PEORIA RIVERFRONT DEVELOPMENT
(ECOSYSTEM RESTORATION) STUDY, ILLINOIS
FEASIBILITY REPORT WITH INTEGRATEDENVIRONMENTAL ASSESSMENT

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APPENDIX C
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PURPOSE

The study purpose is to evaluate the Federal and State interest in enhancing aquatic habitat and reducing sediment delivery and deposition within the Peoria Lakes. This appendix presents site geology and specific geotechnical analyses relevant to the study. To support the preparation of this appendix, Rock Island District, Engineering Division, Geotechnical Branch personnel reviewed literature, obtained soil borings, performed laboratory analysis and interpretation, and provided geotechnical analyses and recommendations.

LOCATION

The project study area is located in Peoria Lakes between Illinois River Mile 181.0 near Chillicothe, Illinois, and Peoria Lock and Dam at approximate River Mile 158.0. An additional area of study is located upland of Peoria Lakes on agricultural land immediately east of the city of Washington, Illinois. Refer to the main report for detailed study area locations.

PROJECT FEATURES

The proposed project features include the following:

Peoria Riverfront: Dredging and island construction upstream and downstream of McClugage Bridge.

Farm Creek: Construction of two sedimentation ponds.

PHYSIOGRAPHY

In central Illinois, glacial features are the major landforms of the flat topography of the lower Illinois River basin, generally that area below the city of Hennepin. Geologic evidence indicates that four glacial advances have occurred over what is now the State of Illinois. In order of occurrence, they were the Nebraskan, Kansan, Illinoian, and Wisconsinan glacial epochs. The two most recent glacial advances, the Illinoian and the Wisconsinan, are largely responsible for the uniform flatness that now characterizes much of the state. The basin is located in the Till Plains section of the Central Lowland physiographic province. The Galesburg Plain, Springfield Plain, and Bloomington Ridged Plain are subsections within the Till Plain. Peoria lies along the southwestern edge of the Bloomington Ridge Plain. This Plain includes the Wisconsinan glacial moraines and associated glacial topography. The altitude of land surface in the basin is generally from 600 to 800 feet above sea level. The area of greatest topographic relief is along the river valley, where elevation changes can range from 200 to as much as 400 feet near the confluence of the Illinois and Mississippi Rivers. The majority of the rest of the basin is extremely flat with less than 20 feet of relief. The distinctions in topography between the older Illinoian drift (Galesburg and Springfield Plains) and the Wisconsinan drift (Bloomington Ridged Plain) are the morainic ridges in the Wisconsinan drift areas. The morainic ridges are generally from 50 to 100 feet high, 1 to 2 miles wide, and 100 to 500 miles long. The moraines are separated by areas with more subdued, undulating or "rolling" topography. The type of bedrock divides the pre-glacial physiographic provinces. Subsequent glacial deposits also are controlled by bedrock lithology and structure. The Illinois Basin is underlain by Pennsylvanian age deposits of carbonates, sandstones and shales with interbedded coals. In the areas where the Pennsylvanian Upland and Lowland are covered by the Illinoian drift, the surface topography reflects bedrock surface. The area covered by Wisconsinan drift reflects glacial depositional features. The various glacial advances were responsible for numerous drainage changes, which can be related in part to features of the bedrock topography. The lowlands are areas where the bedrock surface has been eroded below the surrounding area. The broad Havana Lowland was developed at the junction of three important ancient drainage ways, one of which was the ancient Illinois Valley where the project site lies. This pre-glacial valley was carved by the ancient Mississippi River before advancing glaciers to its more western present day course diverted it. This accounts for a valley that is too large for the present day river. This, in turn, results in many bottomland lakes along the lower valley.

GEOLOGY

Many geologic processes have shaped the drainage pattern of the Illinois basin. The bedrock distribution and topography affected subsequent glacial depositional processes, and glacial processes have strongly affected the hydrology of the basin. For example, at the beginning of the Pleistocene Epoch (approximately 1.7 million years ago, or 1.7 MYBP), the rivers and streams in Illinois were not deeply entrenched in bedrock, but Pleistocene glaciation diverted the Mississippi River to its present position and scoured the bedrock surface. In the Peoria area, the present Illinois River occupies the valley of the Ancient Mississippi River, and above Peoria the Illinois River became established in its present position during the later Wisconsinan glaciation.

BEDROCK GEOLOGY

The uppermost bedrock is mostly carbonate rock of Mississippian (325-360 MYBP) and Pennsylvanian age (280-325 MYBP). Mississippian and Pennsylvanian bedrock are present throughout Illinois, but in most areas it is concealed by recent unconsolidated deposits up to 500 feet thick. Many of the Mississippian- and Pennsylvanian-aged formations are made of cyclic beds of sandstone, siltstone, shale, limestone, coal, and clay. These rocks contain 1% to 2% of coal by volume. There are 75 identified coal beds in Illinois. The Herrin (Number 6) Coal Member of the Pennsylvanian Carbondale Formation, mined at two active underground mines in Illinois, ranges from 200 feet below land surface in the northern and western part of the basin to 800 feet in Shelby County (southeastern part of the basin) and is mostly from 28 to 42 inches thick. Coal beds range from 0 to 150 feet thick. The Mississippian bedrock is mostly shale and limestone. The altitude of the bedrock surface changes as much as 600 feet across the basin. Erosion, prior to and during glaciation, from large rivers and tributaries cut two major bedrock valleys that dominate the bedrock topography—the Mahomet system from the east and the Ancient Mississippi system from the north. The pre-glacial Illinois Valley was part of the ancient Mississippi River. The Princeton Buried Bedrock Valley is a large bedrock valley that connects to the Illinois River Valley in the northern part of the area in and near Bureau County. The bedrock valleys are filled by overlying glacial drift. The time of formation of the bedrock valleys is not well defined. The glacial materials in these valleys are some of the most productive aquifers in the basin. The Illinois and Mackinaw Buried Bedrock Valleys underlie the present Illinois River.

