Mid-Barataria Sediment Diversion

Preliminary Geotechnical Recommendations for Structural Design Technical Memorandum

Coastal Protection and Restoration Authority of Louisiana

November 2013

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Prepared for
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1 Introduction

1.1 Project Location and Description

The Mid-Barataria Sediment Diversion (MBSD) project site is located in Louisiana's Plaquemines and Jefferson Parishes, approximately 26 miles south of New Orleans. The site extends from the western bank of the Mississippi River to the eastern portion of the Mid-Barataria Basin. The northern and southern boundary limits are approximately 500 feet offset from the project centerline. The site location is depicted on Figure 1.

The region is generally low-lying, relatively flat agricultural land crisscrossed with drainage ditches, marshy areas, and woodlands. Belle Chasse Highway (LA 23) currently bisects the project site at approximately Station 65+00. Currently, the Mississippi River is separated from the project site by the Mississippi River and Tributary (MR&T) Levee at the eastern boundary of the proposed project. At approximately Station 137+50, the U.S. Army Corps of Engineers (USACE) New Orleans to Venice (NOV) Non-Federal Levee (NFL) traverses the project site. These structures are discussed in greater detail in subsequent sections of this report.

1.2 Purpose and Scope

The purpose of the proposed MBSD is to divert sediment from the Mississippi River to the Mid-Barataria Basin to replace land lost within the basin. This report summarizes and presents preliminary geotechnical recommendations in support of the project's 30% design. Specifically, this report provides preliminary recommendations for the following:

- inlet system structures foundations
- railroad bridge foundations
- LA 23 bridge foundations
- back structure foundations

Recommendations in this report are based on the Phase 1 geotechnical investigation program of HDR Engineering, Inc. (HDR), and on input from the project team's structural engineers. The purpose of the Phase 1 investigation was to provide an initial site characterization to identify geotechnical-related project conditions and constraints, and to support both preliminary engineering planning and conceptual plan development.

1.3 Project Datum and Coordinate System

The project elevation and coordinates reference U.S. Geological Survey North American Datum of 1983 and World Geodetic System of 1984. Stationing was developed specifically for this project.

1.4 Topics Not Addressed

Several issues are covered in a separate report. Topics not addressed in this report consist of the following:

- Analyze slope stability of the existing MR&T Levee and the proposed NOV Levee.
- Conduct a detailed seepage analysis in support of the proposed structures. However, an initial review of the Phase 1 investigation data was conducted as part of this effort. The result of the review indicates that seepage will be a controlling topic moving forward.
- Identify future conveyance channel levee configurations and relative locations and alignments.

2 Background Information

2.1 Previous Studies

General regional geologic studies have been published by the U.S. Geological Survey that encompass the project site. These studies provide the basis for the geology and geomorphology described later in Section 3.

More specific and recent studies have been performed by USACE for a flood control channel with a similarly proposed alignment named the Myrtle Grove Channel. The preliminary geotechnical investigation included limited borings and test pits in the vicinity of the Coastal Protection and Restoration Authority of Louisiana's (CPRA's) currently proposed MBSD alignment. The subsurface investigation performed in the vicinity of the MBSD consisted of relatively shallow borings aimed at characterizing subsurface materials for mining and use as levee embankment fill materials for the Myrtle Grove Channel.

2.2 Flooding History

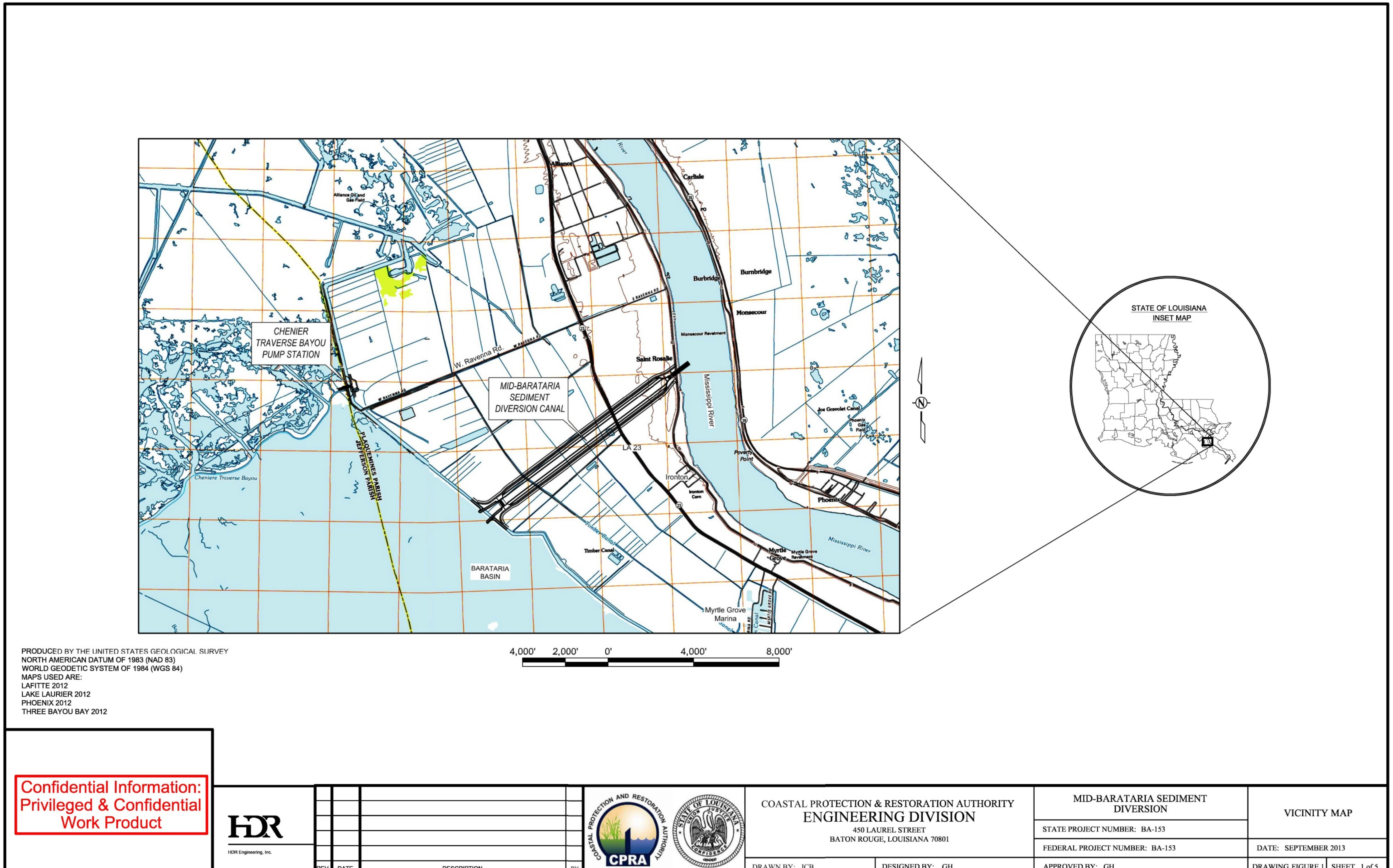
As discussed later in Section 3, the regional land masses are depositional. These deposits resulted from alterations of the Mississippi River channel, overtopping of river banks during high river stages, and storms (specifically, hurricanes). Recent significant flooding has been associated with Hurricanes Isaac and Katrina.

2.3 Nearby Levees

New Orleans to Venice Non-Federal Levee

The NOV-NFL system alignment is located along the eastern land edge of the Mid-Barataria Basin, as shown on Figure 1. USACE is currently designing improvements to the NOV-NFL system. The NOV-NFL improvements include a portion crossing the MBSD proposed alignment at Station 140+00. As part of the proposed improvements, the NOV Levee will be reconfigured to include an increase in crown elevation and stability berms on either side of the levee, as well as a modified toe drainage ditch. The elevation increases have been proposed to be at least elevation +9 feet in the vicinity of the current MBSD conveyance channel. Based on communications with CPRA as of October 11, 2013, the elevation increase will meet the minimum level of protection for a 20-year event within the next 10 years.

Figure 1. MBSD site location



Mississippi River & Tributary Levee

Constructed in stages between 1717 and 1973, the MR&T Levee features various materials and construction methods. It is located on the western bank of the Mississippi River with an alignment that traverses the project site generally in a north-to-south direction. Typical elevations of the levee in the vicinity of the project site are approximately +15.5 feet. This levee crown elevation is designed to contain the MR&T system south of the Bonnet Carré Spillway. Furthermore, an existing railroad spur associated with the ConocoPhillips Alliance Refinery and operated by the New Orleans and Gulf Coast Railway runs parallel to the levee along the landside toe. The spur terminates south of the proposed MBSD project limits.

2.4 Proposed Project Features

Inlet System

The 1,100-foot channel will extend from the Mississippi River to a series of structures to form an inlet to the MBSD channel. The inlet consists of five structures: approach channel, control structure, outlet channel, transition structure, and transition walls, all between approximately Station 22+00 and Station 41+85. A plan and profile of the inlet system illustrating the individual structures' relative locations are presented in Figures 2 and 3. These structures make up the MBSD inlet system and are discussed separately in subsequent sections.

The inlet system is required to maintain the same level of flood protection as the current MR&T Levee system where these two tie in to each other. The control structure, when tied to the MR&T Levee, will become the primary flood control feature and must provide the same level of protection as the current levee. The MR&T Levee design flood level under its current federal authorization is 1,250,000 cubic feet per second (cfs). This is not the 100-year event. The project will modify a federal levee and, therefore, must be permitted under Section 408. The current levee has a freeboard deficiency attributable to ongoing site subsidence that will not be addressed by this project. The structure heights will be set to allow USACE to raise the levee height under future federal authorization.

The current plan is to provide a temporary setback levee to provide primary flood control until the control structure is completed and levees are constructed to tie it into the MR&T Levee.

Control Structure

The control structure is currently proposed to be located between Station 33+85 and Station 35+77. The preliminary footprint has been proposed to be approximately 192 feet by 166 feet. The top of slab is proposed to be the same as the channel invert elevation of –40 feet. The top of walls vary in elevation between +18.5 and +23 feet. As of September 25, 2013, the structural concept consists of three bays with roller gates. Each bay is separate and will be periodically dewatered for gate maintenance. The preliminary dead loads have been determined to be on the order of 3,900 pounds per square foot (psf) in compression at the foundation elevation of –48 feet. Should the structure be completely dewatered, the anticipated uplift pressures attributable to buoyancy effects have been estimated at 4,500 psf.

Outlet Channel

An outlet channel structure is the first stage in the transition from the control structure to the conveyance channel. The outlet channel is a concrete structure consisting of a slab and free-standing, unbraced walls similar in configuration to the control structure, minus the gates. The outlet channel is located downstream and adjacent to the control structure between Station 32+92 and Station 33+67. The preliminary footprint has been proposed to be approximately 75 feet by 166 feet. The top of slab is proposed to be at elevation –40 feet, with top of walls at elevation +13.5 feet. This structure will be permanently submerged without the capability of dewatering and, therefore, will not be subject to buoyancy pressures. The preliminary dead loads have been determined and are presented in Table 1.

Table 1. Outlet channel loading conditions

Loading condition	Dead loads (pounds per square foot)
Immediately after construction	3,000
Effective bearing pressure	1,700
Effective plus high water elevation	3,840

Transition Structure

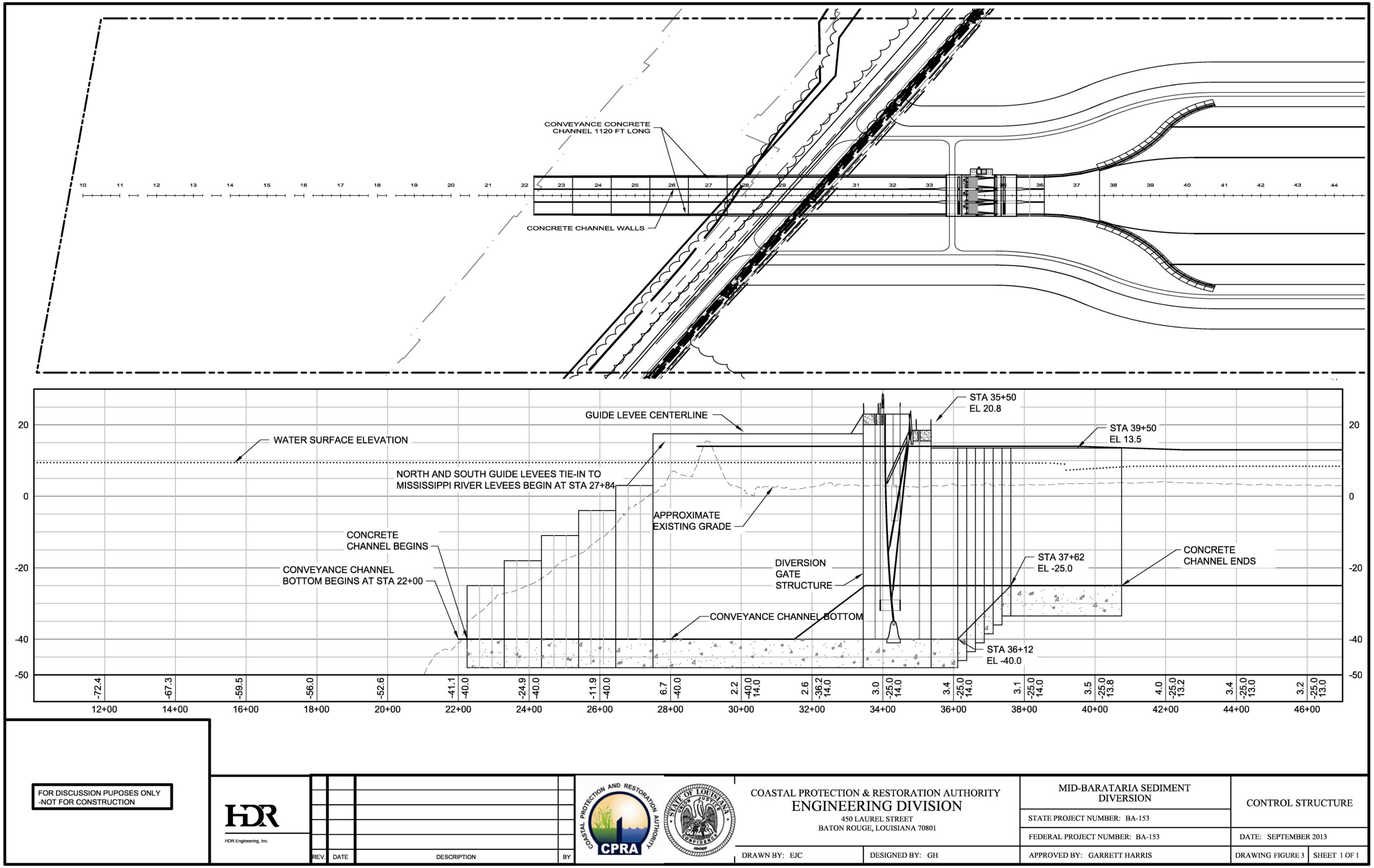
The transition structure is a series of concrete "U"-shaped channels that step up in channel bottom elevations from –40 feet to –25 feet between Station 33+67 and Station 35+17. The wall heights are proposed to be at elevation +13.5 for all steps. The connection between each channel step—and the lengths of each structure—are still being developed. However, the dead load will not exceed 3,000 psf for each step. Since this portion of the channel will be permanently submerged without the capability of dewatering, these structures are not anticipated to experience uplift pressures.

