APPENDIX D

MARSH FILL AND CONTAINMENT DIKE

GEOTECHNICAL REPORT

REPORT OF

GEOTECHNICAL INVESTIGATION
PASS CHALAND TO GRAND BAYOU PASS
BARRIER SHORELINE RESTORATION
PROJECT BA-35
BAY JOE WISE AREA
PLAQUEMINES PARISH, LOUISIANA

FOR

**LOUISIANA DEPARTMENT OF
NATURAL RESOURCES
BATON ROUGE, LOUISIANA**

AND

**SJB GROUPS, INC.
ENGINEERS
BATON ROUGE, LOUISIANA**

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December 23, 2003

Louisiana Department of Natural Resources
c/o SJB Groups, Inc.
P. O. Box 1751
Baton Rouge, Louisiana 70821-1751

Attn: Mr. Stephen Punkay, P.E.

Re: Geotechnical Services
DNR Contract No. 2511-03-09
Engineering Assistance for Coastal Restoration Projects
Pass Chaland to Grand Bayou Pass
Bay Joe Wise Area
Project BA-35
Plaquemines Parish, Louisiana
STE File: 03-1033

Gentlemen:

This report includes the responses and revisions based on SJB and CEC's comments on our report submitted on November 11, 2003. Details are presented in the attached report.

Should you have any questions concerning this report, please contact this office. We appreciate the opportunity to serve you on this project, and look forward to working with you again in the future.

Sincerely,
Soil Testing Engineers, Inc.

Ricardo Abreu
for Dr. Gordon P. Boutwell, P.E.
Senior Consultant

Ching Nien Tsai, Ph.D., P.E.
Chief Engineer

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REPORT OF GEOTECHNICAL INVESTIGATION PASS CHALAND TO GRAND BAYOU PASS BARRIER SHORELINE RESTORATION PROJECT BA-35 BAY JOE WISE AREA PLAQUEMINES PARISH, LOUISIANA

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REPORT OF GEOTECHNICAL INVESTIGATION PASS CHALAND TO GRAND BAYOU PASS BARRIER SHORELINE RESTORATION PROJECT BA-35 BAY JOE WISE AREA PLAQUEMINES PARISH, LOUISIANA

EXECUTIVE SUMMARY

The principal findings of this investigation are summarized below for convenience. Details are contained in the main body of this report, plus its Appendix.

1. **Project.** This project consists of constructing a levee some 12000 feet long to contain and protect dredged fill in Bay Joe Wise.

2. **Scope of Work.** This consisted of drilling four borings in the construction area plus laboratory testing, and engineering analyses to determine stability of slopes, settlement amounts and rates, sedimentation times for the area fill, and cut/fill ratios.

3. **Subsurface Conditions.** All of the soils mentioned are of Holocene (Recent) origin. The project had four borings spaced some 3000 feet apart, so that variations can be expected. The average conditions encountered in the borings were:

- 0' - 8' Water
- 8' - 13' Very soft CLAY and SILTY CLAY
- 13' - 16' Loose SANDY SILT to SILTY SAND
- 16' - 24' Soft to medium SILTY CLAY
- 24' - 40' (+) Soft CLAY

4. **Slope Stability.** Levees with slopes of 1(V):4(H) have satisfactory stability for up to 2 feet of freeboard, and under all conditions analysed using slopes of 1(V):6(H). However, side slopes of 1(V):8(H) are recommended for levees having over 3 feet of freeboard.

5. **Levee Settlements.** Levee settlements due to their own weight are not enough to bring the levees below Elevation +1 foot NAVD unless they are built with a 1 foot or less initial freeboard. When geologic subsidence is included, levees with 3 feet or more of freeboard will not settle below Elevation +1 feet NAVD. For other cases, see Figures 11 through 16.

6. **Dewatering Time.** If the area fill consists of clays as encountered in the borings, some 6 to 8 weeks will be required for dewatering. The area fill may be relatively cohesionless sandy materials from other locations. For these soils, the majority will settle out of suspension in a few days.

7. Deposition Area Settlements. The settlements due to consolidation of the fill and subgrade soils amount typically to around 20-25% of the initial fill freeboard. These settlements will occur rapidly, with the majority occurring within 1 to 2 years after construction. Geologic subsidence adds about 0.5 feet over the 20-year life of the project. See Figures 11 through 16 for details.