GLACIAL GEOLOGY

About 15,000 years ago, the ice from the Wisconsinan glaciation covered most of the northern and east-central parts of the state. It is this glaciation that was responsible for the system of moraines in east-central and northeastern Illinois. The area occupied by the Wisconsinan ice sheet corresponds to what would later become the Grand Prairie. This was the first major expanse of grassland encountered by the settlers after leaving the heavily forested areas of the eastern states. About 12,000 years ago, the climate became warmer and the glaciers began to melt and retreat, forming very large glacial lakes; moraines near the present site of the City of Kankakee contained several of these. As the glaciers continued to melt, the water eventually cut through the moraines and cascaded down what is now the upper Illinois River Valley, resulting in a huge flood known today as the Kankakee Torrent. The waters of the Kankakee Torrent carried tremendous volumes of sand and gravel downstream to the "Big Bend" at Hennepin where the river channel is narrow and entrenched in bedrock. Below Hennepin, where the river valley widens, the water lost its velocity and the sand and gravel was deposited. Moraines are the dominant landform in the northeastern part of the area. The glacial deposits range from 50 to 500 feet thick and are thickest in buried bedrock valleys. The sands and gravels deposited by glacial streams are thick aquifers near the bottom of the valley. The stratigraphic relation of glacial deposits is complex because of different modes of deposition, multiple sources, and lack of regional continuity. Areal distribution of formations and members within formations is generalized because boundaries are inter-tongued and often indistinct. The Wedron Group is the most extensive till unit of Wisconsinan age in the basin, and the type section is largely till with numerous interbedded deposits of outwash gravel, sand, and silt. Other formations with type sections within the area include the Morton and Peoria Loess, which are silty and sandy formations of Wisconsinan age. Major deposits of loess were formed during Wisconsinan glaciations as silt and fine sand was blown from the bottomlands and deposited on adjacent bluffs and uplands. On the bluffs, southeast of the broad lowlands in the Havana area, loess accumulated to nearly 100 feet thick, but elsewhere it is not as thick.

SOILS

Parent materials, climate, plants and animals, topographic relief, and time determine the development of soils. The soils in the Illinois Basin developed mostly in thick loess with some thicknesses greater than 60 inches. Thinner loess (10-40 inches) soil is found in the northeastern part of the basin. Soils developed on sandy to clayey alluvial sediments are found near major streams. The humid, temperate climate of Illinois is conducive to the weathering of soil, formation of clay, and movement of leached chemical constituents downward in the soil profile. Tills commonly weather to depths of 5 feet and sometimes to a maximum depth of 15 feet along soil discontinuities. Most soils in the basin are mollisols, which are dark-colored soils formed under grass vegetation. Mollisols average more than 1% organic matter. The native prairie vegetation under which soils form contributed to the high accumulation of organic materials, which is valuable to agriculture because of the capacity to store water and nutrients. The areas that are not mollisols are along stream valleys where the light-colored alfisols formed under forest vegetation.

Soil type and distribution are two factors that affect the amount of soil erosion that results in sedimentation of lakes and reservoirs. Rates of soil erosion of up to 2% per year of farmland soil have been measured. Sedimentation from soil erosion is particularly serious in Illinois for three reasons: (1) The loess materials blanketing a large part of the state are highly erodible by water, even on the gently sloping land that covers most of the state; (2) under conventional tillage practices for corn and soybeans, the primary crops leave little residue on the surface for much of the year; and (3) rainfall in Illinois is fairly high in the spring when little vegetative cover is present on cropland. Variations in precipitation and temperature may occur in any year because the basin is far from large physical features, such as oceans or mountain ranges, that modify regional weather patterns. Precipitation is normally 35-38 in/yr. In 1993, many Midwestern states set records for annual precipitation, which resulted in massive flooding. Heavy, sustained precipitation in early spring through October contributed to the wettest year on record for Illinois with 50 inches as the state-wide average precipitation for the year. The previous record was 49.5 inches, which was set in 1927. Precipitation ranged from 50% to 70% above the long-term average across the state. The major sources of sediment are watershed, streambank, and bluff erosion, which result, in part, from agricultural land-use practices. Bluff erosion has produced major deltaic intrusions into the valley such that in some places the river has been pinched into a very narrow channel. Deltas produced by Ten Mile Creek and Farm Creek are two such examples of relatively small bluff streams producing sufficient sediment to virtually block the entire valley. These deltas are composed of the relatively coarser fraction of the till and loess source material and are thus mostly sands and gravels. These deltas are responsible for the formation of Upper and Lower Peoria Lakes.

SUBSURFACE EXPLORATION

Subsurface exploration was done to obtain foundation and borrow material samples for determination of their engineering characteristics. All subsurface exploration was done in accordance with Engineer Manual 1110-1-1804.

Personnel from the Rock Island District's Geotechnical Branch performed subsurface exploration during three different time periods as the project feature scope evolved. Six offshore borings were taken in July 2000, three offshore borings were taken in March 2001, and seven hand auger borings were taken on two separate days in May 2001. The hand augers were taken with a 4-inch Iwan auger. The offshore borings were taken with a CME-55 mounted aboard a Rock Island District work barge. These borings were advanced with a 3.5-inch hollow stem auger and a 2-inch split spoon sampler. The boring locations are shown on plate C-1 and the logs are shown on plates C-2 and C-3.

In the event that sand borrow is needed for this project, it would likely be taken from the Ten Mile Creek delta, which lies an average of 1,500 feet upstream of the northern end of the project (see PL-01-5 and PL-01-6). Hydraulic dredging of sands underlying the immediate project area is unlikely due to the amount of fine sediment overlying these sands in the area that was explored.

McClugage Bridge (U.S. 150) transects the Peoria Riverfront project area. Illinois Department of Transportation contract drillers performed the pier foundation exploration for McClugage Bridge. Locations and logs of these borings are shown on plates C-4 through C-6.

The exploration of proposed Peoria Riverfront dredging areas lying nearer to shore was not accomplished due to water depths of only 2 to 3 feet at "normal" river stages. The borings done for McClugage Bridge indicate that the thickness of the fine sediment lying on the surface may be less near the shoreline than that found in the borings taken near the navigation channel (PL-00-1 through 6 and PL-01-01 through 3). Additional subsurface exploration of these "near-shore" areas is anticipated during mid 2002.

LABORATORY TESTING

The results of the laboratory testing are listed with each individual boring log. All laboratory testing on samples taken from the borings located and shown on plates C-1 through C-3 was done in accordance with Engineer Manual 1110-2-1906.

PEORIA RIVERFRONT

Samples taken from borings PL-00-1 through 6 and PL-01-1 through 3 (Lower Peoria Lakes sediment) consisted largely of organic clays (CH-OH and CH). These borings were taken from the sediment surface to depths of approximately 25 feet. The sediment in this upper layer may be used to build sediment confinement embankment and it will form the immediate foundation of all sediment confinement structure options. Atterberg limit testing done on samples from these borings generally plotted very near the "A" line. The average of nine liquid limit and plastic limit tests done on these borings is shown below in Table C-1.