Transition Walls

Transition walls are proposed to transition water from the control structure to the conveyance channel. The walls are proposed to be between Station 35+17 and Station 39+00 with articulated mats in the channel, and will be the downstream terminus for the inlet system. The walls are proposed to have footings founded at elevation –31 feet. The tops of the walls are proposed to be at elevation +13.5 feet. The tops of the articulated mats will be on the bottom of the channel at elevation –25 feet. In plan view, the wall will have an "S" shape that will be the final transition from the 192-foot-wide channel to the 300-foot-wide main channel and channel slope configurations. The main channel slope configurations are proposed to be 4.5:1 (horizontal to vertical).

Figure 2. Conveyance channel

Figure 3. Control structure



Railroad Bridge

An existing New Orleans and Gulf Coast Railway spur parallels the toe of the MR&T Levee along the western embankment of the Mississippi River. The spur currently terminates approximately 1,000 feet south the proposed MBSD conveyance channel centerline. The current proposal is to realign the railroad track to cross the MBSD conveyance channel at approximately Station 63+00. As part of a separate project, the railroad will be extended farther south of the current terminus. Current configurations present the railroad as a bridge with bents at 30 feet on center for the north and south approaches. Bents associated with the conveyance channel crossing will be spaced at 125 feet and 150 feet. The anticipated loading conditions have been provided by the design team as 2,225 psf and 1,219 psf for live and dead loads, respectively. Live loads include a fully loaded freight train, while dead loads include ballast and track.

LA 23 Bridge

LA 23 is a four-lane concrete highway with a north-to-south alignment. The highway is the main link between New Orleans and areas south along the peninsula. Within the limits of the project site, the highway appears to be relatively flat and generally supported on fill that raises the alignment above the surrounding topography. The change in elevation appears to be between 4 and 8 feet above the surrounding topography.

The highway will be reconfigured into a bridge over the proposed conveyance channel near Station 65+00. The current configuration places highway bents at 125- and 150-foot spacing at the channel crossing. The proposed dead loads of the bridge are approximately 2,000 psf.

Where the highway crosses over the conveyance channel, a floodwall will be constructed within the proposed MBSD guide levees. The floodwalls will consist of concrete walls supported on deep foundations under the bridge.

Back Structure

The back structure is currently proposed to be located at the western end of the conveyance channel between Station 120+00 and Station 140+00. The preliminary footprint has been proposed to be approximately 300 feet by 150 feet. The exact location and footprint is still in development. The preliminary dead loads have been determined to be on the order of 2,500 psf in compression at the foundation elevation of –32 feet. Table 2 presents anticipated loading conditions for the back structure.

Table 2. Back structure pile loading conditions

Loading condition	Dead loads (pounds per square foot)
Immediately after construction	2,500
Effective bearing pressure	2,000
Effective plus high water elevation	4,060

3 Geology and Geomorphology

3.1 Geologic Setting

This discussion is based on review of preliminary boring logs, laboratory testing from 30% design investigations, and available USACE exploration data in the site vicinity. Available published reports describing local geomorphology were also reviewed. This discussion should be viewed as a general description of the site geology and will be refined as more data become available. A site plan showing explorations between the MR&T Levee and the back levee (NOV Levee) is shown on Figure 2.

The southern coast of Louisiana has been formed over many thousands of years and in discernible depositional deltaic lobes. The site is located in the Plaquemines complex lobe that is estimated to be only a few hundred years old. Because of its relatively young age, the site is underlain by a relatively thick sequence of fine-grained soils that range in consistency from very soft to medium stiff. These soils have generally compressed under their self weight, termed as normally consolidated. The Plaquemines complex is also undergoing regional subsidence given its young age. Geologic maps of the southern Mississippi River area, prepared by USACE and others, show that the site is crossed by two abandoned distributary channels that splayed from previous courses of the Mississippi River. The two abandoned channels are most likely of different ages and, combined, formed a wide natural levee ridge. West of the ridge, brackish marshes formed on the fringe of the current Barataria Basin. To the east of the ridge lies the Mississippi River and associated point bar deposits.

The site can be characterized/divided into four major geomorphologic areas/reaches progressing from east to west: (1) point bar, (2) natural levee/ abandoned distributary channel, (3) marsh/backland area, and (4) the Barataria Basin/ Marsh. Beginning at the Mississippi River, the site is underlain by deep point bar deposits extending to a depth of about 125 feet, overlain by about 10 feet of fine-grained natural levee deposits. The western limit of the point bar deposits is estimated to be located approximately 1,500 feet west of the MR&T Levee (Station 45+00); however, the actual location will need to be confirmed as part of the 60% design explorations. Recent surveys indicated that the ground surface is generally between elevation +2.5 to +4 feet (North American Vertical Datum [NAVD] 1988) except for interior channel flow lines that are near elevation 0 feet. The MR&T Levee has a crown elevation of approximately +15.5 feet.

The site is mapped, and HDR's explorations encountered two abandoned distributary channels that cross the conveyance channel alignment at approximately Station 55+00 and Station 80+00 (east and west channel, respectively). The eastern channel has been mapped by USACE as being about 850 feet wide, and current explorations indicate it may have a depth of about 58 feet. The western channel appears to be a relic of the Chenière Traverse Bayou and had been mapped with a width about 436 feet; current explorations indicate it may have a depth of about 43 feet. Both the lateral limits and depths of these channels need to be confirmed in the 60% design phase of investigation.

Between the point bar deposits (about Station 45+00) and the marsh backland (about Station 97+50), the near-surface soils consist of natural levee deposits to a depth of about 30 to 40 feet. Ground surface elevations range from +0 to +2 feet west of LA 23 (Station 65+00), up to elevation +3.8 feet in the median of the highway, and vary from elevation +0.5 to +0 feet west of the highway. Below the surficial natural levee deposits, the explorations encountered relatively uniform clay soils, referred to as interdistributary deposits, to depths of about 50 to 65 feet. These soils are underlain by, in order of increasing depth, stiffer fine-grained prodelta deposits extending to depths of about 100 to 108 feet, near-shore deposits extending to depths of about155 to 130 feet, and older very stiff Pleistocene-aged deposits encountered at depths of about118 to 130 feet (corresponding to the maximum depth explored). The sequence of interdistributary, prodelta, and near-shore deposits overlying the Pleistocene-aged soils is present beneath the conveyance channel from Station 45+00 on the east to the back levee on the west (Station 140+00). These soils are absent beneath the point bar deposits that extend to the Pleistocene soil interface at a depth of about 120 feet.

The western portion of the site is overlain by very soft, organic rich marsh deposits (Station 97+50 to Station 140+00). In this area, the ground surface elevation ranges from –2 to –5 feet (NAVD 88), and groundwater was encountered within 12 inches of the ground surface. Marsh deposits (primarily organic clays) were encountered to a depth of about 16 feet. Beneath the marsh deposits, a sequence of soft fine-grained interdistributary deposits was encountered with variable thickness layers of sand and silty sand to a depth of about 32 to 38 feet. The layers of more granular soil lenses are commonly identified as interdelta deposits. This interbedded zone was found to be between 12 to 16 feet thick. However, in one exploration, the interdelta soils were found to be 4 feet thick.

Soil shear strengths are softest (lowest) on the west near the back levee, measured as being as low as 100 psf in the upper 20 feet, and were found to generally increase toward the natural levee ridge formed by the abandoned interdistributary ridge. Vertically, shear strengths were found to be fairly uniform in the upper 15 to 20 feet and then uniformly increased with depth.

3.2 Regional Groundwater

The site is located between the Mississippi River and the Barataria Basin that is, in turn, connected to a series of lakes west of the site. The water surface in the Mississippi River varies and is typically highest in March to June. The water surface within the Barataria Basin is relatively uniform with some minor tidal influence. Both water surfaces are typically higher in elevation that the site ground surface elevations. Groundwater is maintained below the ground surface by a number of drainage ditches and collection canals that cross the site. Forced drainage is used to pump into the basin.

In terms of dewatering the site, the site is bounded by two infinite sources of water, the Mississippi River and the Gulf of Mexico. Results of initial pump testing performed as part of the current investigations indicate that a series of closely spaced wells would be needed to provide substantial drawdown, and the dewatering system would need to be operated continuously to maintain drawdown. Groundwater levels would be reestablished to their current levels relatively quickly should the dewatering system be turned off. Therefore, it is anticipated that the conveyance channel will be excavated in the wet by dredging methods.

4 Site Investigations

HDR and its subconsultant, GeoEngineers, are in the process of completing Phase 1 of geotechnical explorations and laboratory testing for the MBSD. Phase 1 investigations will provide an initial site characterization to identify geotechnical-related project conditions and constraints and will support both preliminary engineering planning and conceptual plan development. Exploration consisted of drilling and sampling soil borings (both 5-inch diameter and 3-inch diameter borings), in-situ field vane strength testing, cone penetration tests (CPTs), piezometer installation, laboratory testing, and two pump tests.

Explorations in the Barataria Marsh consisted of advancing 16 borings throughout a widespread area to provide a general characterization of the future land construction area. The explorations focused primarily on the area west and south of the MBSD conveyance channel outfall. Explorations along the MBSD alignment consisted of advancing 19 explorations points (borings, piezometers, in-situ field vanes, and CPTs) that were conducted at approximately 1,000-foot intervals along the conveyance and intake channel alignments. Explorations along the MR&T west bank levee consisted of advancing 17 borings and CPTs through the levee crown and at the levee landside toe. Limits of the MR&T explorations extended approximately 2,000 feet north and south of the conveyance alignment. Explorations for the river and along the west bank batture were recently completed. No explorations were performed along the back levee since the design team was provided the USACE explorations related to its NOV Levee Improvement Project, which included a significant number of explorations at the conveyance channel outfall. Geotechnical laboratory testing of samples, data reduction, and interpretation of the river and batture borings are ongoing and will be included in a geotechnical data report by GeoEngineers to be published later this year.

A Phase 2 investigation is currently in the development stages and will be used to provide additional data for more complete designs.

5 Design Criteria

Hydraulically related design criteria are key to meeting the project goal of transporting sediment to the Mid-Barataria Basin from the Mississippi River. These hydraulically related design criteria also play a large role in dictating geotechnical demands, such as foundation loads and pressures. The hydraulic modeling provided at the 30% design stage dictates that the structures will operate at various water elevations during the life of the project.

Furthermore, criteria related to constructibility have also been developed. Hydraulic operating levels for the conveyance project have been developed as follows:

- Low operating water elevation in the channel is a maximum of –5 feet.
- High operating water elevation in the channel is a maximum of +10 feet.

The project must also meet criteria at the junction of the MR&T and NOV-NFL Levees, both of which are subject to federal review (including Section 408 requirements). This includes as a minimum:

- Maintain flood protection redundancy during construction.
- At a minimum, provide the same level of flood protection as the current MR&T Levee.
- Accommodate future federal levee raises to reestablish the authorized crown elevation. The MR&T Levee currently has a deficient freeboard elevation for the project design event.

- Allow the NOV-NFL to provide surge protection for a 20-year storm recurrence interval, currently estimated to require a minimum back levee crown elevation of 9 feet. It should be noted that the levee is anticipated to settle over time and will be overbuilt so that the crown elevation does not drop below the minimum elevation for a period of 10 years. The levee crown may be raised again after 10 years to maintain the level protection, depending on funding.
- Provide rear surge protection.

5.1 Design Water Surface

Water surfaces are influenced by the Mid-Barataria Basin, Mississippi River, and storms. Influences on water elevations vary across the site as a result of geologic conditions and water sources. The design long-term groundwater elevation for the project is 0 feet. The free water elevation in the conveyance channel is designed to be no less than elevation 0 feet for the life of the project. Different structures will have differing operating free water surface conditions. The high water elevations vary for the inlet structures from that of the back structure. It should be noted that recent discussions with the USACE New Orleans District Civil Design Branch revealed the requirement that any modifications to the MR&T Levee System must meet the USACE flood protection criteria at the MR&T Levee System design flood event. The current federal authorization for the MR&T Levee at this location does not meet 100-year flood protection nor does it include the requirement to meet the Hurricane and Storm Damage Risk Reduction Design standards.

At this site, the MR&T Levee is deficient in terms of levee freeboard. USACE requires the top of structures to have 2 feet of structural superiority over the adjacent levee to allow for future modification (raise) without affecting the structure. Currently, the walls of the inlet system structures are proposed to be at an elevation 2 feet higher than the existing MR&T Levee crown elevation, but may need to be raised 2 feet above the authorized height of the MR&T Levee.

The back levee will be federalized as part of the NOV-NFL Project that will provide up to 20-year level hurricane surge protection. The NOV-NFL originally was designed to maintain a levee crown elevation of at least +10 feet for the first 10 years after construction. It is understood that the criterion is under review and the requirement may be reduced to as low as elevation +9 feet.

5.2 Foundations

The project site can be grouped into two areas with similar generalized conditions at the anticipated foundation levels: (1) primarily cohesionless, high-permeability point bar deposits extending from the eastern boundary of the project site adjacent to the Mississippi River to approximately Station 40+00, and (2) a relatively low-strength cohesive, low-permeability material that extends from Station 40+00 to the western boundary of the project site. Strength versus depth plots were developed as part of the Phase 1 investigation, and in-situ characteristics were generally described in Section 3.

It is HDR's understanding that the allowable vertical deformations for the foundations are on the order of 2 to 3 inches during construction as a means of mobilizing the required reaction forces. Allowable long-term vertical deformations are being developed by structural teams and will be analyzed as further field information becomes available.

5.3 Subterranean Walls

As discussed above, the project site generally has two generalized conditions. Wall performance criteria in these conditions have been set at an at-rest (K_0) condition by the project team's structural engineers. Furthermore, in the highly permeable point bar materials, subterranean walls will be needed to penetrate to sufficient depths to allow for dewatering prior to excavations while preventing piping into the subgrade.

6 Analysis Methodologies and Preliminary Recommendations

The structural engineering teams for the inlet system, back structure, and bridge structures provided preliminary loading conditions for the structures.

Design loads were determined by the structural design as described in Section 2.4, in conjunction with the design water surface discussed in Section 5.1. While the inlet control structure, highway floodwall, and back structure would have conditions that would load the soil in compression, only the inlet control structure has the potential to experience a net uplift/pullout loading condition. Uplift pressures that could be experienced in that condition are described in Section 6.2.

6.1 Horizontal and Uplift Wall Pressures

Both temporary and permanent wall loading conditions were analyzed. These conditions were in part subject to the in-situ materials characteristics at the relative locations of each structure and the required performance of each structure. The inlet system structures are proposed to be within saturated sand deposits of the point bar materials. The back structure is within the saturated clay marsh deposits. Therefore, the methods used for the respective structures were selected based on these material characteristics.