8. Cut/Fill Ratios. These will depend on the type of fill material. Granular fills should produce 1.0 cubic yards of in-place fill for about 1.3 cubic yards of borrow. In the case of cohesive fill material, this increases to 1.5-1.8 cubic yards of borrow needed to obtain 1.0 cubic yards of in-place fill.

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REPORT OF GEOTECHNICAL INVESTIGATION PASS CHALAND TO GRAND BAYOU PASS BARRIER SHORELINE RESTORATION PROJECT BA-35 BAY JOE WISE AREA PLAQUEMINES PARISH, LOUISIANA

The findings of this investigation, together with the analyses and conclusions based on them, are discussed below. The field and laboratory investigations are described in Appendix A.

1.0 INTRODUCTION

1.1 Overall Descriptions. The Barataria Barrier Island Restoration Project covered by this investigation covers: the Bay Joe Wise area. It will consist of providing levees which will retain hydraulic fill used to build up the area to resist erosion and assist in dune and marsh creation. The general location of the project is illustrated on Figure G-1.

Information on land loss rates in this area was available from studies by Penland (2003). They indicate that the shoreline at Scofield Bayou in the project area averaged about 10 feet per year for the period 1884-1985, but increased to 21 feet per year for the period 1985-2002. Geologic subsidence in this area was furnished by LDNR for the Complex Project as about 6 inches in 20 years.

The objective of this project is to reduce erosion rates in the project area and create dune and marsh habitat. The project consists of construction of a levee in the Bay Joe Wise area and filling behind that levee system. Construction of the features is to be completed using a hydraulic dredge for material placement. The various project features include protection of barrier islands from an encroaching shoreline by reducing the rates of erosion and creating more land along the shoreline. Specifics of this project involve construction of earthen levees and filling the areas shoreward (north) of the earthen levees. At this time, the levees may be built over the existing dune line, but will probably be built just inland (north) of the existing beach ridge. The dredge borrow areas are located at Quatre Bayou area.

1.2 Scope of Work. STE's scope of work consisted of the following items:

- Drill four soil borings to depths of 40 feet below mudline.
- Perform laboratory tests to determine classification, strength, and compressibility characteristics for engineering analyses.
- Perform laboratory tests for particle size and organic content on samples submitted by the SJB Group and Coastal Environmental.
- Perform slope stability and settlement analyses for the proposed levees.
- Analyze settlements which will be caused by placement of the hydraulic fill.

1.3 Limitations. The analyses and recommendations presented in this report are based on the results of the investigation, and the furnished information as provided by SJB and the Louisiana Department of Natural Resources. While it is not too likely that the general conditions will differ greatly from those observed in the borings, it is always possible that variations can occur between or away from the widely spaced (over 3000 feet apart) borehole locations. If it becomes apparent during construction that subsurface conditions differing significantly from those discussed in Section 2 are being encountered, this office should be notified at once so that their effects can be determined and any remedial measures necessary prescribed. Also, should the nature of the project change considerably, these recommendations may have to be re-evaluated.

This report has been prepared for the exclusive use of the Department of Natural Resources and their consultants for the purpose of designing the proposed Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration Project (Bay Joe Wise area) as generally described in Section 1.1. The recommendations provided are site specific and are not intended for use at any other site.

1.4 Report Organization. The main body of this report is divided into four sections: this Introduction (Section 1.0), the Geologic and Soils descriptions (Section 2.0), the Engineering Analyses (Section 3.0), and supplemental testing on samples submitted by the SJB Group and Coastal Environmental and Engineering, Inc.(Section 4.0).

The field and laboratory programs are covered in Appendix A.

