Mid-Barataria Sediment Diversion

Geotechnical Report
30% Basis of Design

Coastal Protection and Restoration Authority of Louisiana

July 2014

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Geotechnical Report
30% Basis of Design

Prepared for
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Preliminary Documents
Not to be used for construction, conveyance, sales or as the basis for permit issuance.
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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 Barataria Basin. The northern and southern boundary limits are offset approximately 500 feet from the project centerline.

The diversion would consist of a self-contained outfall channel roughly 1.5 miles long that would connect at the Mississippi River and Tributary (MR&T) Levee and flow through the North Forced Drainage Area. The channel would be designed to flow under the Belle Chase Highway (LA 23) bridge and New Orleans and Gulf Coast Railway (NOGC) bridge before discharging into the Barataria Basin.

The proposed MBSD is located in a region characterized by generally low-lying, flat agricultural land crisscrossed with drainage ditches, marsh areas, and woodlands. LA 23 bisects the project site at approximately MBSD project Station 65+00.[1] The Mississippi River is separated from the project site by the MR&T Levee at the eastern end of the site. The site is protected from Barataria Bay on the west by an earthen Non-Federal Levee (NFL). This levee is to be modified by the U.S. Army Corps of Engineers (USACE) as part of the New Orleans to Venice (NOV) Federal Levee project that crosses the conveyance alignment at Station 137+00.

The area between the MR&T Levee and the NFL is referred herein as the forced drainage area and as the polder. Both these terms are used interchangeably and refer to the low-lying areas between the levees that require pumping to control groundwater and interior drainage to prevent flooding.

As currently envisioned, the MBSD would consist of the following main components:

- An inlet system consisting of five continuous structures that would draw water and sediment from the Mississippi River: the approach channel, control structure, outlet channel, transition structure, and transition walls (all located between project Station 22+00 and Station 41+85) with the entrance into the Mississippi River at River Mile Marker 61.
- A conveyance complex extending from approximately Station 41+85 to Station 131+70 (8,985 lineal feet). The proposed conveyance channel would be bounded by north and south guide levees.
- A back structure is proposed to transition from the conveyance channel to the Barataria Basin. The back structure will be gated so it can perform as a surge-protection barrier. As currently envisioned, the back structure will be a seven-bay, pile-supported, reinforced-concrete structure that will tie into the NOV earthen levee to provide a continuous line of flood protection.

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[1] Station is a project-specific stationing along the conveyance channel baseline.

- A pump station located north of the conveyance adjacent to the Cheniere Traverse Bayou. The pump station will provide forced drainage of the interior areas that are cut off from by the conveyance complex and can no longer drain to the south and be removed by the Wilkinson Pump Station.
- The conveyance complex would cross LA 23 and the NOGC tracks. The project would include new bridges for LA 23 and the realigned railroad tracks to span the conveyance complex.

These structures are presented in the plans in Volume 1, General Civil Sitework, and are discussed in greater detail in subsequent sections of this report.

1.2 Purpose and Scope

The purpose of this geotechnical study was to obtain information on subsurface conditions in the project area and to perform engineering analyses. This effort was conducted to determine the project's feasibility and to develop preliminary geotechnical recommendations to support the project's 30% design. This report summarizes geotechnical field investigation and laboratory testing and presents the results of the preliminary analyses and geotechnical recommendations for use in design.

The HDR scope of services for this study included the following:

- compiling and reviewing available geotechnical and geologic data pertinent to the project
- conducting an initial phase of field exploration, including test borings, cone penetrometer tests (CPTs), and laboratory testing to obtain information on subsurface conditions
- performing a pump test to obtain information on the site's dewatering characteristics
- performing engineering analyses of the collected data and developing geotechnical feasibility recommendations and design criteria for the proposed MBSD
- preparing this geotechnical investigation report to present results

GeoEngineers, Inc. (GeoEngineers), a subconsultant to HDR, was tasked with performing the field investigation and laboratory testing for the project. Its geotechnical data report (GDR), entitled *Draft Geotechnical 30% Engineering Data Report, Mid Barataria Diversion (BA-153), Myrtle Grove, Louisiana*, dated November 27, 2013, presents the results of the field investigation findings. The GeoEngineers GDR provides details regarding the field investigation and laboratory testing, along with an interpretation of the geologic and geotechnical conditions encountered. Summaries and additional interpretation of the GeoEngineers findings are presented in this report.

Due to the limited scope of the current investigations, subsequent phases of additional geotechnical field investigation, laboratory testing, and geotechnical analyses will be required to support the design moving forward. Areas that will require additional geotechnical investigations are discussed within the body of this report.

1.3 Project Datum and Coordinate System

The project elevation and coordinates are describes in detail in the *Mid-Barataria Sediment Diversion Alternative 1, Base Design Report, 30% Basis of Design*.

2 Background and Existing Information

2.1 Previous Studies

General regional geologic studies encompassing the project site have been published by the U.S. Geological Survey (USGS). These studies provide the basis for conditions described in Section 4, Geology and Geomorphology.

More specific and recent studies have been performed by USACE for a flood control channel with a similar proposed alignment entitled 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 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

The regional land masses are low-lying depositional areas. These deposits resulted from alterations of the Mississippi River channel, overtopping of river banks during high river stages, and storms (specifically, hurricanes). Available existing geotechnical information for the site and adjacent areas was compiled and presentenced in a report prepared by GeoEngineers entitled *Report of Existing Geotechnical Data, Mid Barataria Diversion (BA-153) Plaquemines Parish, Louisiana*, dated May 2013.

2.3 Nearby Levees

New Orleans to Venice Non-Federal Levee

The NFL system alignment is located along the eastern land edge of the Barataria Basin, as shown on Figure 1 (all figures are located at the end of this report). USACE is currently designing improvements to the NFL system. The improvements include a portion crossing the MBSD's proposed alignment at Station 140+00. As part of the proposed improvements, the NFL will be reconfigured by building a new levee (core levee with stability berms on either side of the levee), as well as modifying a toe drainage ditch. The elevation increases have been proposed to be at least elevation +9 feet in the vicinity of the proposed 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.

Mississippi River and 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 levee crown elevations in the vicinity of the MBSD project site are approximately +15.5 feet. The MR&T Levees are designed to contain the MR&T system flows at 1.25 million cubic feet per second. Furthermore, an existing railroad spur operated by NOGC runs parallel to the levee along the landside toe. The spur terminates south of the proposed MBSD project limits.

The MR&T is separated (set back) from the river by a batture. USACE has placed erosion protection along the river bank, batture, and levee waterside slope consisting of articulated block mats (ABMs) placed on riverbank extending from the batture hinge point to down near the bottom of the river channel, rock rip rap along the top of the river bank slope and the batture surface, and slope pavement along the levee waterside slope. The landside slope is vegetated with grasses. The crown of the levee surface is shell fill to provide an all-weather surface. Due to subsidence, the crown of the levee has settled below the authorized minimum crown elevation of about +16.25 feet.

3 Field Exploration and Laboratory Testing

GeoEngineers performed a field exploration that consisted of drilling and sampling soil borings (both 5-inch-diameter and 3-inch-diameter borings) and conducting in situ field vane strength testing, CPTs, piezometer installation, and two pump tests.

3.1 Field Exploration

The field exploration program for this initial study consisted of a combined total of 80 borings, CPTs, field vanes, pump test wells, and piezometers. Table 3-1 presents exploration locations within the proposed project limits.

Table 3-1. Boring locations

Location of explorations (by Station)	Number of explorations
0+00 to 28+00	11
28+00 to 32+00	17
32+00 to 139+00	36
139+00 on	16
Total	**80**

Details of the field exploration program, including the logs of the test borings, CPTs, field vanes, pump test wells, and piezometers, are presented in the GeoEngineers GDR. Exploration relative locations are presented on Figures 2 and 3.

3.2 Laboratory Testing

Laboratory testing was performed on selected samples of soils retrieved during the test borings. Tests included: mini-vane, dry unit weight, unconfined compression, unconsolidated undrained compression, consolidated undrained compression, sieve and hydrometer analysis, consolidation, permeability, and Atterberg limits. Details and results of the testing are presented in the GeoEngineers GDR.

3.3 Field Pump Tests

GeoEngineers installed wells and piezometers and performed pump tests as part of the field investigation. GeoEngineers installed 14 wells at two locations—two were pumping wells and the remaining 12 were observation wells. Results of the pump tests were recorded by GeoEngineers and are presented in the referenced GDR. Locations of the test wells, designated as PT-1 & 2, and the associated observation wells (piezometers), designated as PZ- 1 through 12, are shown on Figures 2 and 4. The pump tests collected aquifer response data to serve as a basis for designing dewatering systems to assist in construction of the proposed project. The pump test results and related evaluations were provided in a memorandum included in the GeoEngineers GDR.

Pump test wells consisted of well casings and screens with total depths between 45 and 60 feet below the ground surface. The bore holes were observed and logged by GeoEngineers during drilling and well construction. Pumping was performed using a 3-inch-diameter submersible pump for the duration of the testing. Flows were measured with an inline flow meter and a 5-gallon bucket. Water levels were measured using a down-hole transducer and a manual depth sounding tape. Flows and depths were recorded throughout the process.

4 Geology and Geomorphology

4.1 Regional Geology

Louisiana's southern coast has been formed over many thousands of years and in discernible depositional deltaic lobes. The MBSD site is located in the Plaquemines complex lobe—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. On the 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.

As discussed by Gagliano et al. (2003), a series of growth faults have developed in the underlying Pleistocene and older basement soils as the Mississippi River Delta has progressed southward. Due to the young age of the sediments resulting in self weight settlement and movement on the growth faults in response to the growth of the delta, the area is experiencing regional subsidence. The rate of subsidence is reported as being a rate of 2 to 4 feet per century for the MBSD site vicinity.

Regional seismicity is controlled by the New Madrid Fault Zone in Missouri. USGS estimates that peak ground accelerations of less than 2 percent of gravity would occur in the vicinity of the MBSD site. Therefore, the risk of strong ground shaking at the site is judged to be very low.

4.2 Site Geology and Geomorphology

This discussion is based on a 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 locations and profile between the MR&T Levee and the NFL is shown in Figure 4. Explorations along the MR&T Levee are presented in Figures 5 and 6. Explorations in the Mississippi River are shown in plan view on Figure 2, with associated cross sections across the MR&T locations on Figure 4. MR&T cross section profiles are subsequently presented on Figures 7 through 12.

As discussed in the GeoEngineers GDR, eight major geologic deposits were identified at the site, either by geologic maps or through the field investigation. The eight identified deposits are:

1. Point Bar
2. Natural Levee
3. Nearshore Gulf
4. Abandoned Distributary
5. Undifferentiated Interdistributary/Intradelta
6. Prodelta
7. Pleistocene
8. Marsh

The locations of these deposits may vary across the site, and these deposits may overlie one another. A description of each deposit follows.