The inlet and outlet systems will consist of relatively stiff monolithic concrete structures designed to limit deflection and rotation under lateral loading. Therefore, permanent lateral earth pressures were calculated for at-rest and fully drained (long-term) conditions. At-rest earth pressure coefficients (K_0) were evaluated based on assumed drained friction angles of 28 degrees for fat clay (CH) (at the outlet system only) and 30 degrees for lean clay, loose sands, and clayey engineered fill (at both the outlet and inlet systems). Native soils were assumed to be normally consolidated for the purpose of establishing at-rest earth pressure coefficients. Unit weights of native soils ranging from 105 to 120 pounds per cubic foot (pcf) were selected based on laboratory data from Borings NL-3A and NL-9A. Engineered fill was assumed to exhibit a unit weight of 125 pcf with total fill thickness for both systems of 15 feet. For the inlet system, HDR assumed a finished grade elevation of +19 feet, top of slab elevation of –40 feet, and bottom of slab elevation of –47 feet. For the outlet system, we assumed a finished grade elevation of +15 feet, top of slab elevation of –25 feet, and bottom of slab elevation of –30 feet. Groundwater table elevations are assumed to be at +0 feet at the inlet system and –4 feet at the outlet system.

Preliminary lateral earth pressure diagrams and uplift diagrams were developed based on the above assumptions and are presented in Appendix A. Preliminary hydrostatic and earth pressures for the condition when the systems may operate dry or empty were developed and are also presented in Appendix A.

6.2 Shallow Foundation Bearing Capacities

Given the relatively high strength of soil at the base of the inlet control structure, a shallow foundation is a potential option for a foundation solution. To explore this option, bearing capacity analyses were performed.

Four bearing capacity analysis methods—Terzaghi (1943), Meyerhof (1951, 1963), Hansen (1970), and Vesic (1973, 1975)—were used to explore this option. Calculations were performed for the ultimate and allowable capacities using the aforementioned analysis methods; they are summarized in Appendix B. Both the Hansen and Vesic methods are described in Bowles (1996) as being appropriate for depth to foundation width (D/B) ratios greater than 1, while the Terzaghi and Meyerhof methods are not appropriate. Accordingly, calculations for D/B ratios greater than 1 were made using only the Hansen and Vesic methods.

To calculate the allowable capacities, a factor of safety of 3 was assumed. The calculations were taken to a depth of 200 feet from the surface to determine capacities in the bearing area for a shallow foundation and to potentially assist in determining an appropriate group factor for a deep foundation design at the inlet control structure. Preliminary recommended bearing capacities are presented in Appendix B.

6.3 Pile Capacities

Pile capacity analyses were performed using a CPT-based empirical pile capacity method developed by Nottingham and Schmertmann (1975) and modified by the Federal Highway Administration (1998). Pile capacities were developed using assumed sleeve friction (f_s) and tip resistance (q_c) profiles determined directly from the acquired CPT data to develop skin friction resistance and tip bearing capacities. The f_s and q_c profiles for the inlet control, floodwall, and back structure are summarized in Appendix C.

Pile capacities versus depth were calculated at the inlet control structure, floodwall, and back structure for two pile diameters at each. Two loading conditions were considered for the inlet control structure: heave resulting from subsurface head pressures while the structure is empty/buoyant and compression loads resulting from fully flooded operating conditions.

The proposed railroad and highway bridge foundations are within an area identified as natural levee deposits. Deposits have been characterized in regional geologic mapping and specifically in the Phase 1 investigation as soft to stiff clays and silts. The foundation recommendations for these structures and specifically for the highway floodwall are based on the limited data collected between Station 60+00 and Station 75+00.

Between Station 125+00 and Station 140+00, the proposed back structure will be constructed in soft to stiff clay and silt deposits. These materials were encountered to the depths explored and are mapped to be significantly deeper. As described above, the structure is anticipated to be continually flooded in at least five of the seven gate sections at any given time. Therefore, a net positive uplift force is not anticipated at this time. However, the structural conditions presented in Table 2 are anticipated to experience compression dead loads on the foundation and lateral pressures on the structures walls.

Preliminary pile capacities versus depth are presented in Appendix D. The calculations were taken, at a minimum, to a depth corresponding to the deepest representative CPT for the site. Where loading conditions require depths of piles to be greater than the CPT data, the f_s and q_c profiles were extrapolated without incorporating further strength gain as depth increased. When the profiles were extended, the calculated pile capacity figures are noted from which depth this was done.

Given the length and resulting weight of the steel pipe piles, the calculated capacities were adjusted accordingly, where noted. The resulting capacities in compression and uplift/pullout were decreased and increased, respectively.

In addition to the pile weights, the added uplift/pullout resistance from the soil plug in the pile was also included. The bottom 25 percent of the piles was assumed to be plugged and the minimum of either the soil resistance on the inside of the pile or the weight of the plug itself was added to the uplift/pullout resistance. The added resistance from the plug resulted in a minimal increase in capacity, and thus was not distinguished from the increase in resistance attributable to the weight of the pile in the pile capacity figures.

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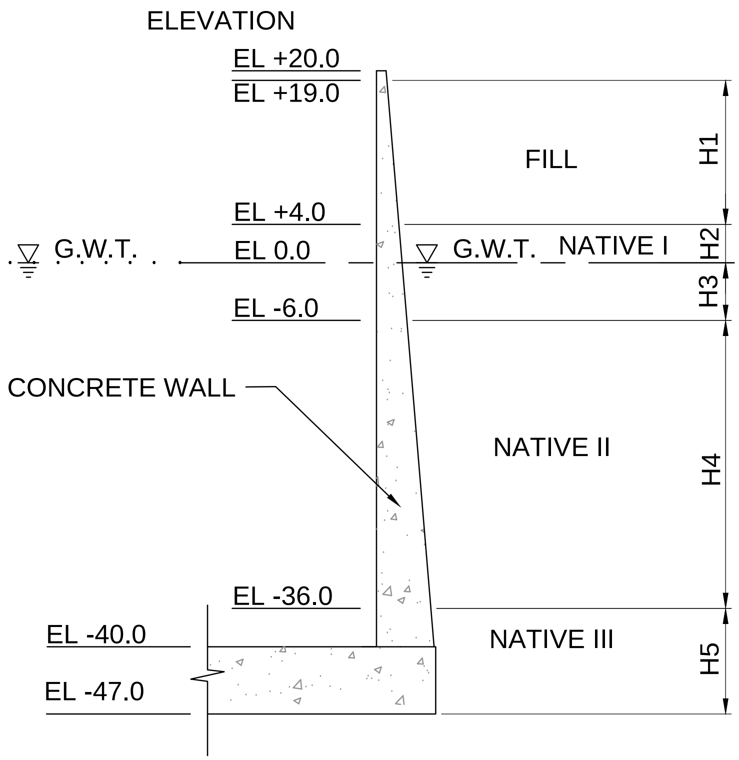
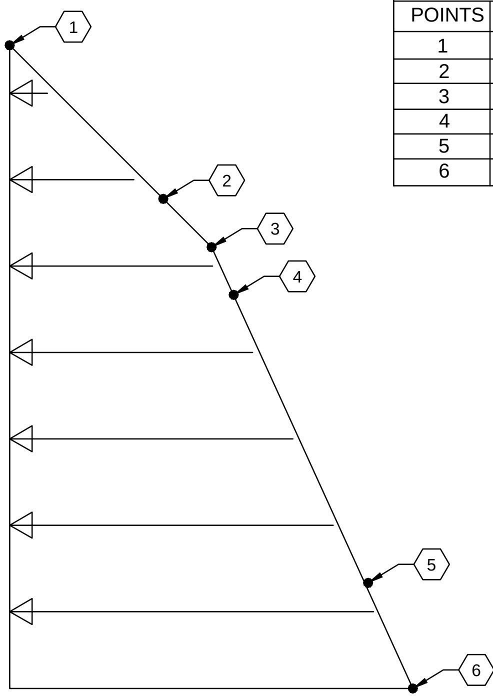
Appendix A. Horizontal and Uplift Wall Pressures

INLET SYSTEM (OPERATING CONDITION)



INLET STRUCTURE



LATERAL EARTH PRESSURE

SOIL PRESSURE POINTS		
POINTS	ELEVATION (ft)	SOIL PRESSURE (psf)
1	19	0
2	4	938
3	0	1168
4	-6	1325
5	-36	2189
6	-47	2506

SOIL PROFILE DESCRIPTION

LAYER	THICKNESS	SOIL TYPE	EFFECTIVE FRICTION ANGLE	SATURATED UNIT WEIGHT
FILL	H1 = 15ft	FILL(CL)	30°	125psf
NATIVE I (ABOVE G.W.T.)	H2 = 4ft	CL	30°	115psf
NATIVE I (BELOW G.W.T.)	H3 = 6ft	CL	30°	115psf
NATIVE II	H4 = 30ft	CL/ML/SM	30°	120psf
NATIVE III	H5 = 11ft	SM/ML	30°	120psf

DRAWING IS NOT TO SCALE

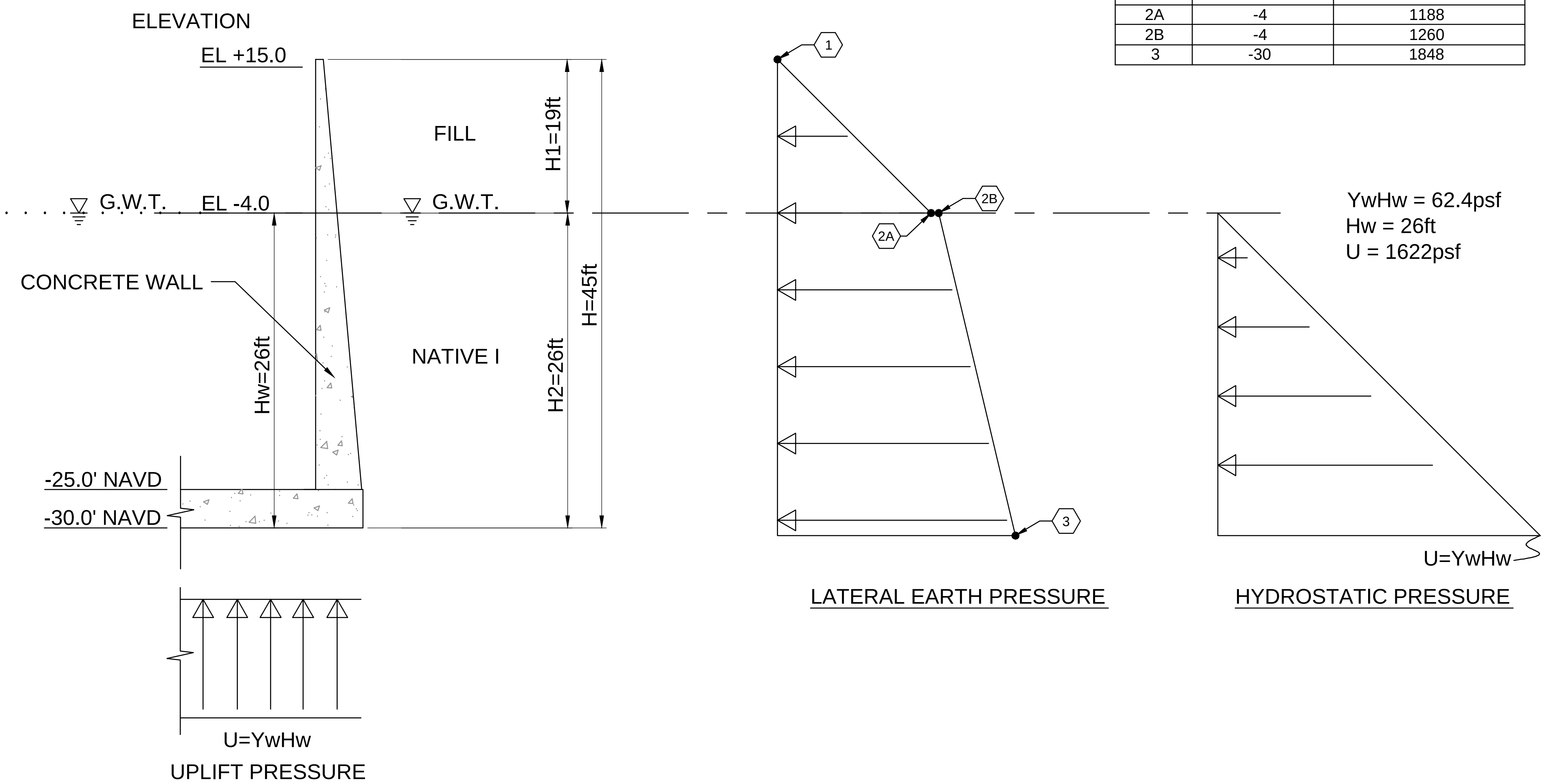
NOTES:

1. G.W.T. IS GROUNDWATER TABLE.

HDR Engineering, Inc.	COASTAL PROTECTION & RESTORATION AUTHORITY ENGINEERING DIVISION 450 LAUREL STREET BATON ROUGE, LOUISIANA 70801	MID-BARATARIA SEDIMENT DIVERSION	INLET STRUCTURE
		STATE PROJECT NUMBER: BA-153	
		FEDERAL PROJECT NUMBER: BA-153	DATE: OCTOBER 2013
	DRAWN BY: JCB / DESIGNED BY: GH	APPROVED BY: GARRETT HARRIS	DRAWING FIGURE 1 / SHEET 1 OF 4

INLET SYSTEM (EMPTY CONDITION)

SOIL PRESSURE POINTS		
POINTS	ELEVATION (ft)	SOIL PRESSURE (psf)
1	15	0
2A	-4	1188
2B	-4	1260
3	-30	1848



INLET STRUCTURE

NOTES:

1.UPLIFT PRESSURE IS EQUAL TO HYDROSTATIC PRESSURE AT THE BOTTOM OF SLAB ELEVATION. THIS PRESSURE IS UNIFORM ACROSS THE BOTTOM OF FOUNDATION.

2. REFER TO SHEET 2 FOR SOIL PROFILE DESCRIPTION.

DRAWING IS NOT TO SCALE

HDR

HDR Engineering, Inc.