TABLE C-1 Peoria Riverfront Atterberg Limit Data				
Boring	**w(%)**	**LL**	**PL**	**PI**
PL-00-1	44	56	23	33
PL-00-2	73	72	33	39
PL-00-2	79	70	33	37
PL-00-3	78	72	32	40
PL-00-3	49	62	24	38
PL-00-5	73	69	31	38
PL-01-1	89	65	32	33
PL-01-1	54	71	31	40
PL-01-2	83	88	35	53
Averages	**69**	**69**	**30**	**39**

The average water content of all samples taken from these borings was 69% and the average coarse fraction was 11%. The soils were very soft when handled in the lab, and blow counts ranged between 0 and 3 throughout the depths of these borings.

Bucket samples taken from borings PL-01-1 and PL-01-2 were sent to an independent laboratory for column settling analyses. Results of these analyses will be used to estimate the various sediment confinement structure option capacities in the event that hydraulic dredging is chosen as the most cost-effective method to move fine sediment.

The borings taken for McClugage Bridge (plates C-5 and C-6) range in depth between 65 and 75 feet. The upper 70 feet generally consist of various sediment layers. The layers vary between soft, silty clays, organic silts, organic clays, silty sand, sand, and gravel. Soft organic clays and silts dominate the upper 25 feet, which is consistent with the soil characteristics found in borings PL-00-1 through 6 and PL-01-1 through 3. The lower 45-foot-deep layer consists of approximately 50% soft silty clays and clayey silts and 50% sands and gravels. Dense shale bedrock is generally found at a depth below 65 feet from the surface. The average moisture contents of the organic silts and clays found in the upper 25 feet of the McClugage Bridge borings are in general agreement with those derived from the more recent sampling done by Rock Island District personnel (69%). Atterberg limit testing was not done on the McClugage Bridge samples. Q_u testing done on samples taken from the upper 25 feet of these borings indicates an average unconfined compressive strength of 1,250 psf (ranging between 600 and 2,000 psf).

Samples taken from borings PL-01-4 through 6 (Farm Creek delta borrow and Ten Mile Creek delta borrow) were identified as coarse to fine sands (see Table C-2).

TABLE C-2 Grain Size Analysis of Sediment Samples Percent Finer by Weight				
	SAMPLE NUMBERS:	**PL-01-4**	**PL-01-5**	**PL-01-6**
	1 1/2"			
S	**3/4"**	100.0%		
I	**3/8"**	99.9%	100.0%	100.0%
E	**#4**	99.2%	99.6%	95.3%
V	**#10**	97.8%	98.5%	82.6%
E	**#16**	96.1%	98.1%	71.2%
	#30	90.5%	95.6%	54.1%
S	**#40**	79.6%	83.1%	42.8%
I	**#50**	42.8%	42.8%	27.9%
Z	**#70**	11.8%	12.8%	14.6%
E	**#100**	3.2%	3.4%	7.2%
S	**#200**	0.2%	0.7%	1.5%
	CLASSIFICATION:	SP, MEDIUM TO FINE SAND	SP, MEDIUM TO FINE SAND	SP, COARSE TO FINE SAND, TRACE GRAVEL

Grain size analyses done on samples from these borings indicate a D_{10} size ranging between 0.18 and 0.20 (see plates C-7 through C-9). Blow counts were not recorded in these borings due to the drilling method employed. However, at the time of sampling, these soils were observed to be of generally medium density.

FARM CREEK

Samples taken from borings PL-01-7 through 10 (Farm Creek sedimentation ponds) were identified as lean and fat clays (CL and CH). Water contents ranged between 24% and 37%, and were generally much closer to the plastic limit than the liquid limit. Blow counts were not recorded due to the drilling method employed. However, these soils were generally stiff in consistency.

STRATIGRAPHY/SITE CHARACTERIZATION

PEORIA RIVERFRONT

The borings taken in the Peoria Riverfront sediment indicate an average surface elevation of 438.0 IL State Plane Coordinate System (NAD 83). The upper 15 feet of sediment generally consists of soft clayey silts and silty clay (CH-OH and CH). Clay, silt, sand, and gravel layers of varying thickness comprise the underlying sediments to approximate elevation 370, where shale bedrock is encountered.

The calculated in situ saturated density and void ratio of the upper 25 feet of sediment, based on the average in situ water content, was 98 pounds per cubic foot (pcf) and 1.86, respectively.

Vane shear tests were taken on the sediments at the time borings PL-01-1 through 3 were taken. The vane shear testing was done on the sediments ranging in depth between 5 and 9 feet in accordance with procedures outlined in ASTM D2573 - Standard Method for Field Vane Shear Test in Cohesive Soil. The testing was done on the sediment at these depths since the project features will be excavated in, built on, and may be built using this surficial material. The test results are shown below in Table C-3, and indicate an average undrained strength of 219 pounds per square foot (psf). To provide a value of undisturbed shear strength comparable to that provided by laboratory tests on undisturbed samples, a correction factor can be applied as outlined by Duncan (page 58). Application of this factor results in a corrected undrained shear strength of 200 psf. For purposes of the sediment embankment stability, sediment foundation stability, and sediment foundation bearing capacity analyses performed in this appendix, the undrained shear strength of the Peoria Lakes sediments is taken as 200 psf. This is considerably lower than the unconfined strengths found for the samples taken from the upper 25 feet of the McClugage Bridge borings (1250 psf). However, the analyses for this appendix are done using the most conservative sediment characteristics.

TABLE C-3. Peoria Riverfront Vane Shear Data

ASTM D2573 S=3T/28pir^3

					Remolded
Boring	**Depth (ft)**	**T**	**Su (psf)**	**T**	**Su (psf)**
PL-01-01	5	250	153	175	107
PL-01-01	9	525	322	220	135
PL-01-02	5	230	141	110	68
PL-01-02	7.5	400	246	180	110
PL-01-03	5	300	184	150	92
PL-01-03	7.5	440	270	220	135
		AVG=	**219**	**AVG=**	**108**

Characteristic residual shear strength was obtained using Bovis' correlation between plasticity index and residual angle of internal friction shown in Duncan, page 89. The residual shear strength of the sediments is taken as 15 degrees.

The Illinois River water surface typically lies 2 feet above the top of the sediments, or elevation 440.0.

Additional discussion of soil strength relevant to specific project features will be provided below under the *Project Features Design* sections of this appendix.

Based on borings taken for McClugage Bridge, the sediments will be taken as 70 feet thick. For purposes of settlement design, the 70-foot-thick sediment layer will be considered to consist entirely of soft silty clay and doubly drained (due to the presence of porous layers throughout).

The sand borrow is characterized with a density of 120 pcf and an angle of internal friction of 34 degrees when placed as sediment confinement embankment.

FARM CREEK

The borings taken at the Farm Creek site indicated an average surface elevation of 743.5 IL State Plane Coordinate System (NAD 83).

