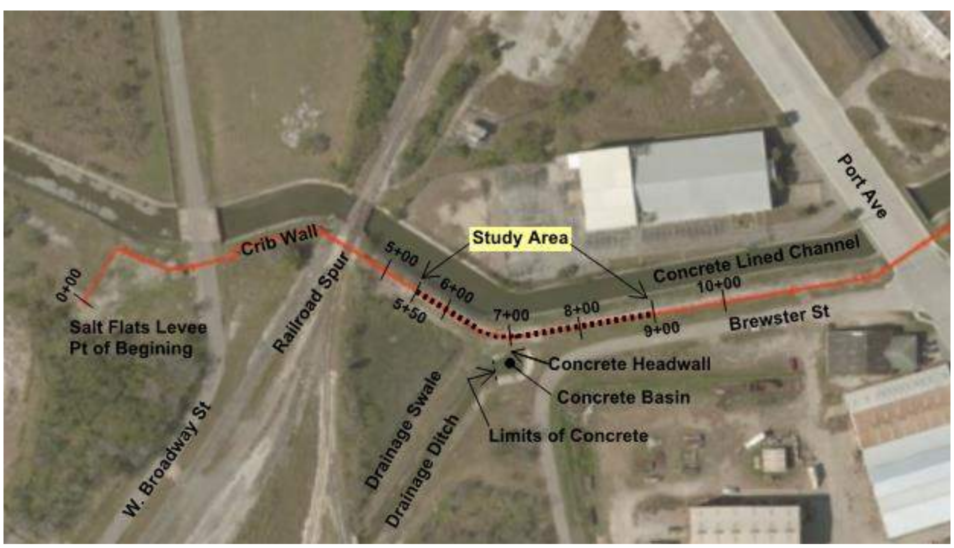


Figure 3 *– Study Area*

7.2 LEVEE AND CHANNEL CONFIGURATION (WITHIN STUDY AREA)

Levee Design

Based on the 1956 design drawings, the earth embankment levee within the study area was constructed of compacted earth fill and topped with 12 inches of compacted select fill. Both layers were specified to be compacted to 90% of the Standard Proctor maximum dry density.

No material specification or placement control of the compacted earth fill is documented on the available historical design drawings. The select fill was, however, identified as a material containing up to 20% sand and a 20% admixture of mudshell. Mudshell is a historical term used to describe an estuary deposited soil that is rich in shell content and usually contains a significant fraction of fine soil particles.

The levee was designed to have a crest width between 14 feet and 18 feet. The maximum specified sideslopes were 1.5H:1V, with 2H:1V described as "usual." The crest of the levee was set at +14 feet Mean Low Tide. To maintain a constant crest elevation, the height of the levee varied depending on natural grade.

Concrete Channel

Based on the original design drawings, the channel was lined with 5 inches of wire-mesh reinforced concrete. The side slopes were 1H:1V and the total bottom width were approximately 28 feet. The depth of the channel was a nominal 6 feet and the bottom elevation was generally between -1 feet and -1.75 feet, as referenced at that time to the Mean Low Tide. Expansion joints were installed at the end of each day's concrete pour. Three inch diameter weep holes were installed in the lower portion of the sideslope at 40-foot spacing.

Urban Engineering designed modifications to the channel in 2009. The project was constructed in 2011 and consisted of lowered the channel bottom through the study area by approximately 1 to 2 feet. The new bottom elevation of the channel is -2.5 feet per NAVD 88. The new bottom consists of 6 inches of #4 bar steel reinforced concrete. Subgrade was established at El. -3.0 feet. The bottom width of the revised channel is now approximately 20 feet, at least within the study area. The top elevation of the sideslope lining typically ranges between El. 5.5 feet and El. 6.0 fee within the study area.

The tie-in with the existing concrete included a saw cut, #4 load transfer bars, and a waterstop material. Expansion joints were also included, with the intent to match existing expansion joint locations, but in no case greater than 40-foot spacing. Based on the design drawings, the expansion joints consisted of asphalt impregnated fiber board that was capped with a paving seal.

A schematic of the original levee design with the adjacent lined channel is provided in Figure 4.

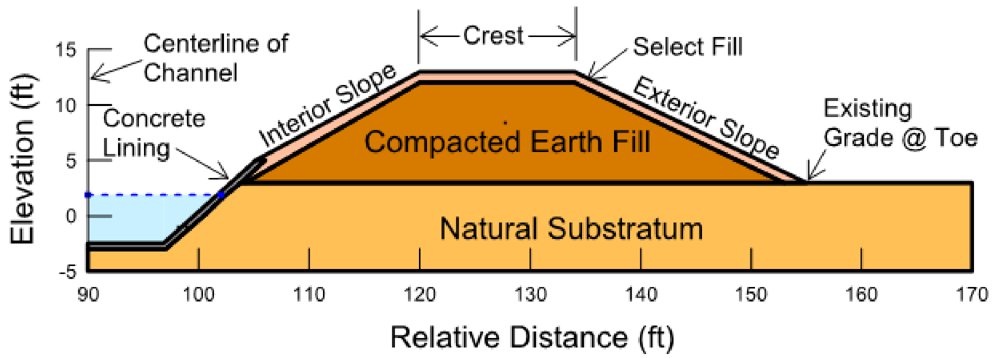


***Figure 4** – Schematic of Levee Design and Concrete Channel*

As previously noted, Maverick Engineering designed a concrete inlet/headwall near Station 7+00, for which the location is shown in Figure 3 and a schematic cross-section is shown in Figure 5. The headwall supports two pipe penetrations that extend through the levee, each discharging stormwater into the Salt Flats Channel. One of these two pipe was installed as part of the original levee construction in 1956 and consists of a 36-inch reinforce concrete pipe (RCP). The other pipe was installed are part of the Maverick Engineering design package and consists of 36-inch diameter HDPE. Based on the design drawings, this particular pipe was installed by a bore method.

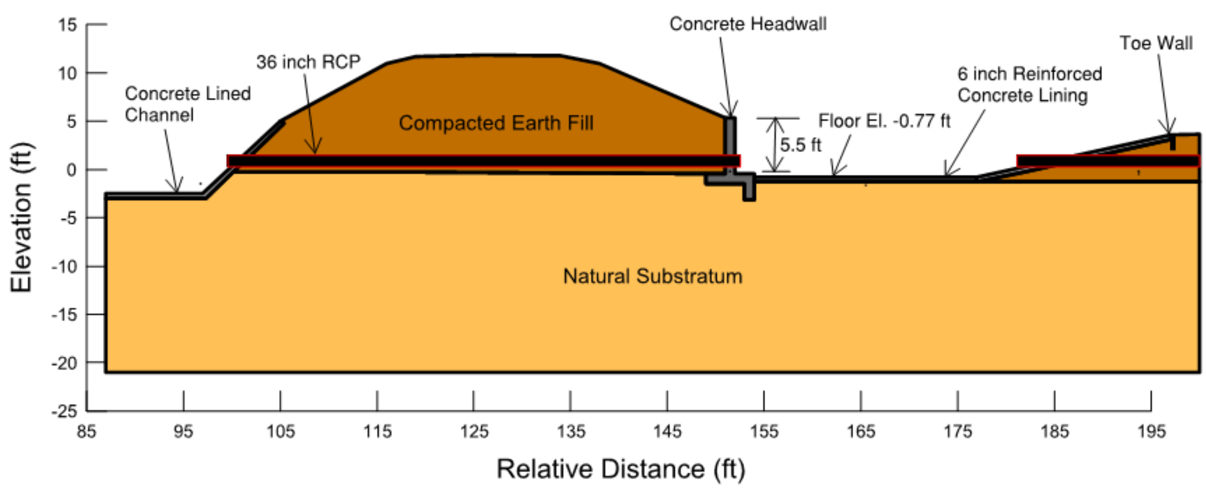


***Figure 5** – Schematic of Headwall Design*

7.3 Condition of Existing Levee

Based on observations made during July and August of 2016, the crest of the levee consists of non-vegetated soil containing a significant fraction of shells. The surface has minor rutting with slight undulations.

Recent topographic survey indicates the top or peak elevation generally ranges from approximately El. +11 to El. +13.5 feet NAVD within the study area or approximately 1.3 to 3.8 feet above the FEMA BFE. The toe elevations typically range from -0.5 feet to +6.5 feet. The overall height of the levee relative to the toe ranges from approximately 7.5 to 12.5 feet. Interior slopes typical range from 1.6H:1V to 2.6H:1V. The exterior slopes within the study area typically vary between 2.8H:1V to 3.7H:1V, except at the headwall, where the exterior slope is approximately 2.1H:1V. Both the interior and exterior slopes are grass covered, with the condition of the grass generally good, except during prolonged dry spells as illustrated in photographs provided in Figures 6 and 7. No slope movement or sloughing was noted during the recent site observations.

Figure 6 – *Interior Slopes (August, 2016)*

Figure 7 – *Exterior Slopes (August, 2016)*

A series of discontinuous tension cracks were noted near the top of the levee slopes during the site observations. Based on limited measurements, the tension cracks on the east edge of the crest typically ranged from 0.5 to 1.0 feet in depth. The tension cracks on the west edge of the crest (channel) were deeper, typically ranging from 1 to 3 feet and in places over one-inch wide at the surface. The tension cracks were noted during the driest and warmest time of the year (late summer), when tension cracks are likely to be most visible and reach their maximum depths. Some self-healing of these cracks may occur during wetter periods, though they will become more apparent again during a subsequent warm/dry periods. Tension cracks were not observed on the face of the slopes, except on the channel side where tension cracks were noted one- to two-feet downslope. Figures 8 and 9 provide examples of the tension cracks.

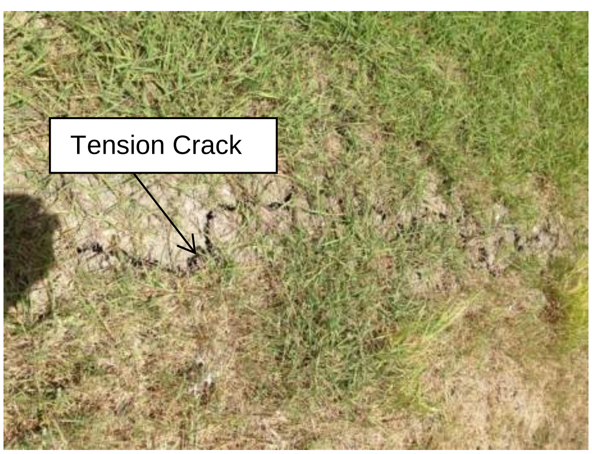


Figure 8 – *Tension Crack (July, 2016)*

Figure 9 – *Tension Crack (August, 2016)*

7.4 CONDITION OF HEADWALL

Based on limited observations, the headwall and associated concrete-lined basin appear to be in good condition and functioning properly. Based on recent topographic survey data, the top of headwall is at El. +5.33 NAVD. The two drain pipes have an invert elevation of approximately -0.35 feet.

The headwall and associated basin represent a discontinuity because the levee slope in this particular area is essentially vertical below approximate El. +5.33 feet. Figures 10 and 11 provide recent photographs of the headwall and basin area.

Figure 10 – *Concrete Headwall and Basin*

Figure 11 – *Interior Drainage Ditch and Basin Area*

7.5 ADDITIONAL PIPE PENETRATION

A third pipe penetration exists south of the headwall at approximate Sta. 6+50. This particular penetration discharges stormwater from the drainage swale that was identified in Figure 3. There does not appear to be a concrete headwall associated with this particular penetration. The topographic survey indicated the pipe is installed at the toe of the levee at an invert elevation of -0.81 feet (bottom of drainage swale). The pipe was identified as being a 30-inch diameter reinforced concrete pipe. Figure 12 and 13 illustrate the drainage swale and the approximate location of the pipe.

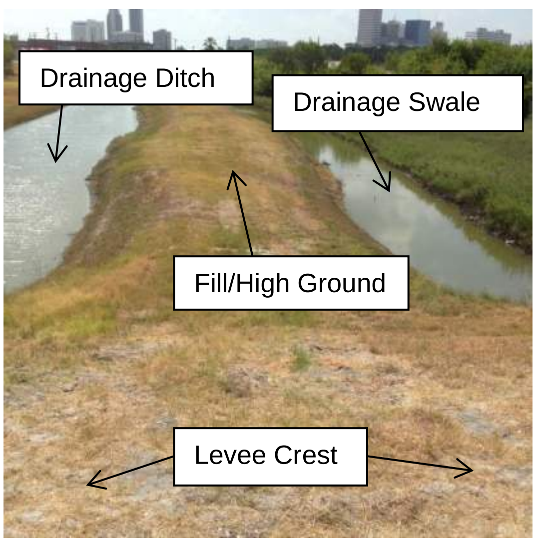


Figure 12 – *Drainage Ditch and Swale*

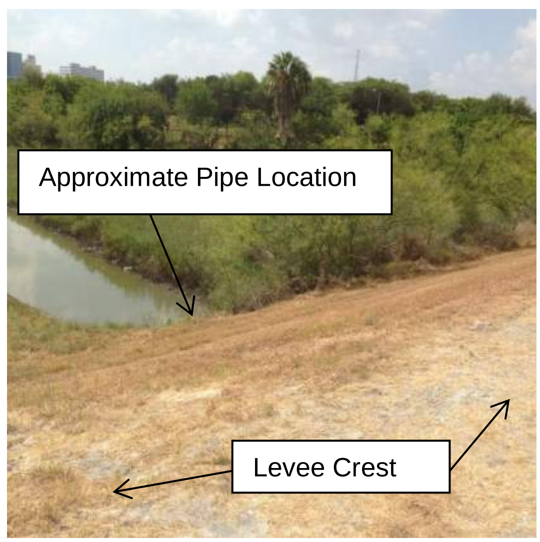


Figure 13 – *Approximate Pipe Location*

8. LEVEE STABILITY ANALYSES

Analyses performed for the Phase 2A assessment (HDR, 2015) indicated the levee embankment and foundation were potentially unstable during a 100-year flood. The reach considered most vulnerable was identified between Stations 5+50 and 8+00. Additional analyses have been performed specifically within this reach of higher vulnerability.

The primary purpose of the additional analyses is to either confirm or reduce the length of levee that may need improvement, and, if appropriate, develop design concepts for such improvements. Given the purpose of this assessment, the study area was extended 100 feet north to Station 9+00, with the intent of better confirming the northern limits requiring improvement.

While the present assessment was not performed specifically for a FEMA levee accreditation process, HDR generally followed the criteria listed under 44 CFR 65.10(b)(4) for "*Embankment and Foundation Stability*." The most notable exception is analyses of pipe penetrations, which was beyond the current scope of work. As referenced in 44 CFR 65.10, HDR applied relevant guidelines presented by U.S. Army Corps of Engineers (USACE) manuals. The manuals used in completing the analyses and defining acceptable performance criteria are given as follows:

- USACE EM 1110-2-1913, Design and Construction of Levees (2013)
- USACE ETL 1110-2-569, Design Guidance for Levee Underseepage (2005)
- USACE EM-1100-2-1902, Slope Stability (2003)
- USACE EM-1110-2-1901, Seepage Analysis and Control for Dams (1986)

8.1 SEEPAGE RELATED FAILURE MECHANISMS

Fundamentally, seepage occurs when there is a total hydraulic head differential between one side of a levee and the other. Seepage or seepage-related forces can cause an earthen levee to fail either by a piping-failure type mechanism and/or by reducing the shear strength of the soil to a point where a deep-seated or shallow-sloughing type failure could occur along the side slopes of the levee.

Piping in particular can occur when flood waters seep through and/or underneath the earthen levee. Once the seepage rate has become constant or nearly constant, the system is said to be in steady-state. During a steady-state condition, water seeps to the surface (either through the levee or natural ground) and exits under a certain hydraulic gradient, which is defined as the loss in total head over an incremental length. When the exit gradient is too great, soil particles may be removed from the area. This phenomenon, called flotation, can cause piping (the removal of soil particles by moving water). Piping can lead to an undermining and failure of the levee.

Slope instability is a condition that occurs when the shear strength of the soil is insufficient to overcome the gravitational forces associated with the mass of the levee embankment. Under steady-state seepage the soils are said to be in a drained condition, meaning there is no excess pore water pressure within the soil matrix. Under a rapid flood drawdown, certain soil types, such as clays, may experience excess pore pressures. In these situations fine grained soils are considered to be in an undrained condition and their shear strength is represented by undrained soil properties.

8.2 APPROACH

To assess both piping and slope failure mechanisms, representative cross sections of the levee were developed at four locations within the study area. Soil boring and associated laboratory test data were used to develop the soil stratigraphy at each location, and engineering properties were assigned to each unique soil type/layer. The cross sections were then analyzed for piping by assuming a prolonged flood elevation of +8.0 feet on the channel side of the levee, and a water surface elevation on the tail water side of the levee of +5.75 feet. The analyses were performed on each cross section using the computer program SEEP/W, a two-dimensional finite-elements modeling program developed by Geo-Slope International, Ltd. The program generates total head contours through the embankment and foundation soils. The spacing or gradient of the total head contours near the toe were used to estimate the exit gradient, which in turn was used to calculate the factor of safety against piping.

Slope stability analyses were performed on each cross section using the computer program SLOPE/W, also developed by Geo-Slope International, Ltd. The program provides several options for calculating the factor safety against slope failure. Spencer's Method was chosen because it satisfies both moment and force equilibrium and it is widely accepted by USACE. The program generates the potential slip surface using an entry-and-exit search method, and both circular slip surfaces and optimized (non-circular) slip surfaces were considered. The following two conditions were analyzed:

- Stability of the exterior slopes was analyzed for a steady state seepage condition. In this case the total head distribution developed from the corresponding seepage analysis was imported into the program and used to determine pore water pressures within the soil. Drained parameters were used for a range of soil types to depict the stability of the levee under a steady state seepage condition.

- Stability of internal slopes was analyzed for a rapid drawdown condition. In this case the prolonged flood elevation was assumed to instantaneously drop to the normal water level within the channel. Undrained strength parameters were used for all soils with appreciable amounts of silt and clay sized particles.

8.3 Surface Profiles

Surface profiles were developed at each of the cross section locations using the recent topographic information obtained within the study area. The four cross section locations are illustrated on Figure 14. Pertinent geometric data at each location are provided in Table 1.

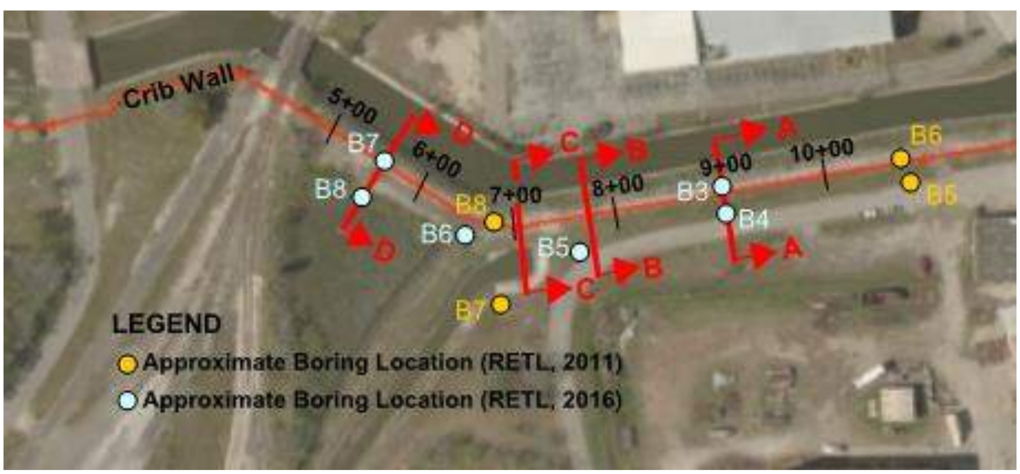


Figure 14 – *Cross Section and Soil Boring Locations*

Table 1 – *Pertinent Geometric Data at Cross Sections*

Cross Section	Location	Top El. (ft)	Outside Toe El. (ft)	Channel Bottom El. (ft)	Inside Ave. Slope(1)	Outside Ave. Slope	Crest Width (ft)
A-A	9+00	13.2	6.86	-2.5	2H:1V	2.8H:1V	11
B-B	7+75	12.8	6.38	-2.5	2H:1V	3H:1V	14
C-C	7+10	11.9	5.33(2)	-2.5	2.6H:1V(3)	2.1H:1V	15
D-D	5+50	11.2	2.57	-2.5	1.6H:1V	3.7H:1V	18

Notes:
1. *Average slope above top of 1H:1V concrete lined channel.*
2. *Elevation at top of headwall. Ground at bottom of headwall = El. -0.77 feet.*
3. *Average slope to top of concrete headwall (designed by Maverick Engineering).*

8.4 Subsurface Conditions

The subsurface conditions applied for each cross section were generated from soil borings completed within the study area, including laboratory test data.

Previous Soil Borings

A previous subsurface investigation was conducted by RETL (2011) for Urban Engineering (2012). The investigation included eight soil borings along the Salt Flats levee. They were generally located between the north end of Stroman Drive and Closure Gate 2, which is adjacent to the crib wall (shown Figure 1). Samples collected in the field were logged and classified by soil type. Select samples were tested in the laboratory for both index properties and strength characteristics.

Of the eight previous borings, four are located within or near the study area (B5 thru B8), as shown on Figure 4. Borings B5 and B7 were completed near the toe of levee and extended to a depth of 60 feet. Borings B6 and B8 were completed on the crest of the levee and also extended to a depth of 60 feet. The information/data obtained from these four borings were applied for the present assessment. To avoid confusion, these previous borings are referenced herein as “2011” borings, e.g. B8 (2011). The approximate locations of the four previous boring are shown in Figure 14.

Recent Soil Borings

The more recent subsurface investigation (RETL, 2016) included nine additional borings (B1 through B9). Two of these borings (B1 and B2) were completed for a separate assessment at the Museum of Science and History (HDR, 2016c) and thus not considered herein. One of the borings (B9) was completed near the crib wall and is discussed in section 9.0 (Crib Wall Replacement).

Of the six borings completed within the study area, two (B3 and B8) were completed on the crest of the levee, while the other four borings (B4 through B6) were completed near the toe of the levee. The crest borings extended to a depth of 65 feet and the toe borings extended to approximately 35 feet. The approximate locations for the six boring are shown on Figure 14.

Samples collected in the field were logged, visually described and classified by soil type. Field testing included standard penetration tests (SPT) in granular and cohesive soils, and pocket penetrometer (PP) tests were conducted on cohesive soils collected with thin walled push tubes. The SPT and PP test data were used to estimate the density of the granular soils and consistency of the cohesive soils.

A significant number of samples were tested in RETL's laboratory for index properties (e.g. moisture content, Atterberg limits, grain size, and unit weight) and unconfined compressive strength. A fewer number of advanced tests were conducted to better determine the drained and undrained strength properties of the soil, the hydraulic conductivity (permeability), and dispersive characteristics. HDR participated during selection of the samples for testing, which included both field visits during completion of the borings, as well as laboratory examination of select samples. The number and types of tests were prioritized to stay within the limits of RETL's scope of work with the City.

Embankment Soils

Based on the two recent crest borings (B3 and B7), the embankment material consists mostly of stiff to very stiff Fat Clay, with a certain amount of Sandy Lean Clay near the surface. The Sandy Lean Clay appears to be the "select" fill material that was placed over the core of the levee. The findings in these two borings are similar to the soils encountered in crest boring B8-2011. Based on review of the crest borings, the thickness of the embankment fill is approximately 8 to 12 feet within the study area.

Collecting quality thin-walled tube samples of these soils generally proved to be challenging, since they tended to be slightly dry and over-consolidated (i.e. well compacted). Two sufficiently undisturbed tube samples of this material indicated an unconfined compressive strength greater or equal to 3.3 tons per square foot.

Natural Substratum

Based on the borings, the embankment is generally founded on a 3 to 7 foot layer of clay (CH/CL) that extends laterally beyond the toe. The soils below this blanket layer of clay are extremely complex, which is typical of a coastal or estuary environment. Those soils extending 20 to 25 feet below the clay blanket layer can broadly be described as inter-bedded, non-uniform, discontinuous layers of Clayey Sand (SC), Silty Sand (SM), Fat Clay (CH), and Lean Clay (CL). The SC/SM materials tend to be more prevalent and were encountered in all borings.

The consistency of the natural cohesive soils is generally very soft to medium stiff, while the density of the Silty Sand is typically loose to very loose. The percentage of secondary soil particles is quite variable, for example, the Silty Sand has a fines content that typically ranges between 15% and 40%. Similarly, samples of the Clayey Sand indicated highly variable amounts fine particles, at times bordering on being classified as a Sandy Lean Clay.

Firmer and denser soils were encountered 20 to 25 feet below the toe of the levee, corresponding to elevations -15 feet to -25 feet. The firmer soils included stiff to very stiff Lean Clay (CL) and Fat Clay (CH), while the denser granular soils included medium dense to dense Silty Sand (SM) and Poorly Graded Sand (SP).

