Appendix D. Geotechnical Considerations

Geofoam or Lightweight Fill Memorandum

To	Bob Beduhn, Mark Stanley		
From	Garrett Harris, PE, GE		
CC	Neil McLellan		
Date	May 7, 2014	Job No.	BA 153-01

Introduction

This memorandum presents the option to use geofoam and or lightweight materials as fill on the Mid-Barataria Sediment Diversion. As discussed in detail in HDR's 2013 geotechnical report, the project consists of a guide channel with gate structures at the Mississippi River end and at the Mid-Barataria basin end of the system, which is underlain by relatively young, soft, weak strength soils. As discussed in a memorandum dated January 14, 2014, settlements are anticipated to be on the order of 2 to 7 feet as a result of the compressibility of the on-site soil conditions. Large magnitudes of settlements, such as those calculated for this project, result in long-term maintenance of the facility associated with adding new fills, longer construction schedules, and more complicated systems such as wick drains and specialty equipment. With the intent of minimizing costs associated with settlements due to consolidation, alternative fills are discussed herein.

Settlements along the guide channel embankments are the main focus of this memorandum but may be applicable in other scenarios such as fill associated with the proposed Louisiana State Route 23 abutments, rail crossings, etc. Currently, civil drawings dated February 2014 illustrate that the guide channel embankments will be configured to consist of 4.5:1 slopes, long term crown elevation of +13 feet, and a crown width of 15 feet to accommodate vehicle access. Current recommendations for compacted earthen embankments are a result of the compressibility and relatively low strength of the foundation materials. As mentioned above, settlements between 2 and 7 vertical feet have been calculated for earthen embankment construction. As a means of construction and to facilitate a reasonable construction schedule, it will be necessary to place wick drains; place soils in stages so the soils can settle and the foundation materials can gain sufficient strength prior to the placement of the next stage; and overbuild the embankment to maintain the design crown elevation of +13 feet. A detailed discussion of this process is presented in HDR's January 14, 2014 memorandum.

Geofoam is expanded polystyrene (EPS) material with unit weights ranging from 0.7 to 3 pounds per cubic foot (pcf). Geofoam has an elastic modulus between 220 and 1860 pounds per square inch (psi) with an associated compressive resistance of 320 to 2680 pounds per square foot (psf) at 1 percent deformation. Geofoam is manufactured in typical dimensions of:

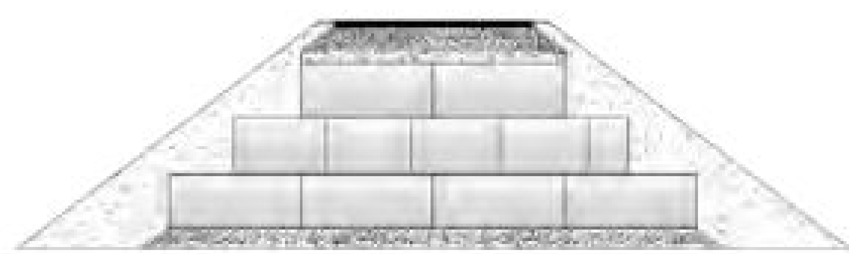
- 4-foot widths
- 8 to 16-foot lengths
- 1 to 36-inch thickness

Geofoam manufactured in accordance with ASTM D6817 is resistant to mechanical alterations when exposed to water moisture. However, petroleum-derived fluids such as gasoline and solvents have been known to degrade or “attack” geofoams. Where there is a potential for geofoam exposure to attack, Gasoline Resistant Geomembrane (GRG) products are recommended. This would be suitable for use in the vicinity of and including in the abutments for the Louisiana State Route 23 overpass. Detail geofoam technical information is presented at the end of this memorandum.

A comparative settlement analysis between the initial, more traditional engineered fill as presented in the January 14 memorandum and geofoam was performed. Earthen fill was designed to have a unit weight of 125 pcf, assuming that onsite fill was to be reused in the core of the embankment. For comparison, geofoam EPS 46 with a unit weight of 3 pcf was selected. In order to fully develop a loading and settlement comparison, it was necessary to consider other geofoam characteristics.