The MBSD site can be characterized/divided into four major geomorphologic areas/reaches progressing from east to west:

- Point Bar (Station 0+00 to Station 49+00)
- Abandoned Distributary Channel (Station 52+00 to Station 60+00, and Station 77+00 to Station 82+00)
- Interdistributary/Intradelta (Station 49+00 to Station 52+00, Station 60+00 to Station 77+00, and Station 82+00 to Station 93+00)
- Marsh/backland area (Station 93+00 and westward)

The above limits are referenced to the conveyance channel centerline stationing and are approximate. Additional explorations are proposed for the 60% design phase to better define the limits of the geomorphologic areas. The following paragraphs briefly discuss the site geology and geomorphology, based on the 30% design phase explorations and published literature (references). More detailed discussion is presented in the GeoEngineers GDR.

Geomorphology Models

For analysis purposes, HDR used the data presented in GeoEngineers reports and developed geologic models of various areas along the project alignment. Six geotechnical "models" have been developed to represent current subsurface conditions at the site and do not account for the effects of proposed fill placement or phased construction. Engineering judgment must be used in evaluating the effects of surcharge and associated induced settlements at the site. The effects of construction activities such as phase construction and dewatering activities also need to be carefully considered. The following local naming conventions have been developed for the different soil units that underlie the site and, in general, can be described as follows (after Heinrich 2005).

Pleistocene Soils

Older sediments that once were subaerially exposed as the Louisiana continental shelf.

Gulf Near Shore

Relatively thin near-shore sediments composed of shelly marine sands and clays placed along the former gulf shoreline and overlaying the older Pleistocene soils.

Pro Delta

The lowermost layer of deltaic sediments that consists of a gulfward thickening blanket of high plasticity clay. This layer accumulated within the Gulf of Mexico as clay carried by currents out of the delta mouth and into the Gulf of Mexico and settled from suspension on its bottom.

Intra Delta and Interdistributary

The sediments underlying and composing the bulk of the delta plain consist of a mixture of interfingering and interlayered intradelta and interdistributary sediments overlying the prodelta sediments. Interdistributary deposits consist largely of under consolidated fat clay. Intradelta deposits typically consist of a heterogeneous mixture of normally consolidated silt and clayey silt, silty clay, fat clay, and about one-fourth fine-grained well-sorted sand.

Natural Levee

Natural levees are asymmetric ridges, which are highest adjacent to their associated channel and slope gently away and downward in elevation from it until they merge with marshes and swamps of lower elevation.

Abandoned Distributary Channels

Abandoned distributaries are channels that once branched off of the modern and relict courses of the Mississippi River. They are called "distributaries" because when active, they distributed floodwaters away from the Mississippi River into the surrounding deltaic plain. The channels of these distributaries and their natural levees radiate outward in a fan-like network from either the modern Mississippi River or its former delta lobe trunk channel.

Marsh/Swamp Deposits

A layer composed of swamp and marsh deposits overlies the intradelta and interdistributary deposits and forms the surface of the delta plain. Swamp sediments occupy a narrow strip of delta plain adjacent to the natural levees of major channels. Marsh sediments underlie the remaining majority of the delta plain. These sediments typically range in thickness from 5 to 10 feet.

Point Bar

The eastern portion of the site comprises point bar deposits that have accreted to the west Mississippi River bank. The point bar material at the site varies considerably, both laterally and vertically, and extends to the Pleistocene-aged soils. The soils vary in age and consistency but generally consist of silts, clays sands, silty sands, and sands at depths below 45 feet and become increasingly fine grained (more clay and silt) above a depth of 45 feet. The point bar deposits are overlain by natural levee deposits. The MR&T Levee is founded on natural levee deposits overlying point bar deposits. The point bar deposits significantly increase in stiffness below elevation –65 feet.

5 Site Conditions

5.1 Surface Conditions

Recent surveying indicates that the ground surface is relatively flat between the Mississippi River and Barataria Basin with the major topographic changes being at the NFL, MR&T Levee, and LA 23. The ground surface between the major topographic features has general elevations between +2.5 and –4 feet (North American Vertical Datum [NAVD] 88), except for interior channel flow lines that are near elevation 0 feet. The NFL has a current crown elevation and width of approximately 5.2 feet and 14 feet, respectively. The existing side slopes consist of 3.5:1 (horizontal to vertical) on the waterside and 3:1 on the polder side. The MR&T Levee has a crown elevation and width of approximately +15.5 feet and 10 feet, respectively. The existing side slopes consist of 3:1 (horizontal to vertical) on the waterside and 4:1 on the polder side. LA 23 is built upon fill materials on a raised elevation of approximately +4 feet. The site is drained by a series of interconnected drainage ditches and pumps, and these serve the low-lying areas.

5.2 Subsurface Conditions

The following paragraphs describe the subsurface soil conditions, focusing on the geological and geomorphological deposits described previously in Section 4.2, Site Geology and Geomorphology.

Beginning at the eastern end of the site at the Mississippi River, the near-surface soil consists of natural levee deposits. These deposits extend westward to about Station 92+00. The natural levee deposits generally consist of lean to fat silty clay, interbedded with sand and silt, and extend to depths of about 10 to 15 feet. Approximately 2 to 5 feet of the upper stratum consists of a relatively thin, desiccated, overconsolidated crust that is generally medium stiff to stiff. Below this crust, the stratum is normally consolidated to slightly overconsolidated, and is generally very soft to soft.

From about Station 92+00 west to about Station 140+00, the near-surface soil consists of marsh deposits extending to depths of about 20 to 25 feet. They generally consist of very soft to soft fat clay with some organics. The stratum is normally consolidated to slightly overconsolidated. Approximately 2 to 5 feet of this upper stratum consists of a relatively thin, desiccated, overconsolidated crust that is generally medium stiff to stiff.

From the Mississippi River to about Station 49+00, the natural levee deposits are underlain by point bar deposits, which generally consist of loose to medium dense silty sand, sandy silt, and clayey sand, extending to the maximum depth explored of about 135 feet.

From about Station 49+00 west to about Station 140+00, the natural levee deposits and marsh deposits are generally underlain by interdistributary/intradelta deposits extending to depths of about 40 to 50 feet. These deposits generally consist of very soft to soft lean to fat clay, with occasional silt and sand lenses. The stratum is normally consolidated to slightly overconsolidated.

Between approximately Station 52+00 and Station 60+00 and between Station 77+00 and Station 82+00, two abandoned distributary channels cut through—and extend to approximately the same depths as—the interdistributary/intradelta deposits. The abandoned distributary channels generally consist of interbedded loose to medium dense silt and silty sand, and soft to medium stiff lean clay with sand and silt lenses.

The western portion of the point bar deposits and the interdistributary/intradelta deposits are underlain by prodelta deposits, which extend to depths of about 110 feet. These deposits generally consist of soft to stiff lean to fat clay, and are normally consolidated to slightly overconsolidated.

The prodelta deposits are underlain by Pleistocene deposits extending to the maximum depth explored of about 135 feet. These deposits generally consist of stiff to very stiff fat clay.

5.3 Groundwater

The site is located between the Mississippi River and the Barataria Basin. The water surface in the Mississippi River varies and is typically highest from March to June. The water surface in the Barataria Basin is relatively uniform, with some minor tidal influence. Both water surfaces are typically higher in elevation than 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 water from the polder drainage ditches to 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 indicated 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 would be excavated in the wet by dredging methods.

6 Discussions and Design Considerations

6.1 Geotechnical Feasibility

From a geotechnical engineering basis the MBSD project is judged to be feasible. However, there are significant short-term and long-term constraints associated with constructing large civil works projects within the relative young lower Mississippi River delta. The geotechnical site conditions and associated constraints must be incorporated into the civil, structural, hydraulic design, and construction phasing. The primary geotechnical site conditions include:

- presence of the very soft to soft, compressible and weak foundation conditions across the entire conveyance complex alignment
- occurrence of high groundwater, and engineering challenges that this would pose on constructibility and long-term performance of the project
- occurrence of long-term regional subsidence that must be factored into setting the conveyance complex levee and structure crown/top elevations
- potential geologic fault activity

Because the MBSD project required the input from a highly diversified multidisciplinary team, the different design disciplines will need to be briefed to thoroughly understand the conditions that constrain the design of each project feature. Due to the complex geomorphologic processes that have formed the polder which the MBSD complex will cross, each project feature will require individual analysis and specific geotechnical recommendations to take the project into final design, including the conveyance channel excavation and transition structures, guide levee, Mississippi River inlet and control structures, Barataria Basin outlet structures, highway and railroad crossing, pump stations, and structure-to-levee tie-ins at both the MR&T Levee and the soon-to-be constructed NOV back levee.

Portions of the project will also penetrate federal flood control levees and will require additional geotechnical studies to demonstrate that the MBSD will meet or exceed the currently authorized level of flood protection during the construction phase, on completion, and during operation. USACE has determined that the MBSD project meets the requirements to be classified as a major revision to the federally authorized project. Additional geotechnical investigations and analysis will be required to take the project into final design and provide the level of detail required to support a Section 408 permit for the project. The final project configuration has not been determined to a level of detail sufficient to develop the full scope of the additional studies.

An example of the project's geotechnical complexity is the design of the conveyance inlet and outlet structures. These structures will most likely require:

1. Deep foundations to provide both compression and tension (uplift) support,
2. Dewatering to facility construction within the deep excavations,
3. Temporary support structures to support excavations in the underlying weak soils,
4. Ground improvement or modification to support new fills and reduce anticipated large differential settlement between the structures and new adjacent fills,

5. Underseepage cutoffs to prevent the development of high seepage gradients that may lead to piping of surrounding soils, and
6. Specific structural elements that tie the structures to existing flood control levees that maintain the continuous line of flood protection provided by the MR&T and the future NFL levees.

Other key geotechnical considerations that must be carried through to final design include but are not limited to:

1. Site preparation including the development and sequencing of working areas with poor bearing/support.
2. Internal drainage and temporary flood control protection around construction areas such as the inclusions of setback levees and in situ cutoff walls. Such measures must meet USACE levee design criteria.
3. Design of guide levees for conveyance of diverted river flows as well as for conditions where flooding within the polder could affect guide levees.
4. Design of floodwalls that may be required at conveyance crossings (highway and rail) or as part of pump station discharge facilities including deep foundation support, underseepage cutoff systems, and tie-ins to adjacent structures or earthen levees.
5. Channel design including channel side slope inclination, foundation soil erodibility, and armoring requirements.
6. Design and construction of river bank penetrations including articulated concrete block (ACB) revetment removal, temporary construction structures, and permanent inlet structures.
7. Channel guide levee, bridge approach fills, structure building pads, new fills placed around structures, and roads/site access fills all require special analysis and design due to soft and compressible foundation soils along the entire length of the conveyance complex. These conditions require specific geotechnical recommendations for design and construction including requirements for improvement (strengthening) of weak foundation soils and/or the high-strength geotextiles to support new fills, sequencing of fills to accommodate settlement-induced construction schedules impacts, and, if consolidation settlements cannot be accommodated, the need for staged construction and surcharging to strengthen soft foundation soils and reduce differential settlements.
8. Temporary excavation support for both shallow and deep excavations below river and groundwater levels and associated approaches to dewatering excavations.
9. Groundwater control during construction and during extreme events (including hurricane storm surge).
10. Underseepage and end-around seepage and control of excessive seepage, and backward erosion (piping).
11. Deep foundation design (inlet structures, outlet structures, bridge support, and pump stations).
12. Settlement of marsh sediments within the Barataria Basin as part of land building activities.