The calculated in situ saturated density and void ratio of the foundation soil, based on the average in situ water content of 30.1%, was 118 pounds per cubic foot (pcf) and 0.78, respectively. The moistures of the upper lean clays did not vary significantly from those of the underlying fat clays.

Due to the exploration method and slow movement of water through cohesive soils, the exact groundwater elevation at this site is unknown. Groundwater was observed at approximately 5 feet below the boring surface elevations at the time the borings were taken. The levels were rising at a rate of approximately 1 inch per hour at the time the boring holes were filled. Groundwater levels are taken at 3 feet below ground surface for settlement calculations, and at ground surface for stability calculations and for characterization of the entire foundation as a saturated soil.

Based on the water content vs. undrained shear strength correlation developed for Rock Island District projects (see Figure C-1), foundation soil strength is taken as 800 psf for the underlying fat clays. The proposed sedimentation embankments will be built using the upper lean clays, and are assigned an undrained strength of 500 psf.

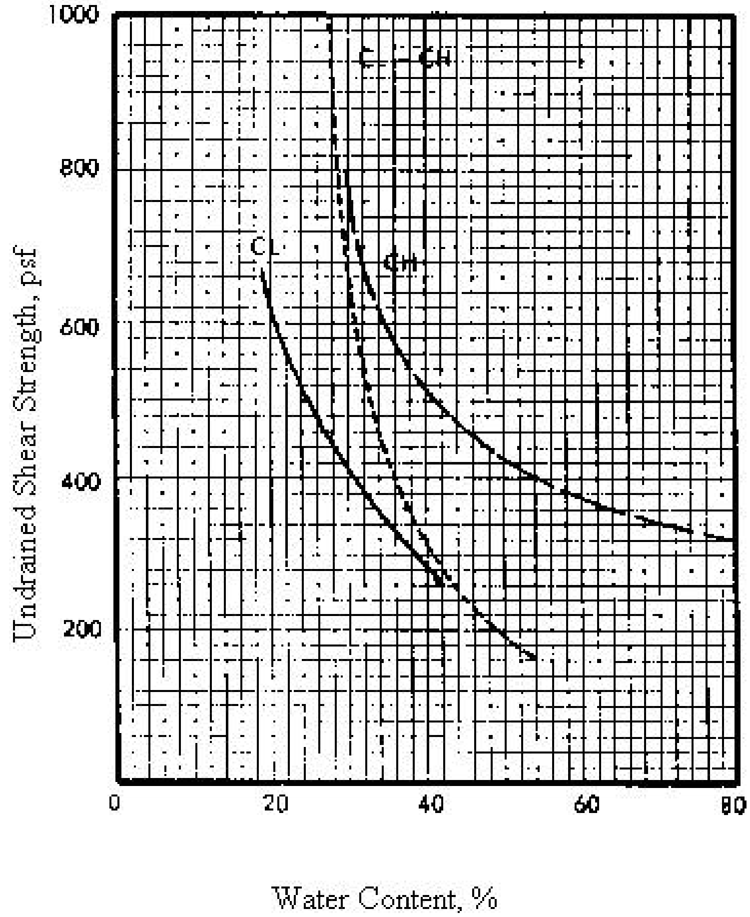

FIGURE C-1. Water Content Versus Undrained Shear Strength

The correlation presented in Figure C-1 represents a compilation of data from normally consolidated river valley clays. The in situ materials found at the Farm Creek project site are glacially deposited clays, and expected to exhibit higher undrained strengths.

Additional discussion of soil strength relevant to specific project features will be provided below under *Project Features Design* sections of this appendix.

PROJECT FEATURES DESIGN

PEORIA RIVERFRONT

General

Several alternatives were considered for construction of the project features at the Peoria Riverfront site. Deepwater habitat construction will be accomplished by a combination of mechanical, hydraulic, and/or "high solids" dredging techniques. Detailed drawings of proposed deepwater habitat are found in the main report. The sediments removed by dredging will be used to construct islands near the deepwater habitat sites.

The general scenario for construction of the islands will be to first construct sediment confinement structures. The sediment confinement structures will be built by: (1) placing mechanically dredged fine sediments to form a confinement embankment; (2) placing hydraulically dredged coarse sediments taken from nearby sand deltas to form a confinement embankment; and (3) placing geotextile containers filled with dredged sediments to form a sediment confinement structure. Detailed drawings of all sediment confinement options are found in the main report.

After construction of the sediment confinement structures, fine sediments will be placed within the confinement structures to form the islands. The purpose of the main report is to evaluate the relative benefits of varying the number and sizes of islands and the quantities of dredging and the habitat benefits gained with each scenario. The purpose of this appendix section is to evaluate the feasibility of deepwater habitat dredging and the various methods of sediment confinement structure and island construction.

<u>Deepwater Habitat</u>

General

As previously noted, deepwater habitat will be constructed by hydraulic, mechanical, and/or "high solids" dredging. If mechanically dredged fine sediment is the preferred alternative for construction of sediment confinement embankments, then the excavation of these sediments will create part of the deepwater habitat. Regardless of the method chosen to construct sediment confinement, deepwater habitat will be created by dredging of fine sediments into sediment confinement structure(s). The main geotechnical concerns for construction of deepwater habitat are the slope stability of these excavations in fine sediment and the proximity of the excavations to sediment confinement structures.

Slope Stability

The stratigraphy at the Peoria Riverfront site was characterized previously in this appendix. The fine sediment undrained shear strength was taken as 200 psf. Selection of undrained strength is considered appropriate for this application, since the sediment is normally consolidated. Normally, consolidated clays develop positive pore water pressures upon unloading, as opposed to over-consolidated clays, where the opposite is true. Increased pore water pressure induces a decrease in effective strength, and the undrained strength is the most critical for this analysis. An idealized dredge cut section was developed to determine stability using the UTexas4 slope stability package. The dredge-excavated slope was analyzed in accordance with Engineer Manual 1110-2-1902 and will not be subjected to pool fluctuation, seepage, or earthquake forces. The stability analysis resulted in a factor of safety against sliding for the 4H:1V cut slopes of 1.42. The UTexas4 program was run in the search mode, and numerous other surfaces were calculated, but only the model scenario depicted on plate C-10 and is considered relevant.

Non-uniform, or "stepped," dredge cuts and associated minor surface sloughing is expected to occur during dredging of the deepwater habitat. However, "stepped" dredge cuts are not expected to affect the overall stability of the deepwater habitat slopes.

The top of the excavated slope should be placed no closer than 30 feet from the outer toe of the sediment confinement structure(s) in order to avoid influence on both the confinement structure(s) and the dredge cut stability.