Soil Stratigraphy

An idealized soil stratigraphy was developed for each cross section using the most relevant and nearest boring information. As an example, the surface profile and soil stratigraphy for Cross Section BB are shown in Figure 16. The surface profile and soil stratigraphy for all four cross sections are provided in Appendix B.

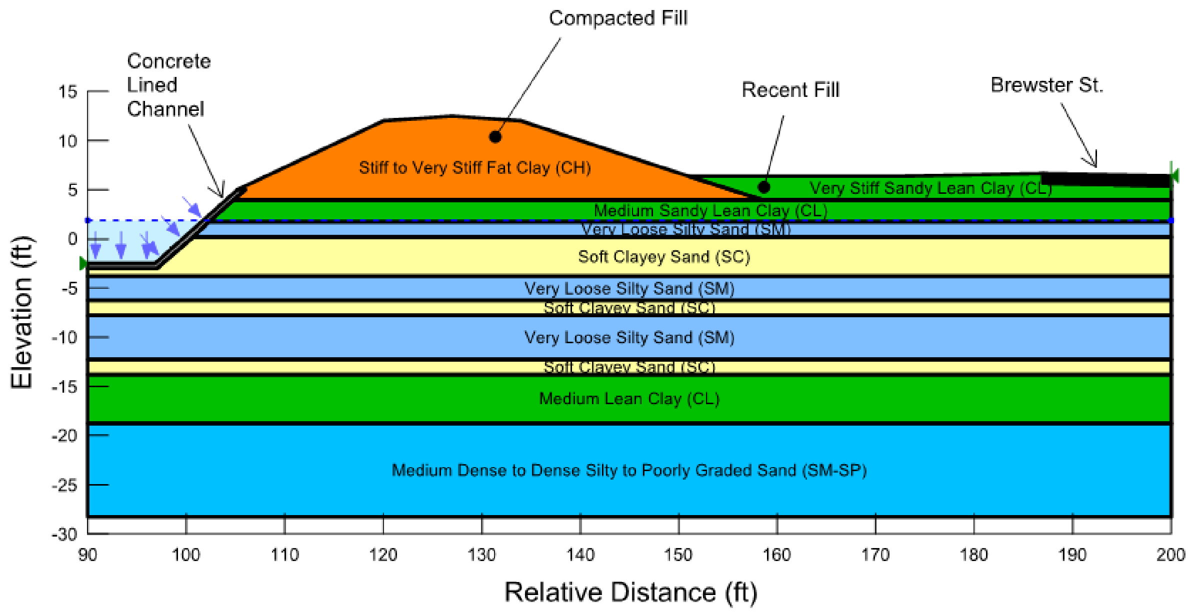


Figure 15 – *Surface Profile and Idealized Soil Stratigraphy (Cross Section BB)*

Engineering Properties

The engineering properties relevant for this study were estimated for each unique stratigraphic layer. The values were selected based on the field data, laboratory test data, typical values (for the given soil type), and published correlations. Table 2 provides a summary of the properties used for Cross Section BB. Tables indicating soil properties for all four cross sections are provided in Appendix C.

Table 2*- Engineering Properties for Each Soil Layer (Cross Section BB)*

Description	Unit Weight (pcf)	Drained Strength		Undrained Strength		Permeability (cm/sec)	
		Φ_d (deg)	c_d (psf)	Φ_u (deg)	c_u (psf)	k vertical	k horizontal
Stiff to Very Stiff Fat Clay (CH)	120	30	150	0	1750	2EE-06	8EE-06
Very Stiff Sandy Lean to Fat Clay (CL/CH)	120	32	100	0	1500	2EE-06	8EE-06
Medium Sandy Lean Clay (CL)	115	28	100	0	500	5EE-06	2EE-05
Soft Clayey Sand (SC)	114	26	75	12	200	1EE-05	3EE-05
Very Loose Silty Sand (SM)	112	27	0	20	0	1EE-04	2EE-04
Medium Lean Clay (CL)	115	28	100	0	650	5EE-06	2EE-05
Medium Dense to Dense Silty Sand to Poorly Graded Sand (SM-SP)	120	32	0	22	0	5EE-04	1EE-03

Groundwater Table

The groundwater table elevation was assumed to equal the water elevation in the channel prior to the storm surge (i.e. +1.92 feet). This assumption coincides well with the depths groundwater was encountered in the soil borings. All soils below this elevation are assumed to be currently saturated.

8.5 Prolonged Flood Elevation

Based on the hydrograph generated by HDR (Figure 2), it is estimated that FEMA's computed water surface elevation (within the channel) may remain near the peak elevation of +9.7 feet for approximately 6 hours, and above +6.5 feet for approximately 24 hours. Given that the earthen levee is constructed of low permeable soil, the wetting or seepage front is unlikely to advance through the levee and achieve steady state seepage within a 24-hour period or duration. Thus using an average elevation over this 24-hour duration would be conservative. For the purposes of this study, a prolonged flood elevation of +8 feet is assumed because it conservatively estimates the average flood elevation during the stated 24-hour period.

8.6 Seepage Analyses

The Seep/W computer program requires the user to define a 2-dimensional space consisting of the following boundaries:

- Left (or West) Horizontal Extent of the Model
- Right (or East) Horizontal Extent of the Model
- Land/Ground Surface Between the two Horizontal Extents
- Vertical Extent of the Model

The resulting interior 2-D space is then defined by the soil stratigraphy and assigned hydraulic soil properties.

<u>Boundary Conditions</u>

Each unique boundary is assigned a certain condition that influences the results. The boundary conditions that were applied are given in Table 3.

Table 3 – *Summary of Boundary Conditions*

Boundary	Description	Condition
Left Horizontal	Centerline of Channel	Constant head = El. +8.0 ft(1)
Right Horizontal	Approximately 1,900 feet from Centerline of Channel	Constant Head = El. +5.75 ft or ground surface elevation, whichever is greater.
Vertical Boundary	El. -28 to El. -40, depending on soil conditions.(2)	No Flow
Surface:		
• Segment A	Channel Bottom to Elevation +8 (interior side)	Constant Head = El. +8.0 feet
• Segment B	Elevation +8 (interior) side to Elevation +5.75 Exterior Side	Potential Seepage Face. Includes natural ground surface if above El. +5.75 feet
• Segment C	Elevation +5.75 to Natural Ground	Constant Head = +5.75 feet if ground elevation is less than +5.75 feet.(3)

Notes:

1. Constant Head equals prolonged flood elevation.
2. Vertical boundary established based on the presence of underlying clay strata or El. -40 feet.
3. Constant head equals tailwater flood elevation.

Key Assumption

The concrete channel was not considered perfectly impermeable in the seepage analyses. Instead the liner was modeled as a 6-inch layer of clay with a permeability of 1EE-06 cm/sec. The rationale for not incorporating a lesser permeable layer (i.e. concrete) are as follows:

- Three-inch diameter weep holes were installed in the lining.
- The original (1971) portion of the channel has likely shifted and settled over time, possibly opening up the expansion joints and/or causing small cracks to form.
- The original expansion joints have been in-place since 1971 (45 years) and the in-fill material (of unknown type) has likely degraded and can no longer be considered "water tight."
- The new portion of the lining was assumed to be in good condition. However, the subgrade was almost certainly soft/loose, and wet. Therefore, even the new portion has probably been subjected to a certain amount of settlement or shifting, potentially opening up the expansion joints and interfaces. Even a very slight opening, possibly on the order of 0.10 inch would be sufficient to transmit water to the subgrade.
- Demonstrating the liner is perfectly impervious may not be feasible. Similarly, estimating the amount of leakage is also unfeasible. Therefore, a more practical and straightforward approach is to model the concrete liner as a low permeability clay.

Seepage Results

The results from the seepage analyses are provided in a graphical format that depicts the total head distribution through the various soil strata during a steady state seepage condition. The computed head distribution for Cross Section BB is shown in Figure 16. The graphical results for all cross sections, including a modified version of DD, are provided in Appendix D.

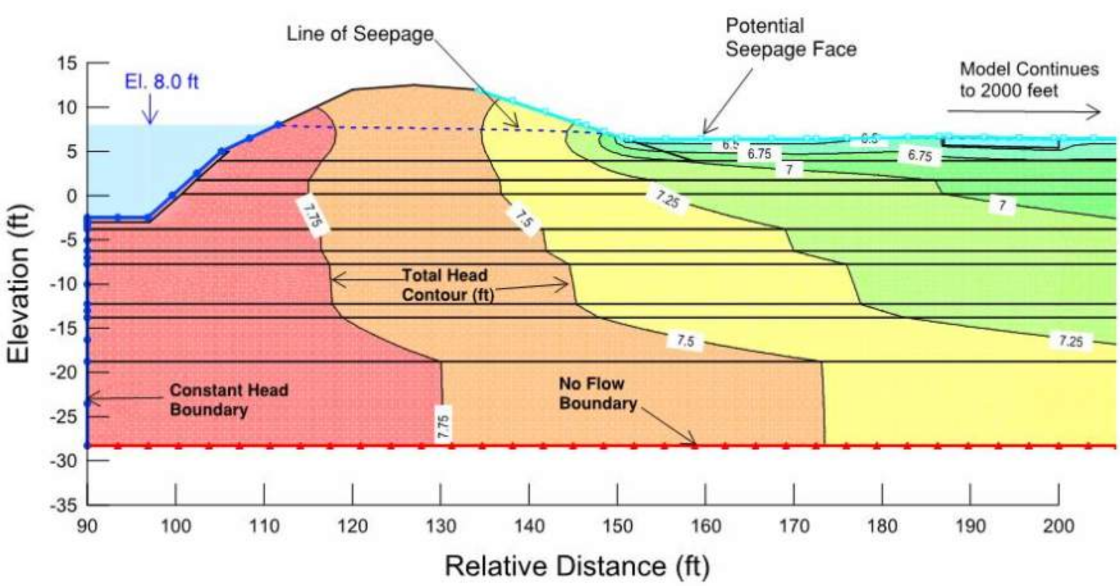


Figure 16 – *Total Head Contours and Boundary Conditions (Cross Section BB)*

8.7 Exit Gradients and Piping

Method and Results

The seepage exit gradients were calculated for each applicable cross section. The methodology generally consisted of calculating the loss in total head across the upper clay layer(s), nearest the toe of the slope. A graphical depiction of the methodology is shown in Figure 17.

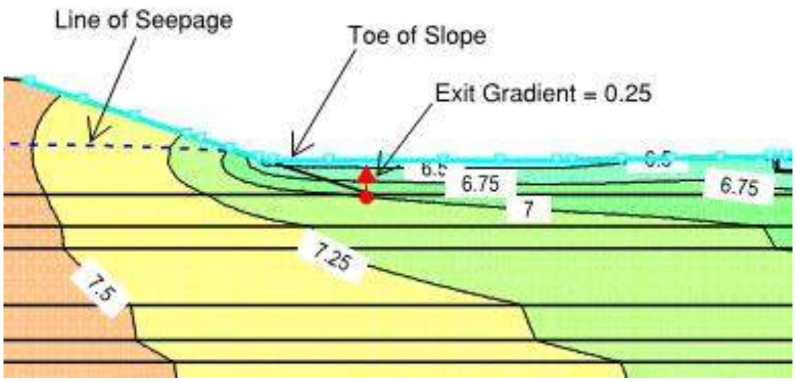


Figure 17 – *Exit Gradient at Cross Section BB*

To determine the factor of safety against piping the critical gradient first needs to be established. This was determined by dividing the saturated unit weight of the clay blanket soil by the unit weight of water. Table 4 provides a summary of the computed exit gradients, critical gradients, and factors of safety against piping.

Table 4 – *Gradients and Factors of Safety*

Cross Section	Critical Gradient	Exit Gradient	Factor Safety
A-A	0.92	0.10	9.2
B-B	0.92	0.25	3.6
C-C	NA(1)	NA	NA
D-D (base)	0.79	0.11	7.2
D-D (modified)(2)	0.76	0.22	3.4

Notes:
1. Refer to discussion Item 1 below.
2. Refer to discussion Item 2 below.

Discussion Items

1. Cross Section C-C incorporated the reinforced concrete slab that exists within the storm water collection basin/headworks. The slab was considered nearly impervious in the seepage analyses and thus piping through this material was not a consideration. The seepage analyses performed at Cross Section C-C were used to determine the total water head distribution for slope stability analyses. The seepage results were also used to verify that hydraulic uplift pressure on the underside of the slab would not be sufficient to cause the

slab to fracture. Based on total and elevation heads, the unbalanced uplift pressure on the basin slab is less than 0.25 pounds per square inch, which is unlikely to cause the reinforced concrete slab to crack or fracture.

2. The soil stratigraphy used in developing Cross Section D-D represents some of the more critical soil conditions encountered within the study area. However, the outer slope at D-D is less critical than other locations. The outer slope and toe area of Cross Section D-D were modified to include more critical slope geometry, such as the drainage swale that exists approximately 100 feet north. The effective outer slope of Cross Section D-D was modified to approximately 2.2H:1V and the toe elevation was lowered to El. -0.75 feet.

Criteria and Conclusions

USACE guidance documents recommend a minimum factor of safety against piping of 1.6. The computed factors of safety for the stated cross sections exceed 1.6, thus satisfying piping criteria for the modeled conditions.

Dispersive Soils

Certain fine cohesive soils can be susceptible to internal erosion when the seepage exit gradient is below the critical gradient. These soils are often referred to as dispersive, and can erode or float in the presence of very slow moving water. The dispersive characteristics of the uppermost site soils were considered during this study.

The laboratory crumb dispersive tests were performed on nine samples (RETL, 2016). Of the nine tests performed, one was performed on the embankment material, while the other eight samples were conducted on material collected near the toe of the levee. The majority of the eight samples were collected between the surface and a depth of 4.5 feet.

The crumb dispersion test classifies clayey soil between Grade 1 and Grade 4. Grade 1 is considered non-dispersive, Grade 2 is intermediate, Grade 3 is dispersive, and Grade 4 is classified as highly dispersive. The purpose of these tests was to qualitatively determine whether the cohesive soil is highly prone to erosion/piping, in particular during a low velocity/steady-state seepage condition.

The test on the embankment material indicated a non-dispersive characteristic, which is generally consistent with a Fat Clay. The results on the other eight samples generally indicate non-dispersive or intermediately dispersive. Of the eight samples, one sample was classified as highly dispersive. The sample in question was collected from 1.5 to 3.0 feet below the surface. A sample collected above this depth interval indicated non-dispersive characteristics, while two different samples collected below this interval indicated intermediate dispersive characteristics.

Overall the results indicate that the cohesive soils are unlikely to be prone to erosion/piping under the presence of small seepage gradients or flow.

8.8 Slope Stability Analyses

Prolonged Flood Conditions

As previously mentioned, steady state slope stability analyses were performed using the computer software program Slope/W. The phreatic surface and pore pressure from the seepage analyses were used in the stability analyses. Because steady-state seepage is a long-term condition, drained strength parameters were assigned to all soils, both coarse-grained and fine-grained. A key assumption was applied in assigning four-foot deep tension cracks across the entire width of the crest. Based on drained soil properties, the maximum theoretical tension crack depth is approximately 4 feet.

The results from the steady-state slope stability analyses are provided in graphical format, which depict the most critical slip surface as determined by Slope/W. As a representative example, the stability result for Section BB is shown in Figure 18. The graphical results for all cross sections, including the modified version of DD, are provided in Appendix E. Table 5 provides a summary of the computed factors of safety.

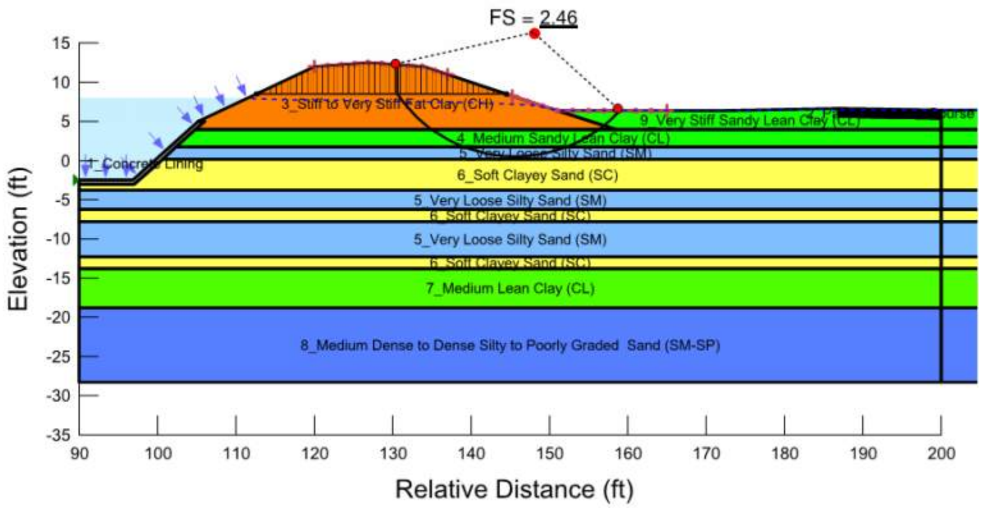


Figure 18 – *Steady State Seepage Stability Result (Cross Section BB)*

Table 5 – *Summary of Slope Stability Results (Steady State Seepage)*

Cross Section	Factor of Safety
AA (Sta. 9+00)	2.67
BB (Sta. 7+75)	2.46
CC (Sta. 7+10)	1.68
DD (Sta. 5+50)	2.29
DD (modified)	1.67

Rapid Drawdown

Levee stability analyses were also performed for rapid drawdown. During rapid drawdown, the stabilizing effect of the water on the upstream/interior face is lost, but the pore-water pressures within the embankment may remain high. As a result, the stability of the interior face of the levee can be much reduced. The dissipation of pore-water pressure in the embankment is largely influence by the permeability and the storage characteristic of the embankment materials. Highly permeable materials drain quickly during rapid drawdown, but low permeability materials take longer to drain.

Slope stability analyses of the interior slopes under a rapid drawdown condition were performed using Slope/W. The approach was simplified by incorporating the following conservative assumptions:

- Phreatic surface through the entire embankment is constant at El. +8.0 feet during prolonged flood.
- Prolonged flood decreases instantaneously to the normal water elevation in channel of +1.92 feet.

- The tailwater elevation of +5.75 feet creates a reverse steady state seepage condition.

Another key and likely conservative assumption was the 4-foot deep tension crack was extended part-way down the interior slope.

Both drained and undrained soil parameters were input into the program. The program then selected soil shear strengths based on the more critical from the two sets of parameters. The results from the rapid drawdown stability analyses are provided in graphical format, depicting the most critical slip surface as determined by Slope/W. As a representative example, the stability result for Section BB is shown in Figure 19. The graphical results for all cross sections, including the modified version of DD, are provided in Appendix F. Table 6 provides a summary of the computed factors of safety.

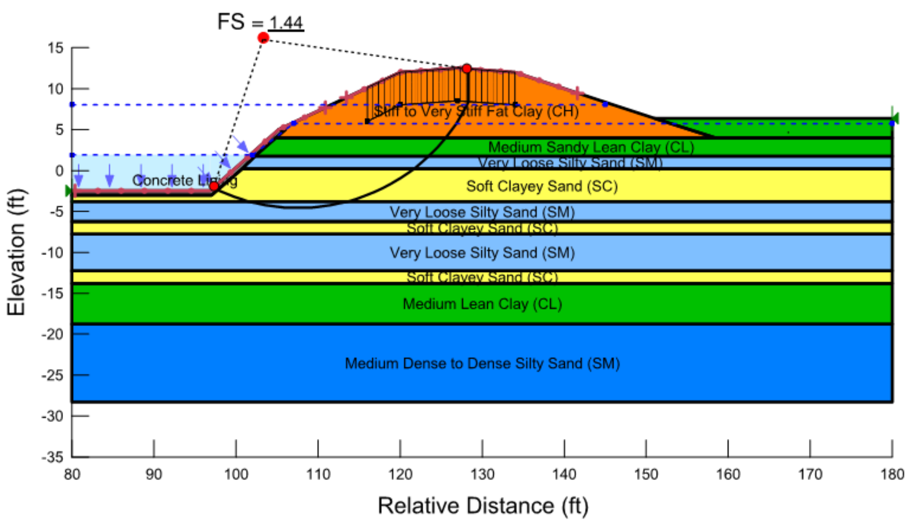


Figure 19 – *Rapid Drawdown Stability Result (Cross Section BB)*

Table 6 – *Summary of Slope Stability Results (Rapid Drawdown)*

Cross Section	Factor of Safety
AA (Sta. 9+00)	1.49
BB (Sta. 7+75)	1.44
CC (Sta. 7+10)	1.42
DD (Sta. 5+50)	1.33

Criteria

USACE guidance documents recommend a minimum factor of safety of 1.4 for steady state seepage, and between 1.0 and 1.2 for a rapid drawdown condition. The computed factors of safety for the examined cross sections exceeded the minimum, thus satisfying slope stability criteria for the modeled conditions.

8.9 Conclusions and Recommendations

Based on the results of this conceptual-level analysis, stability improvements to the basic earthen levee section between Stations 5+50 and 9+00 do not appear warranted at this time. These results indicate less vulnerability with regards to seepage and stability then was indicated in the Phase 2A study (HDR, 2015), which was based on more limited site data. The more positive findings of the current assessment are primarily attributed to following factors:

1. Inclusion of a tailwater flood elevation (+5.75 feet), as supported by results from the recent hydraulic flood modeling (HDR, 2016b).
2. More favorable soil parameters, as supported by the recent geotechnical investigation (RETL, 2016).

The stability results are dependent of the tailwater elevation. Therefore should the City implement interior drainage and/or pump station improvements that would decrease the tailwater elevation, these results should be re-evaluated.

8.10 OTHER CONSIDERATIONS

Headwall

The stability of the levee at and near Station 7+00 is highly dependent upon the stability of the concrete headwall (refer to Figure 5). Analysis of the headwall was beyond the scope of the current assessment. During future detailed design of the levee improvements, the headwall should be analyzed in more detail for other external modes of failure (including sliding and overturning), as well as internal structural capacity, using active loadings generated during the 100-year flood event.

Pipe Penetrations

Overall there are at least nine storm water pipe penetrations through or underneath the SFL, of which three are located within the study area. Each of the stormwater drain pipes is fitted with a flap gate to prevent reverse flow from the channel. In addition to the nine known pipe penetrations, there may be other abandoned or not-in-service pipes.

Pipe penetrations through or underneath a levee inherently create potential for seepage paths to develop, especially if the soil around the pipe is not in firm contact with the pipe. High seepage velocities at the interface, and/or the presence of highly dispersive clays and silts, can cause an internal erosion conduit to form along the longitudinal direction of the pipe. Over time the conduit can enlarge, possibly causing a breach or collapse of the levee.

Most of the known pipe penetrations were likely installed in in open trench during original construction of the levee in 1956. The type and compaction level of the trench backfill is unknown. The most recent pipe penetration (designed by Maverick Engineering) was installed by a jack and bore method. Recent past practice was to install anti-seep devices or water stops at intervals along the pipe. If installed by jack-bore, the water stops could only be included at the downstream and upstream ends of the pipe. Current practice is to encase the landside one-third of the pipe with a granular drainage fill. Based on review of readily-available historical records including engineering drawings obtained from the City, HDR has not been able to determine whether the existing pipe penetrations included any special provisions or measures to reduce the potential for seepage to occur at the soil/pipe interfaces.

Based on the limited soil dispersive test data that are currently available, a slow moving or small quantity of seepage along a particular pipe penetration appears unlikely to create an enlarged internal erosion conduit, unless the seepage has been occurring frequently over an extended period of time. Such failure modes are often gradual, continuing until seepage achieves a more critical velocity or quantity.

Although detailed evaluation of pipe penetrations was beyond HDR's current scope of work, the qualitative evaluation described above indicates that the pipe penetrations represent an unknown vulnerability with regards to the integrity of the Salt Flats Levee and should continue to be investigated. HDR recommends that the City conduct a detailed inventory/survey of the pipe penetrations, as well as any pipes and manholes located near the toe of the levee. The inventory should include gathering or confirming invert elevations, pipe material and diameter, a conditions assessment (to the extent practical), and, if appropriate, the use of geophysical techniques to locate the exact alignment and depth of both active and abandoned pipes. Geophysical techniques are also available with enough resolution to locate significant voids or discontinuities, which would be beneficial in evaluating the vulnerability of the pipe penetrations. Another possible assessment tool is to televise the interior of the pipe(s) to assess for any damage or openings in the joints or accumulation of sediment FEMA has published a guidance document (FEMA 484, 2005) associated with conduits through dams that provides different assessment techniques that could potentially used in this case.

Should levee certification eventually be pursued by the City, the certifying firm or engineer would need to demonstrate that the existing pipe penetrations do not pose a threat to the structural integrity of the levee for up to a 100 year flood event. Documenting a positive performance history of the levee at the penetrations, along with a detailed inventory/survey of the penetrations, could prove to be beneficial in this regards. Nevertheless, the pipe penetrations do represent an unknown vulnerability that should be further investigated.