13. Allowances for regional subsidence effects over the life of the structures.

This geotechnical study has been conducted to characterize geotechnical conditions across the site and to provide initial engineering analysis for setting project geometry and constraints as well as to support preliminary estimated project probable costs. The following sections discuss the current understanding of the MBSD project. The results of preliminary geotechnical analysis based on this understanding and resulting design recommendations are provided in Sections 7 and 8.

6.2 Design Considerations

Weak and Compressible Soil

The majority of the project site is underlain by relatively soft, weak, and compressible soil deposits. This is particularly the case in the western portion of the site, which is underlain by soft marsh deposits. Both in situ field vane strength testing and laboratory unconsolidated undrained strength testing was conducted as part of the 30% design studies. Undrained strength versus depth profiles have been developed for each of the cross sections (locally referred to as strength line plots). The soils at this site are typical of recent delta/embayment deposits in that they have been placed in a relatively rapid manner and have consolidated under their own weight; therefore, they are normally consolidated to slightly overconsolidated. The developed strength profiles have the same general shape where the upper 10 to 20 feet of the profile has nearly uniform shear strength and then increases at a fairly constant rate with depth. Undrained shear strength in the upper zones of the foundation was found to range from a low of 150 pounds per square foot (psf) (very soft) to 300 psf (soft). In some areas, an overconsolidated higher-strength crust of desiccated soil has formed over softer soils.

The low shear strength of the weak and compressible foundations soils will control the depth and inclination of the channel excavation and the height of fill that can initially be placed during levee construction. Placement of new loads, in the form of fills or new structures, will induce consolidation of the compressible foundation soils resulting in site settlement. Due to the thickness of these soils the amount of consolidation can be very large. However, as the soils consolidate there is an associated increase in soil strength, allowing the settling soils to gain strength following the placement of additional fill. The sequence of fill placement with associated settlement periods and strength gain is referred to as staged construction.

Due the large thickness of soft and compressible soils beneath the site, the rate of settlement and associated strength gain may take years to decades. Reducing the effective thickness of the soft soils can be achieved by installing wick drains to selected depths. Current analysis used wick drains 60 feet below the existing ground surface.

Dispersive soils were identified during this investigation program using double-hydrometer laboratory tests. In general, dispersive soils are clays that are susceptible to erosion and piping. When dispersive clays are immersed in water, the clay fraction behaves like a single-grained particle and erodes, creating slope stability issues and flow/piping failures.

Because the site is primarily underlain by soft and compressible soils, the undrained soil strength and soil unit weights are two geotechnical parameters critical in assessing channel slope and guide levee stability. The relatively low undrained shear strengths and correspondingly low bearing capacities of the underlying soil would limit the raises in grade that could be achieved when constructing the guide levees. These limiting characteristics would also affect the slope inclinations at which the levee and channel side slopes could be established, with steeper slopes resulting in lower factors of safety (FOS) against stability.

The foundation soils have very low strengths and are highly compressible. Loads imposed by constructing guide levees and other appurtenant structures would cause the foundation soils to consolidate and induce large settlements.

Long-term performance of the channel is a concern given the natural variability of the near-surface soils and the relatively low shear strengths of the in situ soils within and below the depth of the proposed channel. Because staged construction methods would likely be required for this project, the occurrence of strength gain associated with soil consolidation could be beneficial. However, because of the thick deposits of compressible soils and their relatively slow rates of consolidation, a considerable amount of time would be needed to achieve the strength gain needed (many decades). Installing wick drains to a selected depth within the compressible layer in a predetermined pattern could accelerate consolidation and strength gain in compressible soils. Wick drains are prefabricated strips of corrugated plastic surrounded by a geotextile filter that provide a shortened drainage path for water exiting the soil as the soil consolidates and, when properly designed, can accelerate consolidation.

Alternatively, the foundation soils could be modified through the use of aggregate columns or soil cement columns to reinforce and strengthen the soils. In situ modification methods require importing aggregate and/or cement. Such methods are relatively expensive, and it is anticipated that such an approach would be cost-prohibitive if applied along the length of the conveyance project.

The rate of consolidation could significantly affect project construction schedules and require a phased approach over consecutive years to allow for settlements and strength gain. As for the guide levees, the levees could be raised periodically to maintain the specified crown elevation. Alternatively, for areas that cannot be phased or cannot be delayed, alternative containment designs may be necessary, such as replacing a levee with flood walls supported on improved ground or supported by deep foundations. Seepage mitigation measured, such as drained or undrained seepage berms, may also be required for floodwalls.

For this assessment, HDR assumed that a significant portion of the channel alignment could be treated with wick drains and surcharged to strengthen the channel slope soils to allow conventional channel slope inclinations and to allow placement of the guide levees within a reasonable setback (offset) from the top of slope. Setting the guide levees away from the channel would be required to maintain channel stability where surcharge requirements would entail significant costs or would adversely affect the schedule.

The existing soils have very low strengths and would not allow significant fill thickness to be placed in a relatively short period. Therefore, construction of surcharges and structural fill would need to be placed in maximum stages of 4 to 6 feet. Strength gain between stages would be required and would be a function of the consolidation properties and thickness of the underlying compressible layers. As stated previously, staging periods could be significantly accelerated with the incorporation of wicks.

The design considerations outlined in this section are suitable for the 30% design level. As stated previously, additional field information is needed to fully address the above design considerations.

Water Surface Elevations

HDR used varying water surface elevation (WSE) scenarios (CPRA 2013) to review potential worst-case scenarios with regard to seepage, slope stability, and settlements. These scenarios included the following:

- Overtopping of either the MR&T Levee or NFL and flooding of the north and south polders to elevation +10 feet. WSEs within the channel were assumed to be at elevation 0 feet in this scenario.
- High water at elevation +10 feet within the conveyance channel. WSEs within the polders were assumed to be at elevations consistent with groundwater elevations observed in the piezometers installed during this field investigation stage.
- WSEs associated with an MR&T Levee breach.

Foundation Support

The project site can be grouped into two areas with similar generalized conditions at the anticipated foundation levels: (1) primarily cohesionless, relatively high-permeability point bar deposits extending from the eastern boundary of the project site adjacent to the Mississippi River to approximately Station 45+00, and (2) a relatively low-strength cohesive, low-permeability material that extends from Station 45+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 5, Site Conditions.

HDR assumed 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.

As stated above, additional information is required to fully address design considerations presented herein. Specifically, pile recommendations should be supported with full pile tests at the back structure.

Subterranean Walls

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

6.3 Design Criteria

Hydraulically related design criteria are key to meeting the project goal of transporting sediment to Barataria Basin from the Mississippi River. These hydraulics-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 would 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 project have been developed as follows:

- low operating water level in the channel: elevation –5 feet
- high operating water level in the channel: elevation +10 feet

The project must also meet criteria at the junctions with the NFL and the MR&T Levee, both of which are subject to federal review (including Section 408 requirements). These include, at a minimum:

- Maintain flood protection redundancy during construction.
- 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 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. 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 hurricane surge protection at the juncture with the NOV project.

Water surfaces are influenced by the 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 different 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 USACE's 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 System 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.

6.4 Project Features

The proposed MBSD would have three main features and several associated features, from a geotechnical standpoint, as listed below. The *Mid-Barataria Sediment Diversion Alternative 1, Base Design Report, 30% Basis of Design*, provides additional details on project features.

- channel (inlet structure, guide levees, outlet structure)
- pump station
- basin depositional area
- associated structures (bridges, ditches, etc.)

With the exception of the pump station, these features/structures are interlinked in crossings and loading conditions. This report presents the design and feasibility specifically for the channel and associated features and presents the feasibility of the pump station and bridge concepts.

Channel

The channel, as currently proposed, would to consist of an inlet structure, guide levees, and an outlet structure. As of the date of this report, conceptual drawings and loading conditions have been reviewed for the inlet structure and the guide levees. The outlet structure system was not included in the original scope of this phase because the need had not been fully determined by hydraulic modeling until recently. Therefore, the outlet system will be discussed from a general geotechnical feasibility standpoint here, with a more developed analysis proposed for the 60% design phase of the project.

Inlet System

The 1,100-foot channel would extend from the Mississippi River to a series of structures to form an inlet to the MBSD channel. The inlet system is currently proposed to consist 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 the Volume 1 civil plans. The 130-foot-wide inlet system would have an invert of elevation –40 feet through the control structure and then transition to elevation –25 feet in the transition structure. Loadings for these structures are discussed separately in Section 6.5.

The inlet system would be required to maintain the same level of flood protection as the current MR&T Levee system where these two would tie in to each other. The control structure, when tied to the MR&T Levee, would become the primary flood control feature and must provide the same level of protection as the current levee. The MR&T Levee design flood flow under its current federal authorization is 1,250,000 cubic feet per second (cfs). This is not the 100-year event. The proposed MBSD would 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 would not be addressed by this project. The structure heights would 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.

Guide Levees

Sediment-laden river water collected from the Mississippi River and channelized from the inlet system would be conveyed in large part by the conveyance channel and the guide levees. The guide levees would allow for variations of water levels within the channels. Levee toes would be founded near elevation 0 feet and would have long-term levee elevations of approximately 13.5 feet (CPRA 2013). Currently, guide levees are proposed to be non-federal levees with a primary objective of guidance (with flood protection as a secondary objective). The crest of the levee will be designed to have a service road with a minimum 15-foot width. Given the project site’s relatively soft subsurface conditions, the guide levees are currently proposed to use stability berms.

Discussions regarding the analysis of guide levees are detailed in Section 7.1 of this report.

Pump Station

A pump station is required as part of the construction of the proposed MBSD conveyance channel to maintain an unwatered condition within the polder to the north. The current pump station configuration is modeled after existing USACE pump stations in the region. These configurations generally consist of pile-supported pipe lines and pump equipment that span the NOV Levee and the associated drainage ditch. The exact location and structural details have not been determined as of the date of this report. However, preliminary general vertical pile capacities have been developed as part of this report.

6.5 Loading Considerations for Proposed Improvements

Loading conditions considered for the conveyance channel features were developed through both geotechnical and structural engineering means. The loading conditions were applied with the hydraulic modeling completed at the time of this report.

Guide Levees

Guide levees are anticipated to consist of locally generated, compacted soil fill. The loads on the foundation associated with these fills in conjunction with hydraulic loading conditions were used to develop bearing capacities, total settlements, seepage, and slope stability. Fill loads are associated with the placement of compacted soils generated from local borrow sites and channel excavation efforts. Currently, up to 21 feet of fill is anticipated in the western 4,700 feet to maintain a long-term guide levee crown elevation of +13 feet.

Control Structure

The control structure is currently proposed to be located between Station 33+45 and Station 35+37. The location is shown on Figure 2, and the general plan view of the system is presented on Figure 13. The proposed preliminary footprint would be approximately 192 by 164 feet. The top of the slab is proposed to be the same as the channel invert, at elevation –40 feet. The top of the walls vary in elevation between +18.5 and +23 feet. According to the plans in Volume 2, Diversion Structure, the structural concept consists of three bays with lift gates. Each bay would be separate and would be periodically dewatered for gate maintenance. The preliminary dead loads have been determined to be on the order of 3,900 psf in compression at the foundation elevation of –49 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 would be the first stage in the transition from the control structure to the conveyance channel. The outlet channel would be 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 would be located downstream and adjacent to the control structure between Station 35+37 and Station 36+12. The footprint has been proposed to be approximately 75 by 166 feet. The proposed top of slab would be at elevation –40 feet, with top of walls at elevation +13.5 feet. This structure would be permanently submerged without the capability of dewatering and, therefore, would not be subject to buoyancy pressures. The preliminary dead loads are presented in Table 6-1.