Sediment Confinement Structures

General

As discussed above, sediment confinement structures will be constructed by placement of mechanically dredged fine sediment, hydraulically dredged sand, or sediment-filled geotextile containers. The geotechnical feasibility of the alternatives is evaluated by addressing the stability, bearing capacity, settlement, and erosion protection characteristics of each alternative. Selection of the preferred sediment confinement structure alternative will be made based on overall cost and benefit analyses for each alternative.

Sediment Embankments

General. Sediment embankments will be constructed to heights of up to 12 feet by mechanical dredging fine sediment using large clamshell (7 cu yd or greater) excavation. Clamshell dredging is the best way to construct embankment with soft sediments in the river in order to maintain what little strength already exists in the soil. However, constructing embankment in the river with soft sediments and placing the material in the water is very difficult and not an ideal situation.

None of the embankment will be compacted by conventional construction techniques. Sediment embankment strengths were difficult to estimate due to the nature of the proposed placement method. Shear strengths may vary between residual and unconsolidated-undrained. The undrained sediment shear strengths described above in the *Stratigraphy/Site Characterization* section are considered applicable for sediment embankment design since generally undisturbed foundation soils would be used to build the embankment. Residual shear strengths for sediment embankment design were also considered, since the soil would be at least partially disturbed and remolded by the dredging operations, and failure could potentially occur between clamshell sediment placements.

Placement of sediment embankments by large clamshell has been accomplished at the Peoria Lake Enhancement project site, located approximately 13 miles upstream of the Peoria Riverfront site. The geotechnical design for the project is included in the *References* section of this appendix. It is considered useful here to discuss the design and construction of the Peoria Lake project as a model for the design of the Peoria Riverfront sediment embankments. The foundation sediments at the Peoria Lake site were characterized as a three-layer system using vane shear data—the upper 5-foot-thick layer ("fluff") assigned a 50 psf undrained strength, the second 15-foot-thick layer assigned a 320 psf undrained strength, and the foundation assigned a 600 psf undrained strength. The upper weak 5-foot-thick layer was excavated and sidecast prior to placement of the 12-foot-high embankment on the lower 320-psf material (the lower 320 psf material was also used to build the embankment). Construction of the majority of the 6H:1V-sloped embankment was completed successfully in three passes (contrary to original design, which recommended two passes based on bearing capacity). The contract also required a 30-day minimum wait between passes to enable consolidation and strength gain. The average liquid limit of the 320 psf material was 54%, and the average water content was 41%.

In contrast, the sediments at the Peoria Riverfront site are weaker (as previously characterized in this appendix). Peoria Riverfront sediment is uniformly characterized with average liquid limit of 69% and average water content very near the liquid limit.

Peoria Riverfront will be built by first removing the upper 2 to 3 feet of weak "fluff" type sediment from the deepwater habitat excavation areas and sidecasting it to a position approximately 20 feet from either side of the excavation. The underlying material will then be excavated from the deepwater habitat areas and used to build the sediment embankment. The embankment will be placed during two separate construction seasons, with at least 30 days between the two 3-foot-tall lifts planned for each season. The material that was sidecast at Peoria Lake was expected to erode. However, it has not eroded as badly as expected, and has served as wave protection for the embankment that was eventually built. The sidecast material at Peoria Riverfront is expected to behave in much the same way.

***Stability*.** The slope stability package UTexas4 was used to determine sliding factors of safety of the Peoria Riverfront sediment embankment and foundation. Two soil strength scenarios were modeled: (1) undrained strength of 200 psf for both the foundation and embankment, and (2) undrained strength of 200 psf for the foundation and residual strength of 15 degrees for the embankment. These strengths are consistent with those described in the *Stratigraphy/Site Characterization* section of this appendix. The UTexas4 program was run in the search mode and numerous failure surfaces were examined, but the model scenarios depicted on plates C-11 and C-12 are considered the most critical. These scenarios both resulted in a sliding factor of safety of 1.07. The tiered embankment cross section shown in the main report plates is expected to result in a more conservative safety factor.

The overall embankment strength can be preserved by various construction techniques. The contractor will not be allowed to "throw" the material from the clamshell, but must "place" the clamshell and then release the material in order to preserve the maximum strength of the sediment. The 3-foot-tall embankment lifts should be built starting at the inside of the embankment footprint and working outward. In this way the embankment load will be applied as uniformly as possible, and soft "fluff"-type soils will be progressively pushed outward from beneath the embankment. Instantaneous isolated embankment and shallow foundation failures can be expected due to the unpredictable nature of the sediment strength and placement method. Embankment and foundation soils should gain strength and greater stability with time as the cohesive soils are allowed to consolidate and drain.

As discussed previously, sediment embankments should be placed no closer than 30 feet from deepwater habitat dredge cuts to avoid stability failures both features.

***Bearing Capacity*.** The bearing capacity of the entire maximum 12-foot-high sediment embankment placed on the sediment foundation is approximately modeled by the previous stability analysis. In accordance with Engineer Manual 1110-2-1905, hand calculation of the 12-foot-high sediment embankment load spread over the embankment footprint results in a bearing capacity safety factor in excess of 1.5 for the embankment foundation. As the embankment is built, the upper 3 feet of soft material ("fluff") is expected to be pushed laterally from beneath the embankment to form a small mud wave on either side of the embankment. The "fluff" displacement will end once the lower portion of the embankment founds on competent foundation sediment (at least 200 psf unconfined strength). However, whether or not the lower portions of the sediment embankment can support the additional loading required to build the embankment to its maximum height requires additional analysis. Taking 100 psf as the sediment embankment strength (remolded), Table C-4 shows bearing capacity safety factors for sediment placed at different heights. It is seen that at heights of between 5 and 6 feet, bearing capacity failure can be expected within the lower portions of the embankment. Accordingly, it is recommended that the sediment embankment be built in two passes over two construction seasons. In this way, the lower one half of the embankment will have time to regain some of the strength it lost due to the placement process, to dewater, and to gain more strength due to consolidation.

TABLE C-4 Sediment Embankment Bearing Capacity

Depth (ft)	qallow	FS
1	98	5.24
2	196	2.62
3	294	1.75
4	392	1.31
5	490	1.05
6	588	0.87

cohesion (psf)=	100
qult=(5.14)*(cohesion)=	514

Settlement. The 70-foot-thick sediment layer was previously characterized as a homogeneous layer of soft silty clay for purposes of settling characteristics. The layer is modeled with double-drainage at 10-foot-thick increments since porous layers are found consistently throughout the entire layer. The settlement analysis was done in accordance with Engineer Manual 1110-1-1904, and is shown below in Table C-5. Table C-5 shows that 75% of the 4-foot total settlement will occur in less than a year under a 12-foot-high sediment embankment load. Since the soils in the sediment layer vary widely, some differential settlement can be expected. Also, since approximately 25% of the entire layer consists of porous material, a reasonable estimate of 12-foot-high embankment load settlement is 2 feet within the first year and an additional 1 foot during the next 12 to 15 years. In addition to embankment foundation settlement, the quantity of material required for sediment embankment construction should include consideration of the displaced 3-foot-thick "fluff" layer mentioned previously and desiccation and consolidation of the embankment itself. Sediment embankment desiccation and consolidation is estimated at 10%.