- Geofoam wants to float when submerged. As a result, it is necessary to counteract buoyancy forces. EPS 46 has a buoyancy force of 59.5 pcf. For ease of calculations and to be conservative, a buoyant force of 60 pcf was used. In order to provide an appropriate factor of safety against buoyancy forces a ratio of 1.5 feet of soil fill cover above 1 foot of geofoam was developed.
- Friction between geofoam blocks and other surfaces is relatively small. Therefore, good block to block connections, staggered block configuration and block-to-foundation interface is important. Therefore, the use of gripper plates or “staples” in conjunction with intertwining geofabric (Mirafi 600x or equivalent) are critical to create a more homogeneous core. An example of a typical cross section is shown below in Figure 1.

Figure 1. Typical cross section of geofoam core embankment with compacted fill cover



Source: reproduced from FoamControl Technical Data Sheet, 2014

Using the above characteristics, use of geofoam will provide a foundation pressure of approximately 650 psf per foot length of embankment. Total settlements associated with this configuration have been calculated to be between 0.4 and 1.7 feet along the length of the project. Table 1 provides the settlements per reach. Furthermore, this configuration has been calculated to have a resistance to sliding of approximately 3,125 psf. When laterally loaded by water to elevation +10 feet, the factor of safety against sliding is above 7.

By comparison, compacted earthen embankment has foundation pressure of approximately 2,625 psf per foot length of embankment. As mentioned initially, total settlements associated with this configuration are estimated between 1.4 to 7 feet along the length of the project. Table 1 provides the settlements per reach.

This configuration is also significantly above a factor of safety of 7 against sliding when channel water elevations are at +10 feet.

Table 1. Summary of calculated primary settlement, by model location

Modeled Location		Ground Surface Elevation (feet)	Required Levee Total Soil Fill (feet)	Ultimate Primary Settlement Soil Fill (feet)	Required Levee Total Geofoam Fill (feet)	Ultimate Primary Settlement Geofoam Fill (feet)
Reach	Station					
1	30+00	+3.0	10	1.40 (3.0)[a]	10	0.4
2	55+00	+2.0	13	1.6	11	0.4
3	67+00	+0.0	15	1.8	13	0.5
4	82+00	+0.0	16.5	2	13	0.5
5	110+00	-4.0	21	7	17	1.7

[a](value) - Settlement with groundwater drawn down to El -50 feet.

Note: Guide levee fill was modeled with 4.5:1 slopes and a 15-foot-wide crown width.

Seepage

Seepage, both through and under the embankment, will most likely occur as the embankment cover weathers and the structure settles. Therefore, it is important that an appropriate soil fill cover be constructed, and that foundation contact be well established. Since elevation of the water surface in the channel is not intended to be at elevation +10 feet for extended periods of time, saturation and through seepage are considered negligible and should be accommodated in a maintenance program. However, monitoring wells should be installed at intervals along the crown of the embankment to monitor pore water pressure build up within the core. This scenario is a result of waterside weathering and cracking and landside slopes containment of water in the core. Water confinement may develop uplift pressures behind the landside slope face, causing sloughing or sliding. There are design implementations such as weep holes that allow for drainage that may be considered as the design progresses.

Removal of Ground Improvement Zone Memorandum

To	Bob Beduhn		
From	Mark Stanley, PE, GE		
CC	Neil McLellan		
Date	May 28, 2014	Job No.	BA 153-01

Purpose

This memorandum addresses the value engineering (VE) alternative of removing the proposed channel ground improvement zone for the length of the channel as a cost savings measures. Associated guide levee and conveyance channel geometry changes are also evaluated.

Background

Initial HEC-RAS hydraulic analysis indicated that the channel geometry should be as narrow as possible to maintain relatively high water velocities to maintain sediments suspension across the polder. A channel side slope of 4.5:1 (horizontal:vertical) and a channel bottom elevation of El. -23 feet (North American Vertical Datum, NAVD) was determined to be an optimized channel section with respect to sediment transport. Geotechnical stability analysis found that the foundation soils in their current conditions could not provide adequate slope stability factors of safety with the guide levee located at the top of slope (the original conveyance concept design for MBSD conveyance channel). However, it was determined that channel geometry with a factors of safety of 1.5 or greater could be achieved by improving the soils by installing wick drains, placing a soils surcharge to consolidate and strengthen the foundation soils, and setting the guide levees back at least 80 feet from the top of the excavated channel slope. The approach to soil improvement design consists of installing a 200-foot wide improvement zone centered on the proposed conveyance channel toe of slope. Wick drains would be advanced within the improvement zone at 6 to 8-foot triangular spacing to a depth of 50 to 55 feet and a soil surcharge would be placed for a period of 1 to 2 years.