Table 6-1. 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 would be a series of cast-in-place concrete rectangular-shaped channels that consists of three step-up sections in channel bottom elevations from –40 to –25 feet between Station 36+12 and Station 37+62. The wall heights are proposed to be at elevation +13.5 feet for all steps. The bottom inside width of the channel varies from 164 feet wide at Station 36+12 to 200 feet at Station 37+62. This structure is contiguous to the outlet channel with similar civil design features to those described for the control structure. This structure would be the final structural concrete component to be constructed behind the control structure's temporary construction cofferdam. Based on the structural plans, the dead load would not exceed 3,000 psf for each step. Since this portion of the channel would 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 consist of reverse curvilinear triple sheet pile walls designed to transition the flow from the transition structure to the linear conveyance channel design section between Station 37+62 and Station 42+00. These walls would facilitate the channel section transition from a rectangular section to a trapezoidal channel with overbank stability berms and guide levees. The bottom width would further transition from 200 feet to the full conveyance channel width of 300 feet. The sheet pile wall sections would be designed with a pile-supported concrete relieving platform anchoring system. The wall sections would transition from exposed stepped vertical walls connecting to the transition structure downstream tieback abutment wall at elevation +13.5 feet, to full burial in the downstream trapezoidal channel section. Additional hydraulic, geotechnical, and structural analyses are needed to confirm the final alignment and design of these structures or viable alternatives.

ACB revetment would be installed over a sand and granular filter base encapsulated in geotextile fabric within the limits of the transition walls beyond the transition structure's concrete bottom to have a channel bottom elevation of –25 feet. Channel velocities necessitate that an anchored revetment system be installed within the channel transition section. The installation of ACB revetment would occur after the flood risk reduction system is fully reestablished at the MR&T Levee and control structure.

This would allow a partial degrading of the temporary setback levee across the conveyance channel to facilitate placement of the graded filter and revetment while the dewatering system is operable. ACB would provide scour protection for the full channel section with minimal friction loss. ACB would also allow potential upflow from the point bar deposits through the graded filter media without loss of underlying soils. Hydrostatic pressure differentials between surface and groundwater would be equalized through the filtered ACB.

The limits of the full-width ACB revetment system would extend to the end of the transition walls where the conveyance channel becomes linear. ACB revetment would then be placed for the entire channel length along the stability berms and levee slopes up to the top of levee on each side. Additional hydraulic, geotechnical, and structural analyses are needed to confirm the final design of these scour protection systems or viable alternatives.

Railroad Bridge

An existing NOGC 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 conveyance channel centerline. The current proposal is to realign the railroad track to cross the 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 would be spaced at 125 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.

Belle Chasse Highway 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.

A bridge would carry the highway 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 would cross over the conveyance channel, a floodwall would be constructed within the proposed MBSD guide levees. The floodwalls are envisioned to be typical USACE "T" walls supported on deep foundations below the bridge spans.

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 by 150 feet. The exact location and footprint is being developed. 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 6-2 presents anticipated loading conditions for the back structure.

Table 6-2. 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

7 30% Design Geotechnical Analyses

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 described in Section 6.4, Project Features, in conjunction with the design water surface described in Section 6.5, Loading Considerations for Proposed Improvements. While the inlet control, highway floodwall, and back structure would have conditions that would load the soil in compression, only the inlet control structure would potentially have a net uplift/pullout loading condition. The uplift pressures that could be experienced in that condition are described in Section 6.5.

Existing MR&T conditions were investigated as part of this current program using borings and CPT explorations. The information collected in the field and laboratory testing were used to develop models of the existing MR&T conditions. Cross sections illustrating the developed models are presented on Figures 7 through 12. The models were used in seepage and slope stability analyses. The results of these analyses were subsequently used to develop the feasibility and engineering options for the MBSD inlet system. The results of the seepage and slope stability analyses are presented in Appendixes A and B, respectively. In areas where design FOS are not met in the 30% design phase, additional design work will be needed in subsequent analyses to resolve these areas of concern.

7.1 Conveyance Channel and Guide Levees

Staged Construction Approach

As noted previously, most of the project site is underlain by relatively soft, weak, and compressible clay soil deposits. This clay foundation soil is too weak to support the load imposed by the full heights of the guide levees if they were constructed to full height in one construction operation. A staged construction approach would be required where only a portion of the planned levee is initially constructed. This would allow the underlying clay foundation soil to consolidate and gain strength before additional fill is placed. Because the underlying clay soil deposits are relatively thick and have slow rates of consolidation, the time needed to reach the required levels of consolidation and strength gain would be prohibitively long—many decades or more. To shorten the consolidation period, wick drains have been incorporated into the proposed approach.

In addition, the stability of both the guide levees and the conveyance channel would be affected by their proximity to each other. Where these two are in close proximity, the guide levee would reduce the FOS of the channel slope stability by imposing a surcharge load near the top of the slope. Conversely, the channel would reduce the FOS of the guide levee stability by reducing the available resistance near the toe of the levee.

To account for these variables, an iterative process was undertaken whereby both the number and height of each stage of levee construction—and the distance between the guide levee and channel—were adjusted and analyzed in each case. The iterative analyses also included adjusting the depth and lateral extent of the zone where wick drains would be installed and the wick drain spacing. Results of stability analyses are presented in Appendix B for the selected configuration.

Note that this selected case represents what is judged to be a viable alternative from a preliminary engineering standpoint, but does not a represent a unique solution. Refinements and other alternatives can be developed during subsequent phases of design as more information about the project and subsurface conditions becomes available. Also, in subsequent phases of design, analyses will be needed to more specifically define the levee and channel configuration and the stages of construction, including the height of each stage of filling, the strength gain and time needed for each stage, and the extent of wick drains needed. Recommendations for staged construction are provided in Section 8.1.

Seepage

Steady-state seepage analyses were performed on eight cross sections to assess the potential for underseepage problems along the channel alignment from the inlet system to the western end of the conveyance channel. A summary of the analysis approach and cases analyzed is presented below; additional details are presented in Appendix A.

Seepage analyses were performed using the computer program SEEP/W (2012), part of the geotechnical analysis software package GeoStudio 2012, developed by GEO-SLOPE International, Ltd. SEEP/W is a finite-element modeling program that evaluates both levee underseepage and through seepage. To use SEEP/W, section geometry is entered into the program as distinct soil layers. Permeability or hydraulic conductivity values (horizontal and vertical) are then assigned to the layers as soil type designations.

Seepage analyses were performed for the sections shown in Table 7-1.

Table 7-1. Cross sections selected for stability and seepage analysis

Cross section station location	Location description	Exploration(s) used to develop stratigraphy and soil properties
18+00 to 45+00	Section along project centerline across MR&T Levee, inlet system, and temporary setback levee at approximately Station 42+00; presence of point bar deposits	B-3C, IS-8A, NL-9A
35+00	Section transverse to the project centerline across the inlet system and temporary excavation setback levees; presence of point bar deposits	IS-8A, NL-9A
55+00	Section at conveyance channel and guide levee; presence of abandoned distributary channel deposits	NL-8A
67+00	Section at conveyance channel and guide levee beneath future LA 23 bridge; presence of natural levee deposits	NL-7C, NL-10C
82+00	Section at conveyance channel and guide levee; presence of abandoned distributary channel deposits	NL-6A
90+00	Section at conveyance channel and guide levee; at transition from natural levee deposits to marsh deposits	NL-5C, NL-11C
110+00	Section at conveyance channel and guide levee; presence of marsh deposits	NL-3A, NL-3C
130+00	Section at back structure; presence of marsh deposits	NL-1C

Note: LA 23 = Belle Chasse Highway

Seepage Parameter Selection

Seepage parameter selection was based on a review of the exploration logs associated with each cross section. Horizontal and vertical permeability coefficients were primarily selected based on correlations with soil type and fines content (Carrier 2003); as well as laboratory and field testing results. The document presents suggested permeability coefficients and soil anisotropy ratios for various soil materials using the Unified Soil Classification System (USCS). The soil stratigraphy and corresponding material properties used are presented on the results figures included in Appendix A.

Seepage Analysis Cases and Water Level Conditions

At the inlet system (from Station 18+00 to Station 45+00 and Station 35+00), steady-state seepage analyses were performed for the following cases. Water level data were based on the June 1978 Flood Control, Mississippi River and Tributaries Project Refined 1973 Project Flood Flowline by USACE.

- **Case 1: High Water Mississippi River/Dewatered Inlet System** – Water level in the Mississippi River at elevation +12.25 feet, water level in the inlet system excavation at elevation –50 feet, and water level on the nonexcavation side (polder side) of the temporary setback levee at elevation +3 feet. This case represents the condition when the river is at flood level, the water level within the inlet system area is at the bottom of the excavation, and the water level on the nonexcavation side of the temporary setback levee is at an assumed high groundwater level (ground surface). This analysis case was performed to estimate the rate of flow into the excavation.

- **Case 2: High Water Mississippi River/High Water Inlet System** – Water level in the Mississippi River at elevation +12.25 feet, water level in the inlet system excavation at elevation +12.25 feet, and water level on the nonexcavation side (polder-side) of the temporary setback levee at elevation –3.5 feet. This case represents the condition when the river is at flood level, the water level within the inlet system excavation matches that of the river, and the water level on the nonexcavation side of the temporary setback levee is at a relatively low groundwater level (estimated based on data recorded in piezometers PZ-13 to PZ-15). This case represents the hypothetical condition where there is a breach (such as in the MR&T Levee), causing flooding of the inlet system excavation. Analyses were performed for some time well after flooding has occurred, allowing for the development of steady-state seepage conditions.

Steady-state seepage analyses for each of the other cross sections (Station 35+00 to Station 130+00) were performed for two water level conditions, based on information provided by the HDR hydraulics team. The two conditions analyzed were:

- **Case 1: High Water Channel/Dewatered Polder** – Water level in the channel at elevation +10 feet and water level landside of the guide levee taken as corresponding to typical low groundwater level, which ranges from about elevation –3.5 feet at Station 55+00 to elevation –6.8 feet at Station 130+00. This case represents the condition when the channel is operating at its full design capacity coupled with relatively low groundwater levels in the adjacent areas. Low groundwater levels were estimated based on data recorded in piezometers PZ-13 to PZ-15.
- **Case 2: Low Water Channel/High Water Polder** – Water level in the channel at elevation 0 feet and water level landside of the guide levee at elevation +10 feet. This case is approximately the inverse of Case 1 and represents the condition when there is flooding outside of the guide levees (such as may occur if there is a breach in one of the other levees) while water in the channel is at a normal operating level.

Underseepage and Exit Gradient Calculations

For evaluation of levee underseepage, exit gradients were calculated at the following locations, judged to be critical for the levee and channel configuration under consideration:

- **Station 18+00 to Station 45+00 and Station 35+00**:
    - **Case 1** – For these cases, exit gradients were not calculated because the phreatic surface does not break out into the excavation under the steady-state conditions analyzed. Therefore, seepage gradients are not critical for this seepage case.
    - **Case 2** – For these cases, exit gradients were calculated at the ditch-side levee toe, ditch-side berm toe, and ditch toe (that is, the toe of the ditch slope). Note that a ditch is not modeled behind the setback levee from Station 18+00 to Station 45+00.