COASTAL PROTECTION AND RESTORATION AUTHORITY

CPRA

STATE OF LOUISIANA — UNION JUSTICE CONFIDENCE

COASTAL PROTECTION & RESTORATION AUTHORITY
ENGINEERING DIVISION
450 LAUREL STREET
BATON ROUGE, LOUISIANA 70801

MID-BARATARIA SEDIMENT DIVERSION

STATE PROJECT NUMBER: BA-153

FEDERAL PROJECT NUMBER: BA-153

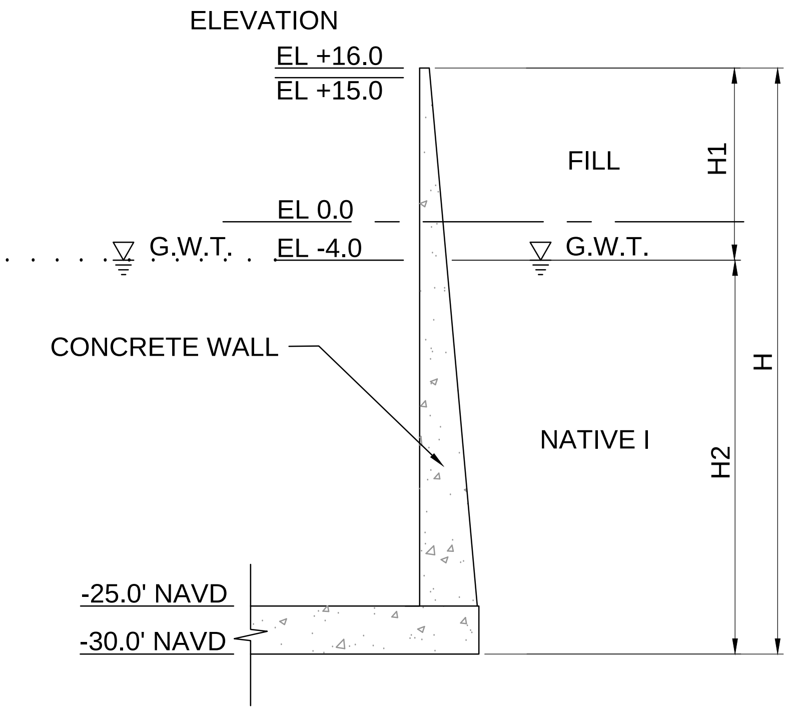
INLET STRUCTURE (EMPTY CONDITIONS)

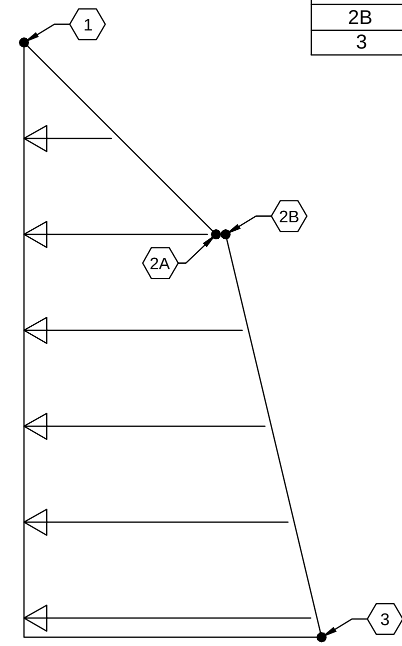
DATE: OCTOBER 2013

DRAWN BY: JCB | DESIGNED BY: GH | APPROVED BY: GARRETT HARRIS | DRAWING FIGURE 2 | SHEET 2 OF 4

BACK STRUCTURE (OPERATING CONDITION)

SOIL PRESSURE POINTS		
POINTS	ELEVATION (ft)	SOIL PRESSURE (psf)
1	15	0
2A	-4	1188
2B	-4	1260
3	-30	1848





BACK STRUCTURE

LATERAL EARTH PRESSURE

SOIL PROFILE DESCRIPTION				
LAYER	THICKNESS	SOIL TYPE	EFFECTIVE FRICTION ANGLE	SATURATED UNIT WEIGHT
FILL	H1 = 19ft	FILL(CL)	30°	125psf
NATIVE I	H2 = 26ft	CL/CH/CHO	28°	115psf

DRAWING IS NOT TO SCALE

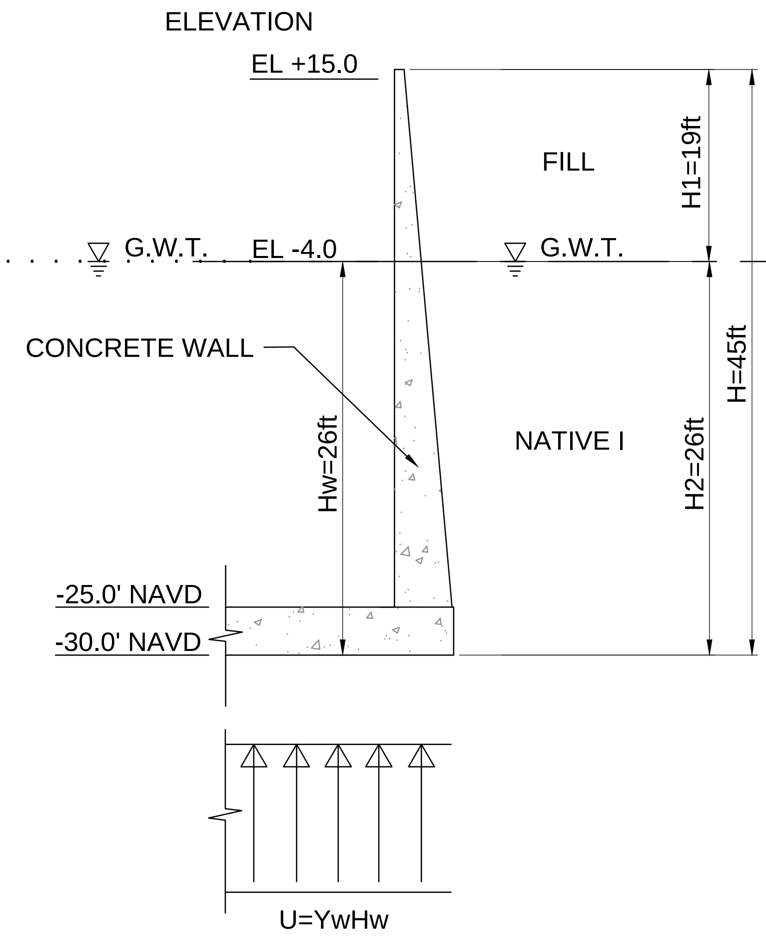
HDR Engineering, Inc.

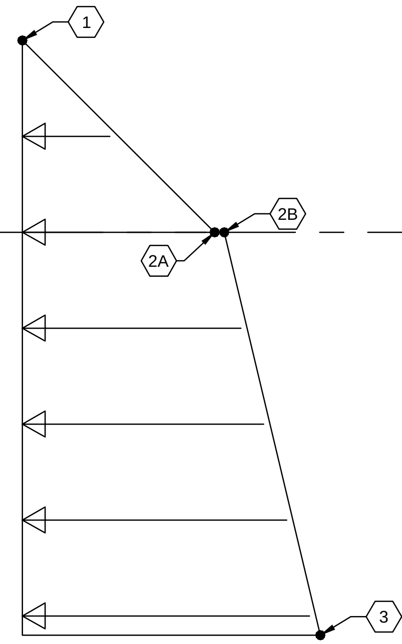
COASTAL PROTECTION & RESTORATION AUTHORITY
ENGINEERING DIVISION
450 LAUREL STREET
BATON ROUGE, LOUISIANA 70801

MID-BARATARIA SEDIMENT DIVERSION		BACK (OUTLET) STRUCTURE		
STATE PROJECT NUMBER: BA-153				
FEDERAL PROJECT NUMBER: BA-153		DATE: OCTOBER 2013		
DRAWN BY: JCB	DESIGNED BY: GH	APPROVED BY: GARRETT HARRIS	DRAWING FIGURE 3	SHEET 3 OF 4

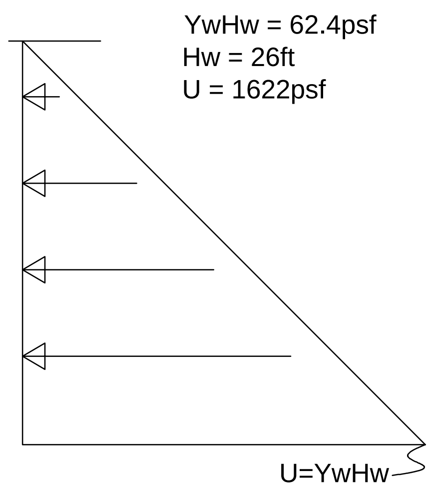
BACK STRUCTURE (EMPTY CONDITION)

SOIL PRESSURE POINTS		
POINTS	ELEVATION (ft)	SOIL PRESSURE (psf)
1	15	0
2A	-4	1188
2B	-4	1260
3	-30	1848





LATERAL EARTH PRESSURE



HYDROSTATIC PRESSURE

BACK STRUCTURE

NOTES:

1.UPLIFT PRESSURE IS EQUAL TO HYDROSTATIC PRESSURE AT THE BOTTOM OF SLAB ELEVATION. THIS PRESSURE IS UNIFORM ACROSS THE BOTTOM OF FOUNDATION.

2. REFER TO SHEET 3 FOR SOIL PROFILE DESCRIPTION.

DRAWING IS NOT TO SCALE

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COASTAL PROTECTION & RESTORATION AUTHORITY
ENGINEERING DIVISION
450 LAUREL STREET
BATON ROUGE, LOUISIANA 70801

MID-BARATARIA SEDIMENT DIVERSION

STATE PROJECT NUMBER: BA-153

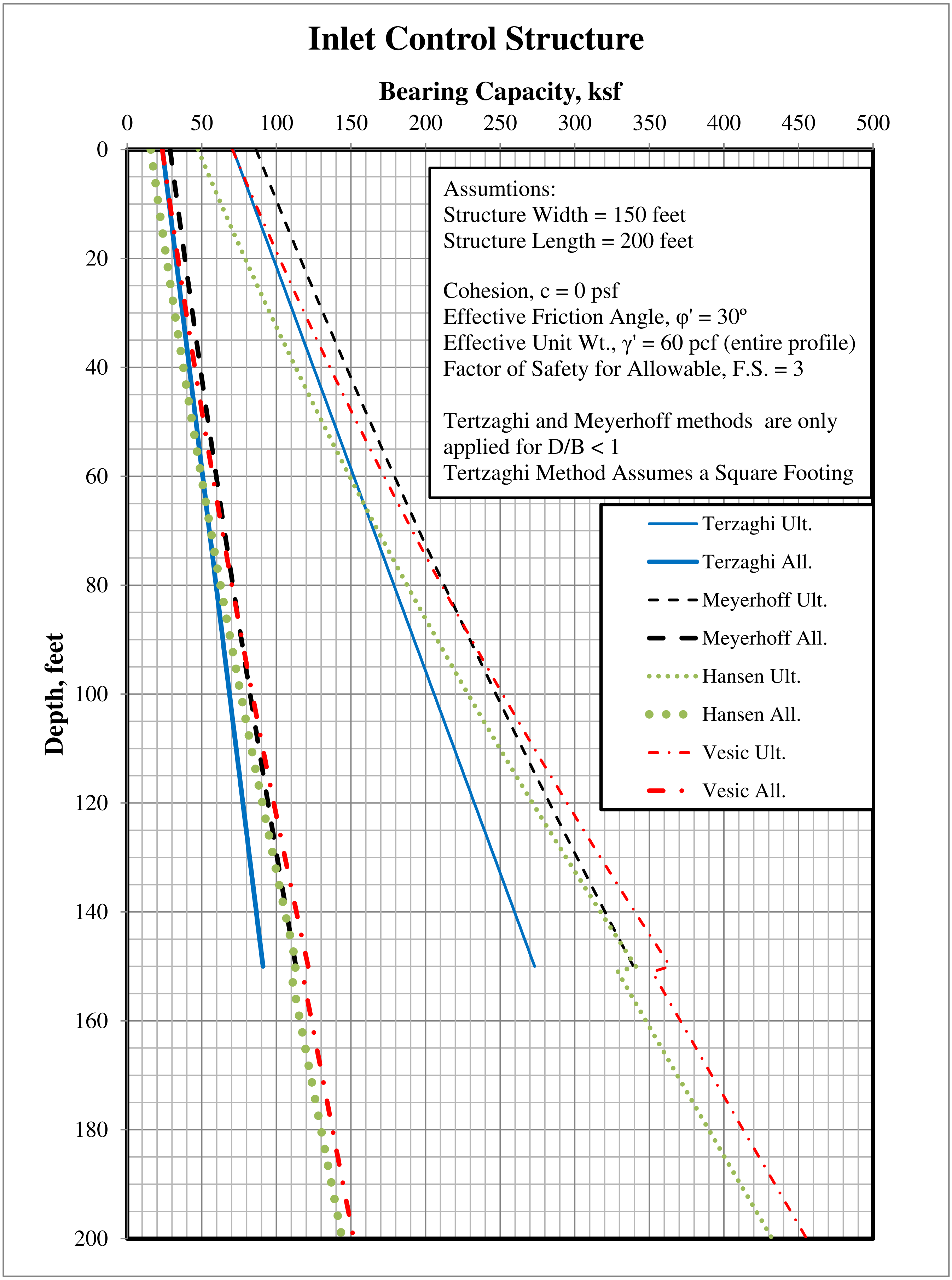
FEDERAL PROJECT NUMBER: BA-153

INLET STRUCTURE (EMPTY CONDITIONS)