9. Crib Wall Replacement

The concrete crib wall located between Stations 2+60 and 3+90 was previously identified as a segment of the Salt Flats levee potentially requiring improvement (HDR, 2015). The wall spans the area between two closure structures associated with West Broadway Street and the Railroad Spur, as previously shown on Figures 3 and 14. It is founded on a shallow embankment, thus technically is considered a floodwall-levee enlargement.

9.1 Crib Wall Configuration

The concrete crib wall was constructed in approximately 1956 as part of the original Salt Flats levee construction. There is one known storm drain that extends underneath the crib wall, a 24-inch diameter reinforced concrete pipe. A schematic of the crib wall, based on the 1956 design drawings and recent site observations, is shown in Figure 20.

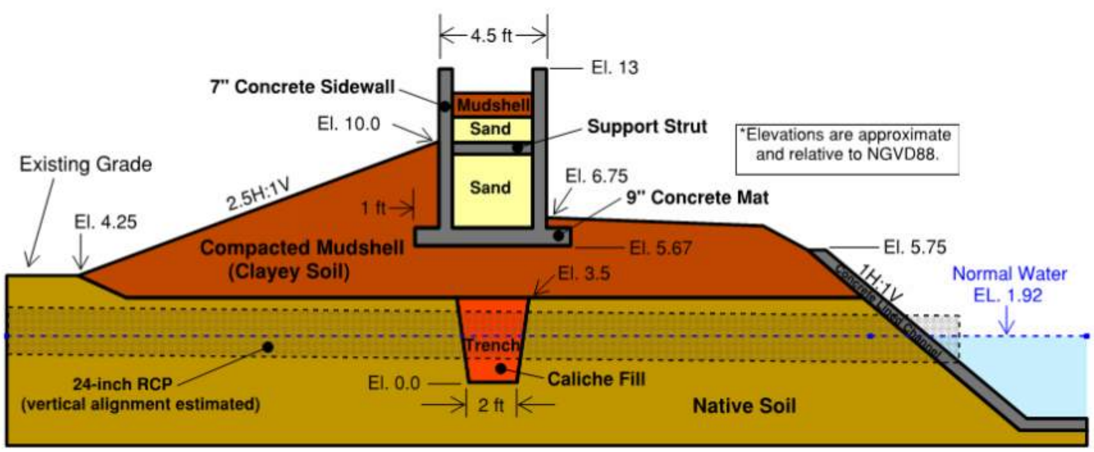


Figure 20 – *Schematic of Existing Crib Wall*

9.2 STRUCTURAL ASSESSMENT AND RECOMMENDATION

Assessment

Exposed portions of the crib wall were observed during a site visit on August 11, 2016. Based on these observations, there are multiple areas where corroded rebar is exposed to the elements. In addition to the exposed rebar, there are locations with large spalls and portions of concrete missing from the wall, most notably at the top corners.

Many of the expansion joints no longer have joint material in the gaps, and open gaps exist between many sections of the wall. In several of the gaps, there are small shrubs and vegetation growing in and through the expansion joint. The wall is no longer plumb with noticeable small bulges and distortions. Examples of the observed conditions of the wall are shown in the Figures 21 through 26.

Figure 21 – *Front Face (spalls/exposed rebar)*

Figure 22 – *Top of Wall (missing concrete)*

Figure 23 – *Backslope of Crib Wall*

Figure 24 – *Exposed Rebar (Front of Wall)*

Figure 25 – *Open Gap with Vegetation*

Figure 26 – *Top View (Missing Concrete)*

Conclusions and Recommendations

Based on the above assessment, the crib wall is in need of significant repairs to make it structurally adequate for a 100-year flood over the next 10 to 20 years. The condition of the crib wall appears to have extended beyond its serviceable life cycle, and making short-term repairs is unlikely to be cost effective. HDR recommends that the crib wall be removed and replaced with an equivalent structure that is structurally compatible with up to a 100-year flood condition, and having a renewed life cycle of at least 50 years.

9.3 Replacement Options

There are several potential options for replacing the existing crib wall. These include:

- Earthen Levee
- Inverted T-Wall
- I-Wall
- Crib Wall

Earthen Levee – Extending the existing foundation embankment up to El. +13 with compacted clay fill is potentially feasible. However, there are several limitations with this approach, as listed below:

1. The interior slopes of the earthen levee section would need to be approximately 1.4H:1V to achieve a crest centerline compatible with the current tie-in locations. This would be steeper than the current interior slopes and potentially unstable, plus difficult to maintain a vegetative cover.
2. Tying-in the levee material to adjacent bunker/wingwall could prove challenging. This would likely require a new vertical section of concrete extending several feet or more into the earthen levee on each side. Achieving adequate compaction of soil at the tie-in could be problematic.
3. Placing and compacting clay fill to construct the levee could be difficult in the confined work area.
4. Heavy construction loads would be imposed upon the adjacent concrete-lined channel during soil placement and compaction, potentially causing damage.

Inverted T-Wall – An inverted T floodwall ("T-Wall") is a reinforced concrete wall whose members act as wide cantilever beams in resisting hydrostatic pressures against the wall. Inverted T-Walls are suitable for floodwall and levee enlargements when walls are higher than 6 or 7 feet. An inverted T-Wall can be constructed in moderately tight confines. Tying into the existing concrete bunker and wingwall should be relatively straightforward.

I-Wall – An I-Wall is a vertical structure consisting of a row of steel sheet piles driven into the existing embankment and underlying native soils. The upper part would consist of reinforced concrete, which would cap the sheet pile. Driving sheet pile would conflict with the existing 24-inch storm drain that currently extends underneath the existing crib wall. Pile driving may also be difficult given the confined/ limited work area. In this case an I-Wall would likely to be more expensive than a T-Wall, and there would be complications with the existing 24-inch storm drain.

Crib Wall – Another crib wall could be constructed that could achieve a similar look and performance to the original crib wall. However, in this case the crib wall is considered a rather heavy structure that would likely be less cost effective than a T-Wall.

9.4 Recommended Wall Type and Conceptual Design

Based on the above considerations, HDR recommends the crib wall be replaced with a inverted T-Wall. A conceptual T-Wall design is provided in Figure 27.

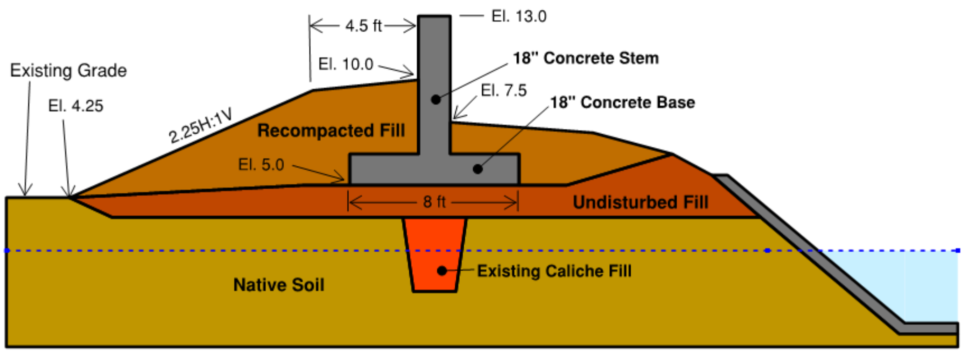


Figure 27 – *T-Wall Concept*

9.5 External Stability

As part of the present assessment, global stability and sliding analyses were performed on the conceptual design configuration shown in Figure 27 to help confirm or verify the T-Wall concept. During detailed design, the T-Wall would need to be analyzed in more detail for adequate factors of safety against these and other external modes of failure, including:

- Sliding
- Overturning
- Bearing Capacity
- Global/Slope Stability

Sliding Analysis

The sliding analysis used a peak flood elevation of +9.7 feet. Soil properties for the existing compacted mudshell and recompacted fill were assumed as follows:

- Unit Weight = 120 pcf
- Internal Angle of Friction = 30 degrees
- Effective Cohesion = 25 psf

The minimum factor of safety for an inland flood wall is 1.5, as referenced from the USACE Engineering Manual 1110-2-2502, “Retaining and Flood Walls” (1989). The results from this analysis indicated a factor of safety of 2.75 against sliding, thus satisfying the criteria. The analysis and results are provided in Appendix G.

Global/Slope Stability Analysis

The global/slope stability analysis utilized the same computer program, Slope/W, as previously described in Section 8.2. The soil stratigraphy was determined from boring B9, which was located near the outside toe of the existing crib wall (RETL, 2016). The properties for each soil layer were estimated based on the field and laboratory data associated with boring B9, as well information obtained from the earthen levee borings. A table of the various soil properties used in the analysis is provided with the results in Appendix G.

The minimum factor of safety for the T-Wall is 1.4 for a steady state seepage condition (the same criteria as discussed in Section 8.8). In this case, a separate steady state seepage analysis was not conducted. Instead the phreatic surface (or line of seepage) was conservatively assumed using the prolonged flood elevation of +8.0 feet on the channel side, and a tailwater water elevation of +5.75 feet on the land side.

The results of the analysis indicate a factor of safety of 2.69 against a deep seated global stability failure extending beneath the T-Wall. The computed factor of safety against a slope failure for the ballast soil on the landside of the wall (i.e. recompacted fill) is 1.60. In both cases the computed factor of safety satisfies the USACE criteria. Graphical outputs of the results are provided in Appendix G.

Conclusion

An evaluation of the results from sliding and global/slope stability analyses indicate the T-Wall could be designed to adequately resist all four modes of external failure.

9.6 SEEPAGE CONSIDERATIONS

Seepage could occur through the supporting embankment materials (mudshell) and underlying native soils during a prolonged flood event. However, based on the seepage analyses performed on the earthen levee, it is unlikely the exit gradients will be high enough to create a piping condition, especially if the backslope is not steepened and/or the toe elevation is not lowered. Still, this is a potential mode of failure that will need to be evaluated in more detail during final design

10. MISCELLANEOUS ITEMS

10.1 STOP LOG COVER PLATES

The stop logs and stop log support posts are stored in concrete structures, referred to as vaults. Based on limited field observations, the stop logs and stop-log support posts appeared to be in good condition.

In the following figure, a cover plate is shown for the hole where the stop log support posts are placed during installation. There are missing bolts in some locations, and other locations contained bolts that were stripped and difficult to remove. Although there are enough bolts to keep the cover plate in place, water and debris are able to enter the openings. Eventually, debris and corrosion from water can prevent the bolts from being replaced. HDR recommends cleaning the holes and replacing the cover bolts.

Figure 28 – *Cover Plate*

10.2 CLOSURE STRUCTURE SUPPORTS

In addition to the stop log cover plates, the top of the closure structure on the south side of East Port Avenue showed signs of delaminated concrete, which can be seen in Figure 29. HDR recommends that the top of the closure structure be repaired to extend the service life of the structure.

Figure 29 – *Delaminated Concrete Support*

10.3 UPSTREAM PLUGS (RAILROAD CROSSINGS)

There are two closure structures that allow rail (train) traffic to pass through the flood protection system. One closure is located between North Port Avenue and the Port of Corpus Christi property. The other closure structure is located directly north of West Broadway Street.

At each of the closure structures, the stop logs are intended to rest on a concrete foundation, which serves as the bottom seal for the closure system. Because there are rails that pass through the concrete foundations, there are gaps that would not be sealed when the stop logs and stop log support posts are in place. An example of one of these gaps is shown in Figure 30. The openings will allow flood waters to flow/leak underneath the stop log gate structure. There have been past attempts to "plug" these openings or gaps with infill material, as shown in Figure 31. HDR recommends neoprene-type plugs (or similar) be fabricated and placed in the openings to provide a better bottom seal for the stop-log gates. The plugs could be designed to be permanently secured in-place or inserted during installation of the stop-logs. The pros and cons of each approach should be evaluated during final design.

Figure 30 – *Gaps Adjacent to Railroad Tracks*

Figure 31- *Asphalt Infill Material*

10.4 FLAP GATES

There are 9 known flap gates located along the drainage outfall, as illustrated on Figure 32.

Figure 32 – *Known Flap Gate Locations*

In general, the metal flap gates are corroded and pitted, and likely no longer open and close as freely as desired. A photograph of a typical flap gate in provided as Figure 33.

Figure 33 – *Typical Flap Gate*

It is apparent the flap gates are nearing the end of their life cycle. The functionality of these relatively simple features are critical to the performance of the flood protection system and interior drainage system. HDR recommends eight of the flap gates be replaced, the exception being flap gate No. 1. Four of the proposed replacements are associated with 24-inch diameter RCP's, and the remaining four are associated with 36-inch diameter pipes. The 36-inch pipes include three reinforced concrete pipes and one HDPE pipe. Based on the 1956 levee design drawings, the reinforced concrete pipes were installed during levee construction. The HDPE pipe was installed in 2007 at the concrete headwall/basin area located near Station 7+00.

HDR recommends the metal flap gates be replaced with duckbill-type check valves (e.g. Tideflex Series 35), and that they be inspected annually. In this particular case, duckbill check valves are likely to be more reliable than metal flap gates.

10.5 Tree Removals

Tree roots have the potential to negatively impact the integrity of an earthen levee. As such, it is standard practice to perform routine maintenance to prevent trees from growing on and near the sideslopes. HDR previously identified several trees that should be removed from the Salt Flats Levee (HDR, 2015). These were identified as follows:

- Cluster of trees near the toe of levee at approximate Station 5+50.
- Dead tree stump on the riverside/channel slope at approximate Station 15+00.

Recent site observations reaffirmed the trees may pose a threat to levee integrity. No others trees were identified for removal during the recent site observations.

The trees should be cut and the root mass removed to the extent practical. Larger trees, such as greater than 24 inches in diameter, may have too extensive of a root system to be completely removed. In these cases, the extent of root removal may require field judgment.

All loose soils present within the resulting cavity should carefully excavated/removed. The resulting void should be scarified and filled with compacted cohesive fill. The final surface area should be re-vegetated with grass.

11. OPINION OF PROBABLE PROJECT COST

A conceptual-level Opinion of Probable Project Cost (OPPC) was developed for the recommended improvements to the Salt Flats Levee. The OPPC is based on design concepts, not detailed design, and should only be used for planning purposes. A summary of the OPPC is provided in Table 7.

Table 7 – *OPPC Summary*

Items	Description	Total
1	Remove and Replace 130-ft Crib Wall with Inverted T-Wall	$416,000
2	Repair Stop Log Cover Plates	$6,000
3	Repair Concrete Support at Closure Structure Near E. Port Ave	$7,000
4	Fabricate and Install Upstream Neoprene Plugs @ RR Crossings	$9,000
4	Remove and Replace Eight Flap Gates with Duckbill Check Valves	$175,000
5	Remove Trees and Repair Disturbed Area	$17,000
	Sub Total	$630,000
6	25% Construction Contingencies	$158,000
7	Survey, Geotechnical, Engineering Design, Permitting, and Construction Phase Services	$189,000
8	Allowance for Evaluating Concrete Headwall and Pipe Penetrations	$120,000
	Project Total	**$1,097,000**

12. RECOMMENDATIONS FOR FINAL DESIGN

Several improvements to the Salt Flats Levee have been recommended, and to the extent applicable, design concepts have been developed. Detailed design, development of construction plans and specifications, applications for any needed regulatory permits, and preparation of bid documents will need to be developed prior to implementing these improvements. A summary of the tasks recommended for inclusion during detailed design is provided below:

1. Develop design plans and specifications for demolishing the existing concrete crib wall between Stations 2+60 and 3+90, and replacing the crib wall with an inverted concrete T-Wall.
2. Develop a detailed work description for repairing the stop log cover plates.
3. Develop design detail(s) and specifications for repairing the concrete support at the closure structure near E. Port Ave.
4. Develop specifications and requirements for installing neoprene plugs at two railroad crossings.
5. Develop plan sheets and specifications for replacing eight flap gates with duckbill check values.
6. Develop plan sheets and requirements for removing the identified trees and repairing the disturbed areas.
7. During the course of completing the present assessment, the stability of the concrete headwall at Station 7+00 was identified as a potential vulnerability. In addition, the vulnerability of the known

pipe penetrations (and possibly additional unknown pipe penetrations) should be assessed in more detail. HDR recommends these two items be further evaluated to determine if future near-term improvements are needed.

13. FEMA ACCREDITATION PROCESS

It is understood by HDR that the City is not currently pursuing FEMA accreditation for the Salt Flats Levee, including its accreditation as a freeboard-deficient levee. Should accreditation be pursued in the future, evidence must be provided that demonstrates adequate levee design, operation, and maintenance systems are in place for the overall levee system. The demonstration must provide reasonable assurance of protection for up to the 100-year flood event.

Note that certification would not warrant that the levee system will protect against future flood events. Even with a certified and accredited flood protection system in place, a possibility of flooding caused by overtopping or other failure modes always exists. Floodplain management measures to reduce the consequences of this possibility, such as elevating structures, maintaining a current flood warning system and evacuation plan, and wisely managing floodplain development, are strongly advised.

Appendix H provides a checklist developed by FEMA that summarizes the design criteria, operation plan, interior drainage plan, maintenance plan, and overall certification requirements for levee accreditation. If the City eventually pursues accreditation for the Salt Flats Levee as a “freeboard deficient” system, all of these requirements would have to be met except for the minimum freeboard requirement. As summarized in Table 8, the current assessment provides a certain amount of support for items under the “Design Criteria” portion of FEMA’s checklist.

Table 8- Excerpts from FEMA's Checklist for Levee Accreditation (Refer to Appendix H for complete list).

FEMA Criteria 44 CFR 65.10(b)	Status for Salt Flats Levee
Freeboard. Minimum freeboard required 3 feet above the Base Flood Elevation (BFE) all along length, and an additional 1 foot within 100 feet of structures (such as bridges) or wherever the flow is restricted. Additional 0.5 foot at the upstream end of a levee. Coastal levees have special freeboard requirements (see 44 CFR 65.10(b)(1)(iii) and (iv)).	The Salt Flats Levee does not meet FEMA's freeboard requirements. However, under the LAMP process, the Salt Flats Levee could be accredited as a "Freeboard Deficient" system. Therefore the freeboard requirement would not apply.
Closures. All openings must be provided with closure devices that are structural parts of the system during operation and designed according to sound engineering practice.	Recommendations for improvement of closure devices are included as part of the current assessment. Additional analyses may be required during an actual certification process, including potential underseepage.
Embankment Protection. Engineering analyses must be submitted that demonstrate that no appreciable erosion of the levee embankment can be expected during the base flood, as a result of either currents or waves, and that anticipated erosion will not result in failure of the levee embankment or foundation directly or indirectly through reduction of the seepage path and subsequent instability.	Not included as part of the current assessment – would need to be documented under a future effort if levee accreditation is pursued.
Embankment and Foundation Stability Analyses. Engineering analyses that evaluate levee embankment stability must be submitted. The analyses provided must evaluate expected seepage during loading conditions associated with the base flood and must demonstrate that seepage into or through the levee foundation and embankment will not jeopardize embankment or foundation stability. An alternative demonstrating that the levee is designed and constructed for stability against loading conditions for Case IV as defined in the U.S. Army Corps of Engineers Engineer (USACE) Manual EM 1110-2-1913, *Design and Construction of Levees*, (Chapter 6, Section II), may be used.	Assessments completed between Stations 2+60 to 3+90 (crib wall) and 5+50 to 9+00 (earthen levee). [Note: FEMA could require a more detailed analysis justifying the prolonged flood elevation of 8.0 feet and tailwater elevation 5.75 feet.] Assessment of pipe penetrations or stability of concrete head wall at Station 7+00 are note part of current assessment. Both items would need to be assessed and documented under a future effort if levee accreditation is pursued.
Settlement Analyses. Engineering analyses must be submitted that assess the potential and magnitude of future losses of freeboard as a result of levee settlement and demonstrate that freeboard will be maintained. This analysis must address embankment loads, compressibility of embankment soils, compressibility of foundation soils, age of the levee system, and construction compaction methods. In addition, detailed settlement analysis using procedures such as those described in USACE Engineer Manual 110-1-1904, *Soil Mechanics Design – Settlement Analysis*, must be submitted.	Not included as part of the current assessment – would need to be documented under a future effort if levee accreditation is pursued. The levee has been in place for over 60 years, so from a practical standpoint all consolidation settlement has likely occurred. A technical memorandum stating why this requirement is not applicable may suffice. An exception would apply if a significant amount of material were added to the levee.
Interior Drainage. An analysis must be submitted that identifies the source(s) of such flooding, the extent of the flooded area, and, if the average depth is greater than 1 foot, the water-surface elevation(s) of the base flood. This analysis must be based on the joint probability of interior and exterior flooding and the capacity of facilities (such as drainage lines and pumps) for evacuating interior floodwaters.	Hydraulic modeling for interior drainage analysis was performed by HDR (2016b). Portions applicable to the levee analysis were applied to the current assessment.

14. LIMITATIONS

This report was developed by HDR for the City's explicit use. Use of this work product by others is at their own risk. The content included in this report is correct to the best of our knowledge and has been developed in accordance with the standard of care that is typical for a practitioner in this industry. The standard of care was followed for collection and analysis of data, and for calculations and modeling performed in support of this report. It is not meant to contain an exhaustive or complete evaluation of all potential or possible design alternatives. Any decisions that are made on the basis of this report are the responsibility of the owner. Decisions by the City should take into account the limitations and residual risks identified or documented in this report. HDR does not warrant or guarantee our work or recommendations.

Some of the information provided in this report was developed or provided by others. Except as specifically identified within this report, HDR has not performed independent validation or verification of exploration data, modeling data, or other analysis on data provided by others.

The conclusions and recommendations in this report are based on the conditions of the project site at the time of this study. Any modifications to the site, man-made or natural, could alter the analysis, findings, and recommendations contained herein and could result in the analysis, findings, and recommendations to no longer be valid. Site conditions, climate changes, vegetation changes, maintenance practice changes, or other factors may change over time. Additional analysis or updates may be required in the future as a result of these changes. Parties other than the City for whom this work was developed under contract, must notify HDR if they would like to use this report for any purpose. HDR may request that additional work be performed and that an updated report be issued.

15. REFERENCES

HDR, 2015. Salt Flats Levee and Museum Floodwall Assessment, Phase 2A Final Report. Prepared for City of Corpus Christi (Project No. E12070), HDR Project 220658, 62 pages (plus appendices).

HDR, 2016a. Technical Memo, Task Order No. 2 – Flood Protection System Vulnerabilities Identification. Memo to Jeff Edmonds, City of Corpus Christi, dated February 12, 2016. Salt Flats Levee System, Phase 2 (Project No. E12070), HDR Project 10026049, 33 pages.

HDR, 2016b. Technical Memo, Task Order No. 3 – Final 2D Hydraulic Model of the Downtown Seclusion Area. Memo to Daniel Deng and Jeff Edmonds, City of Corpus Christi, dated November 30, 2016. Salt Flats Levee System, Phase 2 (Project No. E12070), HDR Project 10031510, 42 pages (plus appendices).

HDR, 2016c. Preliminary Design Memorandum (Draft).– Replacement of the Museum Floodwall. Salt Flats Levee System, Phase 2, Task Order 5 (City Project No. E12070). HDR Project 10026049, 28 pages (plus appendices). Draft report dated December 14, 1016.

Rock Engineering and Testing Laboratory, 2011. Subsurface Investigation, Laboratory Testing Program and Geotechnical Recommendations for the Proposed Salt Flats Drainage Levee Project, Corpus Christi, Texas. City Project No. 3428. RETL Job No. G111102. Report to Mr. Pete Anaya, City of Corpus Christi, dated October 17, 2011, 12 pages plus appendices.

Rock Engineering and Testing Laboratory, 2016. Subsurface Investigation and Laboratory Testing Program for the Proposed Unanticipated Storm Water Capital Requirements (E12193), Stroman Road and Mesquite Street and Harbor Drive, Corpus Christi, Texas. RETL Report No. G116175-REV1. Letter report to Mr. Jeff Edmonds, City of Corpus Christi, Dated November 3, 2016, 8 pages plus appendices.

Urban Engineering, 2012. Salt Flats Levee System (Phase 2), FEMA Certification Guidance Document. Prepared for City of Corpus Christi (Project 3428), Urban Job No. 36688.B0.00, 150 pages.

A

Site Topographic Map

(Urban Engineering)

General Notes:

1.) Grid Bearings and Distances shown hereon are referenced to the Texas Coordinate System of 1983, Texas South Zone 4205, and are based on the North American Datum of 1983(2011) Epoch 2010.00.

2.) Elevations shown hereon are referenced to the North American Vertical Datum of 1988 (NAVD88), Geoid 12A.

3.) Some features shown on this Survey may be out of scale for clarity.

4.) This Survey was prepared from field data obtained on August 8, 2016.

5.) This map was prepared without the benefit of a current title commitment. The surveyor has made no investigation or independent search for easements of record, encumbrances, restrictive covenants, ownership of title evidence or any other facts that an accurate title search may disclose.