- **Station 55+00 to Station 130+00**:
    - **Case 1** – For these cases, exit gradients were calculated at the ditch-side levee toe, ditch side berm toe, and ditch toe (that is, the toe of the ditch slope).
    - **Case 2** – For these cases, exit gradients were calculated at the channel-side levee toe, channel-side berm toe, and channel toe (that is, the toe of the channel slope).

The following maximum average vertical exit gradients were selected as target values for evaluating results of the underseepage analyses:

- at the ditch-side and channel-side levee toes: $i_e \leq 0.5$
- at the ditch-side and channel-side berm toes: $i_e \leq 0.5$ to ≤ 0.8 up to 150 feet from the levee toe
- at the ditch and channel toes: $i_e \leq 0.5$ to ≤ 0.8 up to 150 feet from the levee toe

Levee through-seepage is considered to be a concern if the phreatic surface were to exit along the face of the levee, based on the SEEP/W analysis results, and the levee were to consist of erodible soil (typically silt or granular material). It is anticipated that the levees would be constructed of clayey soil that is not highly susceptible to erosion. Further, for cases where the phreatic surface would exit along the slope, the breakout height is estimated to be about 1 foot or less above the levee toe. On this basis, levee through-seepage is not anticipated to be a significant issue. The phreatic surface exit height is presented in Table 7-2.

Results of the seepage analyses are included in Appendix A. Table 7-2 also summarizes these results. As indicated previously, areas that do not meet criteria in the 30% design will be addressed in subsequent design analyses. The modeled sections at Station 67+88 and Station 82+00 had elevated seepage gradients in the drainage ditch adjacent to the polder side stability berm toe, assuming steady-state conditions develop during operation of the conveyance channel. The ditch is modeled as being dry in accordance with current USACE practice. The elevated seepage gradients are primarily a result of the ditch excavation thinning the foundation blanket layer and the presence of near surface interbedded, permeable silty sand layers related to the abandoned distributary channels. To reduce the seepage gradients, the more permeable layers can be cut off below the levee using either a sheetpile or in situ soil cement wall. Alternatively, the drainage ditch can be set back from the polder stability berm toe. The configuration of the polder side stability berm and the adjacent drainage ditch and localized seepage cutoff walls will need to be evaluated further in final design.

Table 7-2. Summary of seepage analysis results

Cross section designation and station location	Seepage gradient desig-nation	Seepage gradient location	Average vertical exit gradient	Critical gradient	Meets under-seepage criteria?	Phreatic surface exit height above levee toe
A-A' 18+00 to 45+00	2A 2B	Setback levee toe Setback berm toe	0.10 0.27	0.51 0.44	Yes Yes	1.0
32+00	2C	Ditch toe	0.28	0.44	Yes	N/A
55+00	1A 1B 1C 2A 2B 2C	Ditch-side levee toe Ditch-side berm toe Ditch toe Channel-side levee toe Channel-side berm toe Channel toe	N/A N/A 0.53 0.03 N/A 0.08	N/A N/A 0.79 0.79 N/A 0.68	N/A N/A Yes Yes N/A Yes	0.2
67+00 Sens. 1	1A 1B 1C 2A 2B 2C	Ditch-side levee toe Ditch-side berm toe Ditch toe Channel-side levee toe Channel-side berm toe Channel toe	N/A N/A 0.77 0.02 0.12 0.61	N/A N/A 0.68 0.74 0.68 0.68	N/A N/A No Yes Yes Yes	N/A
82+00	1A 1B 1C 2A 2B 2C	Ditch-side levee toe Ditch-side berm toe Ditch toe Channel-side levee toe Channel-side berm toe Channel toe	0.08 0.50 1.68 N/A N/A 0.73	0.76 0.68 0.68 N/A N/A 0.76	Yes Yes No N/A N/A Yes	N/A
90+00	1A 1B 1C 2A 2B 2C	Ditch-side levee toe Ditch-side berm toe Ditch toe Channel-side levee toe Channel-side berm toe Channel toe	N/A N/A 0.20 0.04 0.13 0.35	N/A N/A 0.60 0.65 0.60 0.60	N/A N/A Yes Yes Yes Yes	0.4
110+00	1A 1B 1C 2A 2B 2C	Ditch-side levee toe Ditch-side berm toe Ditch toe Channel-side levee toe Channel-side berm toe Channel toe	N/A N/A 0.22 0.05 0.11 0.02	N/A N/A 0.60 0.70 0.60 0.60	N/A N/A Yes Yes Yes Yes	0.5
130+00	1A 1B 1C 2A 2B 2C	Ditch-side levee toe Ditch-side berm toe Ditch toe Channel-side levee toe Channel-side berm toe Channel toe	N/A N/A 0.31 0.03 0.12 0.58	N/A N/A 0.60 0.69 0.60 0.60	N/A N/A Yes Yes Yes Yes	0.3

Note: N/A = not applicable

Stability

Stability analyses were performed on cross sections to assess the potential for stability concerns or problems along the channel alignment from the inlet system to the western end of the conveyance channel. Stability analyses were performed using the computer program SLOPE/W, part of the geotechnical analysis software package GeoStudio 2012, developed by GEO-SLOPE International, Ltd. SLOPE/W is a two-dimensional limit equilibrium stability analysis program that permits slope stability calculation using various limit equilibrium methods. The Spencer (1967) method was selected for all slope stability analyses because it is a rigorous formulation that satisfies both moment equilibrium and force equilibrium. A summary of the analysis approach and cases analyzed is presented below; additional details are presented in Appendix B.

Stability analyses were performed on cross sections as referenced previously for seepage analyses. Software integration features built into GeoStudio allow the user to import pore water pressure conditions calculated using SEEP/W into SLOPE/W. The steady-state seepage results discussed in Appendix A act as “parent” analyses for subsequent slope stability analyses. Slope-stability analyses are sometimes referred to as “child” analyses, which are dependent on results of the parent seepage analysis. This parent/child coupling facilitates identifying seepage-induced stability issues, examining the sensitivity of slope stability to selection of seepage parameters and evaluating the effect of improvements such as seepage/stability berms or cutoff walls.

Stability Parameter Selection

Two sets of stability parameters were developed for each cross section and were applied using distinct material models. The first set consists of fully drained strength parameters for use in steady-state stability analyses. The second set consists of a combination of drained and undrained parameters depending on soil type (that is, undrained for non-free-draining soils and drained for free-draining soils) for use in rapid flood stability analyses. Stability parameter selection was performed based on a review of the exploration logs associated with each cross section and available laboratory test results.

Drained strength parameters for steady-state stability analyses were selected primarily based on USCS soil classifications. No site-specific drained strength testing was available to directly estimate drained strength parameters; therefore, drained strength parameters for analysis were conservatively assumed based primarily on USCS soil classifications. The USACE New Orleans District Engineering Division developed the *Hurricane and Storm Damage Risk Reduction System Design Guidelines*, revised in June 2012, hereafter referred to as the USACE Hurricane Guidelines. Chapter 3 of the guidelines prescribes conservative drained strength values for various soil types for use in steady-state stability analyses when site-specific data are unavailable. Effective cohesion was conservatively selected as zero for all soil types under drained conditions. Effective friction angles were selected from a range of 23 degrees for clay to 30 degrees for clean sand.

Undrained strength parameters for rapid-flood stability analyses were selected based on a combination of prescribed values and site-specific information. Sands were considered free-draining materials; therefore, drained parameters were used for rapid-flood stability analyses. Undrained strength parameters of silts were selected using the USACE Hurricane Guidelines. Undrained strength parameters in clays and clay/silt mixtures for rapid flood stability were selected based on unconsolidated undrained (UU) triaxial test results, geotechnical index testing, CPT-based strength correlations, estimated overconsolidation ratios (OCRs), an assumed normally consolidated undrained strength ratio (0.22), and field strength testing (field vane). Undrained strengths within normally consolidated clays were increased assuming an 8-foot-thick aerial preload fill (120 pounds per cubic foot [pcf]) left in place for 1 year and 50 percent pore pressure dissipation in foundation clays. A total undrained strength increase of up to 106 psf was applied (that is, 8-foot fill × 50 percent consolidation × 120 pcf × 0.22 = 106 psf).

Undrained strength parameters selected using the above methods were compared for consistency with undrained strength profiles produced by GeoEngineers for each exploration as presented in its GDR. Undrained strengths used in the rapid flood stability analyses do not match the GeoEngineers strength lines exactly, but were found to be similar within the upper soil layers that are expected to most greatly influence the critical slip circles in the stability analyses. Figures presented for each section in Appendix B show side-by-side comparisons of GeoEngineers' strength lines and undrained strength lines used in the analyses.

Stability Analysis Cases and Boundary Conditions

The MR&T, guide levee, and channel configurations analyzed at each cross section location correspond to those shown in drawings in Volume 1, General Civil Sitework. The levee and channel configurations, as well as that of the landside ditch (or polder), are based on operational requirements and include consideration of iterative slope stability analyses to help establish slope inclinations and the extent of the stability berm.

Stability analysis of the existing MR&T Levee was performed on two cross sections. Two cross sections, C-C' and E-E' as presented on Figures 4, 9 and 11, were selected in the vicinity of where the inlet channel structures are proposed to cross into the Mississippi River through the MR&T. The analysis was performed using steady-state conditions calculated in seepage models. Two Mississippi River water surface elevations were used, +5 and +12.5 feet. The focus of the analysis was to review the existing conditions prior to the start of construction. Stability during construction will need to be analyzed in detail in conjunction with the tie-in design as part of the 60% design.

Stability analyses for the inlet system cross sections focused on the temporary (during construction) condition of the setback levees. Both of the analysis cases described below use steady-state seepage conditions calculated in Seepage Case 2 (flooded excavation), as described in Appendix A. Stability analyses were not performed within the excavation because the cofferdam has not yet been included in the excavation model. Therefore, stability analyses of the excavation would not represent actual construction conditions. The following stability analysis cases were evaluated at temporary setback levees for the inlet system excavation (from Station 18+00 to Station 45+00 and Station 35+00):

- **Case 1** – Water level in the Mississippi River at elevation +12.25 feet, water level in the inlet system excavation at elevation +12.25 feet, and water level on the nonexcavation side (polder side) of the temporary setback levee at elevation –3.5 feet. This case represents the condition when the river is at flood level, the water level within the inlet system excavation matches that of the river, and the water level on the nonexcavation side of the temporary setback levee is at a relatively low groundwater level (estimated based on data recorded in piezometers PZ-13 to PZ-15). This case represents the hypothetical condition where there is a breach (such as in the MR&T Levee), causing flooding of the inlet system excavation. Analyses assumes that sufficient time has elapsed after flooding that steady-state seepage conditions and corresponding drained conditions for soil strength are appropriate. Analyses focused on slope surfaces in the temporary setback levee in the direction away from the inlet system excavation. A 65-foot-long stability berm was added on the polder side of the temporary setback levees to meet stability criteria.