It should be noted that this analysis applies only to settlement beneath the sediment confinement structure embankment. The settlement and desiccation of hydraulically dredged material placed within the sediment confinement structure will be addressed separately under the *Sediment Confinement Structure Capacity* section of this appendix.

Settlement plates placed at various locations along and within the sediment confinement structures will be required as part of the construction contract so that short-term and overall foundation settlement can be monitored.

TABLE C-5

Peoria Riverfront Settlement Analysis

Hc(ft)	po(psf)	S(ft)	Stot(ft)	t (days)	St (ft)
10	177	1.54	1.54	199	1.16
20	531	0.83	2.37	199	1.78
30	885	0.55	2.92	199	2.19
40	1239	0.39	3.31	199	2.48
50	1593	0.30	3.60	199	2.70
60	1947	0.20	3.81	199	2.86
70	2301	0.16	3.97	199	2.98

z (ft)	a/z	b/z	I	2I	2Ip (psf)
5	14.40	1.00	0.5	1	1049
15	4.80	0.33	0.46	0.92	965
25	2.88	0.20	0.42	0.84	881
35	2.06	0.14	0.37	0.74	776
45	1.60	0.11	0.34	0.68	713
55	1.31	0.09	0.27	0.54	566
65	1.11	0.08	0.25	0.5	524

INPUT

				Ccavg=	0.518027743
U(%)=	75	Tv=	0.478	Cv (ft^2/day)=	0.06
e0=	1.82	LL (%)=	69	Cc=	0.518
wn (%)=	70	Gs=	2.6	PI (%)=	39
ht (ft)=	12	density (pcf)=	98	dp (psf)=	1049
a (ft)=	72	b (ft)=	5	h2o depth (ft)=	2

NOTES

vertical stress influence value calculation from NAVFAC, 1986, Design Manual 7.01, p.170)
initial void ratio and density calculated from insitu water content and SG (sat'd condition)
coefficient of consolidation estimated from LL correlation (NAVFAC Design Manual 7.01, p.144)
compression index taken as average of eight correlations (Das, 1995, Prin of Fdn Engr, p. 41)
pressure increase calculated from insitu density and confined disposal facility height
time/settlement calcs based on 1-D, single drainage (Das, 1995, Prin of Fdn Engr, pp. 42-50)
calculations checked by Sibte Zaidi, P.E., CEMVR-ED-G

Protection. Erosion protection will be required for the sediment embankments in select areas where wind- and vessel-generated waves are present. Whether or not the sidecast material placed on the sediment embankment foreshore is effective as wave erosion protection will be immediately apparent as construction of the sediment embankment proceeds. The construction contract can include an option for rock embankment protection if the need arises. Erosion due to river current is also expected, but current protection design is not addressed in this appendix (refer to the Hydraulics Appendix). The Automated Coastal Engineering System (ACES) design and analysis system was developed by the Coastal Engineering Research Center at the USACE Waterways Experiment Station. Its wave growth and rubble-mound revetment design applications were used to select a rock gradation and embankment slope able to resist the expected wave attack.

The Wind Adjustment and Wave Growth program calculated a wave height of 2.36 feet and a wave period of 2.86 seconds, as shown on plate C-13. Assumptions used in this analysis included a maximum observed wind speed of 80 mph, fetch depth of 8 feet, and a measured fetch length of 2 miles. Rock Island District's Geotechnical Branch personnel have previously experienced waves of this height at the project site. Using this wave height and period input, along with assumptions of 6H on 1V embankment slope, toe depth of 2.7 feet, and a low damage tolerance level of 2, the Rubble Mound Revetment Design program calculated a suggested 0.85-foot-thick riprap layer with a top size of approximately 100 lbs. This is shown on plate C-14 and represents the minimum acceptable protection requirement. Illinois Department of Transportation (ILDOT) Gradation #4 erosion protection stone is the most commonly produced protection stone having a gradation that approximates the ACES output (ILDOT Gradation #4 has top size of 150 pounds). Since commercial and recreational vessels are also expected to generate waves, especially where the sediment embankments lie close to the channel, ILDOT Gradation #5 (top size 400 pounds) is recommended for use as wave erosion protection for the sediment embankments. ILDOT Gradation #5 has been used successfully in numerous similar Rock Island District projects on both the Mississippi and Illinois Rivers. The size distribution of ILDOT Gradation #5 dictates placement of a 24-inch-thick layer to ensure competent protection. A layer of geotextile fabric will be placed between the 6H:1V embankment and stone protection to prevent migration of material in either direction.

Final selection of embankment riprap gradation and thickness will be based on cost/benefit analysis, as well as the evaluation of the effect of the sidecast material as a breakwater, degree of embankment stabilization through natural vegetation, and degree of embankment stabilization through consolidation.

A 3-foot-high stone breakwater placed approximately 15 feet offshore is an alternative to protection stone placed directly on the sediment embankment. This alternative would also be built using ILDOT Gradation #5, and may provide economic and/or additional habitat benefit. Geotextile fabric would also be placed beneath the breakwater.

Another erosion protection option is to allow the sediment embankment to vegetate naturally. This method may not be as reliable as stone protection, but is worth consideration. The purpose of the project is to establish deepwater habitat. Some degree of sediment containment structure erosion may be acceptable once the structure has served its purpose of containing sediment.

Stone protection incorporated into the design should be constructed of materials with a prolonged service life. The District's Quarry File database was screened for possible suppliers of riprap or other rock products. There are no quarries in the Peoria vicinity that can produce riprap from rock units of acceptable durability. The nearest acceptable quarries are in Logan, La Salle, Scott, and Pike Counties. Material from the last three counties may possibly be shipped by barge on the Illinois River.

Selection of the recommended erosion protection alternative will depend largely on cost/benefit considerations.

Sand Embankments

General. Sediment confinement structures built using hydraulically dredged sand are an alternative to those built with mechanically dredged sediment. The strength characteristics of hydraulically dredged sands are more predictable than the weaker fine sediments for use in embankment construction. However, the sand embankment would similarly be founded on weak in situ fine sediment, and the deepwater habitat benefit gained by using the fine sediment found in the project area for embankment construction would not be realized. The sand embankments will be built to heights of up to 12 feet. Sand sources for this type of construction are located nearby, at the mouths of Farm and Ten Mile Creeks.