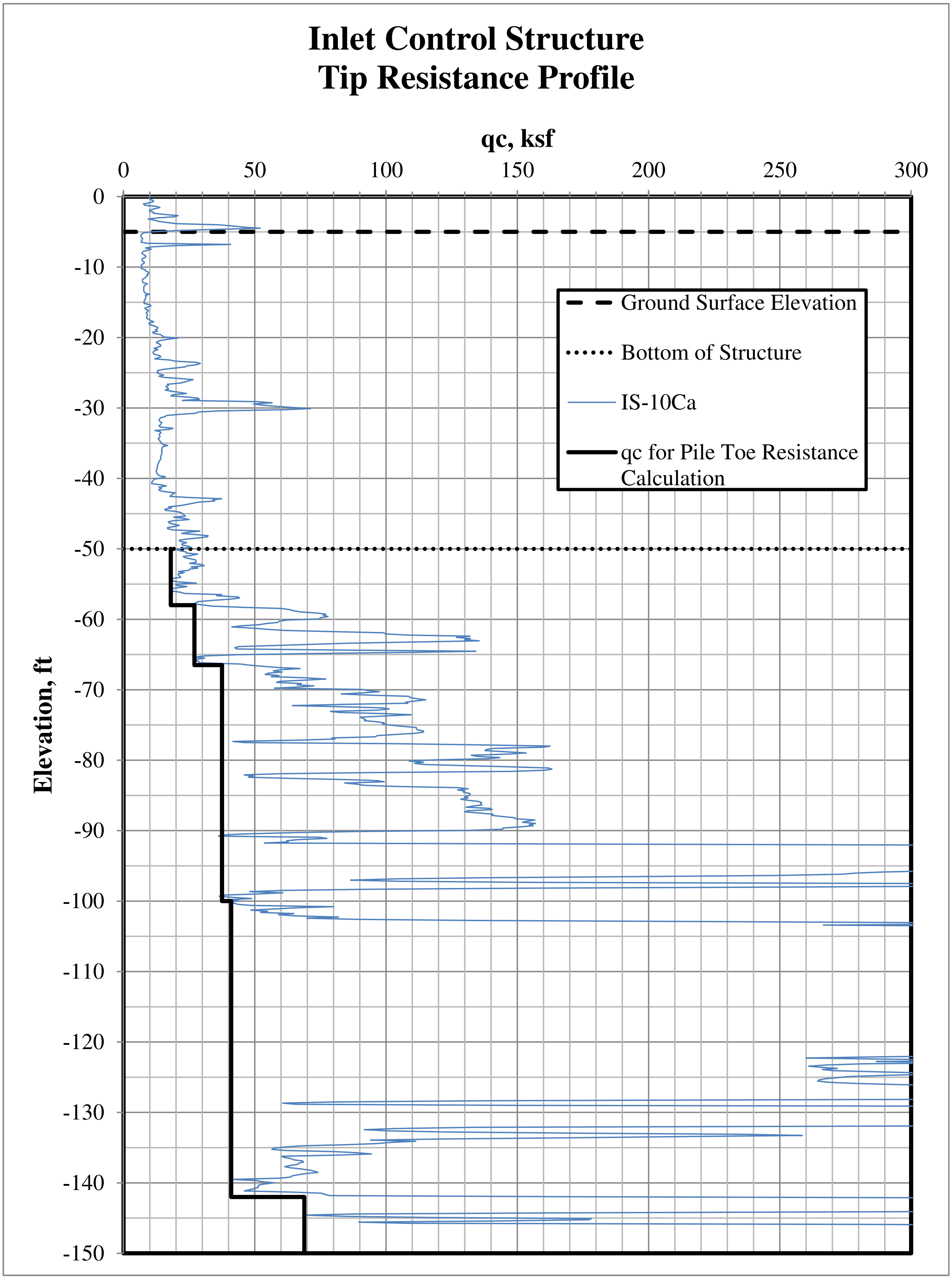
DATE: OCTOBER 2013

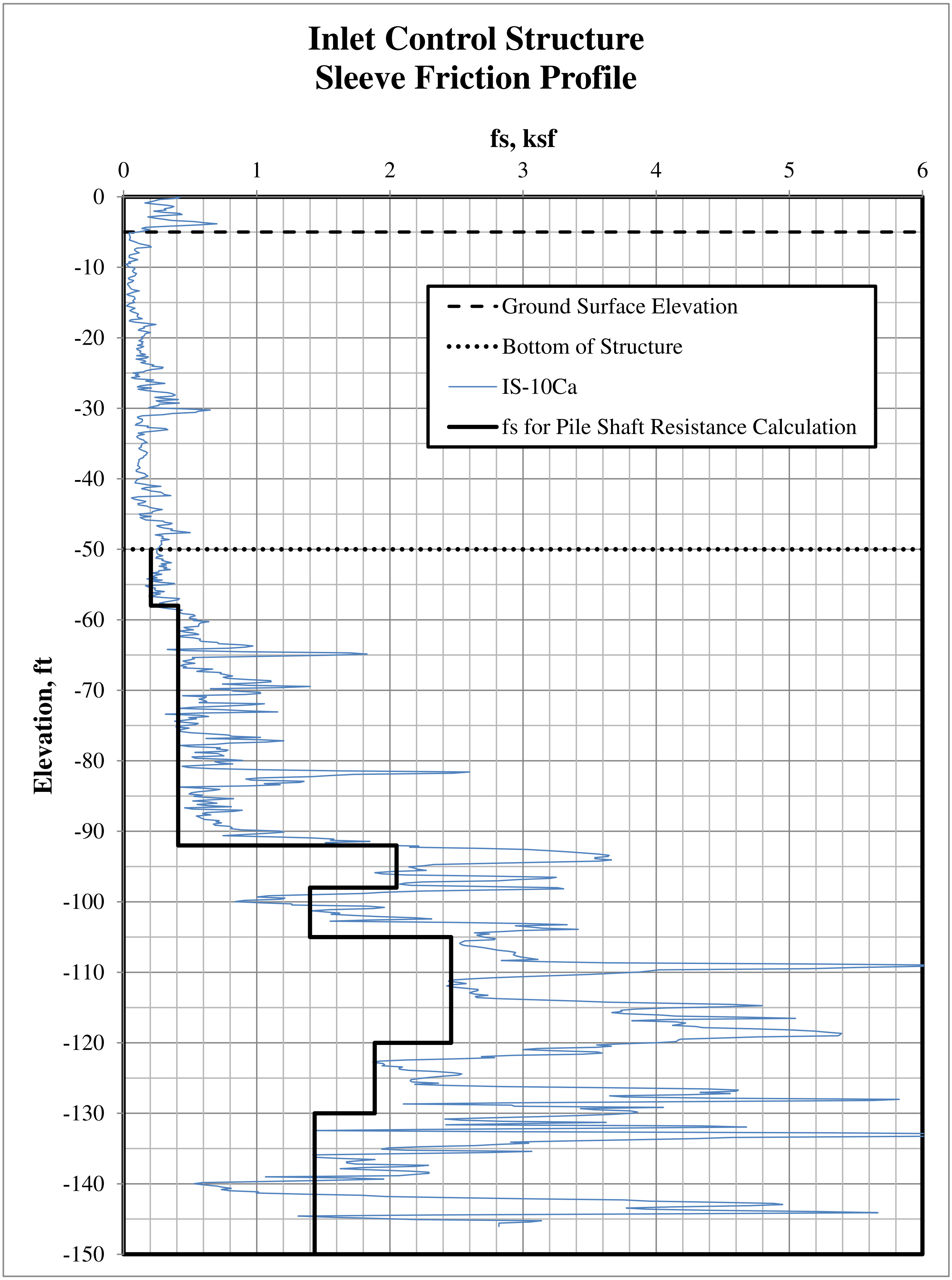
DRAWN BY: JCB | DESIGNED BY: GH | APPROVED BY: GARRETT HARRIS | DRAWING FIGURE 4 | SHEET 4 OF 4

Appendix B. Shallow Foundation Bearing Capacities

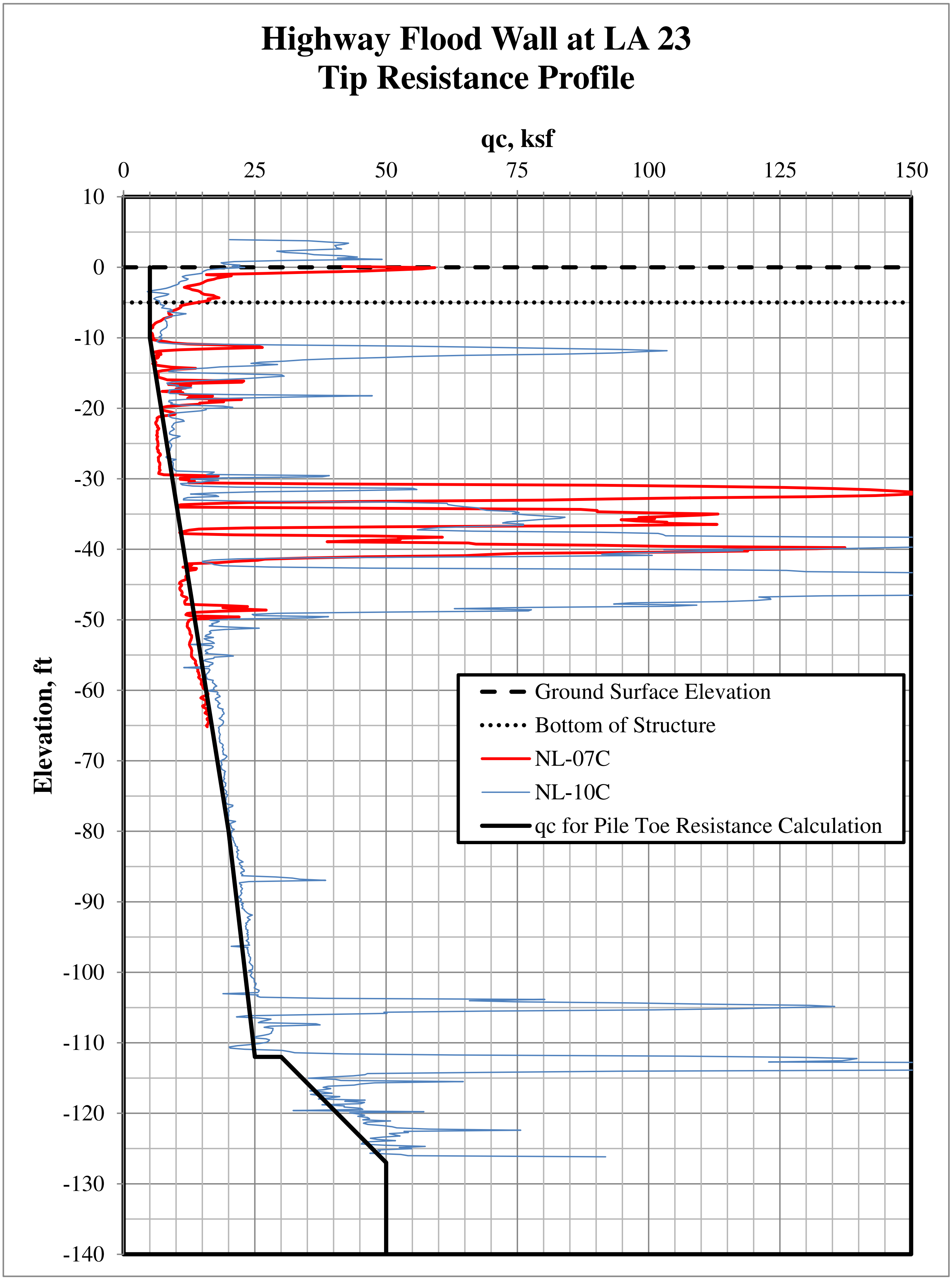

Inlet Control Structure
Bearing Capacity, ksf
0
50
100
150
200
250
300
350
400
450
500
Depth, feet
0
20
40
60
80
100
120
140
160
180
200
Assumtions:
Structure Width = 150 feet
Structure Length = 200 feet
Cohesion, c = 0 psf
Effective Friction Angle, φ' = 30º
Effective Unit Wt., γ' = 60 pcf (entire profile)
Factor of Safety for Allowable, F.S. = 3
Tertzaghi and Meyerhoff methods are only applied for D/B < 1
Tertzaghi Method Assumes a Square Footing
Terzaghi Ult.
Terzaghi All.
Meyerhoff Ult.
Meyerhoff All.
Hansen Ult.
Hansen All.
Vesic Ult.
Vesic All.

Appendix C. Cone Penetrometer Test Sleeve Friction and Tip Resistance Profiles

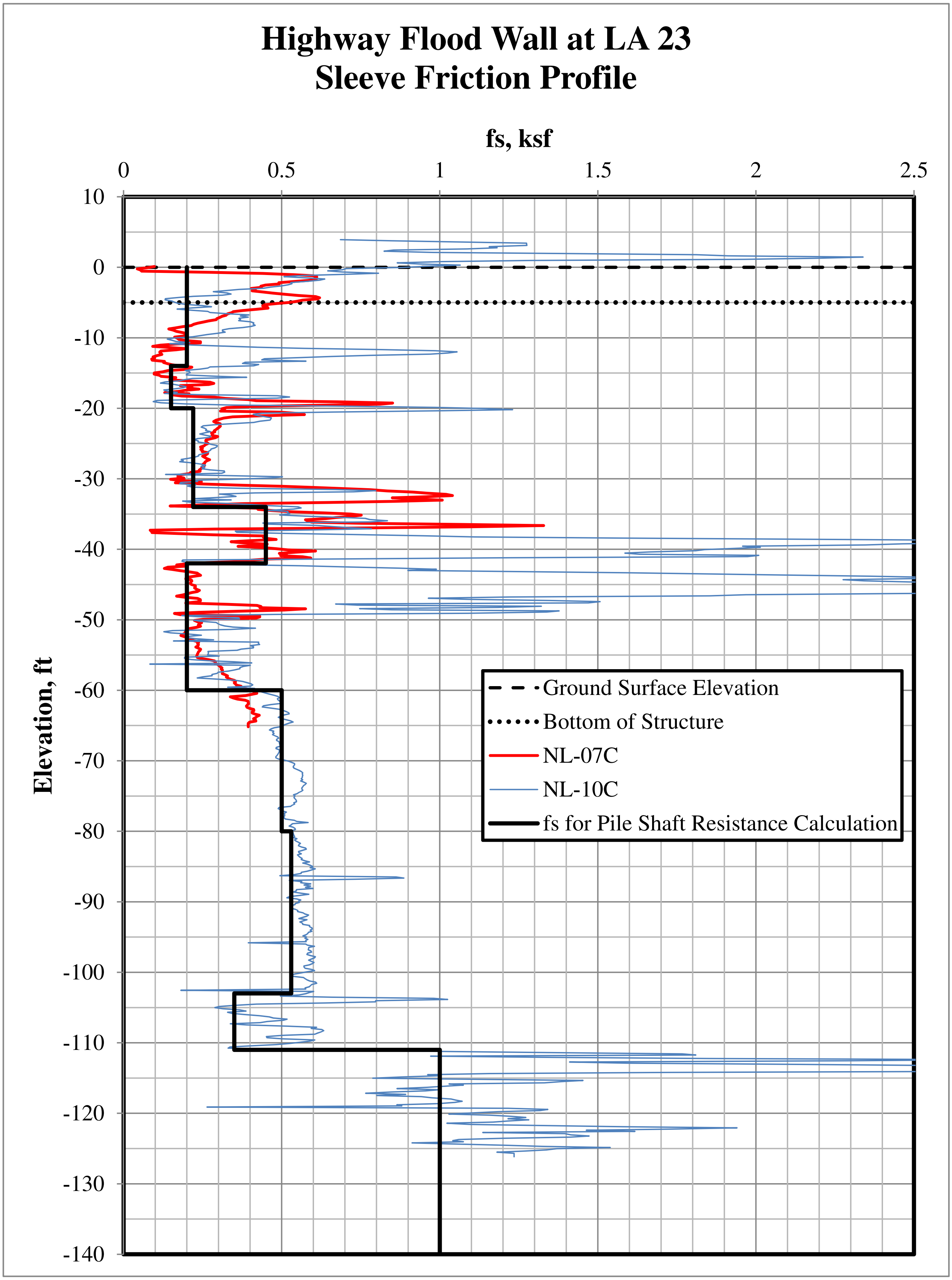

Inlet Control Structure
Tip Resistance Profile
qc, ksf
0
50
100
150
200
250
300
Elevation, ft
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-110
-120
-130
-140
-150
Ground Surface Elevation
Bottom of Structure
IS-10Ca
qc for Pile Toe Resistance Calculation