- **Case 2** – This stability case is subjected to the same flood water levels described previously for Case 1. Analyses were performed assuming rapid-flood loading conditions and corresponding undrained conditions for soil strength. Analyses focused on slope surfaces in the temporary setback levee in the direction away from the inlet system excavation. The stability berm on the polder side of the setback levee (described for Case 1) was included in the analysis.

Stability analyses for the conveyance channel cross sections (Station 55+00 to Station 130+00) were focused on the permanent (long-term) condition of the guide levees and channel. Stability Case 1 uses steady-state seepage conditions from Seepage Case 1. Stability Cases 2 and 3 use steady-state seepage conditions from Seepage Case 2. The seepage cases are described in Appendix A. The following stability analysis cases were evaluated for the conveyance channel cross sections:

- **Case 1** – Water level in the channel at elevation +10 feet and water level landside of the guide levee taken as corresponding to typical low groundwater level, which ranges from about elevation –3.5 feet at Station 55+00 to elevation –6.8 feet at Station 130+00. This case represents the condition when the channel is operating at its full design capacity coupled with relatively low groundwater levels in the adjacent areas. Analyses were performed assuming steady-state seepage conditions and corresponding drained conditions for soil strength. Analyses focused on slip surfaces toward the landside direction (toward the ditch). Both global-scale slip surfaces (from about the levee crown to the ditch) and local slip surfaces (from the stability berm toe to the ditch) were analyzed.
- **Case 2** – Water level in the channel at elevation 0 feet; water level landside of the guide levee at elevation +10 feet. This case is approximately the inverse of Case 1 and represents the condition when there is flooding outside of the guide levees (such as may occur if there is a breach in one of the other levees) while water in the channel is at a normal operating level. Analyses were performed assuming steady-state seepage conditions and corresponding drained conditions for soil strength to represent the case where water levels are sustained at flood levels for a relatively long period of time. Both global-scale slip surfaces (from about the levee crown to the channel toe) and local slip surfaces (from the stability berm toe to the channel toe) were analyzed.
- **Case 3** – Water level in the channel at elevation 0 feet and water level landside of the guide levee at elevation +10 feet. This case has the same water levels as Case 2 but assumes that the water level landside of the guide levee reaches elevation +10 feet quickly, to represent the rapid flood loading condition. Analyses were performed using undrained conditions for soil strength, but the phreatic surface within the levee and pore pressures within foundation layers were conservatively modeled using steady-state seepage conditions. Both global-scale slip surfaces (from about the levee crown to the

channel toe) and local slip surfaces (from the stability berm toe to the channel toe) were analyzed.

The USACE New Orleans District Guidance Document Section 3 served as a reference for developing some of the criteria used for analyses for the MBSD. Based on this document and others, the following target FOS for stability were adopted:

- steady-state stability: FOS $\geq$1.5
  - Case 1 for inlet system (from Station 18+00 to Station 45+00 and Station 35+00)
  - Cases 1 and 2 for conveyance channel (Station 55+00 to Station 130+00)
- rapid flood stability: FOS $\geq$1.3
  - Case 2 for inlet system (from Station 18+00 to Station 45+00 and Station 35+00)
  - Case 3 for conveyance channel (Station 55+00 to Station 130+00)

Results of the stability analyses are included in Appendix B. For each analysis cross section location, the appendix includes (1) tabulation of the soil stratigraphy, soil parameters, water level conditions, and calculated critical FOS, and (2) graphical results of the SLOPE/W analysis, including the locations of the calculated critical slip surfaces. Table 7-3 summarizes the analysis results. The results were used to inform the placement of cutoff walls, drainage ditch locations, and surcharging requirements. As can be seen in the table for Case 1B (polder side stability berm and drainage ditch stability, steady-state seepage during operating conveyance), FOS near 1.0 or below was common for all cases. Case 1A modeled polder side steady-state stability through the levee during operation of the conveyance channel, which met the minimum design criteria. The Case 1B analysis indicates that, although not a levee safety issue, placement of the drainage ditch at the toe of the seepage berm could result in failure of a portion of the berm into the ditch, which could affect surrounding site drainage. Conservative soil strength parameters were assigned to the polder side soils representing current soil strengths. The parameters should be reevaluated during final design to include the potential strength gains attributable to staged construction once the construction sequence/ approach is developed. Stabilization measures such as offsetting the ditch or using stabilization fabrics below the stability berm should be evaluated during final design.

Case 2 models channel side stability for the conditions where the polder has flooded, the conveyance is not in operation, and sufficient time has passed to develop steady state seepage conditions. Case 3 models channel side stability during rapid loading of the levee due to flooding of the polder. For Case 2B, the FOS do not meet the criteria but are not failing (FOS of 1.0 or less). A reduced FOS may be acceptable for the Case 2B condition (stability berm/channel slope stability) in terms of levee safety. The lower FOS suggests that slope slumping or shallow slope failures could occur within the conveyance channel slopes and would need to be repaired. Measures to improve stability include raising the water surface within the conveyance channel to equalize the water pressures. Alternatively, the levee and stability berm can be set back from the top of channel and the water level in the channel can be raised to reduce the differential head between the flooded polder and the channel.

Table 7-3. Summary of stability analysis results

Cross section designation and station location	Analysis		Target FOS	Calculated critical FOS	Slip direction	Critical slip surface description
	Case	Type				
18+00 to 45+00	1	SS	≥1.50	1.54	Toward channel	Setback levee crown to channel
	2	RFL	≥1.30	1.75	Toward channel	Setback levee crown to channel
35+00	1	SS	≥1.50	2.47	Ditch-side	Setback levee crown to ditch
	2	RFL	≥1.30	1.69	Ditch-side	Setback levee crown to ditch
55+00	1A	SS	≥1.50	2.62	Ditch-side	Levee crown to ditch
	1B	SS	≥1.50	0.90	Ditch-side	Berm toe to ditch
	2A	SS	≥1.50	2.15	Channel-side	Levee crown to channel toe
	2B	SS	≥1.50	1.16	Channel-side	Berm toe to channel slope
	3A	RFL	≥1.30	1.79	Channel-side	Levee slope to channel toe
	3B	RFL	≥1.30	1.59	Channel-side	Levee crown to berm
67+00	1A	SS	≥1.50	2.40	Ditch-side	Levee crown to ditch
	1B	SS	≥1.50	0.58	Ditch-side	Berm toe to ditch
	2A	SS	≥1.50	2.01	Channel-side	Levee slope to channel toe
	2B	SS	≥1.50	1.25	Channel-side	Berm toe to channel slope
	3A	RFL	≥1.30	1.72	Channel-side	Levee slope to channel toe
	3B	RFL	≥1.30	2.05	Channel-side	Berm to channel toe
	3C	RFL	≥1.30	1.72	Channel-side	Levee crown to levee toe
82+00	1A	SS	≥1.50	1.73	Ditch-side	Levee slope to ditch
	1B	SS	≥1.50	0.20	Ditch-side	Berm toe to ditch
	2A	SS	≥1.50	2.71	Channel-side	Levee crown to channel toe
	2B	SS	≥1.50	1.36	Channel-side	Berm toe to channel slope
	3A	RFL	≥1.30	2.14	Channel-side	Levee slope to channel slope
	3B	RFL	≥1.30	2.53	Channel-side	Berm to channel toe
	3C	RFL	≥1.30	1.68	Channel-side	Levee crown to levee toe
90+00	1A	SS	≥1.50	2.23	Ditch-side	Levee slope to ditch
	1B	SS	≥1.50	0.93	Ditch-side	Berm toe to ditch
	2A	SS	≥1.50	1.86	Channel-side	Levee crown to channel toe
	2B	SS	≥1.50	1.30	Channel-side	Berm toe to channel slope
	3A	RFL	≥1.30	1.68	Channel-side	Levee slope to channel slope
	3B	RFL	≥1.30	1.52	Channel-side	Levee slope to berm
110+00	1A	SS	≥1.50	2.29	Ditch-side	Levee crown to ditch
	1B	SS	≥1.50	1.04	Ditch-side	Berm toe to ditch
	2A	SS	≥1.50	2.01	Channel-side	Levee crown to channel toe
	2B	SS	≥1.50	1.43	Channel-side	Berm toe to channel slope
	3A	RFL	≥1.30	1.52	Channel-side	Levee slope to channel slope
	3B	RFL	≥1.30	1.99	Channel-side	Berm to channel slope
	3B	RFL	≥1.30	1.20	Channel-side	Levee slope to berm
130+00	1A	SS	≥1.50	2.02	Ditch-side	Levee crown to ditch
	1B	SS	≥1.50	0.89	Ditch-side	Berm toe to ditch
	2A	SS	≥1.50	1.95	Channel-side	Levee slope to channel toe
	2B	SS	≥1.50	1.40	Channel-side	Berm toe to levee toe
	3A	RFL	≥1.30	1.24	Channel-side	Levee slope to berm
	3B	RFL	≥1.30	1.91	Channel-side	Berm to channel slope

Notes: FOS = factor of safety, RFL = rapid flood loading condition, SS = steady-state seepage condition

For Case 3B (rapid loading levee stability), soil strengths below the levee were assumed to be the current soil strengths with no allowance for strength gain due to staged construction. Station 110+00 and Station 130+00 are within the western portion of the site and have the weakest near-surface soils. Accordingly, these sections have FOS that do not meet criteria. The stability of each stage of construction, including associated strength gains attributable to consolidation of the underlying soils and use of high-strength geotextile fabrics, will need to be evaluated during final design.

Settlement

Settlement analysis were conducted using laboratory data obtained from current site characterization efforts as well as available data from past USACE geotechnical studies for the Non-Federal New Orleans to Venice project and the Myrtle Grove Sediment Diversion project. A memorandum that presents details of the current 30% design level settlement analysis is presented in Appendix C.

Settlement analysis was conducted using the computer program Consol3 (Virginia Tech). The program calculates the change of stress due to irregular surcharge loads (such as an earthen embankment) and calculates settlement over time for a vertical soil column. The conveyance channel and guide levees are assumed to be linear features that are constructed on “virgin” ground and can be represented by a one-dimensional model. This is judged to be appropriate for 30% design. More detailed two- and three-dimensional analyses will be required as project details are developed and construction phasing is evaluated.

Five cross sections representing five reaches of the conveyance channel were developed to represent the variation of foundation conditions along the conveyance channel. Table 7-4 summarizes the five reaches of levee modeled with the associated foundation condition and dominant features. Settlement analyses were conducted as follows:

- Calculate total primary settlement for each model.
- Evaluate the time rate of consolidation and the variation of time rate with depth.
- Calculate effect of wick drains on the consolidation time rates within the foundation soils.
- Evaluate secondary compression effects.
- Evaluate minimum height of surcharge for stability berm and excavation phasing.
- Provide initial recommendations for project phasing and site preparation for wick drain installation.

To investigate the contribution of settlement within contributing substrata, the percent of ultimate settlement that occurs between the depths of 0 to 30 feet, 30 to 60 feet, and below 60 feet were calculated and are summarized in Table 7-4, which is reproduced from Table 2 in the settlement memorandum presented in Appendix C.