Stability. The hydraulically dredged sand embankments that form the perimeter of the sediment confinement structure(s) will be built with 4H:1V interior slopes and 5H:1V exterior slopes. This configuration has proven effective in its resistance to both wave and through-seepage erosion in numerous Rock Island District applications. The slope stability package UTexas4 was used to determine sliding factors of safety for the sand embankment and foundation. Again, 200 psf undrained strength was taken as the foundation strength, and a friction angle of 34 degrees was taken as the sand embankment strength. The UTexas4 program was run in the search mode and numerous failure surfaces were examined, but the model scenario depicted on plate C-15 is considered to be the most critical. This scenario resulted in a sliding factor of safety of 1.265—slightly higher than that calculated for the sediment embankment.

Settlement. Although sediment confinement embankments built using sands will have slightly higher unit weights than those built with fine sediment, they will also be built on slightly steeper slopes. Manipulation of these variables shown in the Table C-5 spreadsheet resulted in slightly less settlement beneath sand embankments than beneath sediment embankments. It should again be noted that this analysis applies only to settlement beneath the sediment confinement structure embankment, and not the island interior.

In addition to embankment foundation settlement, the quantity of material required for sand embankment construction should include consideration of the displaced 3- to 4-foot-thick "fluff" layer mentioned previously.

The settlement and desiccation of hydraulically dredged material placed within the sediment confinement structure will be addressed separately under the *Sediment Confinement Structure Capacity* section of this appendix.

Protection. The ACES analysis discussed previously for the sediment embankments results in essentially the same wind- and vessel-generated wave protection requirements. ILDOT Gradation #5 protection stone and geotextile fabric will be used.

Rock groins are an alternative to protection stone placed directly on the sand embankment. Groins have proven to be an effective methodology for protection of sand embankments at the Pool 8 Islands project in U.S. Army Corps of Engineers

St. Paul District. The groins are essentially breakwaters placed perpendicular to, and extending 30 to 40 feet from, the sand embankment. They are typically spaced at five times their length. This alternative would also be built using ILDOT Gradation #5, and may provide economic and/or additional habitat benefit. Geotextile fabric would also be placed beneath the groins.

Vegetative erosion protection will not establish in sand as readily as in fine sediment, and is therefore not an option for sand embankment erosion protection.

Selection of the recommended erosion protection alternative will depend largely on cost/benefit considerations.

Geotextile Containers

General. The use of geotextile containers as a third alternative for sediment confinement is also considered. Geotextile containers have been similarly applied elsewhere as described by Fowler (1994) for shoreline protection, dredged material disposal containment, breakwaters, scour protection, sedimentation prevention, and river training. The containers proposed for Peoria Riverfront would be approximately 7 feet high and 19 feet wide. They would be built by placing either hydraulically dredged sand or "high solids"-dredged fines into each individual container, and each container would be sequentially connected to form a sediment confinement structure. The geotextile selection is made based on the desired height of the structure and the forces exerted by the material placed in the container.

After placement of the geotextile container as a sediment confinement structure, adjacent fine sediments can be mechanically dredged to raise the confinement to a maximum height of 12 feet, depending on the required hydraulic dredging storage requirements. Stacked geotextile containers are not considered here due to a variety of factors, including foundation instability, internal container instability, and geotextile container "rolling" resistance.

A geotextile container design typical to what is proposed for Peoria Riverfront is shown in Figure C-2.

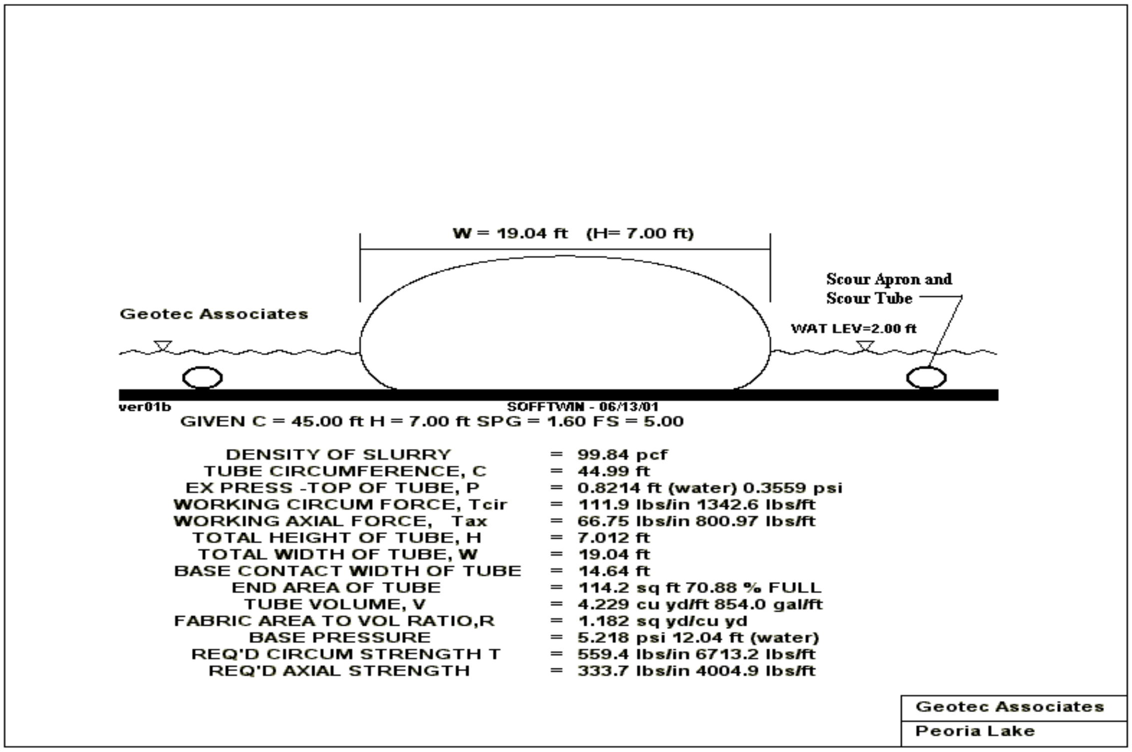

FIGURE C-2. Geotextile Container Design (Courtesy of Jack Fowler, Geotec Associates)

Bearing Capacity. As shown in Figure C-2, the base pressure (or allowable bearing pressure) of the geotextile container is 5.218 psi, or 751.4 psf. The ultimate bearing capacity of the sediment foundation is expressed as:

qult = cNc = (200)(5.14) = 1028 psf

The factor of safety for bearing failure of the geotextile container is expressed as:

FS = qult/qallow = 1028/751.4 = 1.37

Protection. The geotextile system selected to withstand the forces listed in Figure C-2 will be extremely durable. These types of systems have resisted waves, current, ultraviolet light attack, and vandalism for 10 or more years in a variety of harsh environments. The T. C. Mirafi-manufactured GC1000 *Geotube*, or its equivalent, is an appropriate product for this application. In the event that the geotextile containers will be covered by mechanically dredged sediment or otherwise protected, the less durable T. C. Mirafi GC500 *Geotube* (or equivalent) would be an appropriate choice. The selected geotextile container will be fabricated with a scour apron on both sides, as shown in Figure C-2. The scour apron will serve to protect the foundation from undermining due to wave and current erosion.