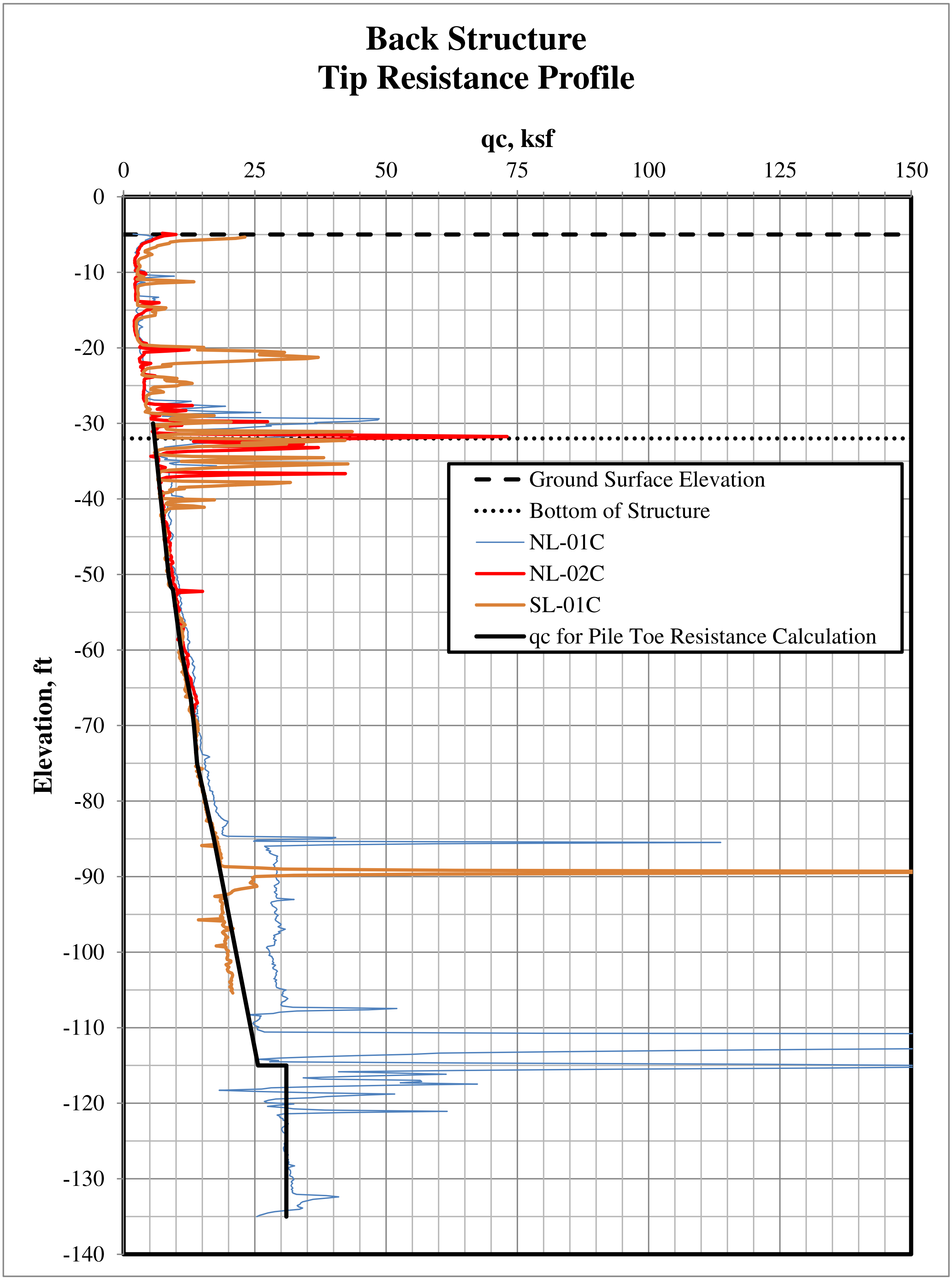


Inlet Control Structure
Sleeve Friction Profile
fs, ksf
0
1
2
3
4
5
6
Elevation, ft
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-110
-120
-130
-140
-150
Ground Surface Elevation
Bottom of Structure
IS-10Ca
fs for Pile Shaft Resistance Calculation

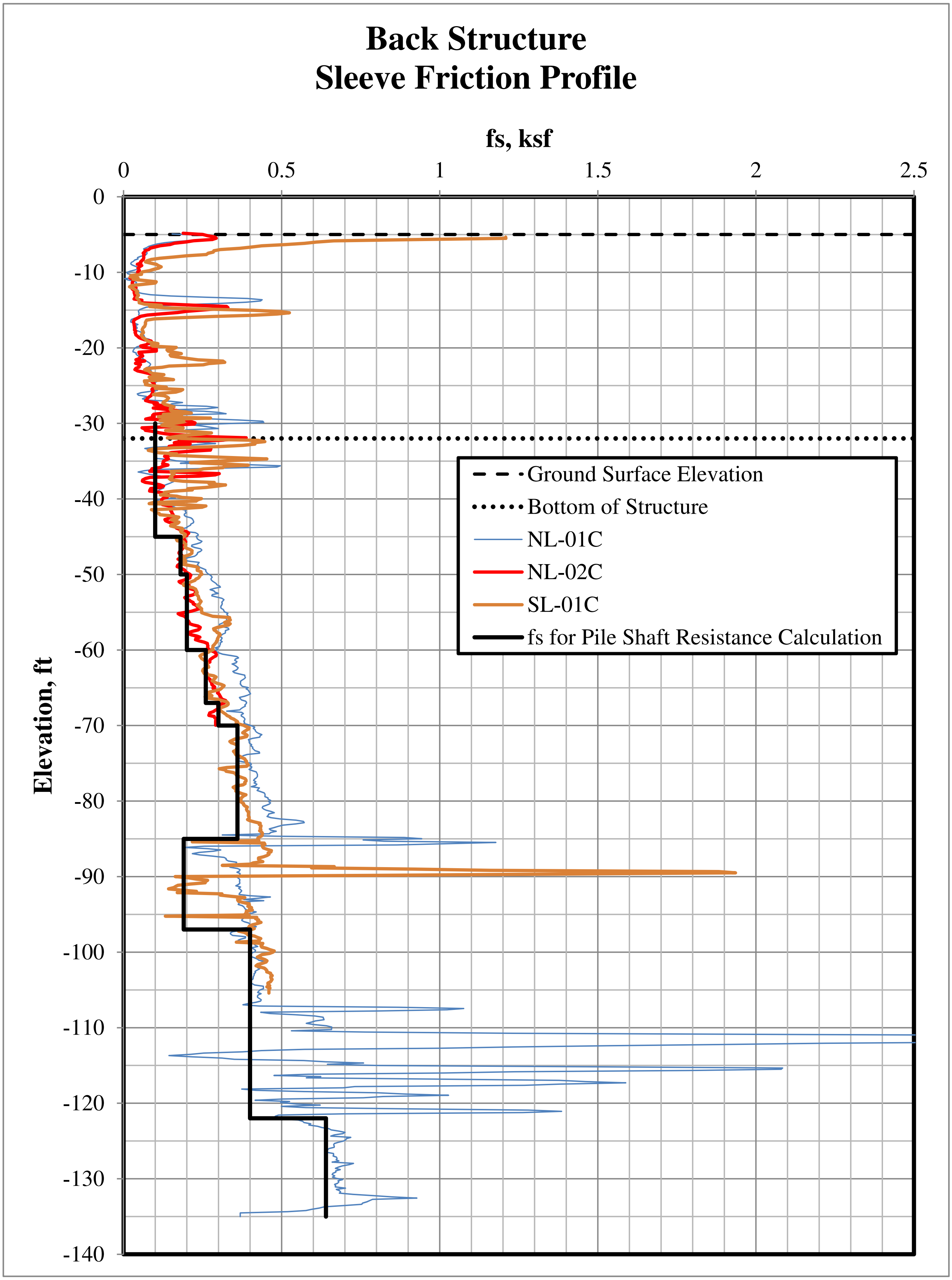


Highway Flood Wall at LA 23
Tip Resistance Profile
qc, ksf
0
25
50
75
100
125
150
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-110
-120
-130
-140
Elevation, ft
Ground Surface Elevation
Bottom of Structure
NL-07C
NL-10C
qc for Pile Toe Resistance Calculation

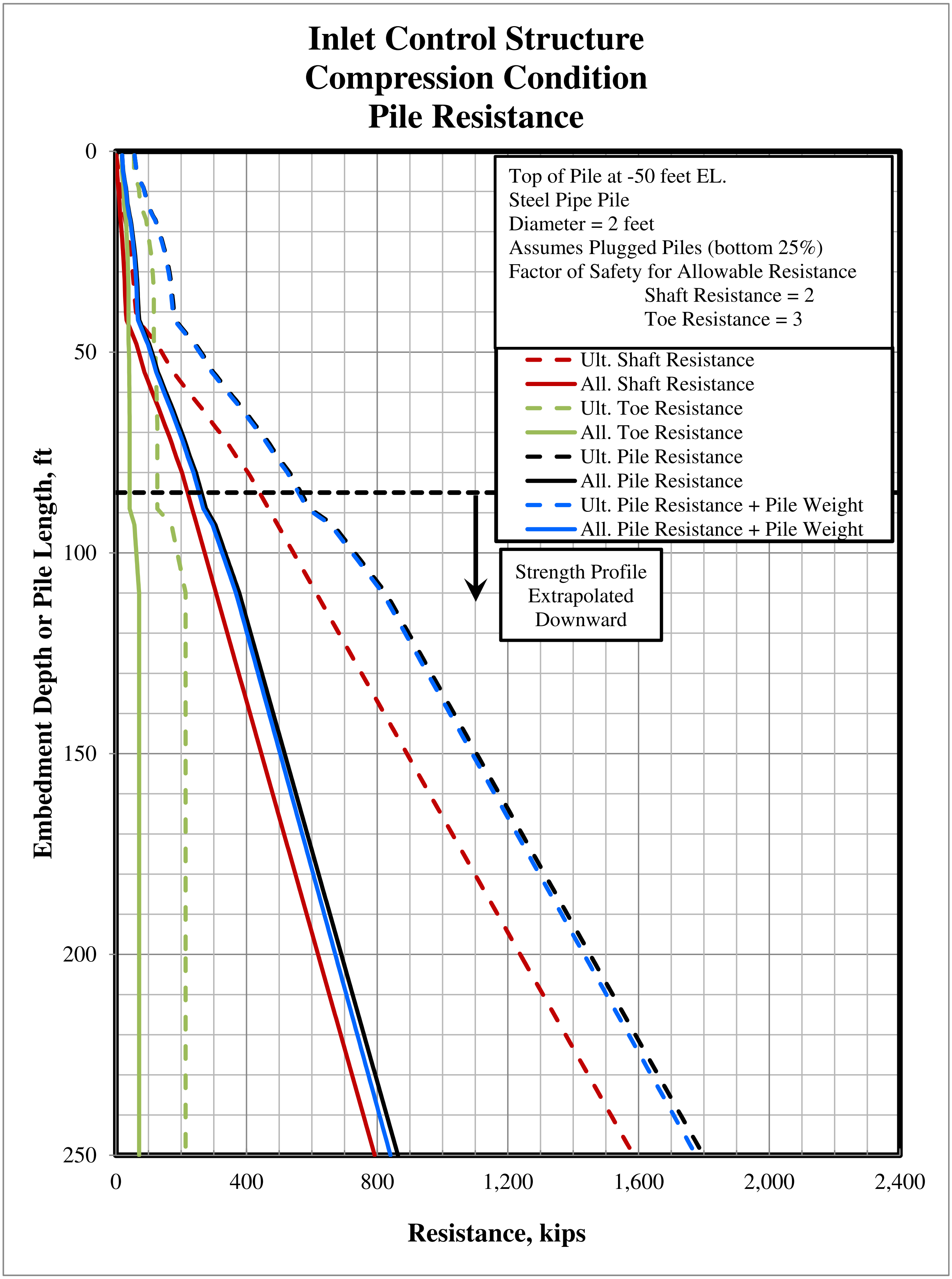

Highway Flood Wall at LA 23
Sleeve Friction Profile
fs, ksf
0
0.5
1
1.5
2
2.5
Elevation, ft
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-110
-120
-130
-140
Ground Surface Elevation
Bottom of Structure
NL-07C
NL-10C
fs for Pile Shaft Resistance Calculation

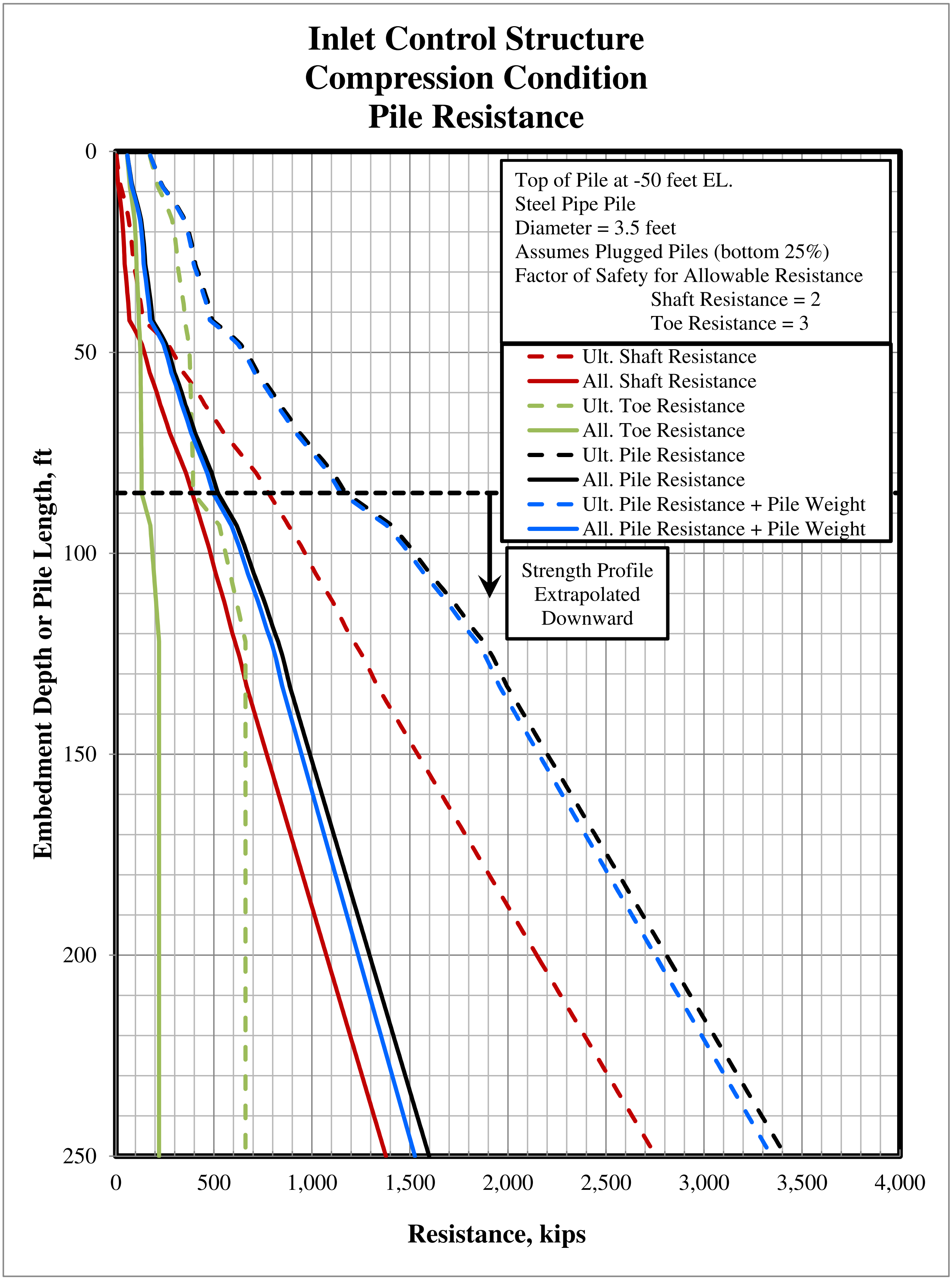


Back Structure
Tip Resistance Profile
qc, ksf
0
25
50
75
100
125
150
Elevation, ft
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-110
-120
-130
-140
Ground Surface Elevation
Bottom of Structure
NL-01C
NL-02C
SL-01C
qc for Pile Toe Resistance Calculation