More detailed hydraulic analysis has since been conducted and indicated that the location of the guide levees is not as critical to the sediment transport as originally shown. Specifically, the guide levees could be set back a greater distance from the top of excavation with limited loss in performance. Setting back the guide levees could potentially remove the need for the conveyance channel soil improvement zone.

Current Configuration

The currently proposed guide levee geometry is similar to that proposed by the USACE for the New Orleans to Venice non-federal levee (NFL) that consists of a central core levee with stability berms constructed on both the flood and protected side of the levee. The levee section has 15-foot wide crown width and 5:1 side slopes. The stability berms are at least 5 feet thick at the levee toe and are 80 to 100 feet wide. The top of the stability berms are sloped at 40:1 away from the levee.

The current conveyance geometry is as follows:

- Channel bottom set at elevation -25 feet with 300-foot bottom width

- 4.5:1 side slopes
- Water level within the channel is not lower than 1 foot below existing grade
- Side slopes daylight at elevation +2.0 (toe of guide levee stability berm)
- 80-foot wide stability berm (surface inclined at 40:1)
- Core levee with a crown elevation of +13 feet
- Polder side stability berm
- Longitudinal drainage ditch at the toe of the polder side stability berm.

Approach

The approach to assessing the VE alternative was to:

1. Confirm the channel slope could be excavated with a 4.5:1 slope inclination in the wet with no ground improvement and no guide levee construction.
2. Determine the minimum setback required to decouple the guide levee stability from the conveyance channel cut stability.
3. Determine additional measures required to provide guide levee stability limiting phased construction.

As a modification to the original channel/guide levee geometry, the drainage ditch that was originally designed to run along the polder side stability berm toe was relocated to follow the limits of the project right-of-way. At this location, the drainage ditches are now offset from the levee and are not incorporated into this analysis.

For this analysis, the same three original site models developed to represent the variation of foundation conditions across the site were re-evaluated after replacing the "improved" undrained strength profile with the strength profiles developed for existing conditions. The strength profiles developed for this analysis are presented in HDR's report entitled "30% Geotechnical Investigation Report," dated July 2014. The results of this analysis are discussed below.

Slope stability analysis was conducted to first assess the feasibility of excavating the conveyance channel in the wet with no guide levee. Once the feasibility of excavating a stable channel slope was confirmed, the guide levee stability was modeled independently of the channel excavation. The findings of those analyses were then used to develop a minimum setback distance where the guide levee construction would not adversely impact conveyance channel stability.

It was assumed that without improving the in-situ undrained strength, the use of high strength geotextile fabric (HTGF), as is being proposed by the USACE for the NFL levee, would likely be necessary to allow the construction of the guide levee to the initial crown elevation. Table 1 summarizes the tensile strength and extents of the HTGF assumed in the evaluation of guide levee stability.

Table 1. Summary of the use of high strength geotextile fabrics in stability analyses

Channel Reach	Model Station	Geotextile Strength	Cover at Extents	Width of Geotextile
Station 45+00 to 70+00	Station 55+00	10,000#	11 feet	105 feet
Station 70+00 to 93+00	Station 82+00	10,000#	10 feet	184 feet
Station 93+00 to 140+00	Station 130+00	20,000#	10 feet	190 feet

Findings

Table 2 presents the results of the channel excavation stability analysis assuming no ground improvement and the channel is excavated in the wet. The horizontal distance behind the top of excavated slope where critical failure surface intersects the ground surface was recorded for each model. This analysis found that the global stability of the excavated channel slope with no surcharge load has a calculated factor of safety of slightly above 2, which is judged to be appropriate for unimproved slopes that may be subject to long term slope creep.