Borehole Locations

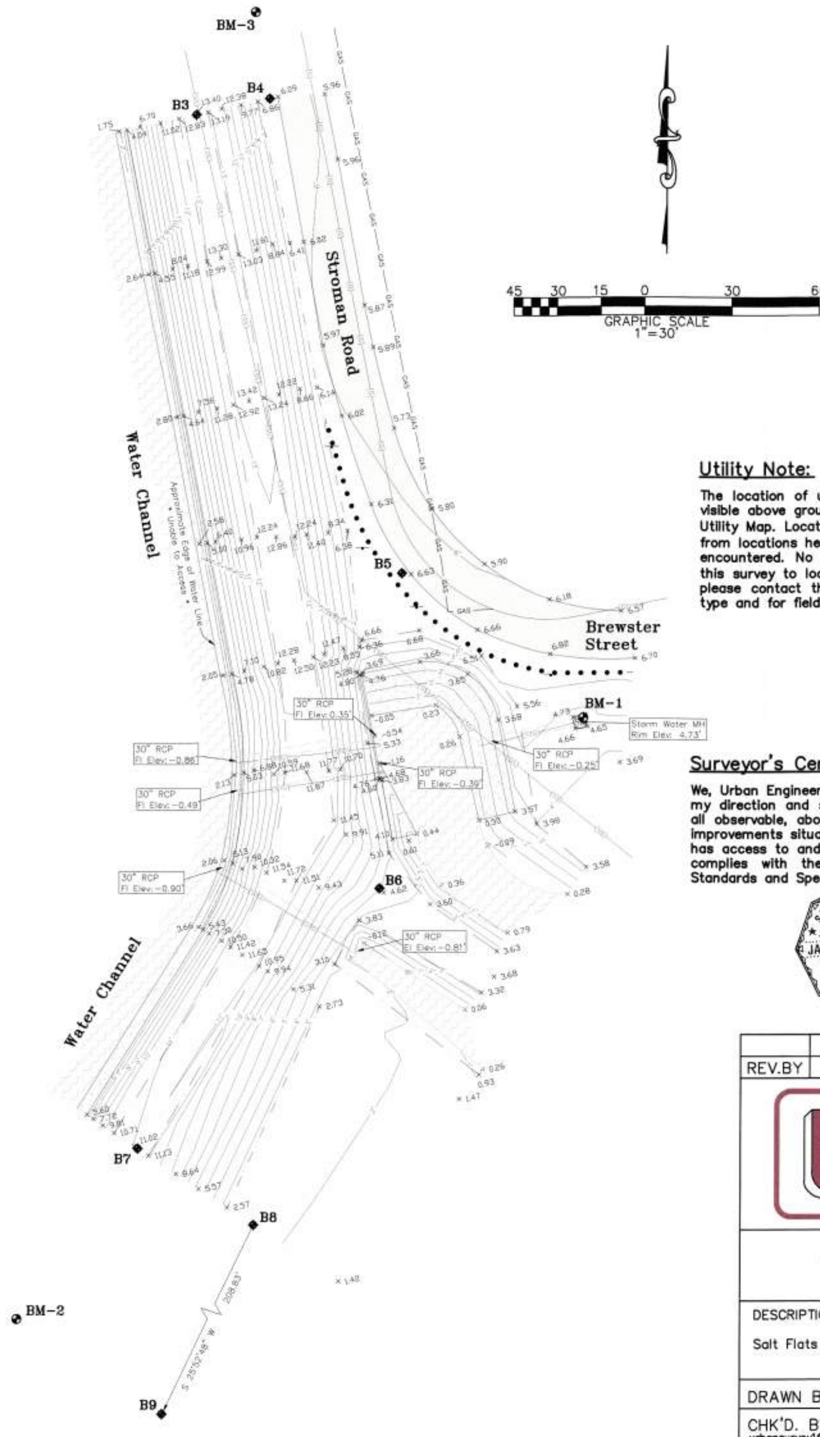
Point	Northing	Easting	Elevation
B3	17183822.06'	1338101.73'	13.21'
B4	17183827.62'	1338126.91'	6.27'
B5	17183664.95'	1338172.75'	6.73'
B6	17183556.94'	1338165.11'	4.81'
B7	17183467.55'	1338082.24'	11.16'
B8	17183441.28'	1338122.12'	1.63'
B9	17183253.39'	1338030.96'	4.43'

Site Benchmark / Control

Point	Northing	Easting	Elevation	Description
BM-1	17183615.43'	1338235.07'	4.74'	"□" Set on Concrete
BM-2	17183408.81'	1338040.90'	12.68'	"□" Set on Concrete
BM-3	17183857.39'	1338121.94'	6.11'	5/8 Inch Iron Rod Set

COUNTY OF NUECES
CITY OF CORPUS CHRISTI

LOCATION MAP N.T.S.



LEGEND:

- Site Benchmark/Control
- Borehole
- Sign
- Storm Water Manhole
- Guard Rail Fence
- Top of Bank
- Toe of Slope
- Gas Line (As Marked)
- Gas Line (City GIS)
- Storm Water Line (City GIS)
- Sanitary Sewer Line (City GIS)
- Concrete
- Asphalt Road
- Water

Utility Note:

The location of underground utilities shown hereon are based on visible above ground structures and the City of Corpus Christi GIS Utility Map. Locations of underground utilities/structures may vary from locations hereon. Additional buried utilities/structures may be encountered. No excavations were made during the progress of this survey to locate buried utilities/structures. Before excavation, please contact the appropriate agencies for verification of utility type and for field location.

Surveyor's Certificate:

We, Urban Engineering, have made an on the ground field survey, under my direction and supervision, of the property legally described hereon; all observable, aboveground evidence of buildings, structures and other improvements situated on the premises have been shown; said property has access to and from a dedicated roadway. This Survey substantially complies with the current Texas Society of Professional Surveyors Standards and Specifications for a Category 6, Condition II Land Survey.

STATE OF TEXAS REGISTERED JAMES DAVID CARR 6458 PROFESSIONAL LAND SURVEYOR

URBAN ENGINEERING

James D. Carr, R.P.L.S.
License No. 6458

REV.BY	DATE	DESCRIPTION	APPROVED

URBAN ENGINEERING
TBPE FIRM NO. 145, TBPLS FIRM NO. 10032400
2725 SWANTNER DR. CORPUS CHRISTI, TX 78404
PHONE: 361.854.3101 WWW.URBANENG.COM

TOPOGRAPHIC SURVEY

DESCRIPTION:

Salt Flats Levee Boring Locations and Cross Sections.

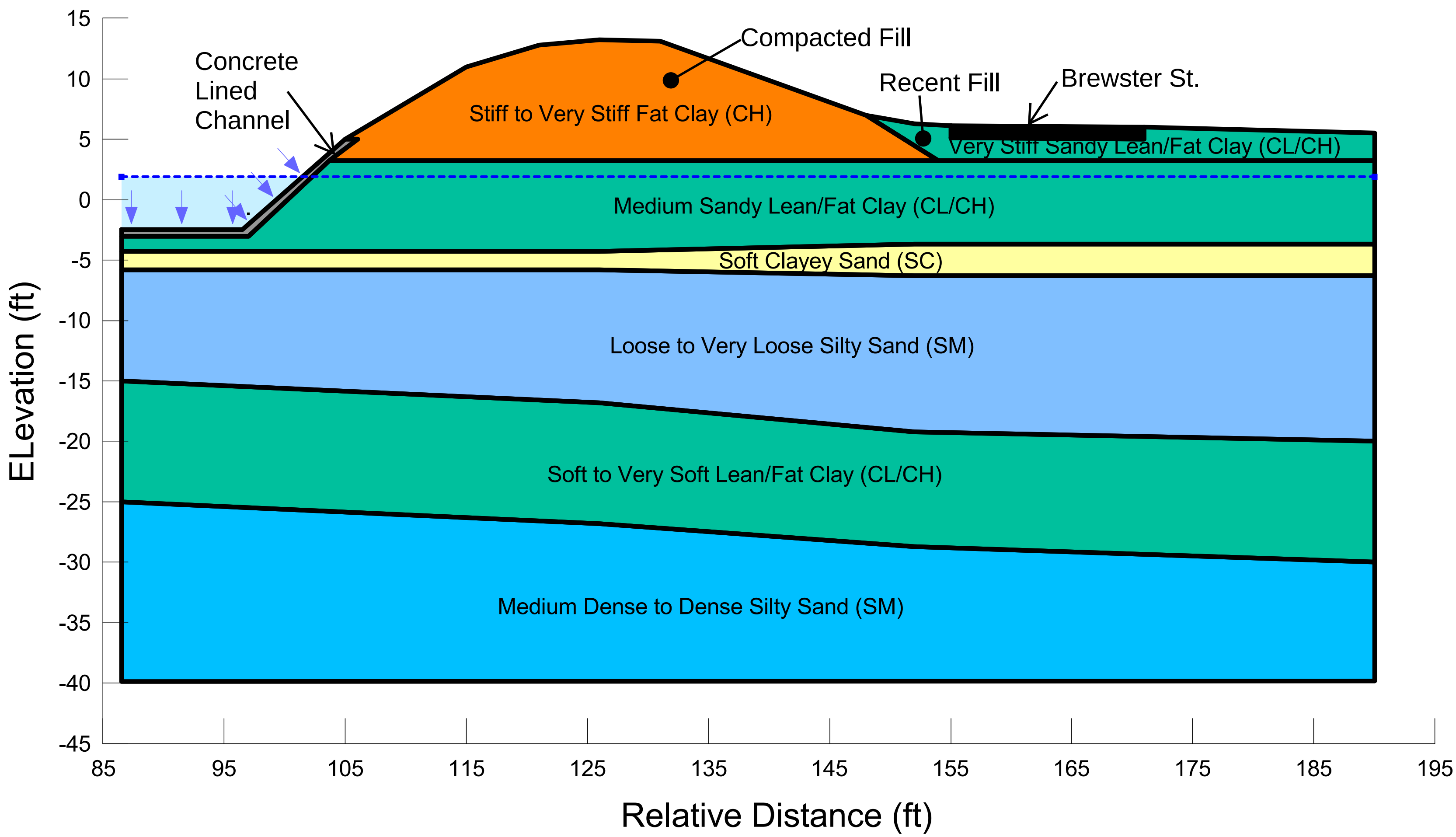
DRAWN BY: CDS SCALE: 1"= 30' JOB NO: 36688.B600

CHK'D. BY: ____ DATE: 08/10/16 SHEET: 1 OF: 1
urbansurvey1@urbaneng.com

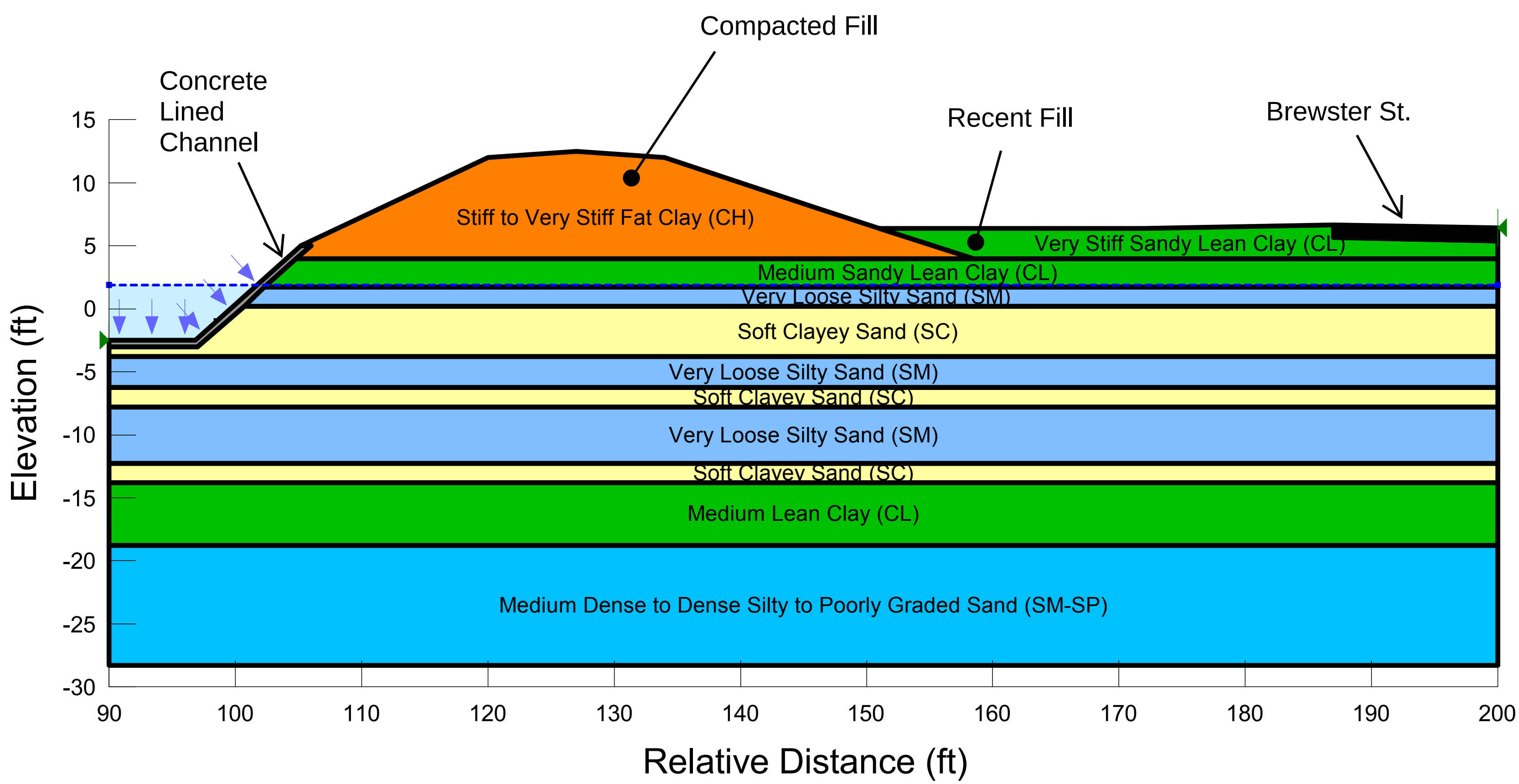
B

Fully Developed Cross Sections of Earthen Levee (Study Area)

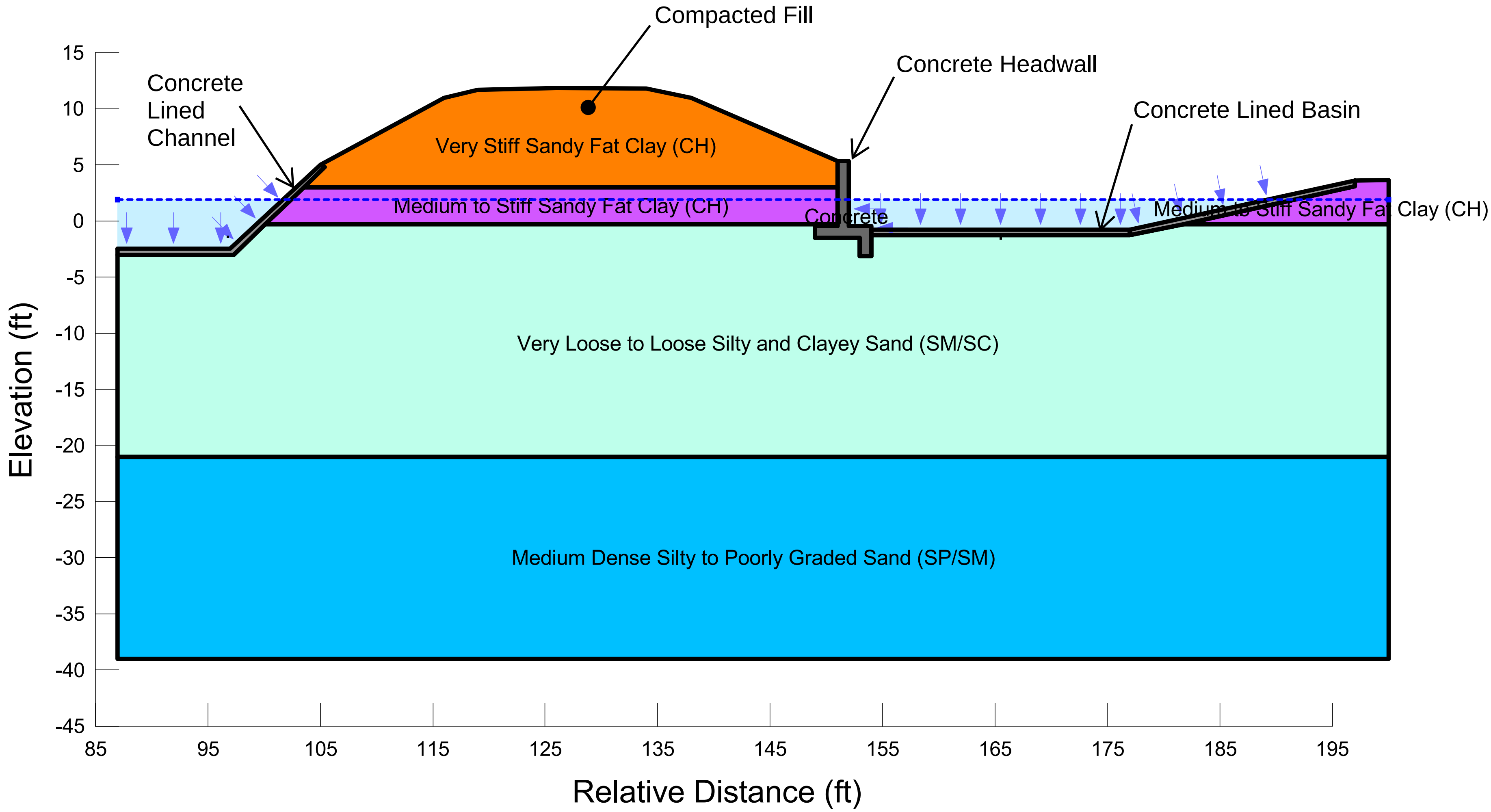
Salt Flats Levee
Soil Stratigraphy
Cross Section AA (Sta 9+00)



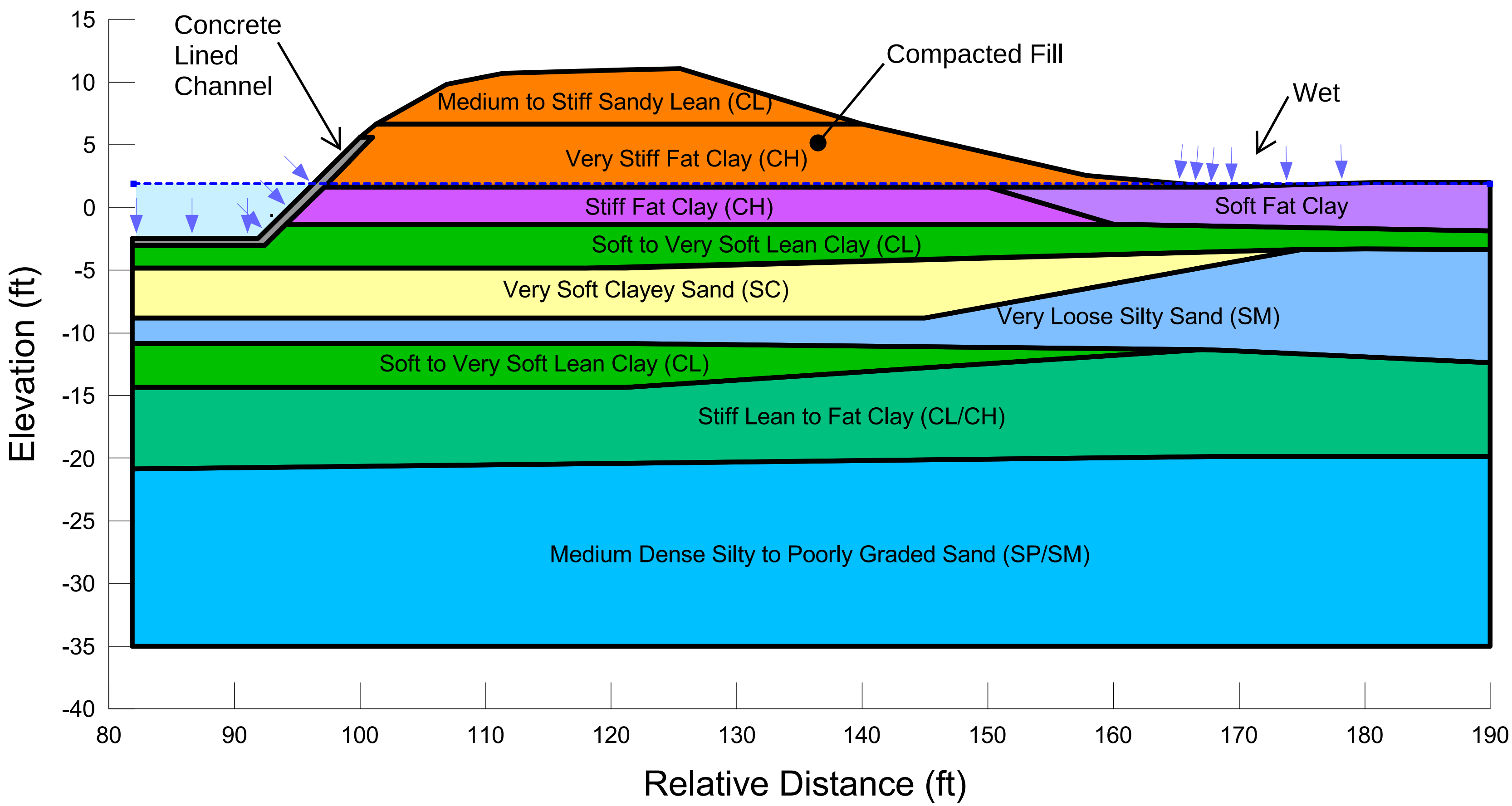
Salt Flats Levee
Soil Stratigraphy
Cross Section BB (Sta 7+75)



Salt Flats Levee
Soil Stratigraphy
Cross Section CC (Sta 7+10)



Salt Flats Levee
Soil Stratigraphy
Cross Section DD (Sta 5+65)



C

Soil Properties of Earthen Levee and Substratum

Soil Properties – Cross Section AA

Reference Borings Embankment: B3, B7, B6 (2011), B8 (2011)

Reference Borings Substrata: B3, B4, B5 (2011), B6 (2011)

Stratum	N	PP (tsf)	Uc (tsf)	PI	P200 (%)	Unit Weight (pcf)	Undrained Strength S_u (psf)	Drained Strength		Undrained Strength		Permeability (cm/sec)		Vol. Moist. (%)
								Φ_d (deg)	C_d (psf)	Φ_u (deg)	c_u (psf)	k_v (cm/sec)	K_h/k_v (cm/sec)	
Stiff to Very Stiff Fat Clay (CH)	10	2.75	4.2	42	63	120	1750	30	150	0	1750	2EE-06	4	40
Stiff to Very Stiff Sandy Lean to Fat Clay (CL/CH)	10	--	--	31	56	120	1500	32	100	0	1500	5EE-06	4	40
Medium Sandy Lean to Fat Clay (CL/CH)	8	1.30	--	40	--	115	750	28	100	0	750	2EE-06	4	45
Soft Clayey Sand (SC)	3	--	<0.5	26	36	114	300	26	75	12	200	1EE-05	3	45
Loose to Very Loose Silty Sand (SM)	4	--	--	--	20	112	0	28	0	22	0	1EE-04	2	45
Soft to Very Soft Lean to Fat Clay (CL/CH)	1.5	--	-	22	--	110	250	27	50	0	200	5EE-06	4	45
Medium Dense to Dense Silty Sand (SM)	26	--	--	--	17	120	0	32	0	22	0	1EE-04	2	40
Concrete Channel Lining	--	--	--	--	--	150	4500	--	4500	0	4500	1EE-06	1	40

Soil Properties – Cross Section BB

Reference Borings Embankment: B3, B7, B6 (2011), B8 (2011)

Reference Borings Substrata: B5, B6

Stratum	N	PP (tsf)	Uc (tsf)	PI	P200 (%)	Unit Weight (pcf)	Undrained Strength S_u (psf)	Drained Strength		Undrained Strength		Permeability (cm/sec)		Vol. Moist. (%)
								Φ_d (deg)	C_d (psf)	Φ_u (deg)	c_u (psf)	k_v (cm/sec)	K_h/k_v (cm/sec)	
Stiff to Very Stiff Fat Clay (CH)	10	2.75	4.2	42	63	120	1750	30	150	0	1750	2EE-06	4	40
Very Stiff Sandy Lean to Fat Clay (CL/CH)	23	--	--	27	52	120	1500	32	100	0	1500	2EE-06	4	40
Medium Sandy Lean Clay (CL)	5	--	--	27	52	115	500	28	100	0	500	5EE-06	4	45
Soft Clayey Sand (SC)	2	<1.0		14	43	114	300	26	75	12	200	1EE-05	3	45
Very Loose Silty Sand (SM)	4	--	--	--	24	112	0	27	0	20	0	1EE-04	2	45
Medium Lean Clay (CL)	7	<2.0	-	14	--	115	650	28	100	0	650	5EE-06	4	45
Medium Dense to Dense Silty to Poorly Graded Sand (SM-SP)	28	--	--	--	16	120	0	32	0	22	0	5EE-04	2	40
Concrete Channel Lining	--	--	--	--	--	150	4500	--	4500	0	4500	1EE-06	1	40