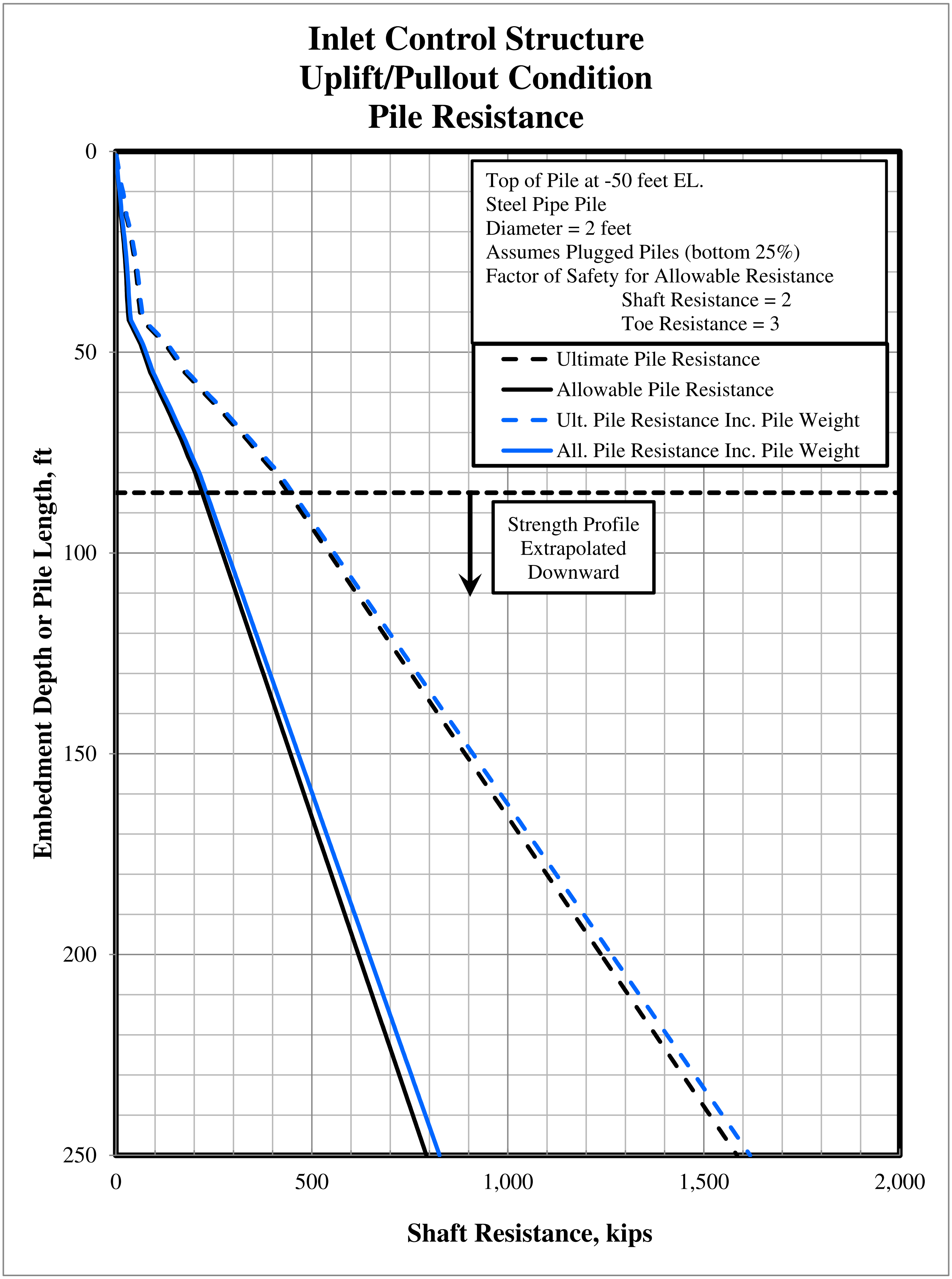

Back Structure
Sleeve Friction Profile
fs, ksf
0
0.5
1
1.5
2
2.5
Elevation, ft
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-110
-120
-130
-140
Ground Surface Elevation
Bottom of Structure
NL-01C
NL-02C
SL-01C
fs for Pile Shaft Resistance Calculation

Appendix D. Pile Capacities

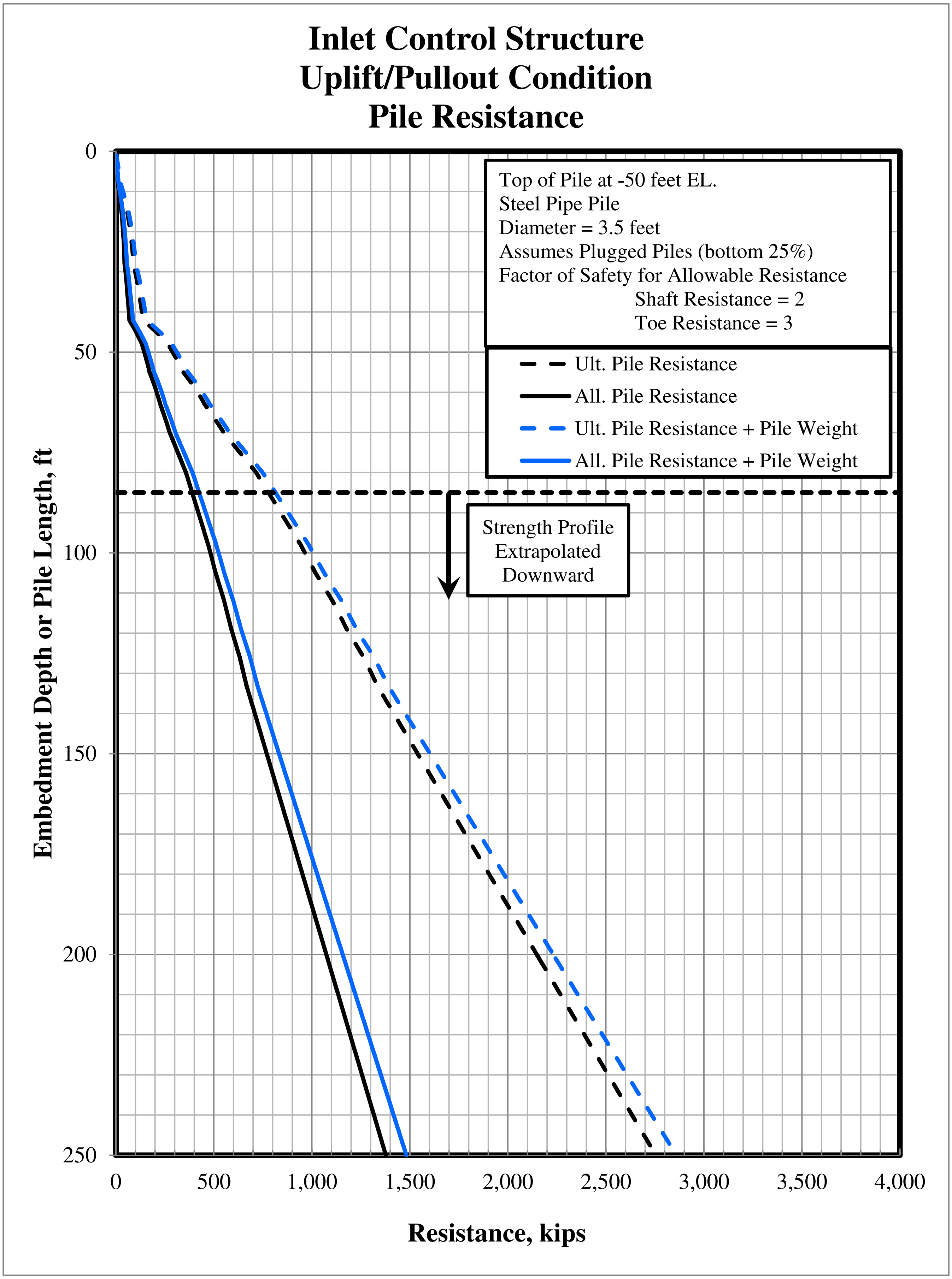

Inlet Control Structure
Compression Condition
Pile Resistance
Top of Pile at -50 feet EL.
Steel Pipe Pile
Diameter = 2 feet
Assumes Plugged Piles (bottom 25%)
Factor of Safety for Allowable Resistance
Shaft Resistance = 2
Toe Resistance = 3
Ult. Shaft Resistance
All. Shaft Resistance
Ult. Toe Resistance
All. Toe Resistance
Ult. Pile Resistance
All. Pile Resistance
Ult. Pile Resistance + Pile Weight
All. Pile Resistance + Pile Weight
Strength Profile
Extrapolated
Downward
Embedment Depth or Pile Length, ft
0
50
100
150
200
250
0
400
800
1,200
1,600
2,000
2,400
Resistance, kips

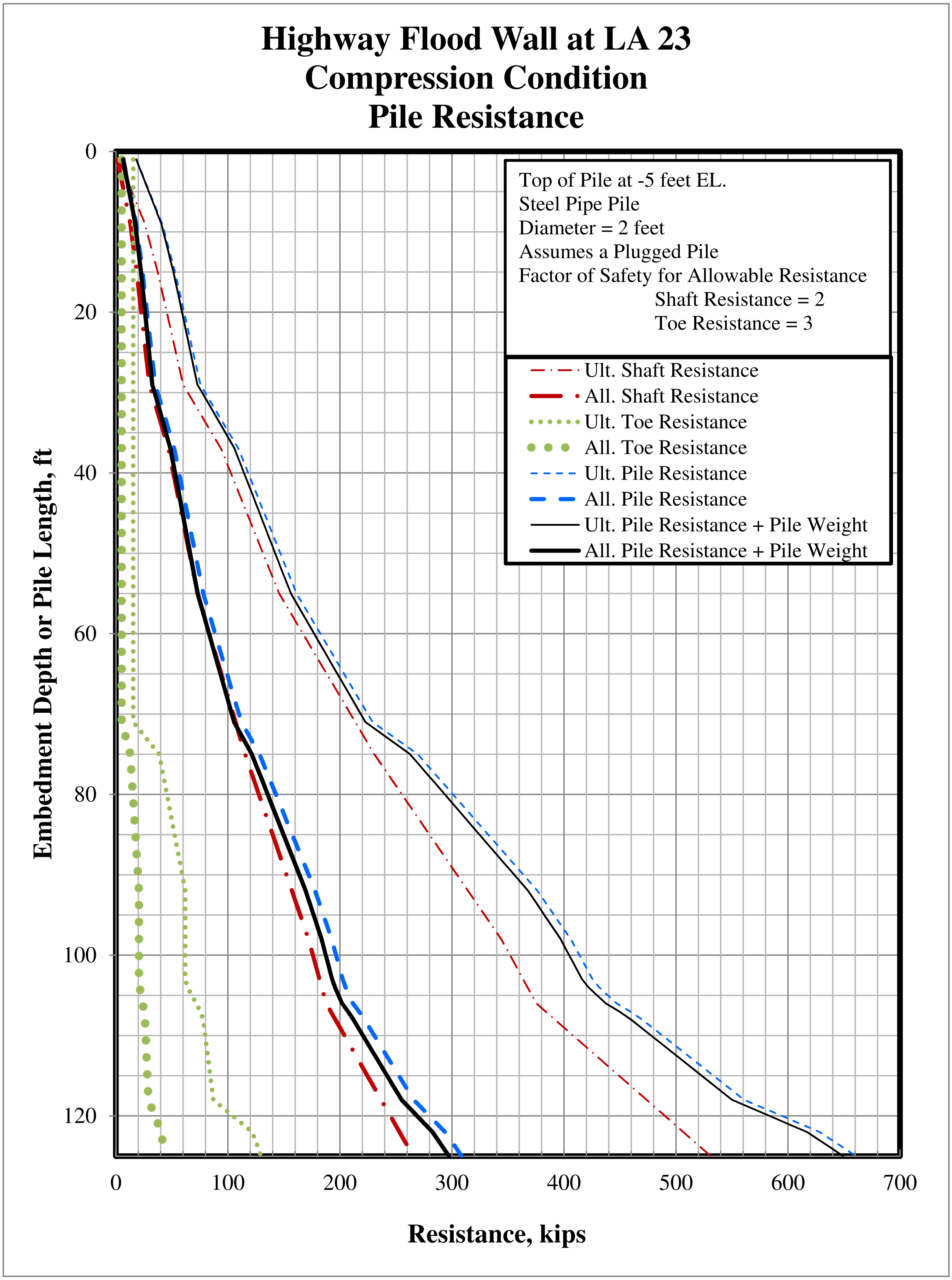


Inlet Control Structure
Compression Condition
Pile Resistance
Top of Pile at -50 feet EL.
Steel Pipe Pile
Diameter = 3.5 feet
Assumes Plugged Piles (bottom 25%)
Factor of Safety for Allowable Resistance
Shaft Resistance = 2
Toe Resistance = 3
Ult. Shaft Resistance
All. Shaft Resistance
Ult. Toe Resistance
All. Toe Resistance
Ult. Pile Resistance
All. Pile Resistance
Ult. Pile Resistance + Pile Weight
All. Pile Resistance + Pile Weight
Strength Profile
Extrapolated
Downward
Embedment Depth or Pile Length, ft
0
50
100
150
200
250
0
500
1,000
1,500
2,000
2,500
3,000
3,500
4,000
Resistance, kips