Sediment Confinement Structure Capacity

The outer perimeters of all sediment confinement structure alternatives were established by hydraulic analyses. However, the sediment confinement structures considered in this appendix will enclose areas of differing size since the structures have dissimilar cross-sectional geometry. Twelve-foot-high sediment confinement has been considered previously in this appendix due to the critical nature of this loading. However, a smaller confinement structure height may prove to be preferred due to cost/benefit analyses. Typical confinement design is accomplished by inputting a known volume of material to be dredged. The purpose of this study is to maximize the cost/benefit ratio. Therefore, the various confinement structure options are analyzed for final dredged material capacities (or quantity of deepwater habitat) so that a cost/benefit relationship can be established using this information.

Sediment confinement design is done using techniques described in EM 1110-2-5027, "Confined Disposal of Dredged Material." Hydraulically dredged sediment confinement design depends on many factors, including the previously described in situ sediment characteristics at Peoria Riverfront. The design methods described in EM 1110-2-5027, however, require additional input that is either unavailable or unknown at this time. Column settling analyses on bucket samples taken from borings in the project area were not complete at the time this appendix was written. Column settling test data are required to accurately design confinement options for the discrete, zone, and flocculent settling characteristics of the sediment. Also, at this feasibility stage of design, confinement height and dredging events and volumes (presumably contracted over the course of several years) have not been defined, and therefore cannot be incorporated into a final confinement design. The level and sequence of funding is unknown. Without this type of information, estimation of primary consolidation, secondary compression, and desiccation of sediment placed within the confinement during an undefined period of time is not practical. Detailed confinement design will be performed as final project scope and timing information is more specific.

A preliminary design factor of 1.10 will be used to describe the ratio of the volume dredged to the volume placed in confinement. This factor was obtained by weight/volume relationships assuming that the sediment will settle and desiccate to a final void ratio of 1.5 after it is hydraulically dredged into the confinement area. This factor is considered reasonably conservative and appropriate for this stage of design.

FARM CREEK

General

Construction of two sedimentation ponds is proposed for the Farm Creek project site. The pond locations are shown on plate C-1. Detailed drawings of the pond embankment options are found in the main report.

Sedimentation Ponds

General

The proposed sedimentation pond embankments will be approximately 600 feet long, a maximum of 12 feet high, and placed on 3H:1V slopes. They will be built with cohesive borrow material taken from the ponded areas shown on plate C-1. The borrow was characterized previously in this appendix, and will be taken from the surface to a maximum depth of three feet. Borrow locations and pond embankment footprints will be cleared and grubbed immediately prior to construction. Embankment footprints will be scarified immediately prior to placement of the first embankment lift. Embankments will be placed horizontally in lifts no greater than 8 inches. The lean clay borrow will be compacted by conventional methods to 90% of maximum density and within 3% optimum water content. The embankments will be seeded to prevent erosion. Stone protection is not considered necessary for the embankment slopes due to the small wind fetch and intermittent pool conditions. Effluent conduits will be placed through the embankments and protected as shown in the main report drawings.

Stability/Bearing Capacity

UTexas4 slope stability analysis was performed for a typical embankment slope and foundation as shown on plate C-15. Embankment and foundation soil strength was previously characterized in this appendix as 500 and 800 psf undrained strength, respectively. Circular searches were initiated passing through the foundation, and culminated with a critical failure surface located largely in the embankment (see plate C-16). The analysis resulted in a sliding factor of safety of 5.36. Hand calculation of bearing capacity resulted in a factor of safety of approximately 2.85.

Settlement

Settlement for the proposed embankments was done using the same methodology used for sediment embankments. A conservative 60-foot-thick, singly drained foundation layer was considered influenced by the embankment load. Approximately 2 feet of settlement can be expected in slightly less than 3 years following construction. The embankment should be overbuilt by 2 feet where foundation loading corresponds to a 12-foot-high maximum embankment and tapered to no additional height where embankment ties into natural ground surface. The analysis is shown in Table C-6.

TABLE C-6

Sediment Pond Settlement Analysis

Hc(ft)	po(psf)	S(ft)	Stot(ft)	t (days)	St (ft)
10	280	1.32	1.32	797	0.99
20	841	0.69	2.00	797	1.50
30	1402	0.45	2.45	797	1.84
40	1963	0.31	2.77	797	2.08
50	2524	0.24	3.01	797	2.25
60	3085	0.16	3.17	797	2.38

z (ft)	a/z	b/z	I	2I	2Ip (psf)
5	7.20	1.00	0.5	1	1422
15	2.40	0.33	0.46	0.92	1308
25	1.44	0.20	0.42	0.84	1194
35	1.03	0.14	0.37	0.74	1052
45	0.80	0.11	0.34	0.68	967
55	0.65	0.09	0.27	0.54	768

INPUT

				Ccavg=	0.299675714
U(%)=	75	Tv=	0.478	Cv (ft^2/day)=	0.06
e0=	0.78	LL (%)=	50	Cc=	0.300
wn (%)=	30	Gs=	2.6	PI (%)=	33
ht (ft)=	12	density (pcf)=	118	dp (psf)=	1422
a (ft)=	36	b (ft)=	5	h2o depth (ft)=	0

NOTES

vertical stress influence value calculation from NAVFAC, 1986, Design Manual 7.01, p.170)
initial void ratio and density calculated from insitu water content and SG (sat'd condition)
coefficient of consolidation estimated from LL correlation (NAVFAC Design Manual 7.01, p.144)
compression index taken as average of eight correlations (Das, 1995, Prin of Fdn Engr, p. 41)
pressure increase calculated from insitu density and confined disposal facility height
time/settlement calcs based on 1-D, single drainage (Das, 1995, Prin of Fdn Engr, pp. 42-50)

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