Table 7-4. Summary of calculated primary settlement by model location

Modeled location		Ground surface elevation (feet)	Required levee over-build to obtain elev. +13 feet	Required levee total fill thickness (feet)	Ultimate primary settlement (feet)[a]	Percentage contribution by depth range		
Reach	Station					0 to 30 feet	30 to 60 feet	60+ feet
1	30+00	+3.0	Build to +13 feet	10	1.40 (3.0)[b]	85 (63)	14 (16)	1 (20)
2	55+00	+2.0	2	13	1.6	50	34	16
3	67+00	+0.0	2	15	1.8	60	10	28
4	82+00	+0.0	3.5	16.5	2	50	32	18
5	110+00	-4.0	7	21	7	53	26	21

[a] Guide levee fill modeled with 4.5:1 slopes and a 15-foot-wide crown width
[b] (value) – settlement with groundwater drawn down to elevation –50 feet

Primary settlements vary along the conveyance channel alignment and were analyzed for settlements versus depth at 1-year, 2-year, 5-year, 10-year, 20-year, and 50-year intervals. The analysis led to the following conclusions for areas outside of the point bar deposits:

- Approximately 50 percent of the settlement occurs in the upper 30 feet of the foundation soils and about 70 to 85 percent occurs in the upper 60 feet.
- Time rate of settlements analysis calculates the following percentage of ultimate primary settlement as occurring:
  - Year 5 – 45 to 55 percent
  - Year 10 – 60 to 70 percent
  - Year 20 – 80 to 94 percent
  - Year 50 – 80 to 94 percent

Within the point bar (Station 30+00), 85 percent of the settlement occurs within the upper 30 feet of the foundation if groundwater levels are not lowered during construction. The impacts of dewatering need to be considered in guide levee design and for the MR&T Levee but are beyond the scope of this report since the construction sequence has not been defined.

7.2 Inlet System

Shallow Foundations

Given the relatively high strength of soil at the base of the inlet control structure, a shallow foundation may be a potential alternative. 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 D. 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, FOS 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 D.

Pile Foundations

The same approach was used to evaluate pile vertical capacity for the inlet control, floodwall, and back structure, as well as for the railroad and highway bridge foundations, discussed collectively below. Pile vertical 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 D.

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 current 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. Additional geotechnical investigations are required for design of the rail and highway bridge pile-supported foundations and approach fills, which were outside the scope of the current investigations.

Between Station 125+00 and Station 140+00, the proposed back structure would 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 previously, 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 6-2 are anticipated to experience compression dead loads on the foundation and lateral pressures on the structure 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.

Lateral and Uplift Pressures on Retaining Walls and Slabs

Temporary and permanent wall loading conditions were analyzed for both the inlet system and back structure. The same approach was used to evaluate lateral and uplift pressures for both structures. The two loading conditions described below effectively bound the wall pressures and uplift pressures from high (empty condition) to low (operational conditions).

- Temporary – Empty condition where the structure is dry and subjected to full hydrostatic pressure (both lateral and uplift pressure) from adjacent normal groundwater conditions. This represents the dry construction condition with the conservative assumption that groundwater levels outside the excavation have not been drawn down through active dewatering.
- Permanent – Operational condition in which the water level in the structure is at elevation 0 feet and the groundwater level is equal to or lower than the operational water level. This represents a condition where the channel is operating at normal water levels and groundwater levels outside the structure are such that the walls are not subjected to hydrostatic loading in the direction into the structure.

The inlet system structures are proposed to be within the saturated, generally sandy soil of the point bar deposits. The back structure would be within the saturated clayey marsh deposits.

The inlet and back structure systems would 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. Additional details of the analysis approach are presented in Appendix E. Preliminary lateral earth pressure, hydrostatic pressure, and uplift pressure diagrams were developed based on the above assumptions for both the inlet system and back structure for temporary (empty) and permanent (operational) conditions.

7.3 Back Structure

Pile Foundations

The same approach was used to evaluate pile vertical capacity for the inlet control, floodwall, and back structure, as well as for the railroad and highway bridge foundations, as previously discussed collectively. Preliminary pile capacities versus depth are presented in Appendix D.

Between Station 125+00 and Station 140+00, the proposed back structure would 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 previously, 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 6-2 are anticipated to experience compression dead loads on the foundation and lateral pressures on the structure walls.

Lateral and Uplift Pressures on Retaining Walls and Slabs

The same approach was used to evaluate lateral and uplift pressures for the inlet control and back structure, as discussed previously. Preliminary lateral earth pressure diagrams and uplift diagrams were developed based on the above assumptions and are presented in Appendix E. Preliminary hydrostatic and earth pressures for the condition when the systems may operate dry or empty were developed and are also presented in Appendix E.

7.4 Belle Chasse Highway and Railroad Bridges

Pile Foundations

The same approach was used to evaluate pile vertical capacity for the inlet control, floodwall, and back structure, as well as for the railroad and highway bridge foundations, as previously discussed collectively. Preliminary pile capacities versus depth are presented in Appendix D.

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.

8 30% Design Conclusions and Recommendations

8.1 Conveyance Channel and Guide Levees

Configurations

Based on the stability and seepage analyses, HDR concludes that the channel and guide levee configurations included in the drawings in Volume 1, General Civil Sitework, are generally feasible from a geotechnical standpoint. As the project moves toward 60% design, additional field exploration and laboratory testing will be undertaken and additional analyses will be performed. Some specific areas that will need further evaluation, resulting in revisions to the preliminary configuration and design, are discussed below.

With respect to seepage and stability, a number of the preliminary analyses indicated FOS below values established for the project. These cases are discussed below, along with recommendations for additional analyses and approaches for mitigating the condition to acceptable levels.

Guide Levee Stability Berm Toe

For all cross sections analyzed, critical slip surfaces extending from the berm toe into the ditch for the steady-state condition had FOS below target values. Potential approaches (including combinations of these approaches) to increase the FOS to acceptable levels during future phases of the project include:

- Changing the configuration of the berm toe and ditch, including different ditch depths and side slope inclinations.
- Moving the ditch farther away from the levee.
- Reevaluating the design water levels in the ditch and assessing whether they are overly conservative. A higher water level would result in a higher FOS.
- Using soil improvement to increase the soil strength in the critical zone.

With respect to underseepage, seepage exit gradients into the ditch were above the target values at Station 67+00 and Station 82+00. Potential approaches (including combinations of these approaches) to reduce these seepage gradients to acceptable levels during future phases of the project include:

- Reanalyzing the cross section using revised soil stratigraphy, based on information obtained from future phases of field investigation and laboratory testing.
- Changing the configuration of the berm toe and ditch, including different ditch depths and side slope inclinations.
- Moving the ditch farther away from the levee.
- Reevaluating the design water levels in the ditch and assessing whether they are overly conservative. A higher water level would result in lower gradients.

Channel Slope

- For all cross sections analyzed, critical slip surfaces in the channel slope had FOS below the target values. The critical slip surfaces were typically shallow surface failures. Potential approaches (including combinations of these approaches) to increase the FOS to acceptable levels during future phases of the project include:
  - Reevaluating the parameters used for analysis because relatively conservative values were used. In part, this was because laboratory testing for drained conditions was not performed during Phase 1. Such drained strength testing should be performed in future phases to develop drained strength parameters for analyzing this loading condition.
  - Considering measures to improve the near-surface soil strength along the channel slopes.
- For cross sections at Station 130+00 and Station 110+00, corresponding to areas with the weakest near-surface soil, critical slip surfaces through the levee and channel berm had FOS slightly below the target values for the rapid-flood loading condition. Potential approaches (including combinations of these approaches) to increase the FOS to acceptable levels during future phases of the project include:
  - Reevaluating the parameters used for analysis because relatively conservative values were used, and the resulting FOS were only slightly below the target values.
  - Using soil improvement to increase the soil strength in the critical zone.

Borrow Site Evaluation

Guide levees are proposed to be constructed of soils generated from on-site materials. As described above in more detail, the site is generally underlain by silty clay, clay and clayey silts. As the guide levee and areas to receive engineered fill progress in design, suitable fill material types will need to be developed and recommended. Potential borrow sites should be investigated in accordance with the USACE New Orleans District Geotechnical Design Procedure for Earthen Embankments and USACE New Orleans District Standard Specifications 31 24 00.00 12.

Current plans have indicated that excavated materials associated with the conveyance channel may be suitable for use as engineered fill. Furthermore, two sites to the north of the project located on Phillips 66 property may also provide materials suitable for use as engineered fill. Given the limited use and wide spacing of the current exploration program and the level of current design effort, a geotechnical investigation to identify these materials will most likely be needed.

Available laboratory data from the current investigation has been compiled in Tables 8-1 through 8-3 for the purposes of identifying the vertical and lateral extent of soils that may be suitable for use as levee borrow.

The USACE New Orleans District has published levee borrow material requirements as part of the Hurricane and Storm Damage Risk Reduction System design guidelines, as follows:

Classification of Suitable Soils (ASTM D2487 and USCS)

- CH and CL
  - Soil cannot contain more than 35% sand by dry weight
  - Soil cannot contain more than 9% organic material by weight

Plasticity Index

- PI shall be greater than 10%

These requirements have the following limitations:

1. ML soils are not suitable; however, minor amounts of ML may be suitably blended with CH or CL to formulate a CL per ASTM D2487.
2. Borrow soil may not contain excessive amounts of wood; however, isolated wood pieces less than 1 foot long and with a cross section less than 4 square inches are acceptable, although pockets and/or zones of wood are not acceptable. The soil cannot contain more than 1 percent by volume of objectionable material.

Tables 8-1 through 8-3 present the results of available organic content testing, Atterberg Limits tests and a summary of material distribution based on the soils visual classification and thickness were noted on the individual borings logs. Table 8-4 presents the USACE New Orleans District guide for soil moisture and plasticity that is used to classify fine-grained soils in the Lower Mississippi River region.

The distribution of soils that likely meet levee borrow soil requirements vary across the site. Specifically:

- Soils between Station 28+00 and Station 38+00 located above elevation –5 to –10 feet are likely to meet the requirements for levee borrow soils. Below that depth, selective excavations may be needed to avoid more granular soils or will require blending with higher plasticity soils to meet levee borrow soil material requirements.
- Soils between Station 38+00 and Station 93+00 located above elevation –10 feet are likely to meet the requirements for levee borrow soils. Below that depth, selective excavations may be needed to avoid more granular soils or may require blending with higher plasticity soils to meet levee borrow soil material requirements.
- Soils between Station 93+00 and Station 140+00 meeting the requirements for borrow soils are present above elevation –16 feet. However, more selective excavations would likely be required to remove highly organic, highly plastic soils and granular soils. Below elevation –16 feet, it is likely the soil high organic content will preclude the use of these soils as levee borrow material.