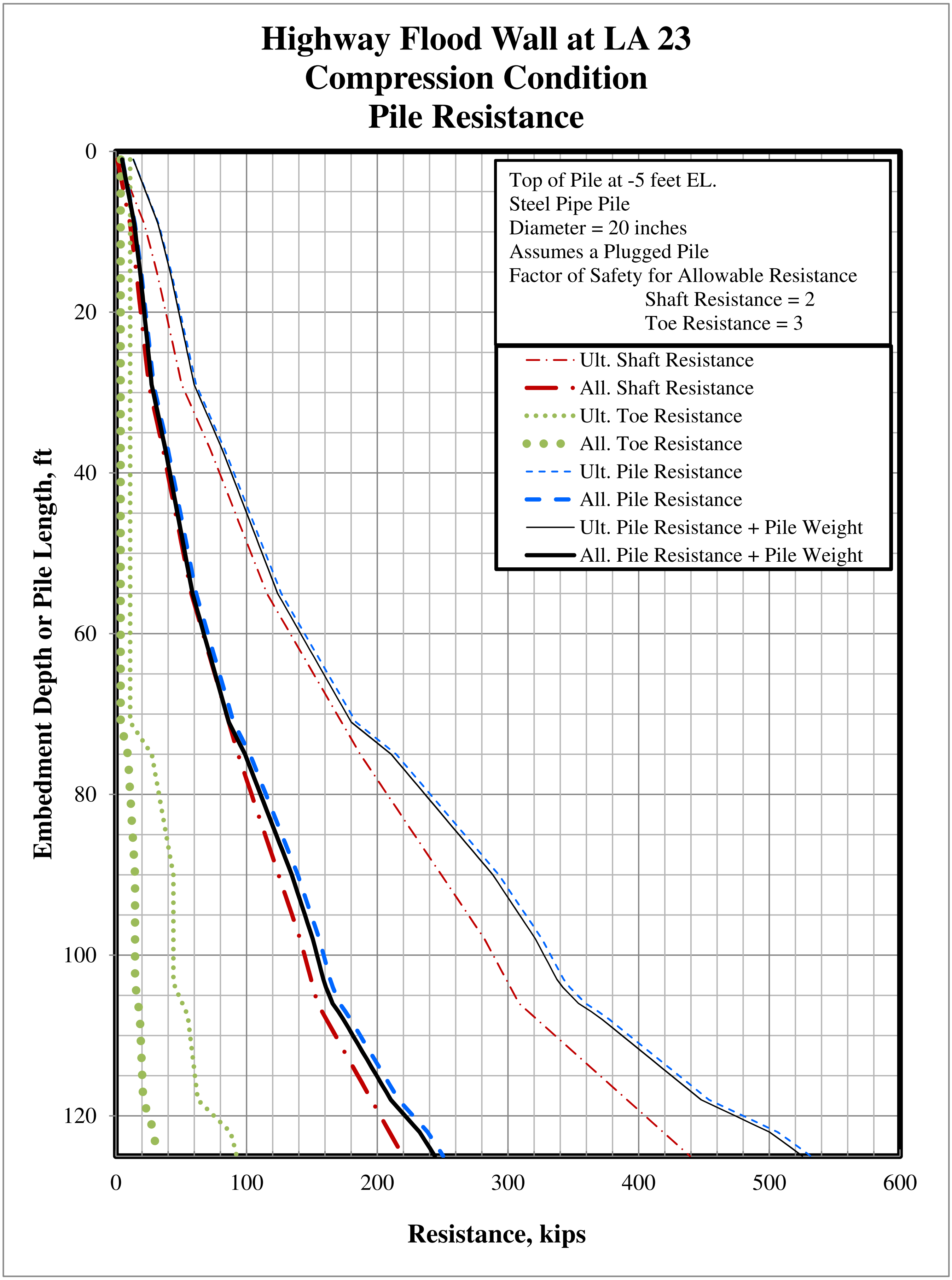


Inlet Control Structure
Uplift/Pullout Condition
Pile Resistance
Top of Pile at -50 feet EL.
Steel Pipe Pile
Diameter = 2 feet
Assumes Plugged Piles (bottom 25%)
Factor of Safety for Allowable Resistance
Shaft Resistance = 2
Toe Resistance = 3
Ultimate Pile Resistance
Allowable Pile Resistance
Ult. Pile Resistance Inc. Pile Weight
All. Pile Resistance Inc. Pile Weight
Strength Profile Extrapolated Downward
Embedment Depth or Pile Length, ft
0
50
100
150
200
250
0
500
1,000
1,500
2,000
Shaft Resistance, kips

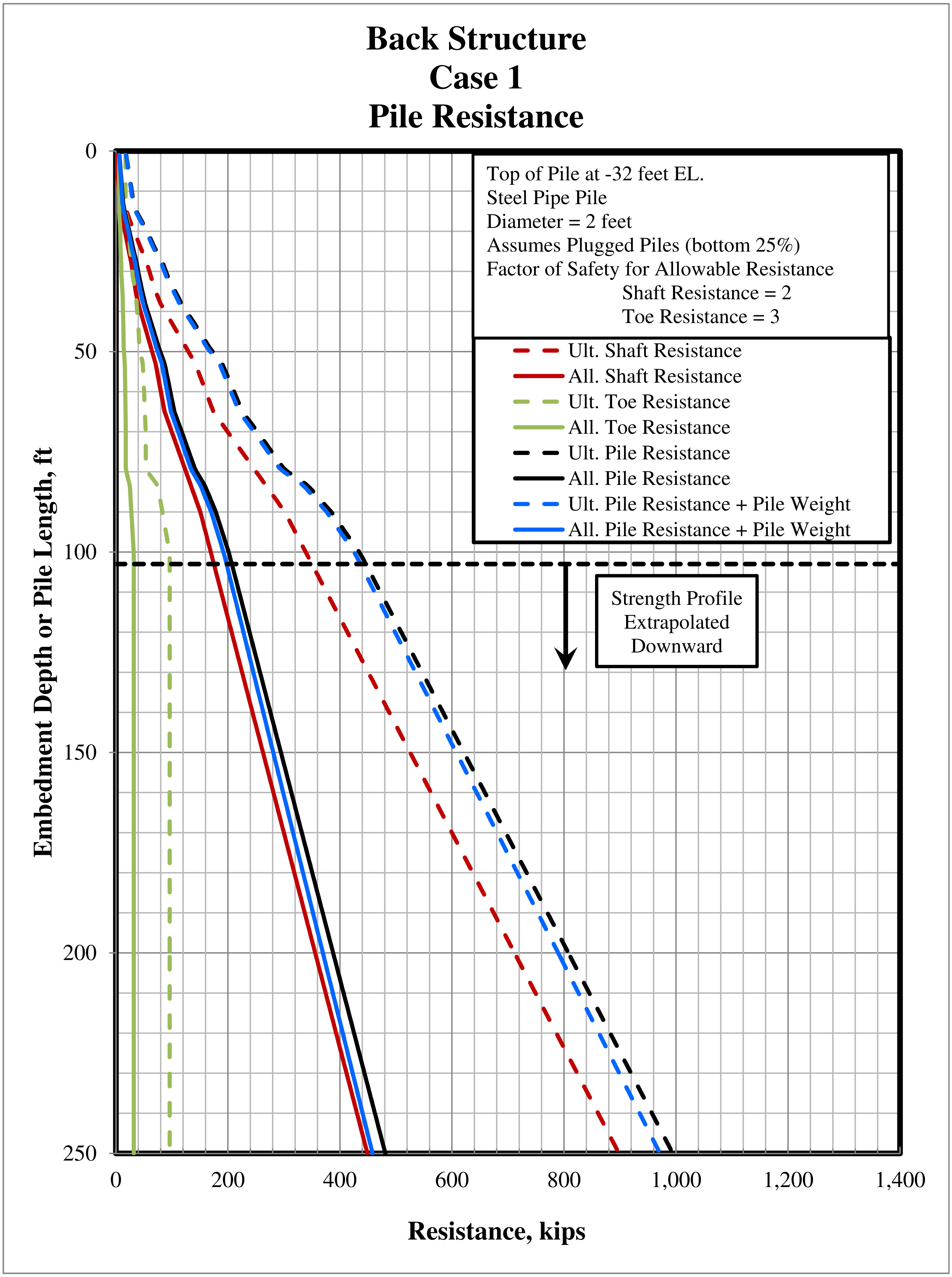

Inlet Control Structure
Uplift/Pullout Condition
Pile Resistance
Top of Pile at -50 feet EL.
Steel Pipe Pile
Diameter = 3.5 feet
Assumes Plugged Piles (bottom 25%)
Factor of Safety for Allowable Resistance
Shaft Resistance = 2
Toe Resistance = 3
Ult. Pile Resistance
All. Pile Resistance
Ult. Pile Resistance + Pile Weight
All. Pile Resistance + Pile Weight
Strength Profile Extrapolated Downward
Embedment Depth or Pile Length, ft
0
50
100
150
200
250
0
500
1,000
1,500
2,000
2,500
3,000
3,500
4,000
Resistance, kips

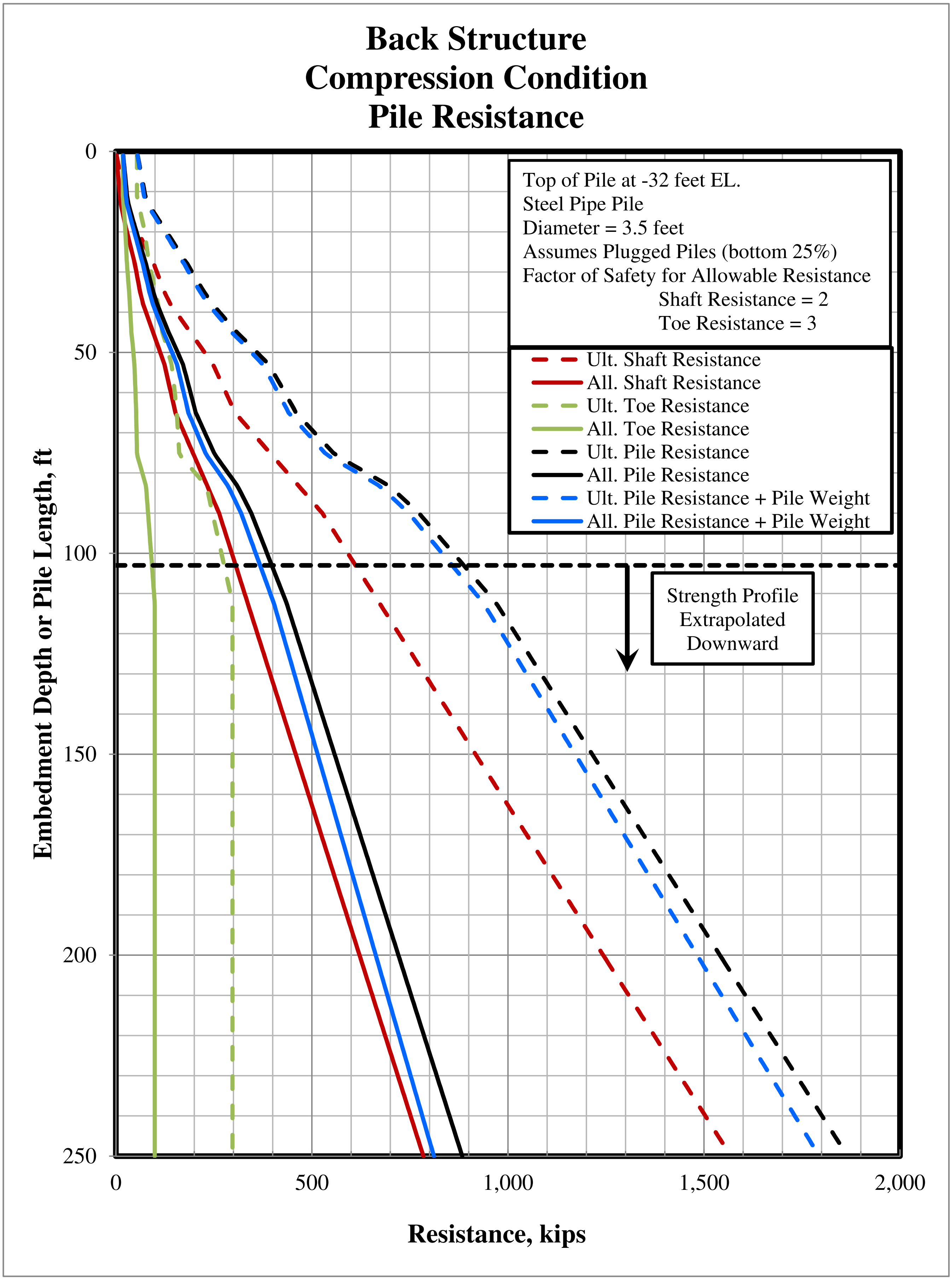


Highway Flood Wall at LA 23
Compression Condition
Pile Resistance
Top of Pile at -5 feet EL.
Steel Pipe Pile
Diameter = 2 feet
Assumes a Plugged Pile
Factor of Safety for Allowable Resistance
Shaft Resistance = 2
Toe Resistance = 3
Ult. Shaft Resistance
All. Shaft Resistance
Ult. Toe Resistance
All. Toe Resistance
Ult. Pile Resistance
All. Pile Resistance
Ult. Pile Resistance + Pile Weight
All. Pile Resistance + Pile Weight
Embedment Depth or Pile Length, ft
0
20
40
60
80
100
120
0
100
200
300
400
500
600
700
Resistance, kips



Highway Flood Wall at LA 23
Compression Condition
Pile Resistance
Top of Pile at -5 feet EL.
Steel Pipe Pile
Diameter = 20 inches
Assumes a Plugged Pile
Factor of Safety for Allowable Resistance
Shaft Resistance = 2
Toe Resistance = 3
Ult. Shaft Resistance
All. Shaft Resistance
Ult. Toe Resistance
All. Toe Resistance
Ult. Pile Resistance
All. Pile Resistance
Ult. Pile Resistance + Pile Weight
All. Pile Resistance + Pile Weight
Embedment Depth or Pile Length, ft
0
20
40
60
80
100
120
0
100
200
300
400
500
600
Resistance, kips


Back Structure
Case 1
Pile Resistance
Top of Pile at -32 feet EL.
Steel Pipe Pile
Diameter = 2 feet
Assumes Plugged Piles (bottom 25%)
Factor of Safety for Allowable Resistance
Shaft Resistance = 2
Toe Resistance = 3
Ult. Shaft Resistance
All. Shaft Resistance
Ult. Toe Resistance
All. Toe Resistance
Ult. Pile Resistance
All. Pile Resistance
Ult. Pile Resistance + Pile Weight
All. Pile Resistance + Pile Weight
Strength Profile
Extrapolated
Downward
Embedment Depth or Pile Length, ft
0
50
100
150
200
250
0
200
400
600
800
1,000
1,200
1,400
Resistance, kips


Back Structure
Compression Condition
Pile Resistance
Top of Pile at -32 feet EL.
Steel Pipe Pile
Diameter = 3.5 feet
Assumes Plugged Piles (bottom 25%)
Factor of Safety for Allowable Resistance
Shaft Resistance = 2
Toe Resistance = 3
Ult. Shaft Resistance
All. Shaft Resistance
Ult. Toe Resistance
All. Toe Resistance
Ult. Pile Resistance
All. Pile Resistance
Ult. Pile Resistance + Pile Weight
All. Pile Resistance + Pile Weight
Strength Profile
Extrapolated
Downward
Embedment Depth or Pile Length, ft
0
50
100
150
200
250
0
500
1,000
1,500
2,000
Resistance, kips