Soil Properties – Cross Section CC

Reference Borings Embankment: B3, B7, B8 (2011)

Reference Borings Substrata: B5, B6, B8 (2011)

Stratum	N	PP (tsf)	Uc (tsf)	PI	P200 (%)	Unit Weight (pcf)	Undrained Strength S_u (psf)	Drained Strength		Undrained Strength		Permeability (cm/sec)		Vol. Moist. (%)
								Φ_d (deg)	C_d (psf)	Φ_u (deg)	c_u (psf)	k_v (cm/sec)	K_h/k_v (cm/sec)	
Very Stiff Sandy Fat Clay (CH)	11	>4	3.4	>35	72	120	1750	30	150	0	1750	2EE-06	4	40
Medium to Stiff Sandy Fat Clay (CH)	--	1.8	1.5	>35	52	117	800	29	100	0	800	5EE-06	4	45
Very Loose to Loose Silty Sand and Clayey Sand (SM/SC)	3	--	--	<9	24	113	--	27	25	18	25	5EE-05	2.5	45
Medium Dense to Dense Silty to Poorly Graded Sand (SM/SP)	>25	--	--	--	16	120	0	32	0	22	0	5EE-04	2	40
Concrete Channel Lining	--	--	--	--	--	150	4500	0	4500	0	4500	1EE-06	1	40
Concrete Headwall	--	--	--	--	--	150	4500	0	4500	0	4500	NA	NA	NA
Concrete Basin Lining	--	--	--	--	--	150	3000	0	3000	0	3000	1EE-09	1	0

Soil Properties – Cross Section DD

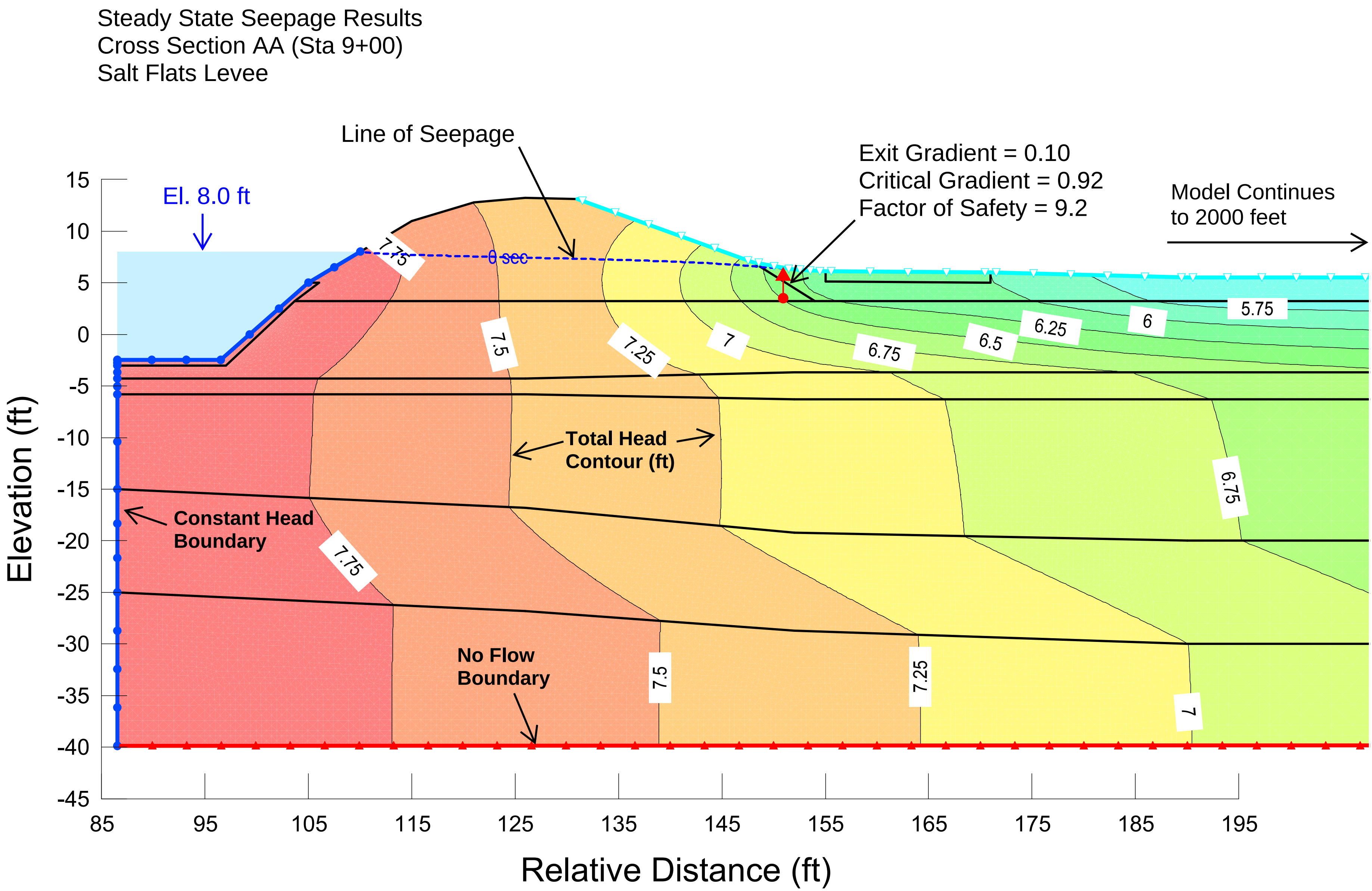
Reference Borings Embankment: B7, B8 (2011)

Reference Borings Substrata: B6, B7, B8

Stratum	N	PP (tsf)	Uc (tsf)	PI	P200 (%)	Unit Weight (pcf)	Undrained Strength S_u (psf)	Drained Strength		Undrained Strength		Permeability (cm/sec)		Vol. Moist. (%)
								Φ_d (deg)	C_d (psf)	Φ_u (deg)	c_u (psf)	k_v (cm/sec)	K_h/k_v (cm/sec)	
Medium to Stiff Sandy Lean Clay (CL)	8	--	--	33	53	118	1000	30	100	0	1000	5EE-06	4	40
Very Stiff Sandy Fat Clay (CH)	--	3.8	3.3	37	62	120	1750	30	150	0	1750	2EE-06	4	40
Stiff Fat Clay (CH)	--	1.5	--	--	--	117	1000	29	100	0	1000	2EE-06	4	45
Soft Fat Clay (CH)	3	--	--	42	85	112	300	26	50	0	300	5EE-06	4	45
Soft to Very Soft Lean Clay (CL)	1.5	<1.0	0.60	--	--	110	200	25	50	0	200	5EE-06	4	45
Very Soft Clayey Sand (SC)	WOH	--	--	31	37	112	100	25	50	12	100	5EE-05	3	45
Very Loose Silty Sand (SM)	1	--	--	--	16	112	0	27	0	20	0	1EE-04	2	45
Stiff Lean to Fat Clay (CL/CH)	8	2.0	1.7	34	92	117	1200	30	75	0	1200	1EE-06	4	45
Medium Dense Silty to Poorly Graded Sand (SP-SM)	30	--	--	--	--	120	0	32	0	22	0	1EE-04	2	40
Concrete Channel Lining	--	--	--	--	--	150	4500	0	4500	0	4500	1EE-06	1	0

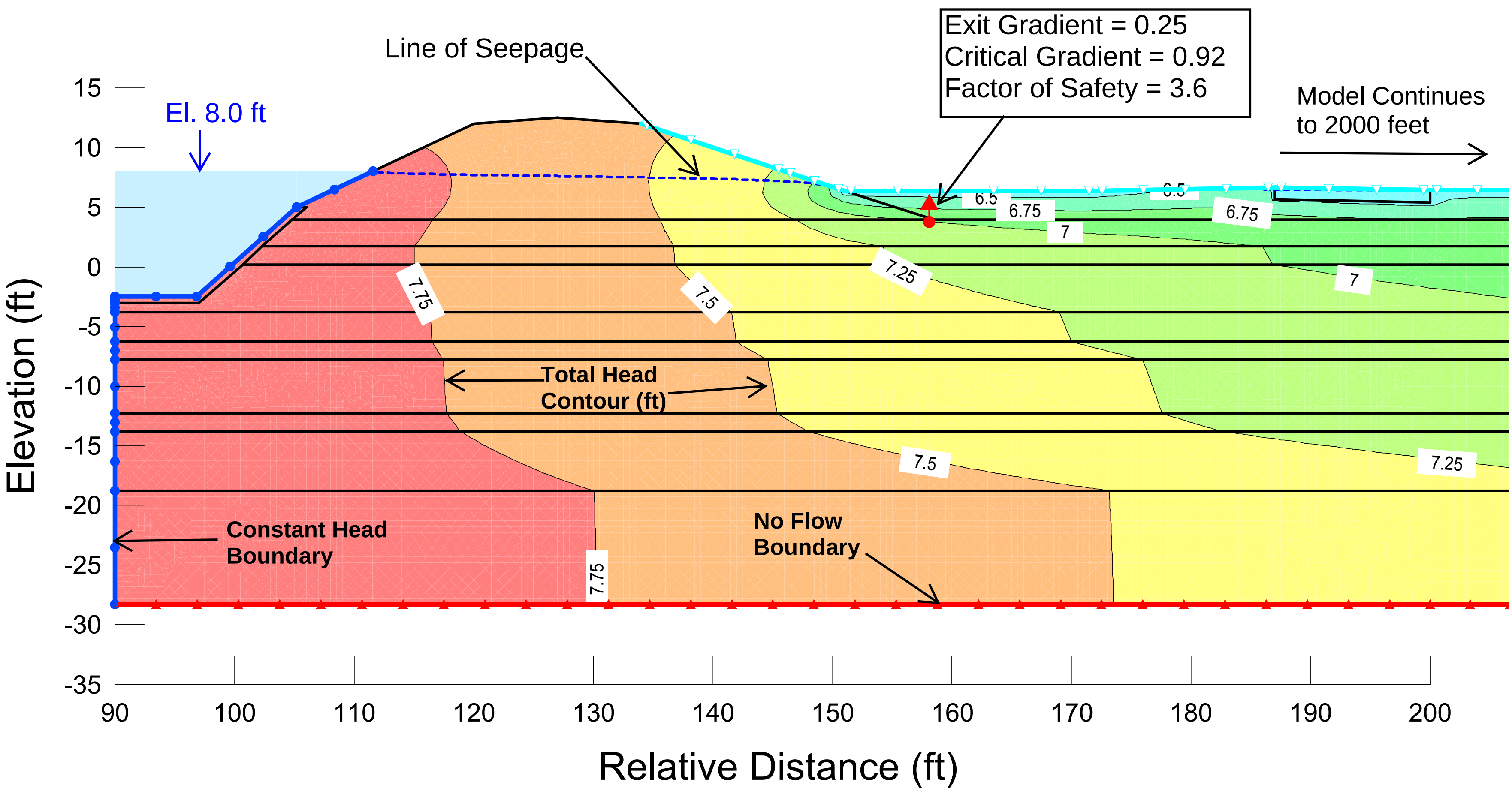
D

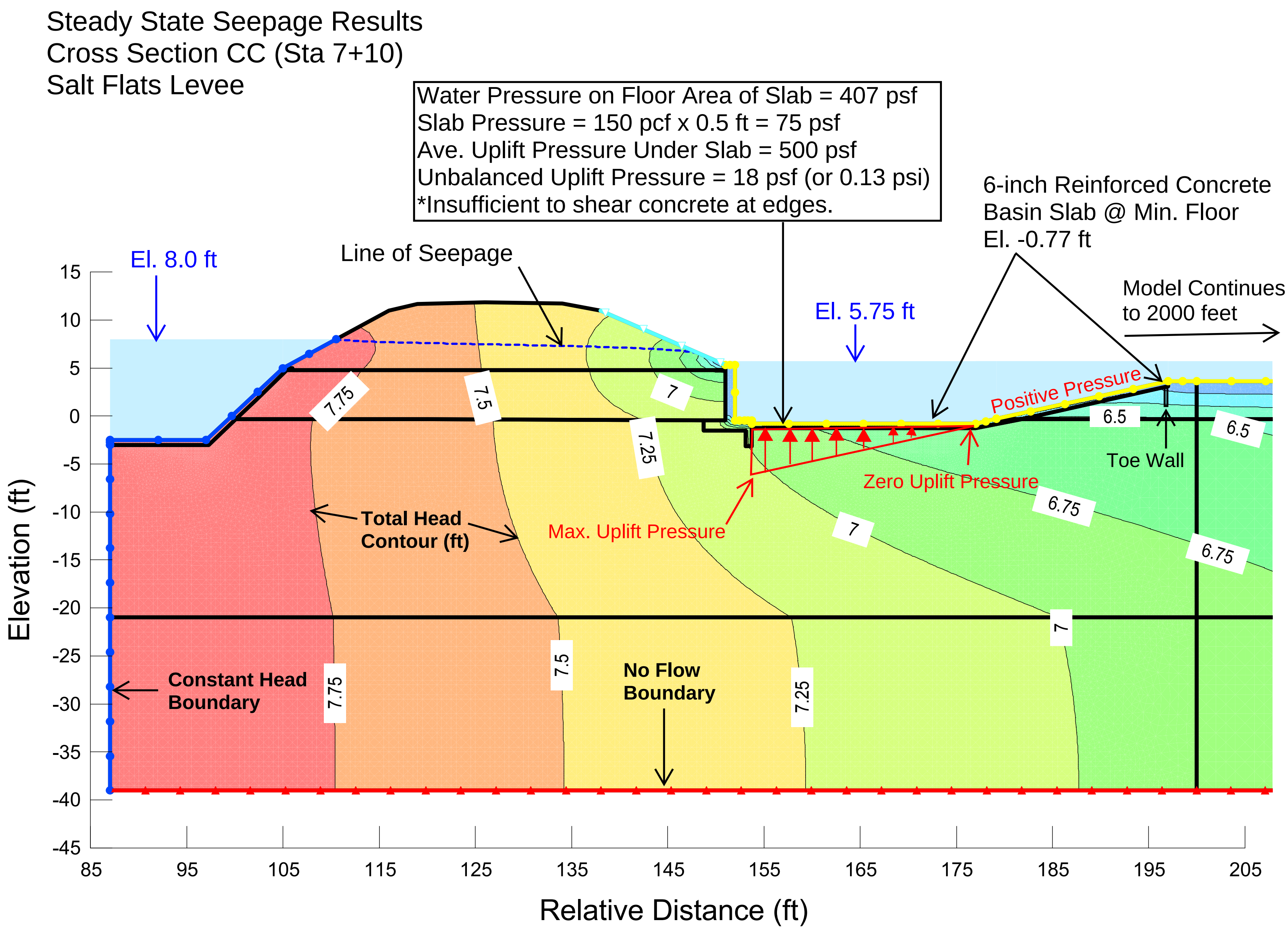
Seepage and Piping Results for Earthen Levee



Steady State Seepage Results
Cross Section AA (Sta 9+00)
Salt Flats Levee
Line of Seepage
El. 8.0 ft
Exit Gradient = 0.10
Critical Gradient = 0.92
Factor of Safety = 9.2
Model Continues
to 2000 feet
Total Head
Contour (ft)
Constant Head
Boundary
No Flow
Boundary
7.75
7.5
7.25
7
6.75
6.5
6.25
6
5.75
Elevation (ft)
Relative Distance (ft)

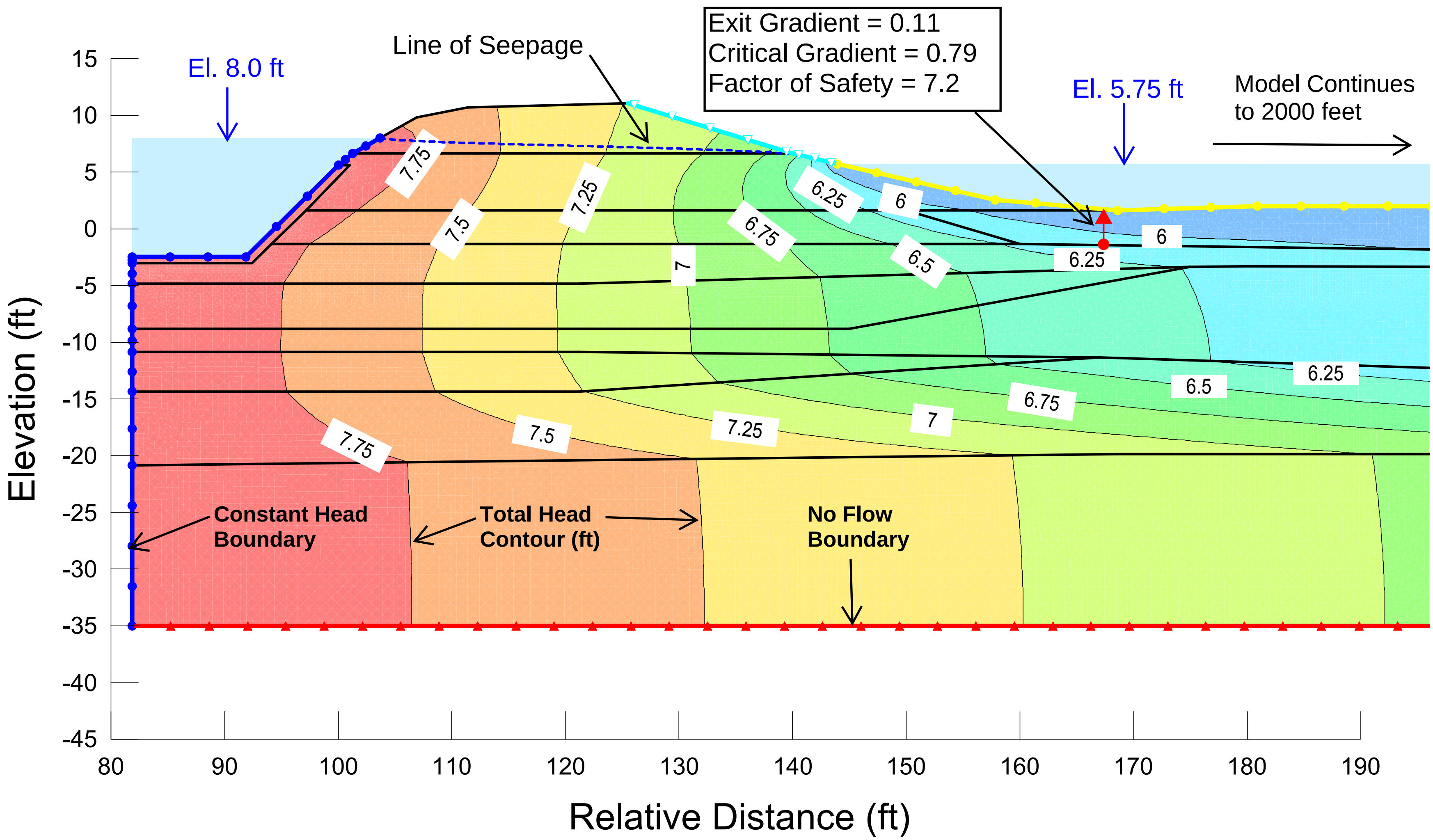
Steady State Seepage Results
Cross Section BB (Sta 7+75)
Salt Flats Levee



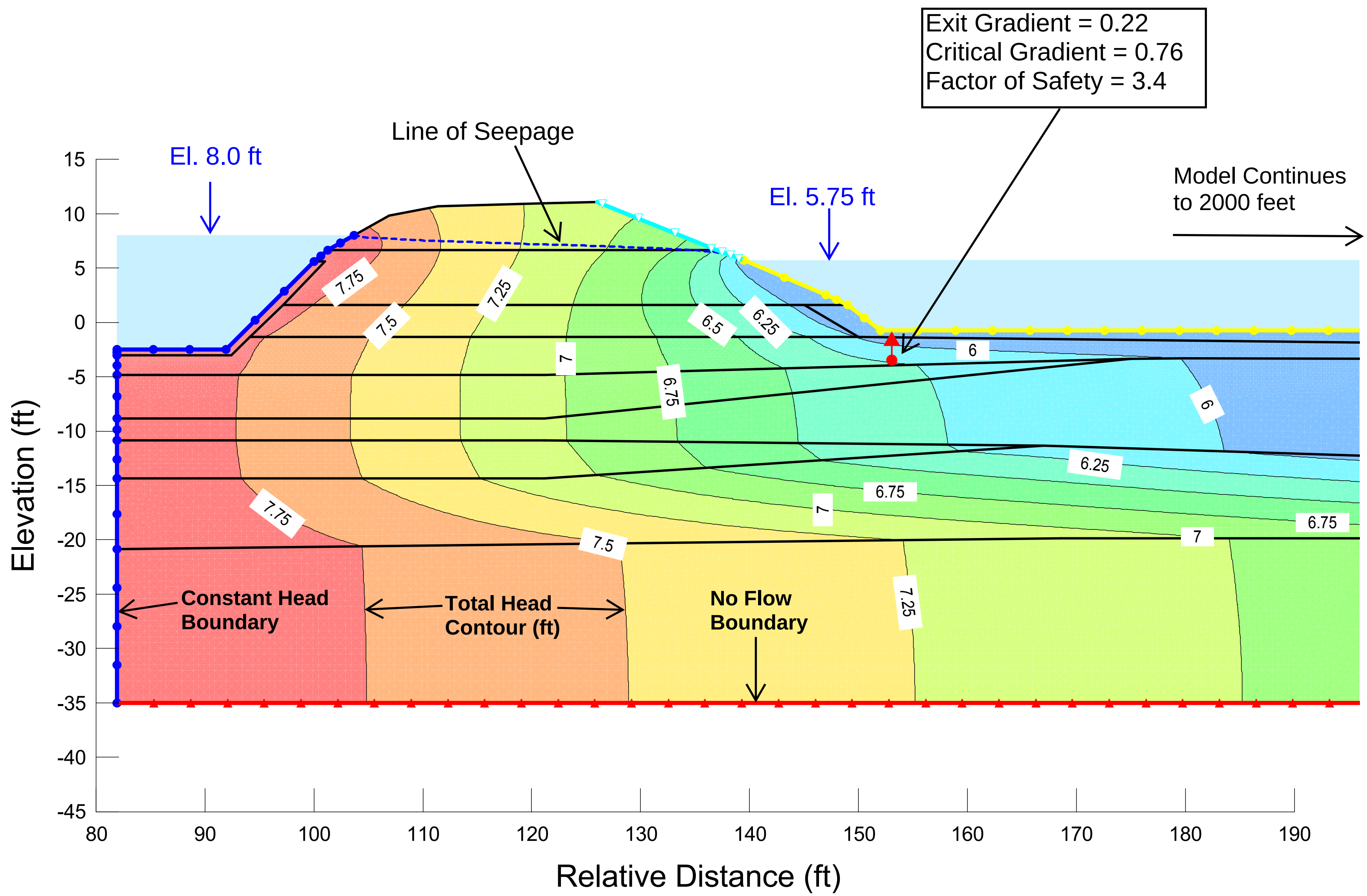


Steady State Seepage Results
Cross Section CC (Sta 7+10)
Salt Flats Levee
Water Pressure on Floor Area of Slab = 407 psf
Slab Pressure = 150 pcf x 0.5 ft = 75 psf
Ave. Uplift Pressure Under Slab = 500 psf
Unbalanced Uplift Pressure = 18 psf (or 0.13 psi)
*Insufficient to shear concrete at edges.
6-inch Reinforced Concrete
Basin Slab @ Min. Floor
El. -0.77 ft
Model Continues
to 2000 feet
El. 8.0 ft
Line of Seepage
El. 5.75 ft
Positive Pressure
Toe Wall
Zero Uplift Pressure
Max. Uplift Pressure
Total Head
Contour (ft)
Constant Head
Boundary
No Flow
Boundary
7.75
7.5
7.25
7
6.75
6.5
Elevation (ft)
Relative Distance (ft)
15
10
5
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
85
95
105
115
125
135
145
155
165
175
185
195
205