Table 3 presents the results of the guide levee stability analysis modeled independently of the excavation. The calculated horizontal distance measured from the stability berm toe to the location where the failure surface intersects the existing grade outside of the levee footprint was recorded for each model.

For the portion of levee between stations 45+00 to 93+00, a 10,000 pound HTGF was included in the stability model. The width of the HTGF was limited to 105 feet between stations 45+00 and 70+00, the approximate bottom width of the core levee. Between stations 70+00 and 93+00, the width of the HTGF was limited to 184 feet, extending equidistant into both stability berms and maintaining a minimum of 10 feet of soil cover. The stability analysis suggests that the HTGF may not be required between stations 45+00 to 70+00, since the failure surface did not intersect the HTGF. However, the HTGF was engaged in the model between Stations 70+00 and 93+00. Due to the high variability of subsurface conditions and presence of abandoned distributary deposits between Stations 45+00 to 93+00, we judge that the 10,000 pound HTGF be included in the preliminary design. The high factors of safety reported in Table 3 are due to the presence of the stability berms and sensitivity analysis should be conducted during final design to optimize the width of the stability berm.

For the portion of levee between stations 93+00 and 140+00, a much stronger HTGF was required to provide a minimum factor of safety of 1.4 or greater. The 20,000 pound HTGF was modeled as being 190 feet wide, extending equidistant into both stability berms and maintaining a minimum of 10 feet of soil cover. The width and/or strength of the HTGF may be reduced once the levee construction sequence is defined and an evaluation of foundation strength gain over time can be more accurately estimated.

Table 4 presents calculated overlap distance which is the horizontal distance measured from the top of cutslope to the toe of the guide levee stability berm where the two failure surface overlap. At this point a combined failure surface could develop resulting in a large slope failure into the excavated conveyance channel with a lower factor of safety. A minimum separation distance of 50 feet has been selected to provide the recommended minimum levee setback distance listed in Table 4.

Table 2. Summary of channel excavation stability analysis

Channel Reach	Model Station	Factor of Safety	Location of Failure[a]
Station 45+00 to 70+00	Station 55+00	2.16	42 feet
Station 70+00 to 93+00	Station 82+00	2.07	25 feet
Station 93+00 to 140+00	Station 130+00	2.07	27 feet

[a] Location refers to the distance behind the top of excavated slope where critical failure surface interests the ground surface

Table 3. Summary of guide levee stability

Channel Reach	Model Station	Geotextile Strength	Factor of Safety	Location of Failure[a]
Station 45+00 to 70+00	Station 55+00	10,000#[b]	3.83	60 feet
Station 70+00 to 93+00	Station 82+00	10,000#	3.30	60 feet
Station 93+00 to 140+00	Station 130+00	20,000#	1.63	43.5 Feet

[a] Horizontal distance of critical failure surface where it intersects existing grade measured from toe of stability berm

[b] Failure surface does not engage high strength geotextile fabric

Table 4. Summary of setback distances

Channel Reach	Model Station	Overlap Distance[a]	Recommended Minimum Setback Distance[b]
Station 45+00 to 70+00	Station 55+00	102 feet	152 feet
Station 70+00 to 93+00	Station 82+00	85 feet	135 feet
Station 93+00 to 140+00	Station 130+00	70 feet	120 feet

[a] Overlap distance equal the sum of location distance from tables 1 and 2 equal to the distance where the guide levee and excavated slope failure surfaces will just begin to overlap.

[b] Recommended minimum setback distance equal to the overlap distance plus a 50 feet separation.

Discussion

Stability analysis presented herein suggest that removal of the conveyance channel slope ground improvement zone and setting back the guide levees (core levee with stability berms) between 120 feet to 150 feet from the excavated channel top of slope is a viable alternative and should provide a cost savings. Stage construction techniques could also potentially be implemented to allow placement of the guide levee stability berms first, to allow the foundation soils to consolidate and strengthen under imposed loads prior to constructing the core levee to reduce the guide levee setback. However, this is beyond the scope of this VE analysis and could be evaluated during final design.

This analysis also found that the high strength fabrics at the base of the levee were required to be able to initially build the levee a crown elevation of +13 feet.