Table 8-1. Soil profile characterization – fine-grained soils, Station 38+00 to Station 93+00

Predominant classification	Percentage of volume[a]	Organic content (%)			Plasticity index testing	Plasticity index		Liquid limit		Classification (based on mean PI and LL value)	Visual soil classification			
		Laboratory sample count	Range	Mean	Laboratory sample count	Range	Mean	Range	Mean		% Fine-grained[b]	Classifications	Predominant classification	Percentage of volume[b]
Natural Levee	+3.5 to -10	17	2.1 to 7.2	3.6	41	11 to 57	23	33 to 83	44	CL4/ CL6	100	ML, CL4, CL6, CH2, CH3, CH4	CL4/CL6	60
Inter Distributary	-10 to -60	—	—	—	40	10 to 101	29	19 to 136	47	CL6	92	ML, CL4, CL6, CH2, CH3, CH4	ML	41
Upper Pro Delta	-60 to -85	—	—	—	10	15 to 69	58	37 to 95	85.5	CH4	88	ML, CL4, CL6, CH2, CH3, CH4	CH3/CH4	51
Lower Pro Delta	-85 to -130	—	—	—	12	35 to 72	54	53 to 97	83	CH4	84	ML, CL4, CL6, CH2, CH3, CH4	CH3/CH4	65
Gulf Near Shore	-105 to -117	—	—	—	2	49 to 72	60.5	70 to 97	83.5	CH4	80	CL6, CH2,CH3, CH4	CH3/CH4	65

[a] percentage of length of soil core logged as predominant classification soil for the specific depth interval

[b] does not include material classified as SC, SM, or SP

Table 8-2. Soil profile characterization – fine-grained soils, Station 28+00 to Station 38+00

Deposit	Elevation (feet)	Organic content (%)			Plasticity index testing	Plasticity index		Liquid limit		Classification (based on mean PI and LL value)	Visual soil classification			
		Laboratory sample count	Range	Mean	Laboratory sample count	Range	Mean	Range	Mean		% Fine-grained[a]	Classifications	Predominant classification	Percentage of volume[b]
Upper Levee	+16 to +8	—	—	—	6	15 to 54	24	31 to 75	44.5	CL6	100	ML, CL4 , CL6, CH2, CH3	CL4/CL6	81
Lower Levee	+8 to +2	—	—	—	8	9 to 26	16	31 to 46	37	CL4	99	ML, CL4 , CL6, CH2, CH3	CL4/CL6	87
Natural Levee	+2 to -5.5	—	—	—	21	11 to 41	21	33 to 66	43	CL4/CL6	100	ML, CL4 , CL6, CH2, CH3	CL4/CL6	69
Upper Point Bar	-5.5 to -45	—	—	—	37	6 to 44	19	26 to 68	40	CL4	78	ML, CL4 , CL6, CH2, CH3	ML	44
Middle Point Bar	-45 to -65	1	0.9 to 0.9	0.9	12	7 to 57	26	29 to 85	49	CL6	62	ML, CL4 , CL6, CH2, CH3	ML	42

[a] percentage of length of soil core logged as predominant classification soil for the specific depth interval

[b] does not include material classified as SC, SM, or SP

Table 8-3. Soil profile characterization – fine-grained soils, Station 93+00 to Station 139+00

Deposit	Elevation (feet)	Organic content (%)			Plasticity index testing	Plasticity index		Liquid limit		Classification (based on mean PI and LL value)	Visual soil classification			
		Laboratory sample count	Range	Mean	Laboratory sample count	Range	Mean	Range	Mean		% Fine-grained[a]	Classifications	Predominant classification	Percentage of volume[b]
Marsh	-4 to -15	2	3 to 5.5	4.3	15	17 to 162	61	37 to 206	91	CH4	94	ML, CL4 , CL6, CH2, CH3, CH4, CHOA, CHOB, CHOC	CL4/ CL6	59
Intra Delta	-15 to -36	15	1.8 to 26.1	7.6	19	17 to 149	57.5	41 to 195	84	CH4	83	ML, CL4 , CL6, CH2, CH3, CH4, CHOC	CH2/ CH4	35
Inter Distributary	-36 to -70	—	—	—	22	30 to 73	55	50 to 101	81	CH4	96	ML, CL4 , CL6, CH2, CH3, CH4	CH3/ CH4	63
Upper Pro Delta	-70 to -85	—	—	—	1	65 to 65	65	94 to 94	94	CH4	100	CL6, CH2, CH4	CH2/ CH4	93
Lower Pro Delta	-85 to -110	—	—	—	5	56 to 64	59	85 to 95	88	CH4	98	CH4	CH4	98
Gulf Near Shore	-115 to -120	—	—	—	2	42 to 47	44.5	72 to 72	72	CH3/CH4	58	CH3, CH4	CH3/ CH4	58
Pleistocene	-120 to -135	—	—	—	2	74 to 76	75	104 to 105	104.5	CH4	100	CH4	CH4	100

[a] percentage of length of soil core logged as predominant classification soil for the specific depth interval

[b] does not include material classified as SC, SM, or SP

Table 8-4. USACE guide for moisture content and plasticity

Guide for
* MOISTURE CONTENTS
ADAPTED TO CEMVN-ED-F SOILS

CLASS	STIFF	MEDIUM	SOFT	V. SOFT	LIQUID LIMIT	PLASTICITY INDEX
CH-4	41-53	43-65	55-80	67-130	70-110	45-75
CH-3	32-43	34-55	44-67	55-114	55-80	30-55
CH-2	27-34	30-44	38-55	48-90	50-60	25-40
CL-6	23-30	25-39	33-48	40-79	40-50	20-35
CL-4	20-25	21-33	27-41	35-67	28-43	10-25
CH-OA					110-160	75-97
CH-OB					160-185	97-115
CH-OC					185-	115-

* For brown or oxidized soils, subtract 10% from the above Moisture Contents.

NOTE: We are using this with the Unified Soil Classification System as a guide and supplementation breakdown for CH's and CL's. We use the CHOA, CHOB and CHOC for organic fat clays in lieu of "OH" and CLOA, CLOB and CLOC for organic lean clays in lieu of OL when used for lean clays. Also, double classes are not used, such as SC-SM or CL-ML. The major class governs and the secondary is recorded as a modification or stratum as appropriate.

Figure 8.5 Unified Soil Classification System Modified for New Orleans Soils

Recommendations for Staged Construction

Given soft soils at the site and the proposed loads currently anticipated, staged construction is recommended. As discussed above, the native conditions are highly compressible and have relatively low strengths in the upper substrata of the foundation materials. The previous sections present staged fills and construction per feature. However, a global staging of the features should be considered as well. HDR recommends considering timelines and costs for construction sequencing of features. The following is the current general construction sequencing assumed for this project (details such as permitting have been left out):

1. Clear and grub the areas of the proposed levee foundation, starting with the regions that have been identified as the most compressible.
2. Areas in step 1 to receive wick drains subsequent to placing materials as a working pad. In parallel, efforts to clear and grub areas adjacent to the MR&T Levee in preparation for construction of the setback levee.
3. Construct the setback levee adjacent to the MR&T and begin placing fills on wick drained areas. In parallel, monitoring settlements and placing fills as soon as possible along the conveyance levees.
4. Upon completion of the setback levee, begin construction of the inlet into the Mississippi River.

5. Begin preparation and construction of the outlet and associated structures with the Barataria Basin.
6. Re-grade and construct temporary LA 23 alignment, construct the new highway overpass of the conveyance channel.
7. Realign NOGC railroad and construct the new railway overpass of the conveyance channel.
8. Begin excavation of channel to proposed grades. Excavated materials may be used as fill once appropriately processed.
9. Complete all connections between structures and top of fills.
10. Take steps to begin operations.

There are significantly more steps per item for the above list. However, that level of detail should be developed along with cost estimates and schedules. It should also be noted that fill placement will continue as a maintenance issue for up to 40 years given the compressibility of the foundation materials. Operations will continue while settlements are active and should be monitored by both pore water pressures and surveying.

8.2 Project Structures

Shallow Foundations

Preliminary recommended allowable bearing pressures were presented in Section 7.2, Inlet System, and in Appendix D to evaluate the feasibility of supporting the inlet system structures on shallow foundations. Where these allowable bearing pressures are insufficient or if the corresponding magnitudes of settlement are excessive, pile-supported foundations should be considered.

Pile Foundations

As discussed in Section 7.2, preliminary recommended pile capacities versus depth were developed for the inlet control, floodwall, and back structure, as well as for the railroad and highway bridge foundations, for 2-foot pile diameters. The preliminary recommended pile capacity versus depth curves are presented in Appendix D. Table 8-5 presents preliminary pile lengths based on the current estimated loading conditions presented previously.

The 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). These preliminary pile capacities are considered to be conservative. Pile capacity analyses using methods that are based on boring data (such as SPT N-value) or on laboratory test data (such as undrained shear strength from strength testing), for example, the α-method, were not performed in the Phase 1 design because these data were being developed at the time of the analyses.

Table 8-5. Pile lengths

Structure	Estimated lengths (feet)
Control (Inlet Gates)	70 (tension)
Outlet	70 (compression)
Transition	70 (compression)
NOGC crossing	200 (compression)
Highway crossing	200 (compression)
Back	170 (compression)

HDR recommends that additional analyses be performed in future phases of the project, including conducting a full-scale pile load test program to verify both compression and tension capacities for critical structures. Consideration should be given to performing analyses based on other pile capacity estimation methods; such analyses should incorporate information from additional field explorations and laboratory testing that become available.

It is also noted that these vertical pile capacities are for single piles and assume that piles are a minimum of three pile-diameters apart, measured center-to-center. Should piles be spaced closer together, a reduction of the pile capacities may be needed to account for group effects. Also, pile lateral capacities have not been evaluated as part of the 30% design.

Lateral and Uplift Pressures

Both temporary and permanent wall loading conditions were analyzed. The same approach was used to evaluate lateral and uplift pressures for the inlet control and back structure. Preliminary lateral earth pressure diagrams and uplift diagrams are presented in Appendix E. Preliminary hydrostatic and earth pressures for the condition when the systems may operate dry or empty were developed and are also presented in Appendix E.

9 Summary

This report presents 30% design level parameters based on 80 exploration locations spread over the project area, review of readily available reports and geologic maps, and meetings with local agencies. In general, the project site is located on relatively compressible, weak soils and relatively shallow groundwater. The project goals present loading conditions that require staged placement of fills, large lateral loads, and pile foundations for larger structures. Given the distances between exploration locations and the regional level of available information, the data presented in this report are considered appropriate for 30% design only and provide a concept level of design.

As a means of developing the project to 60% design and further, additional explorations are required. The next phase of investigations should consider those concepts that were developed as a result of this investigation such as anticipation of settlements, pile foundations, and structural tie-ins. Furthermore, future investigation programs should consider portions of the project that are in support of the main project but were not part of the scope of this program, such as the pump station and borrow areas. These portions of the project need to be investigated and designed as the project moves forward. HDR also recommends including full pile tests (compression and tension) within the critical areas of the connection between the conveyance channel and the basin as well as at the highway and railway crossings. In addition, test fills with wick drains and settlement monitoring should be constructed. These two site-specific testing programs will provide designers with site-specific data that will be used in full design to save project budgets and schedules.

10 Limitations

This report presents the findings, conclusions, and recommendations from HDR's geotechnical engineering evaluation of the MBSD project at the 30% design level stage. It has been prepared in accordance with generally accepted engineering practice and in a manner consistent with the level of care and skill for this type of project within this geographical area. No warranty, expressed or implied, is made.

The conclusions and recommendations presented herein are based on research and available literature, the results of field exploration and laboratory materials testing by others, and the results of preliminary engineering analyses.

Geotechnical engineering and the geologic sciences are characterized by uncertainty. Professional judgments presented herein are based partly on our understanding of the proposed construction, partly on our general experience, and on the state-of-the-practice at the time of this evaluation.

11 References

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Figures