Steady State Seepage Results
Cross Section DD (Sta 5+50)
Salt Flats Levee



Steady State Seepage Results
Cross Section DD (modified)
Salt Flats Levee

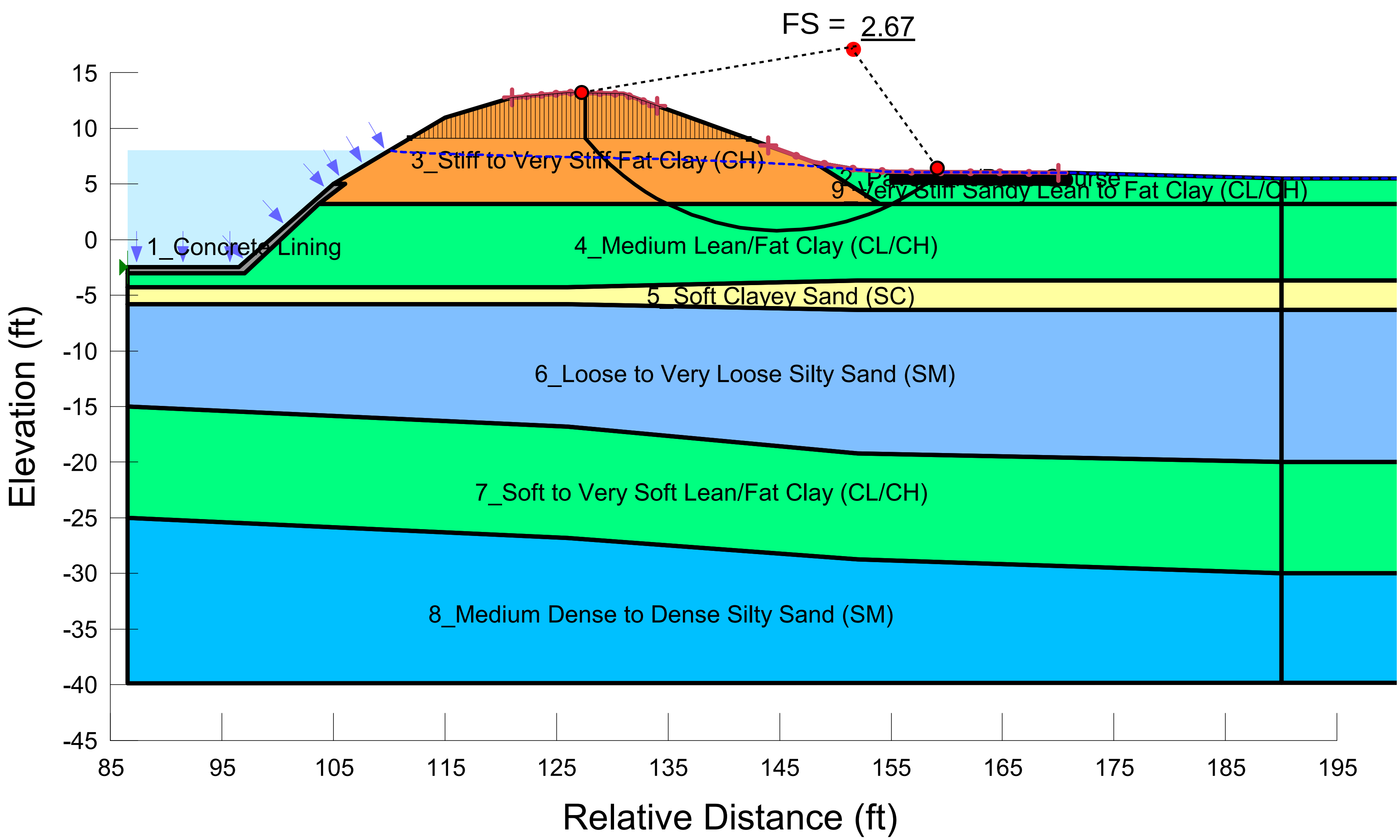


E

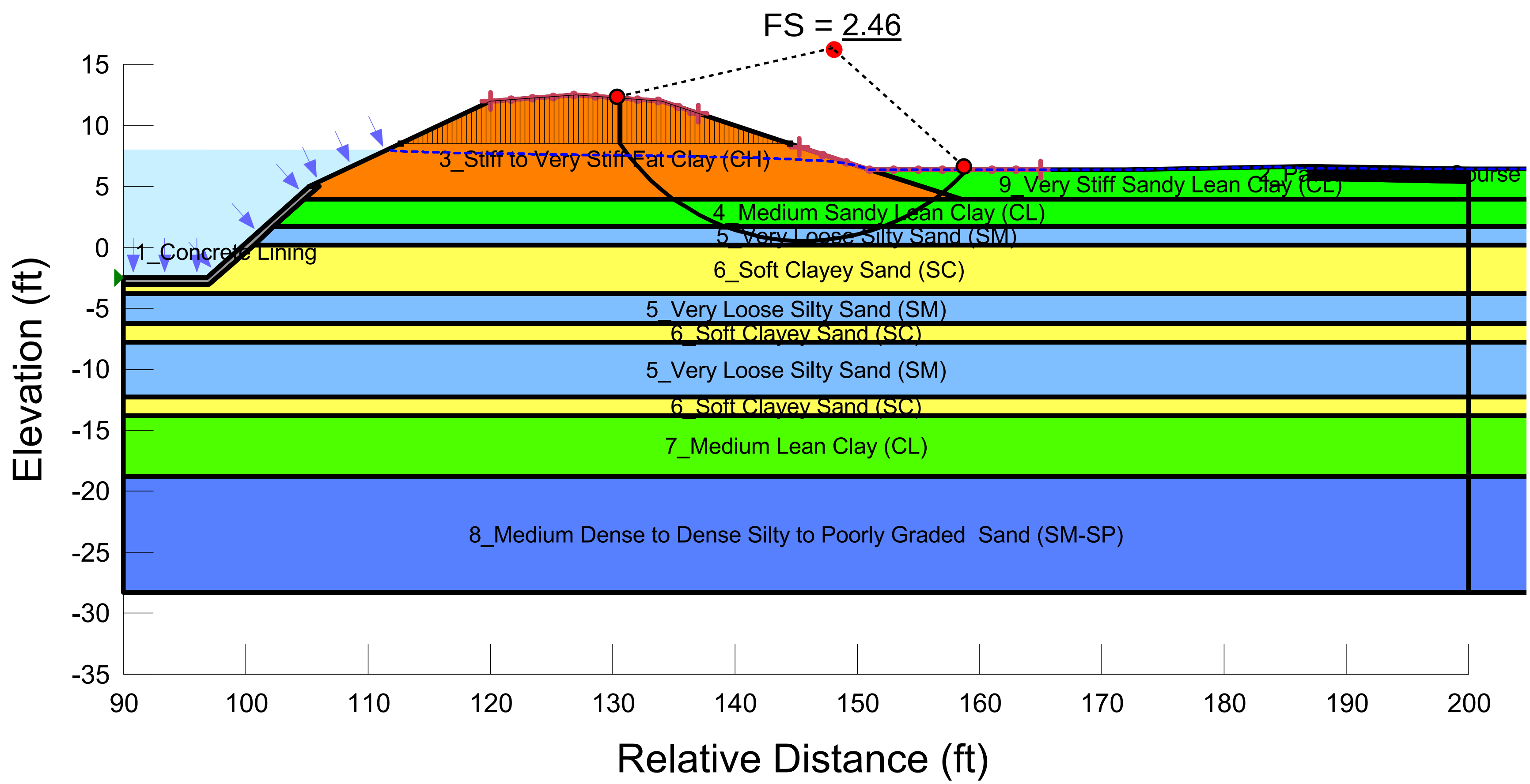
Slope Stability Results for Earthen Levee

(Steady State Seepage)

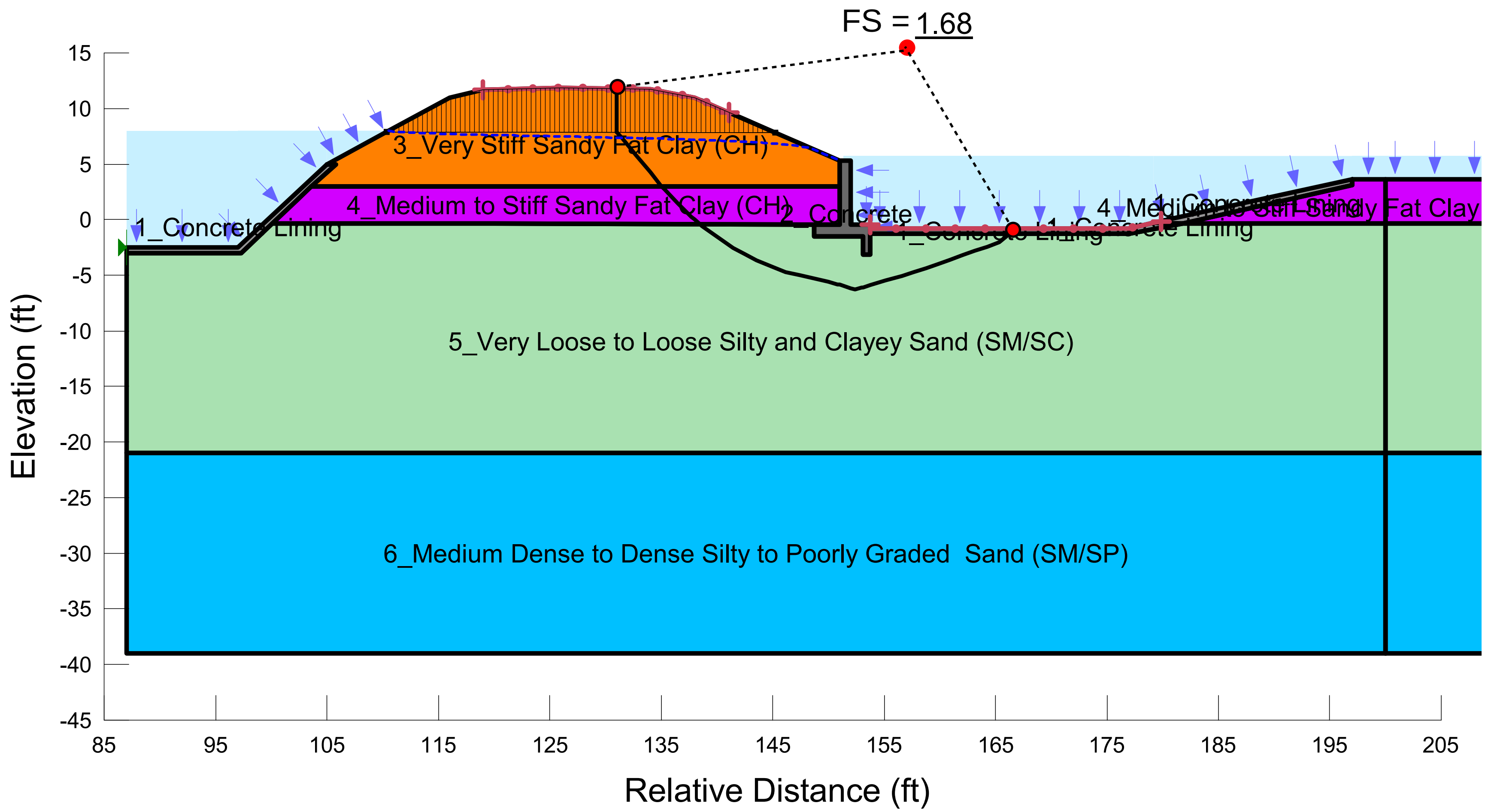
Slope Stability Analysis
Steady State Seepage
Cross Section AA (Sta. 9+00)
Salt Flats Levee



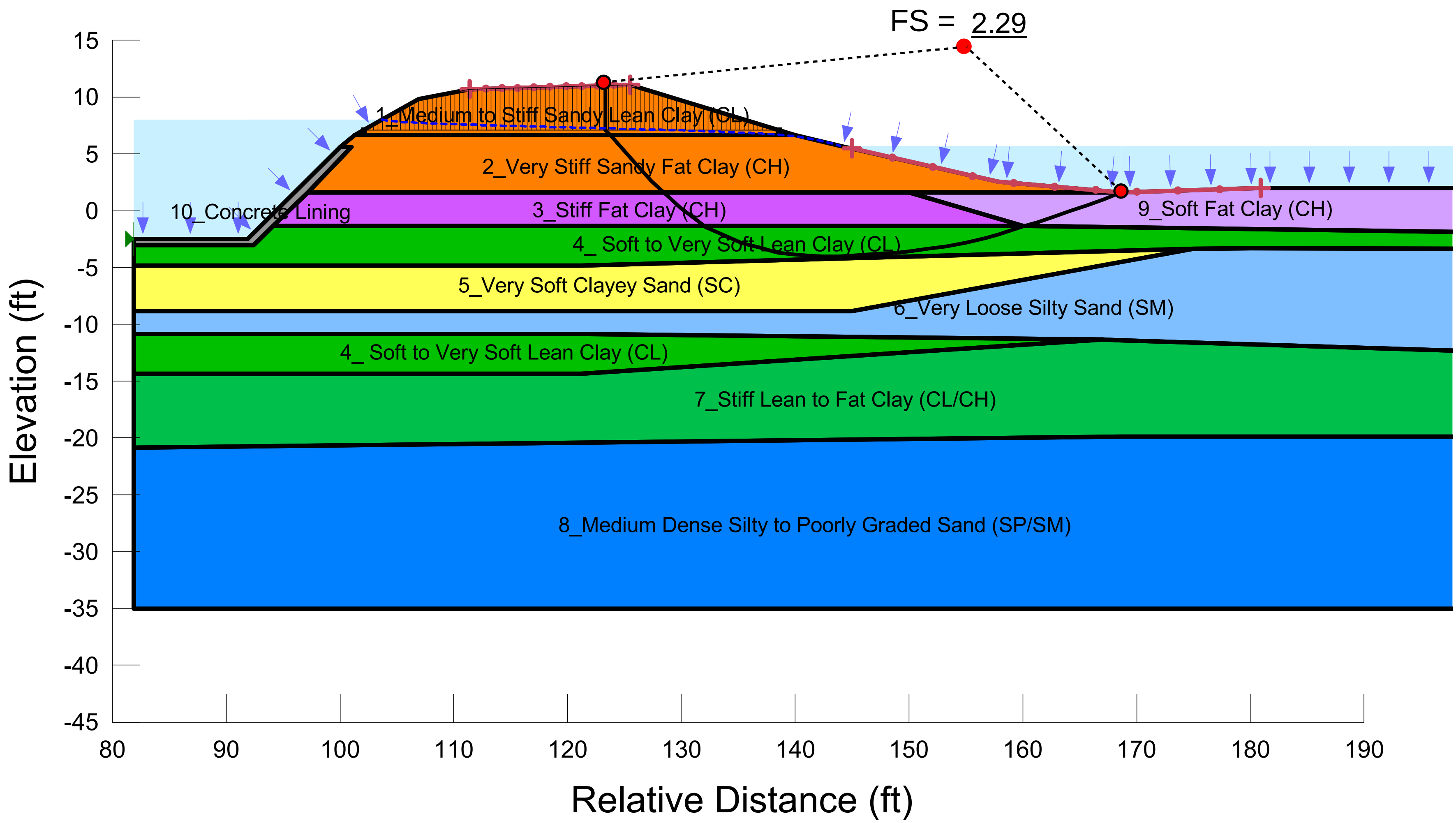
Slope Stability Analysis
Steady State Seepage
Cross Section BB (Sta. 7+75)
Salt Flats Levee



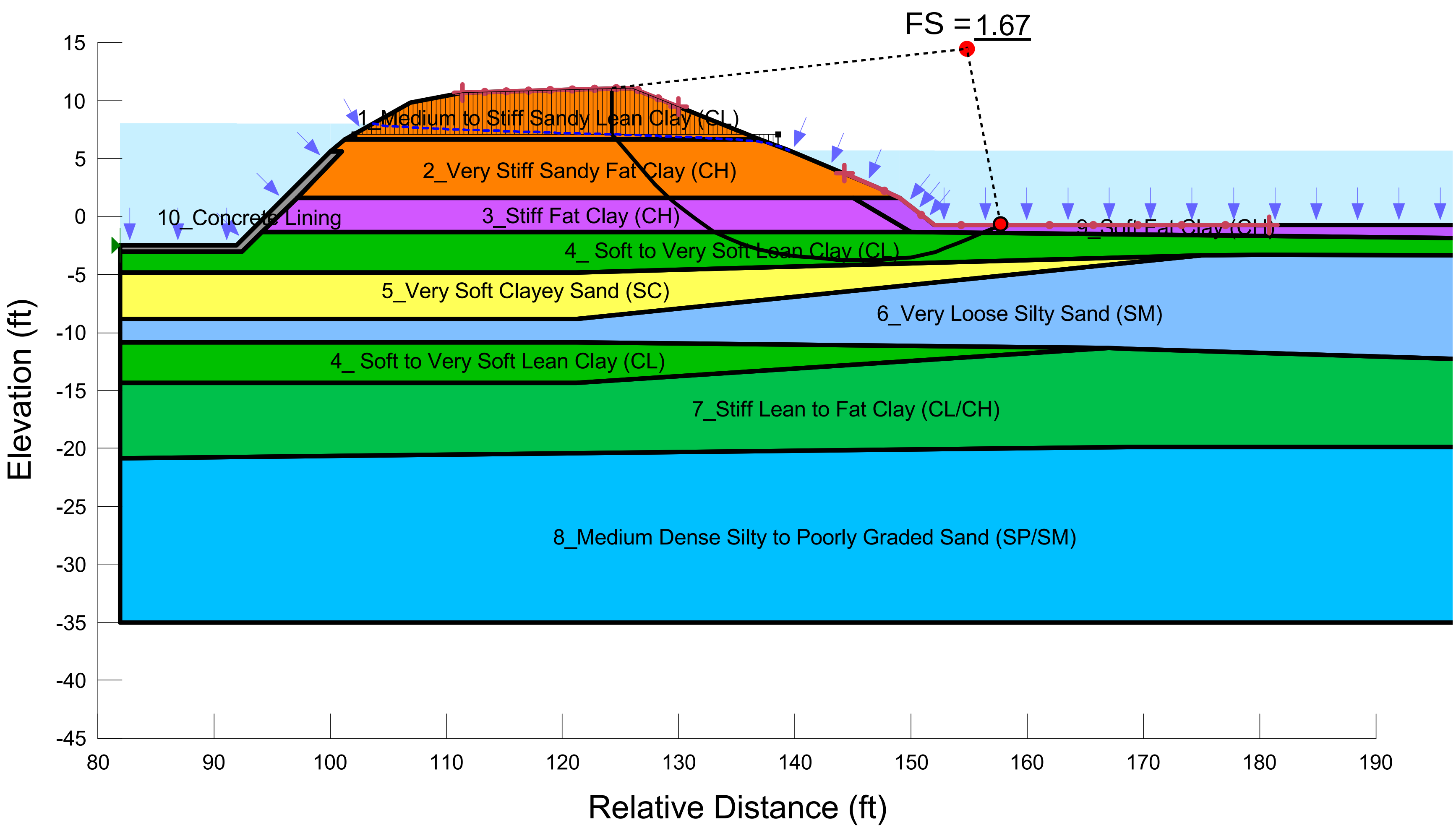
Slope Stability Analysis
Steady State Seepage
Cross Section CC (Sta. 7+10)
Salt Flats Levee



Slope Stability Analysis
Steady State Seepage
Cross Section DD (Sta. 5+50)
Salt Flats Levee



Slope Stability Analysis
Steady State Seepage
Cross Section DD (modified)
Salt Flats Levee

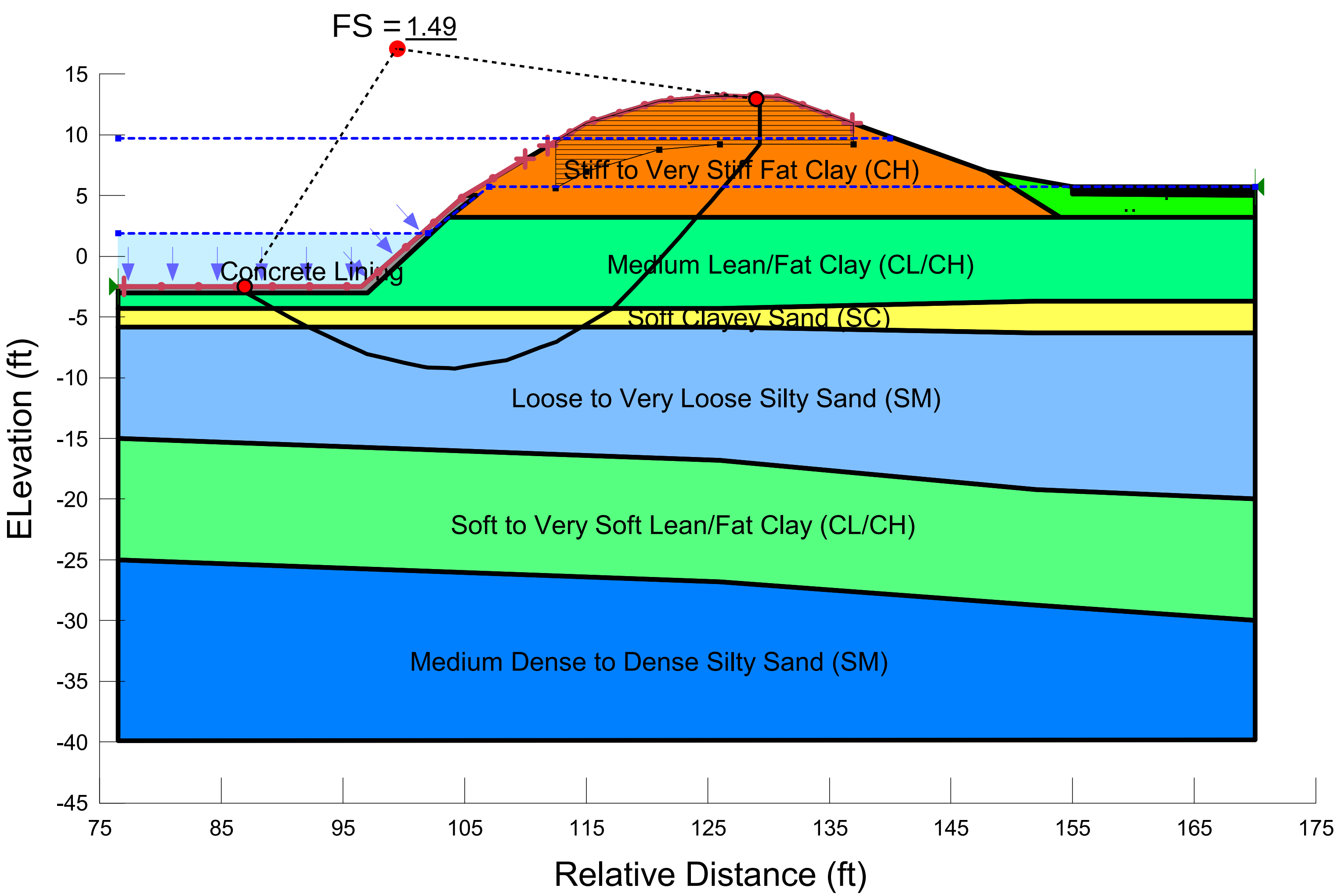


F

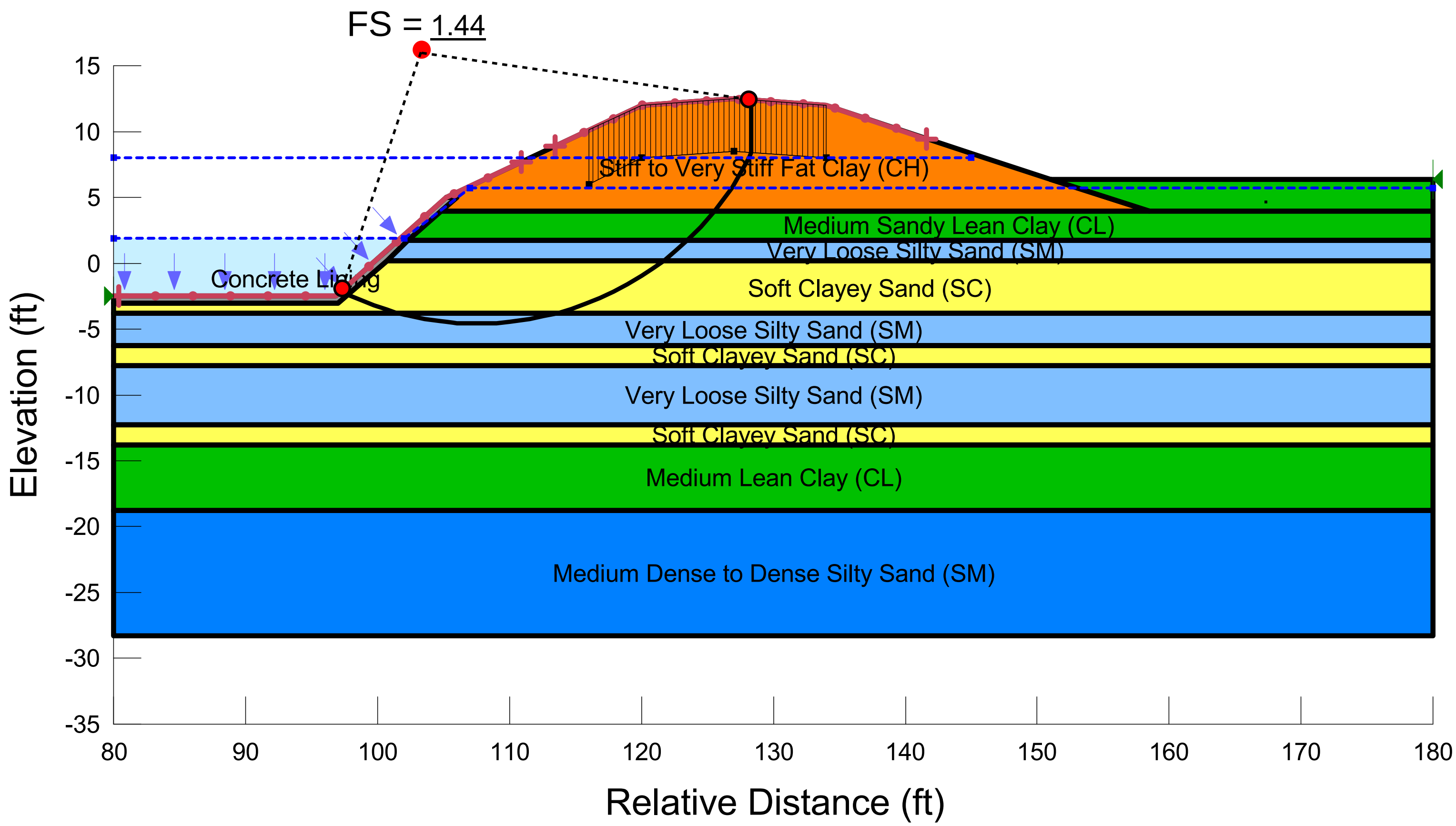
Slope Stability Results for Earthen Levee

(Rapid Drawdown)

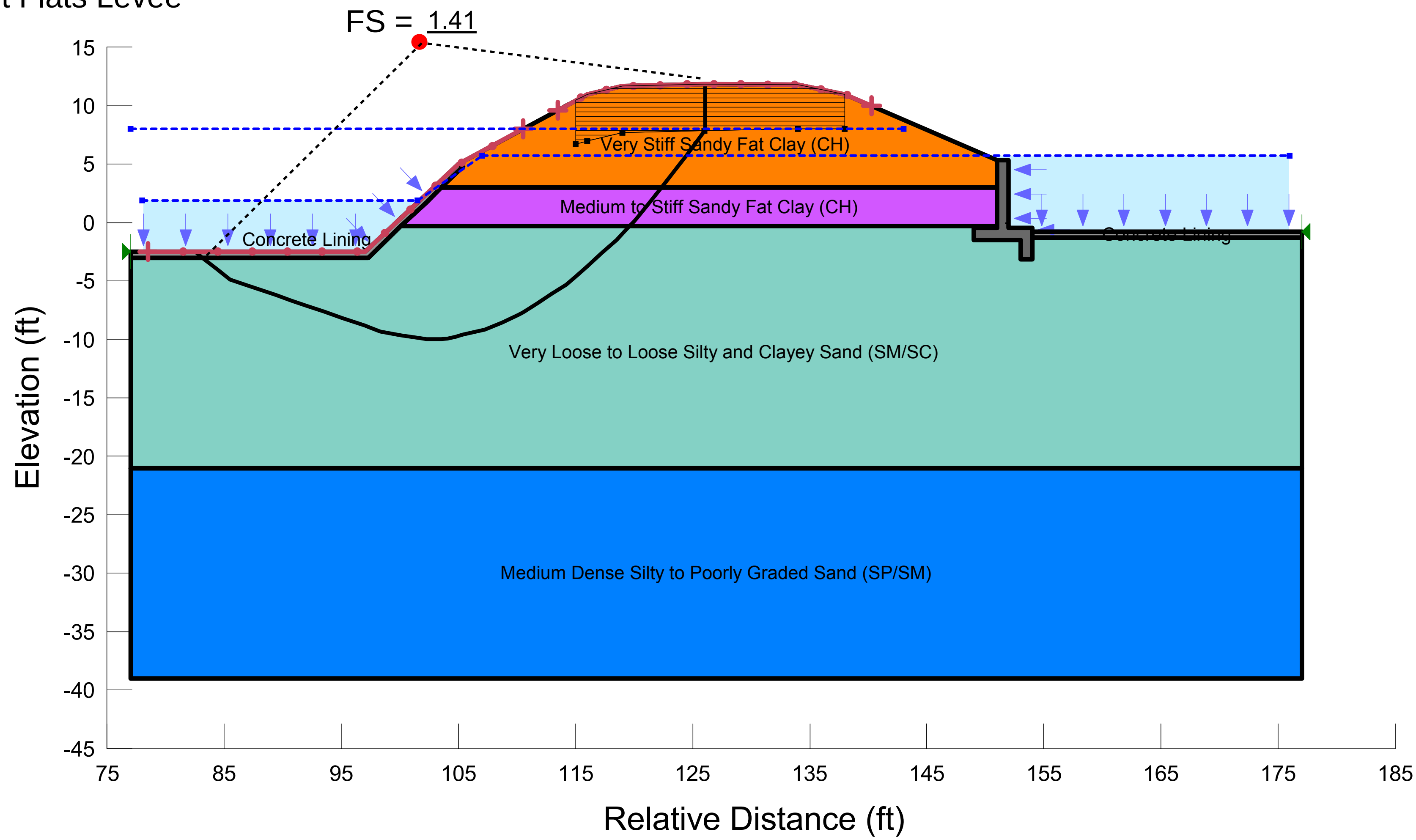
Slope Stability Analysis
Rapid Drawdown Condition
Cross Section AA (Sta. 9+00)
Salt Flats Levee



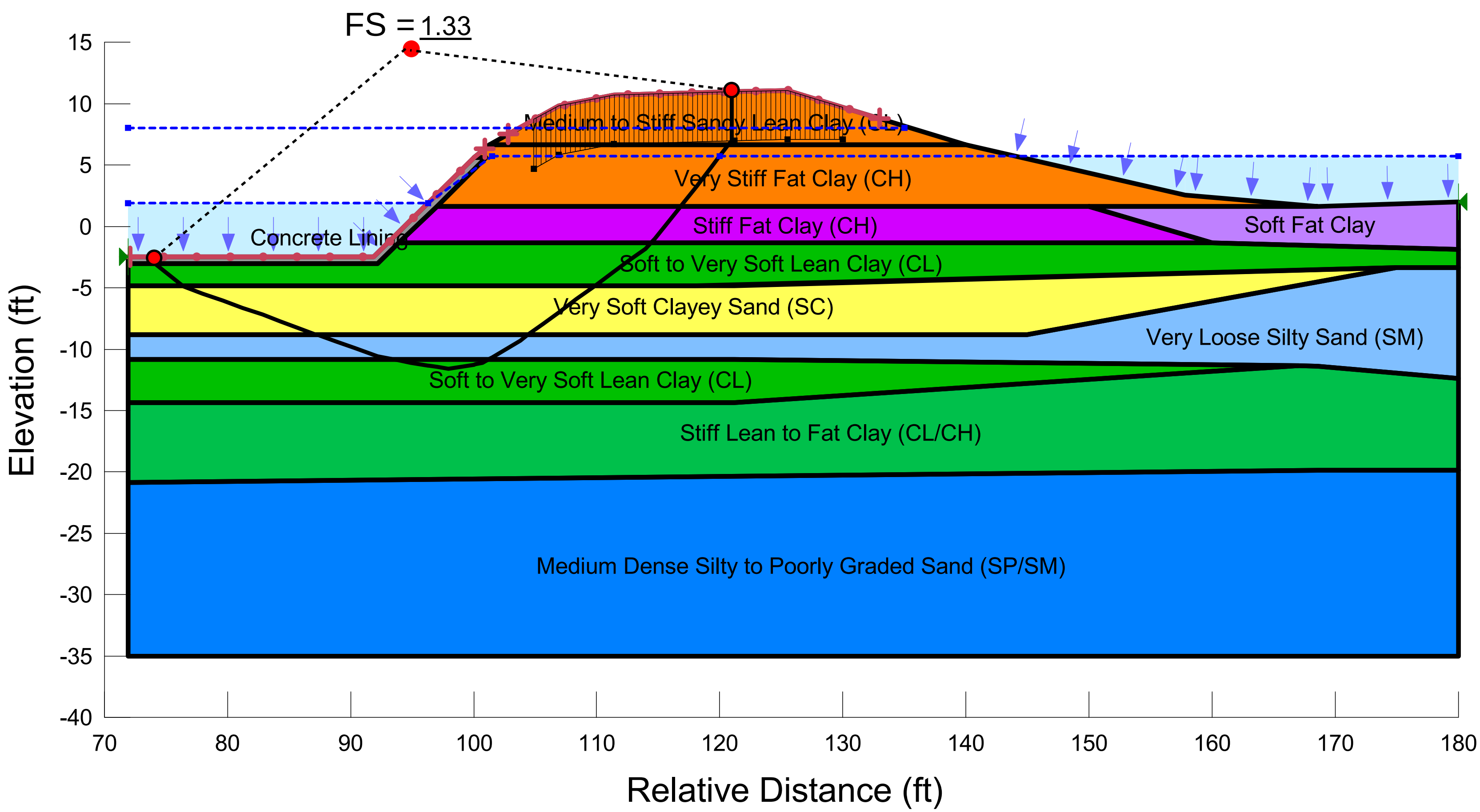
Slope Stability Analysis
Rapid Drawdown Condition
Cross Section BB (Sta. 7+75)
Salt Flats Levee



Slope Stability Analysis
Rapid Drawdown Condition
Cross Section CC (Sta. 7+10)
Salt Flats Levee



Slope Stability Analysis
Rapid Drawdown Condition
Cross Section DD (Sta. 5+50)
Salt Flats Levee



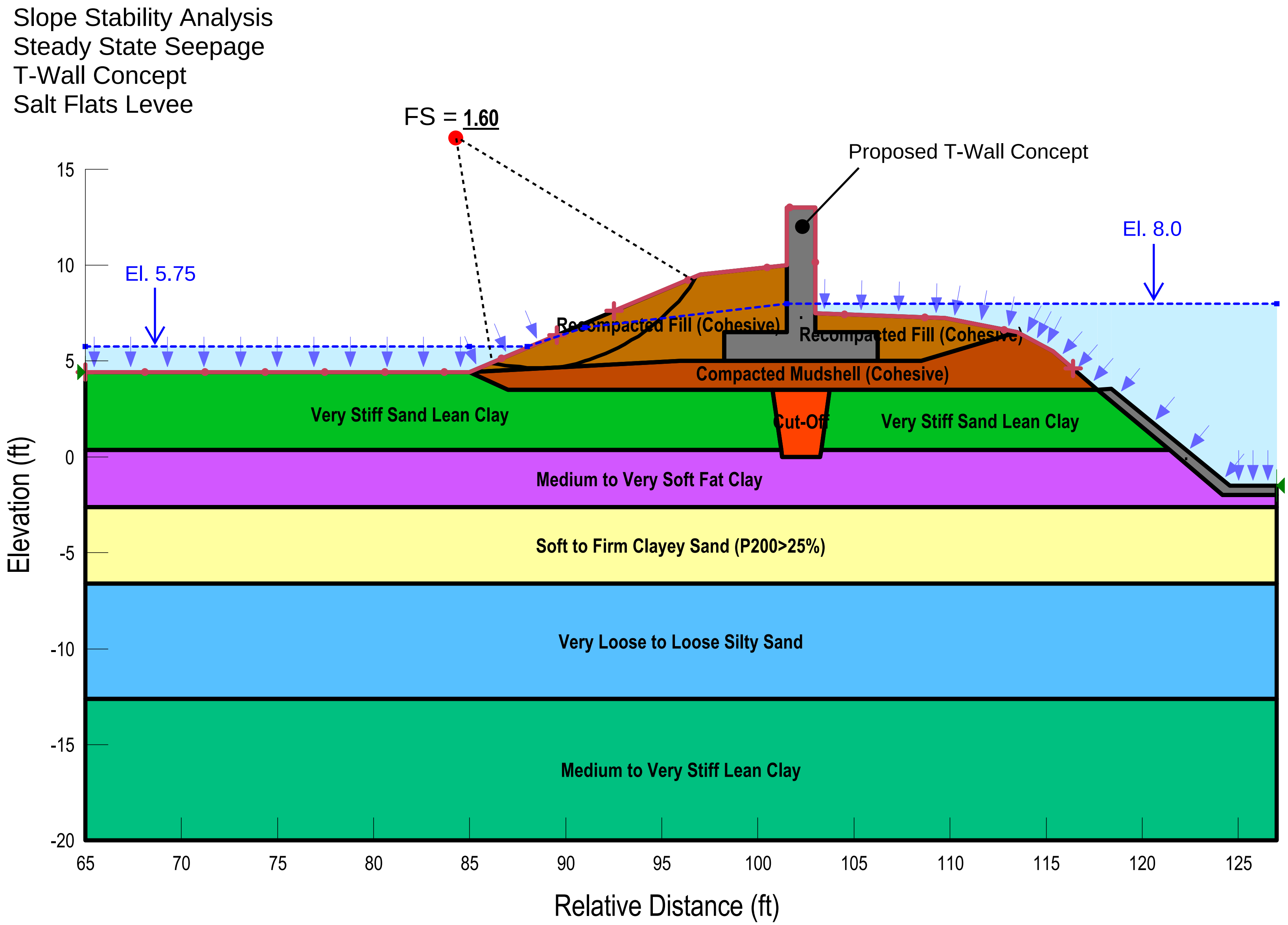
G

External Stability Analyses for Conceptual T-Wall

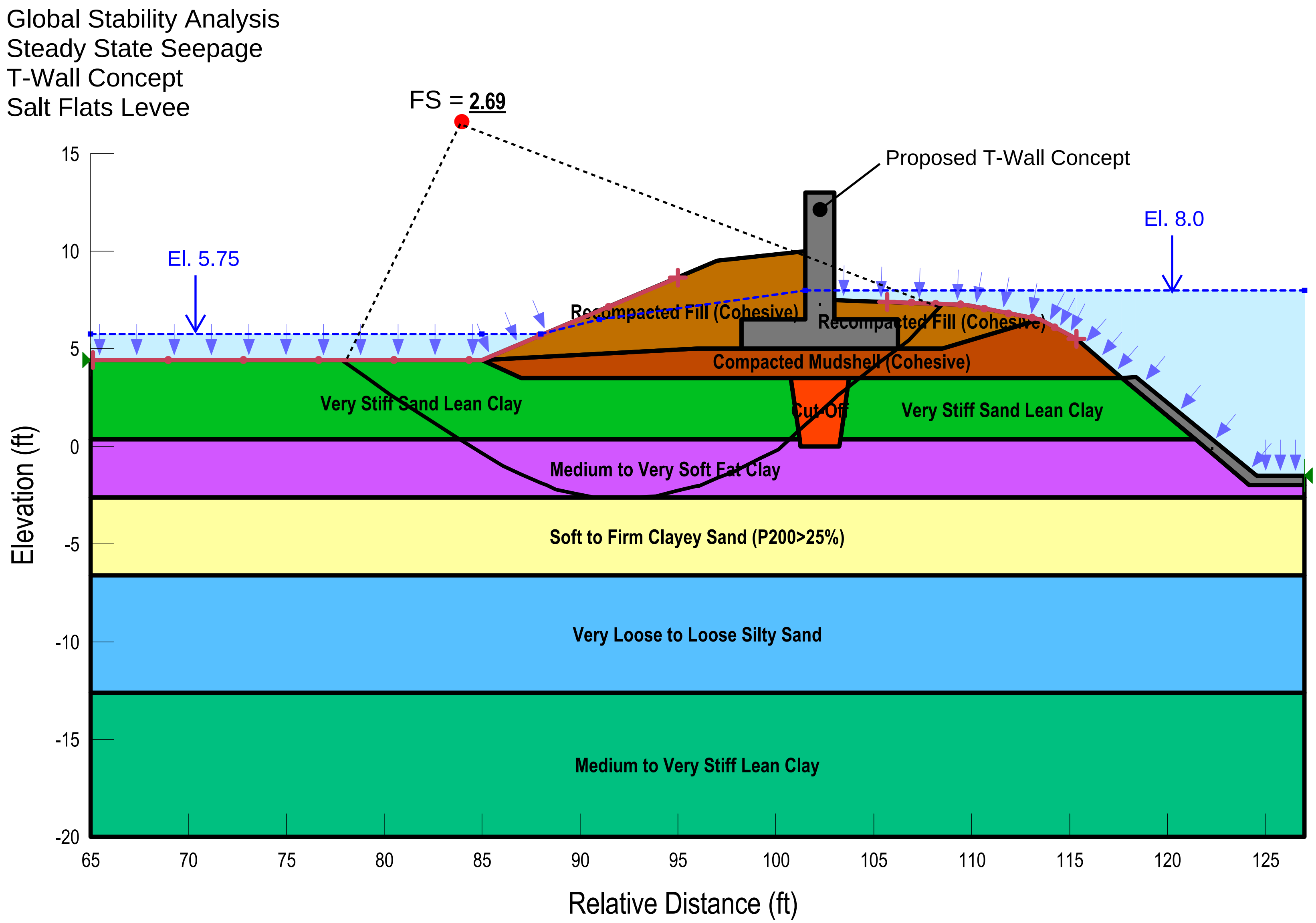
Soil Properties @ T-Wall

Reference Boring: B9

Stratum	N	PP (tsf)	Uc (tsf)	PI	P200 (%)	Unit Weight (pcf)	Undrained Strength S_u (psf)	Drained Strength	
								Φ_d (deg)	C_d (psf)
Recompacted Fill (Cohesive)	--	--	--	--	>50%	120	>1000	30	25
Compacted Mudshell (Cohesive)	--	--	--	--	>50%	120	>1000	30	25
Very Stiff Sandy Lean Clay	18	--	--	--	--	120	1500	32	100
Caliche Cut Off Trenth (Cohesive)	--	--	--	--	>50%	120	1500	32	50
Medium to Very Soft Stiff Fat Clay	4	<0.25	--	51	--	115	250	26	50
Soft to Firm Clayey Sand	4	--	-	8	26	114	300	27	75
Very Loose to Loose Silty Sand	6	--	--	--	--	112	0	27	0
Medium to Very Stiff Lean Clay	6	<3.5	0.60	22		120	750	29	100
Concrete T-Wall	--	--	--	--	--	150	5000	--	5000



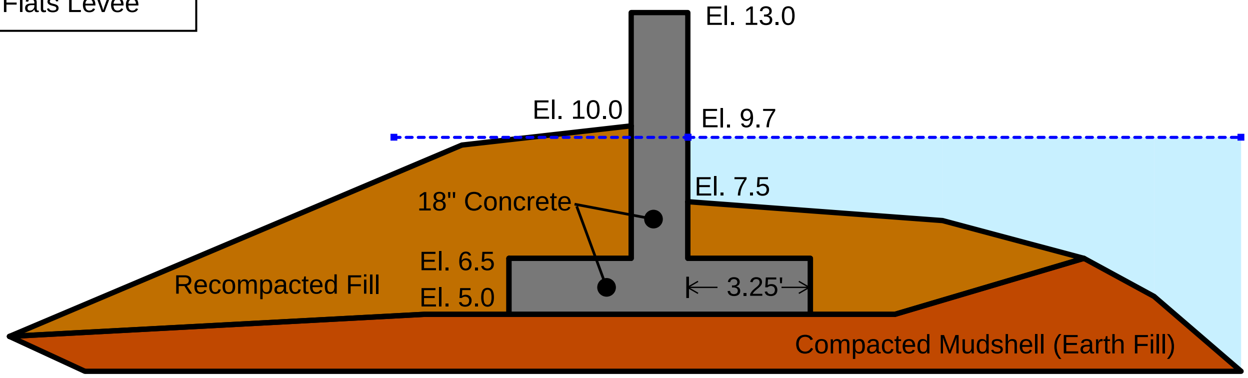
Slope Stability Analysis
Steady State Seepage
T-Wall Concept
Salt Flats Levee
FS = 1.60
Proposed T-Wall Concept
El. 5.75
El. 8.0
Recompacted Fill (Cohesive)
Recompacted Fill (Cohesive)
Compacted Mudshell (Cohesive)
Very Stiff Sand Lean Clay
Cut-Off
Very Stiff Sand Lean Clay
Medium to Very Soft Fat Clay
Soft to Firm Clayey Sand (P200>25%)
Very Loose to Loose Silty Sand
Medium to Very Stiff Lean Clay
Elevation (ft)
15
10
5
0
-5
-10
-15
-20
65
70
75
80
85
90
95
100
105
110
115
120
125
Relative Distance (ft)



Global Stability Analysis
Steady State Seepage
T-Wall Concept
Salt Flats Levee
FS = 2.69
Proposed T-Wall Concept
El. 5.75
El. 8.0
Recompacted Fill (Cohesive)
Recompacted Fill (Cohesive)
Compacted Mudshell (Cohesive)
Cut-Off
Very Stiff Sand Lean Clay
Very Stiff Sand Lean Clay
Medium to Very Soft Fat Clay
Soft to Firm Clayey Sand (P200>25%)
Very Loose to Loose Silty Sand
Medium to Very Stiff Lean Clay
Elevation (ft)
Relative Distance (ft)
15
10
5
0
-5
-10
-15
-20
65
70
75
80
85
90
95
100
105
110
115
120
125

Check T-Wall for Sliding at Peak Flood Condition

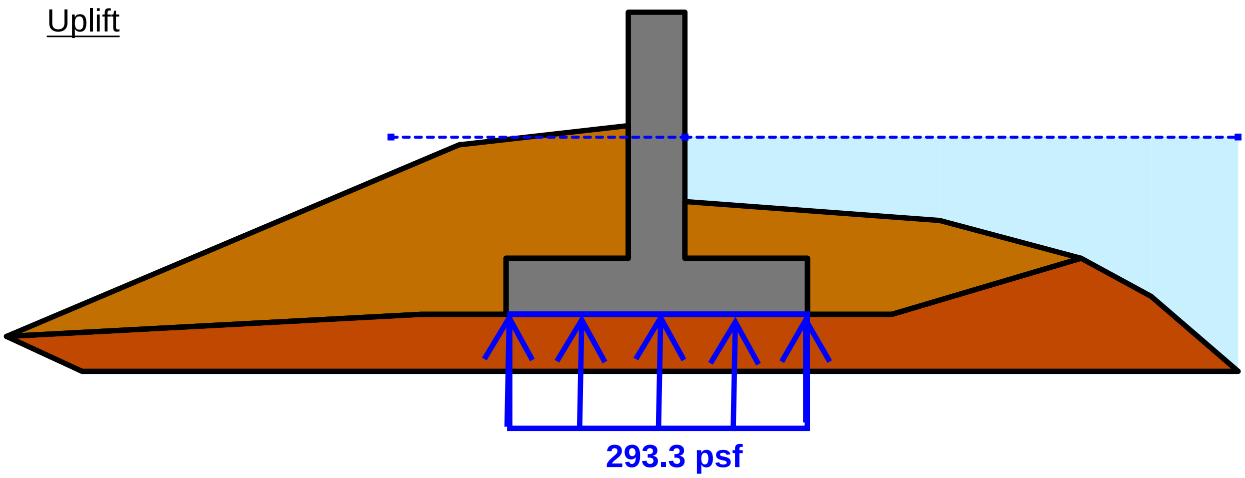
T-Wall Concept
Salt Flats Levee



Approach and Assumptions:

Base Width = 8 feet
Flood Elevation = 9.7 feet
Piezometric Head Under Base = 9.7 feet
Passive Resistance Only Engages Along Front of Base Slab (not stem)
Unit Weight of Compacted Fill and Mudshell =120 pcf
Unit Weight of Concrete = 150 pcf; Unit Weight of Water = 62.4 pcf
Internal Angle of Friction for Both Fills = 30 Degrees
Friction Controls Sliding Resistance (i.e. not undrained cohesion)
Use Rankine Theory: Ka = 0.33 and Kp = 3.0 (@ Friction Angle = 30 degrees)
Neglect Effective Cohesion
Use Buoyant Unit Weight of Soil for Active and Passive Resistance

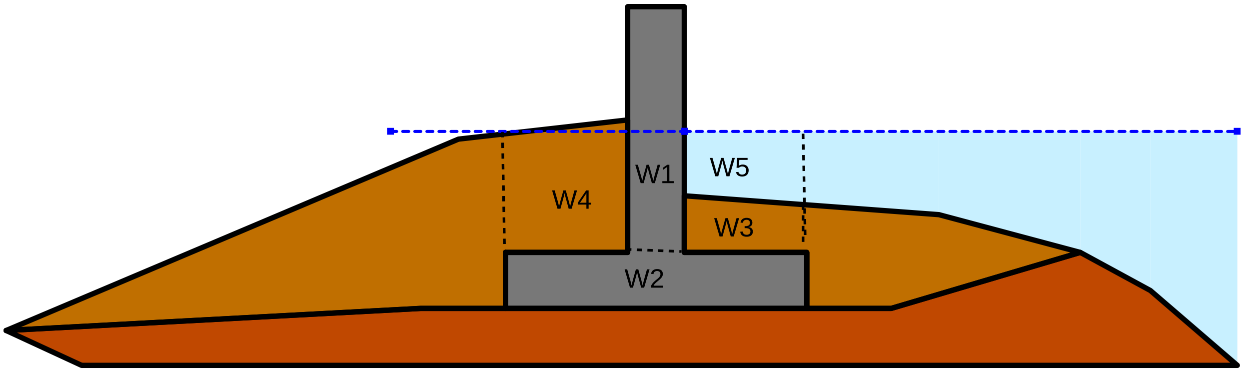
Uplift



Uplift Pressure = (9.7' - 5.0') x 62.4 pcf = 293.3 psf

Total Uplift Force = 8' x 293.3 psf = 2346 lbs/lf

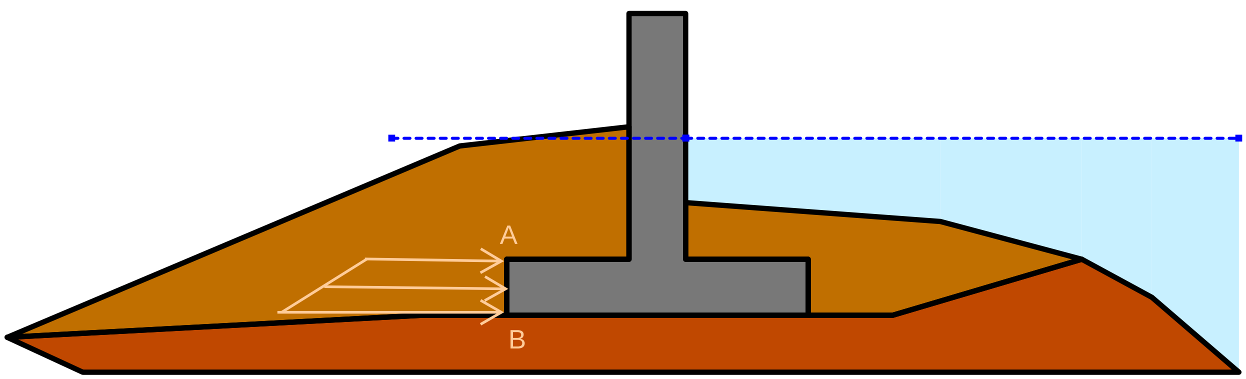
Dead Weight On Bottom of Base Slab



W1 = 1.5 ft x (13'-6.5') x 150 pcf = 1462 lbs/lf
W2 = 1.5 ft x 8 ft x 150 pcf = 1800 lbs/lf
W3 = 3.25 ft x (7.5' - 6.5') x 120 pcf = 390 lbs/lf
W4 = 3.25 ft x (10' - 6.5') x 120 pcf = 1365 lbs/lf
W5 = 3.25 ft x (9.7'- 7.5') x 62.4 pcf = 446 lbs/lf

Total Dead Weight (W) = 5463 lbs/lf

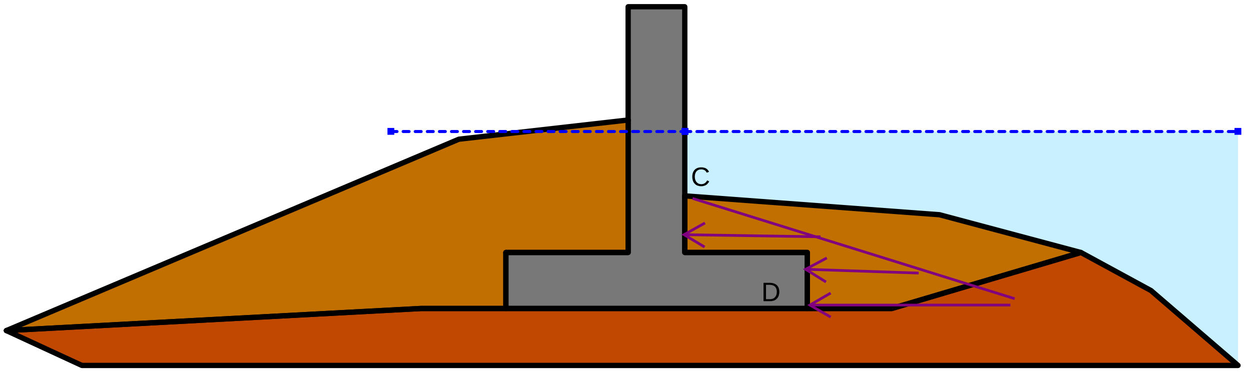
Passive Resistance (face of base only)



Effective Vertical Stress @ A = (120-62.4)pcf x (10'-6.5') = 201.6 psf
Effective Vertical Stress @ B = (120-62.4)pcf x (10'-5.0') = 288.0 psf
Average Effective Vertical Stress AB = 244.8 psf
Average Horizontal Passive Pressure = 244.8 x Kp = 244.8 x 3.0 =734.4 psf

Passive Resisting Force = 734.4 psf x 1.5 feet = 1102 lbs/lf

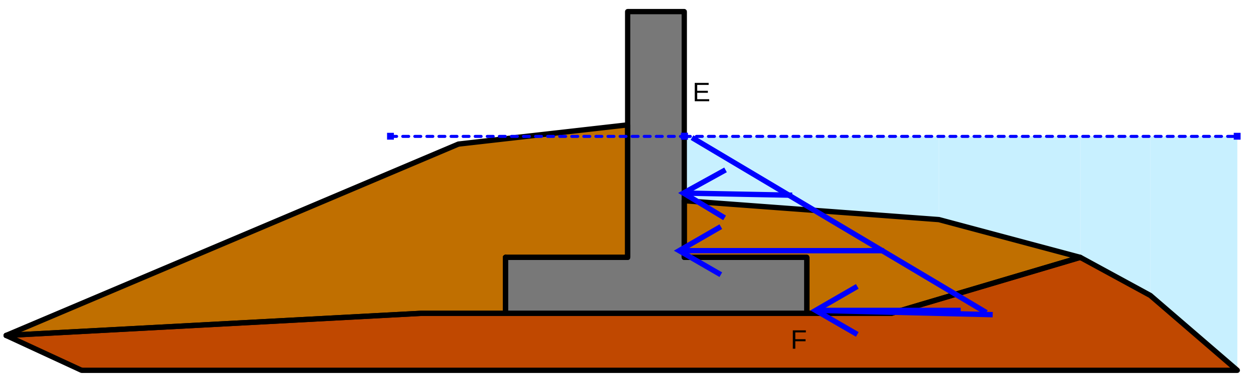
Active Soil Force (front of wall)



Effective Vertical Stress @ C = 0 psf
Effective Vertical Stress @ D = (7.75'- 5.0') x (120 - 62.4) pcf = 158.4 psf
Average Vertical Stress @ CD = 79.2 psf
Horizontal Active Pressure = 79.2 psf x Ka = 79.2 x 0.33 = 26.1 psf

Total Active Force (soil) = 26.1 psf x (7.75 - 5.0') = 72 lbs/lf

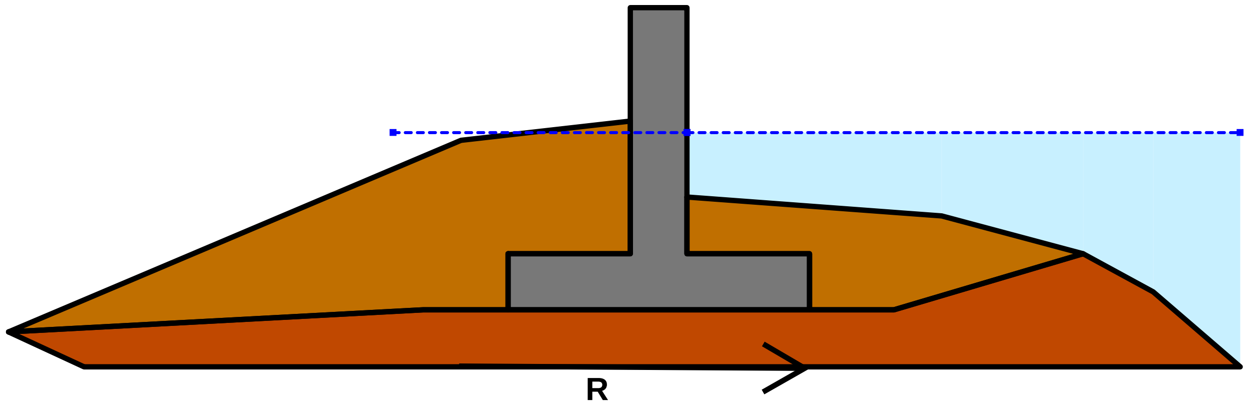
Water Force on Front of Wall



Water Pressure @ E = 0 psf
Water Pressure @ F = (9.7'-5.0') x 62.4 pcf = 293.3 psf
Average Water Pressure @ EF = 146.6 psf

Total Water Force = 146.6 psf x (9.7 - 5.0) = 689 lbs/lf

Sliding Resistance



Slide Friction Factor = Tan (30 x 0.60) = 0.32
Sliding Resistance R = Effective Down Force x 0.32
Effective Down Force = Dead Weight - Uplift Force = 5463 - 2346 = 3117 lbs/lf

Sliding Resistance = 3117 x 0.32 = 997 lbs/lf

Factor of Safety Against Sliding

FS = Forces Resisting - Forces Acting
Forces Resisting = Sliding Resistance + Passive Resistance = 997 + 1102 = 2099 lbs/lf
Forces Acting = Active Soil + Water Force = 72 + 689 = 761 lbs/lf

FS = 2099/761 = 2.75

H

FEMA Checklist

FACT SHEET

LEVEES
IDENTIFYING THE RISK

Meeting the Criteria for Accrediting Levee Systems on NFIP Flood Maps

How-to-Guide for Floodplain Managers and Engineers

A levee system is a flood protection system that consists of a levee, or levees, and associated structures, such as closure and drainage devices, which are constructed and operated in accordance with sound engineering practices. A levee is a manmade structure, usually an earthen embankment, designed and constructed in accordance with sound engineering practices to contain, control, or divert the flow of water so as to provide protection from temporary flooding.

As part of the flood mapping process, the Department of Homeland Security, Federal Emergency Management Agency (FEMA) and its State and local mapping partners review levee system data and documentation.

It is the levee owner's or community's responsibility to provide data and documentation to demonstrate that a levee system meets National Flood Insurance Program (NFIP) requirements as described in Title 44, Chapter 1, Section 65.10 of the Code of Federal Regulations (44 CFR Section 65.10), which you may view on the FEMA Web site at www.fema.gov/plan/prevent/fhm/lv_fpm.shtm.

To be recognized as providing a 1-percent-annual-chance level of flood protection on the modernized NFIP maps, called Digital Flood Insurance Rate Maps (DFIRMs), levee systems must meet *and continue to meet* the minimum design, operation, and maintenance standards (44 CFR Section 65.10)..

To help clarify the responsibilities of community officials, levee owners, or other parties seeking recognition of a levee system identified during a study/mapping project, FEMA issued Procedure Memorandum No. 34 (PM 34), *Interim Guidance for Studies Including Levees*, on August 22, 2005. PM 34 provided clarification of the procedures provided in Appendix H of FEMA's *Guidelines and Specifications for Flood Hazard Mapping Partners*.

FEMA issued Revised Procedure Memorandum No. 43, *Guidelines for Identifying Provisionally Accredited Levees*, on March 16, 2007, which allows issuance of preliminary and, in some cases, effective DFIRMs while communities/levee owners compile and submit required data and documentation. FEMA issued Procedure Memorandum No. 45, *Revisions to Accredited Levee and Provisionally Accredited Levee Notations,* in April 2008 to clarify map notes for accredited and provisionally accredited levee systems.

This document provides information regarding the types of data and documentation that must be submitted for levee systems to be accredited on DFIRMs, including a checklist and an index of further resources you may wish to consult.

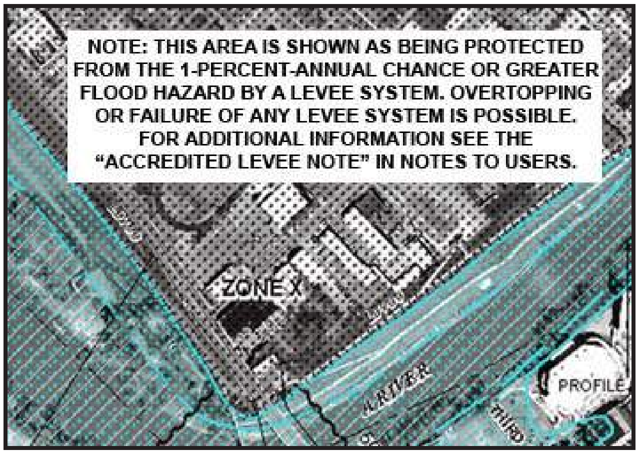
COMMUNITIES WITH LEVEE SYSTEMS SHOULD KNOW:

- The community and/or other party seeking recognition or continued recognition of a levee system must provide data and documentation showing that the levee system provides base (1-percent-annual-chance) flood protection for FEMA to credit the levee system with flood protection on a FIRM or DFIRM.
- Communities *must* actively participate in the levee system documentation process.
- Levee systems without sufficient data and documentation will not be credited with providing base flood protection.
- Some levee systems may qualify for the Provisionally Accredited Levee (PAL) designation.
- Guidance regarding the PAL designation and other levee issues is available at:

www.fema.gov/plan/prevent/fhm/lv_fpm.shtm

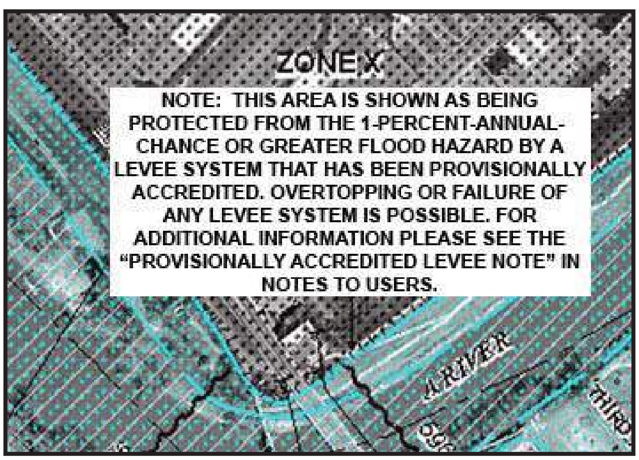
HOW FEMA WILL MAP LEVEE SYSTEMS

FEMA mapping requirements are designed to provide the people living and working behind levee systems with accurate, up-to-date flood hazard and risk information so that they may make wise decisions to minimize damage and loss of life. FEMA does not evaluate the performance of a levee system—this is the responsibility of the levee owner. FEMA is responsible for establishing levee system evaluation and mapping standards, determining flood insurance risk zones, and reflecting these determinations on DFIRMs.



Accredited Levee System

An accredited levee system is a system that FEMA has determined can be shown on a DFIRM as providing a 1-percent-annual-chance or greater level of flood protection. This determination is based on the submittal of data and documentation required by 44 CFR Section 65.10. The area landward of an accredited levee system is shown as a moderate-risk area, labeled Zone X (shaded), on the DFIRM except for areas of residual flooding, such as ponding areas, which will be shown as high-risk areas, called Special Flood Hazard Areas (SFHAs). Flood insurance is not mandatory in Zone X (shaded) areas, but is mandatory in SFHAs. FEMA strongly encourages flood insurance for all structures in levee-impacted areas.



Provisionally Accredited Levee (PAL) System

The PAL designation may be used for a levee system that FEMA has previously accredited with providing 1-percent-annual-chance flood protection on an effective FIRM/DFIRM, and for which FEMA is awaiting data and/or documentation that will show the levee system is compliant with 44 CFR Section 65.10. Before FEMA will apply the PAL designation to a levee system, the community or levee owner will need to sign and return an agreement indicating the data and documentation required for compliance with 44 CFR Section 65.10 will be provided within a specified timeframe. The impacted area landward of a PAL system also is shown as a moderate-risk area, labeled Zone X (shaded). Therefore, flood insurance is not mandatory for insurable structures in the levee-impacted area; however, it is strongly encouraged by FEMA as are other protective measures.

Levee System Not Accredited or De-accredited

If the levee system is not shown as providing 1-percent-annual-chance flood protection on an effective FIRM, the system is considered "not accredited" and the levee-impacted area is mapped as Zone AE or Zone A on a DFIRM, depending on the type of study performed for the area. If the levee system was previously shown as providing 1-percent-annual-chance flood protection on an effective FIRM or DFIRM, but does not meet the PAL requirements or is no longer eligible for the PAL designation, FEMA will de-accredit the levee system and re-map the levee-impacted area as an SFHA, labeled Zone AE or Zone A depending on the type of study performed . Flood insurance will be required for insurable structures with federally backed mortgages in SFHAs.

Design Criteria*	Section of the NFIP Regulations: 65.10(b)
Description: For levee systems to be recognized (i.e., accredited) by FEMA, evidence that adequate design and operation and maintenance systems are in place to provide reasonable assurance that protection from the base flood exists must be provided. The following requirements must be met:	
Checklist for Design Criteria:	
■	**Freeboard.** Minimum freeboard required 3 feet above the Base Flood Elevation (BFE) all along length, and an additional 1 foot within 100 feet of structures (such as bridges) or wherever the flow is restricted. Additional 0.5 foot at the upstream end of a levee. Coastal levees have special freeboard requirements (see Paragraphs 65.10(b)(1)(iii) and (iv)).
■	**Closures.** All openings must be provided with closure devices that are structural parts of the system during operation and designed according to sound engineering practice.
■	**Embankment Protection**. Engineering analyses must be submitted that demonstrate that no appreciable erosion of the levee embankment can be expected during the base flood, as a result of either currents or waves, and that anticipated erosion will not result in failure of the levee embankment or foundation directly or indirectly through reduction of the seepage path and subsequent instability.
■	**Embankment and Foundation Stability Analyses.** Engineering analyses that evaluate levee embankment stability must be submitted. The analyses provided must evaluate expected seepage during loading conditions associated with the base flood and must demonstrate that seepage into or through the levee foundation and embankment will not jeopardize embankment or foundation stability. An alternative analysis demonstrating that the levee is designed and constructed for stability against loading conditions for Case IV as defined in the U.S. Army Corps of Engineers (USACE) Engineer Manual 1110–2–1913, *Design and Construction of Levees*, (Chapter 6, Section II), may be used.
■	**Settlement Analyses.** Engineering analyses must be submitted that assess the potential and magnitude of future losses of freeboard as a result of levee settlement and demonstrate that freeboard will be maintained. This analysis must address embankment loads, compressibility of embankment soils, compressibility of foundation soils, age of the levee system, and construction compaction methods. In addition, detailed settlement analysis using procedures such as those described in USACE Engineer Manual 1110–1–1904, *Soil Mechanics Design— Settlement Analysis*, must be submitted.
■	**Interior Drainage.** An analysis must be submitted that identifies the source(s) of such flooding, the extent of the flooded area, and, if the average depth is greater than 1 foot, the water-surface elevation(s) of the base flood. This analysis must be based on the joint probability of interior and exterior flooding and the capacity of facilities (such as drainage lines and pumps) for evacuating interior floodwaters.

Operation Plan*	Paragraph 65.10(c)(1) of the NFIP Regulations
Description: For a levee system to be recognized (i.e., accredited), the operational criteria must be as described below. All closure devices or mechanical systems for internal drainage, whether manual or automatic, must be operated in accordance with an officially adopted operation manual, a copy of which must be provided to FEMA by the operator when levee or drainage system recognition is being sought or when the manual for a previously recognized system is revised in any manner. All operations must be under the jurisdiction of a Federal or State agency, an agency created by Federal or State law, or an agency of a community participating in the NFIP.	
Checklist for Operation Plan:	
■	**Flood Warning System.** Documentation of the flood warning system, under the jurisdiction of Federal, State, or community officials that will be used to trigger emergency operation activities; and demonstration that sufficient flood warning time exists for the completed operation of all closure structures, including necessary sealing, before floodwaters reach the base of the closure.
■	**Plan of Operation**. A formal plan of operation including specific actions and assignments of responsibility by individual name or title.
■	**Periodic Operation of Closures.** Provisions for periodic operation, at not less than one-year intervals, of the closure structure for testing and training purposes.
■	**Interior Drainage Plan**. See below.
Interior Drainage Plan	**Paragraph 65.10(c)(2) of the NFIP Regulations**
Description: Interior drainage systems associated with levee systems usually include storage areas, gravity outlets, pumping stations, or a combination thereof. These drainage systems will be recognized by FEMA on NFIP maps for flood protection purposes only if the following minimum criteria are included in the operation plan.	
Checklist for Interior Drainage Plan:	
■	**Flood Warning System.** Documentation of the flood warning system, under the jurisdiction of Federal, State, or community officials that will be used to trigger emergency operation activities; and demonstration that sufficient flood warning time exists to permit activation of mechanized portions of the drainage system.
■	**Plan of Operation.** A formal plan of operation including specific actions and assignments of responsibility by individual name or title.

■	**Manual Backup.** Provision for manual backup for the activation of automatic systems.
■	**Periodic Inspection.** Provisions for periodic inspection of interior drainage systems and periodic operation of any mechanized portions for testing and training purposes. No more than 1 year shall elapse between either the inspections or the operations.
Maintenance Plan	**Paragraph 65.10(d) of the NFIP Regulations**

Description: For levee systems to be recognized as providing protection from the base flood (i.e., accredited by FEMA), the maintenance criteria must be as described herein.

Checklist for Maintenance Plan:

■	Levee systems must be maintained in accordance with an officially adopted maintenance plan, and a copy of this plan must be provided to FEMA by the owner of the levee system when recognition is being sought or when the plan for a previously recognized system is revised in any manner.
■	All maintenance activities must be under the jurisdiction of a Federal or State agency, an agency created by Federal or State law, or an agency of a community participating in the NFIP that must assume ultimate responsibility for maintenance.
■	This plan must document the formal procedure that ensures that the stability, height, and overall integrity of the levee and its associated structures and systems are maintained. At a minimum, the plan shall specify the maintenance activities to be performed, the frequency of their performance, and the person by name or title responsible for their performance.
Certification	**Paragraph 65.10(e) of the NFIP Regulations**

Description: Data submitted to support that a given levee system complies with the structural requirements set forth in "Design Criteria" (Paragraphs 65.10(b)(1) through (7) of the regulations) must be certified by a Registered Professional Engineer. Also, certified "as-built" plans of the levee must be submitted. Certifications are subject to the definition given in Section 65.2 of the NFIP regulations. In lieu of these structural requirements, a Federal agency with responsibility for levee design may certify that the levee has been adequately designed and constructed to provide protection from the base flood.

Checklist for Certification Requirement:

■	All data submitted is certified by Professional Engineer or certified by a Federal agency**.**
■	Certified as-built levee plans are included in the submittal.

A NOTE ABOUT FLOOD RISK AND FLOOD INSURANCE

Levee systems are designed to provide a *specific level of protection*. They can be overtopped or fail during larger flood events.

Levee systems also decay over time. They require regular maintenance and periodic upgrades to retain their level of protection. When levees do fail, they often fail catastrophically. The resulting damage, including loss of life, may be much greater than if the levee system had not been built.

For all these reasons, FEMA strongly encourages people in levee-impacted areas to understand their flood risk, know and follow evacuation procedures, and protect their property by purchasing flood insurance protection, by floodproofing, or by taking other protective measures.

CHECKLIST INFORMATION

The checklist provided in this fact sheet is meant to assist local community officials and levee owners in gathering the data and documentation that will be required for FEMA to show a levee system as providing 1-percent-annual-chance flood protection on the community's DFIRM. Where possible, text from the actual NFIP regulations (44 CFR Section 65.10) was used.

The checklist is set up according to the appropriate paragraph of 44 CFR Section 65.10. For example, Design Criteria can be found in Paragraph 65.10(b):

Design Criteria*	Section of the NFIP Regulations: 65.10(b)
Description: For levee systems to be recognized (i.e., accredited) by FEMA, evidence that adequate design and operation and maintenance systems are in place to provide reasonable assurance that protection from the base flood exists must be provided.	

For a comprehensive description of each item in this checklist, please see Appendix H of the *Guidelines and Specifications for Flood Hazard Mapping Partners*. Locations of this resource, and other useful resources, are provided below.

INDEX OF RESOURCES

This fact sheet is accessible, along with an assortment of other levee-related resources, through a dedicated portion of the FEMA Web site. The gateway to the FEMA-provided levee information, which is organized by stakeholder group to assist levee owners, community officials, and other stakeholders, is www.fema.gov/plan/prevent/fhm/lv_intro.shtm. The FEMA resources referenced in this fact sheet, listed below, are directly accessible through www.fema.gov/plan/prevent/fhm/lv_fpm.shtm.

- Procedure Memorandum No. 34, *Interim Guidance for Studies Including Levees*
- Revised Procedure Memorandum No. 43, *Guidelines for Identifying Provisionally Accredited Levees*.
- Procedure Memorandum No. 45, *Revisions to Accredited Levee and Provisionally Accredited Levee Notations*
- Appendix H, "Mapping of Areas Protected by Levee Systems," of *Guidelines and Specifications for Flood Hazard Mapping Partners*.
- Section 65.10. *Mapping of Areas Protected by Levee Systems* of the NFIP regulations.

Flood insurance information can be found at www.fema.gov/business/nfip or on the NFIP's consumer Web site, www.FloodSmart.gov.

Links to the USACE Web site also are provided on the levee-dedicated pages; the resources discussed in this fact sheet are accessible through the USACE Web page at www.usace.army.mil/publications/eng-manuals.