2.0 GEOLOGICAL AND SOIL CONDITIONS

2.1 Site and Geology Conditions. This site is bounded on the west by Bayou Huertes and on the east by Grand Bayou pass; see Figures 1 and 2. The surface is water (at about 0 feet NAVD), or, along the south side, a barrier beach (again, elevations undefined). Reference is made to Figure 2 and the sources cited on that figure. Even there, the subsurface geology is not well defined. A Barrier Beach some 300 feet wide is shown along the south (gulf) shore. Abandoned Courses some 500 feet wide follow old N-S bayous at each end of the project area near its center. A profile on the "Ft. Livingston" USA/COE geologic map passes about 2 miles inland from the site. It indicates Holocene marsh deposits to around elevation -200 feet NAVD, followed by Holocene-Pleistocene sand to around -320 feet NAVD, where Pleistocene-age clays begin. The marsh deposits frequently have sand bodies (relict beaches) in the -20 to -50 feet NAVD range. Barrier Beach sands typically extend to about -10 feet NAVD.

2.2 Soil Conditions. Four (4) borings were made to investigate the subsurface conditions in the Bay Joe Wise project area. All of these borings were given a "BJW" - number; their approximate locations are illustrated on Figure 3. These borings were taken along the then-proposed centerline of the levee. Global Positioning System (GPS) coordinates taken during drilling are shown on the individual logs of the borings in Appendix A. A Soil Profile is given on Figure 4. The average soil conditions at these four borings can be summarized as shown on Table 2.2-1, below.

TABLE 2.2-1
AVERAGE SOILS DATA

Depth* (feet)	Soil Type	LL (%)	PI (%)	W (%)	DD (%)	Su (ksf)	N (b/f)	Content (%) of Sand/Silt/Clay
0-8	Water	-	-	-	-	0	0	0/0/0
8-13	CL-CH	45	24	40	77	0.16	NT	NT
13-16	ML, SM	-	NP	33	82	NT	NT	77/13/10
16-24	CL	33	11	40	74	0.50	6	3/54/43
24-40+	CH	79	47	58	63	0.31	3	NT

* Below waterline
LL: Liquid Limit
PI: Plas. Index
W: Water Content
NP: Non-Plastic
DD: Dry Density
Su: Undrained Shear Strength
N: Standard Penetration Resistance
NT: Not Tested

It should be noted that the relatively cohesionless (ML, SM) stratum is not present in boring BJW-1, and its depth varies among the other borings. Consult Figure 4 and the individual logs of the borings in Appendix A.

2.4 Limitations. The description given above contains averages and is based on boreholes spaced some 3000 feet apart. The soils can be expected to vary between borehole locations. For details at a particular location consult the individual logs of the borings in Appendix A.

3.0 ENGINEERING ANALYSES

This section presents the methodologies used in the analyses, their results, and recommendations for geotechnical construction.

3.1 Assignments. The engineering assignments given to STE were covered generally in Paragraph 3.1.1 of SJB's "Geophysical Technical Memorandum" dated June 27, 2003, and mentioned again in SJB's letter to LDNR dated July 23, 2003. In summary, STE was requested to provide:

- Bearing Capacity and Slope Stability Analyses,
- Settlements and their rates,
- Foundation design criteria for the containment levees,
- Alternate to earthen levee system, and
- Fill Stability and longevity.

STE also secured certain beach and near-shore sediment samples for particle size analyses. Other samples taken by the SJB Group were also tested for particle size and organic content (See Section 4.0).

3.2 Levee Stability Analysis. This section presents the methodology used in the slope stability analyses for the levees, the cases analyzed, and the results. The levee may actually be the outer edge of the pumped fill.

3.2.1 Slope Stability Analysis - General. A slope has two types of forces acting on it. The soil weight and any seepage forces try to make the soil slide; these are called the "driving forces." The weight of soil below the waterline is its "effective" or, buoyant weight. Therefore, a foot of soil above water has 2 to 3 times the driving force of a foot of soil below water. The strength of the soil tries to keep it from sliding; this is called the "resisting force." Both depend on the geometry of the situation: the assumed "Failure Surface." The procedure is to mentally isolate a block of soil whose bottom is the trial "Failure Surface," and compute the resisting and driving forces. Their ratio is called the "safety factor," and is the measure of stability. In practice, one analyses many soil blocks until the block yielding the lowest safety factor is found. This is assumed to govern, and the safety factor for the slope is the lowest safety factor determined. The calculations for any but the simplest conditions are quite laborious. They are therefore now performed on a digital computer, using a proven code such as PCSTABL, XSTABL, UTEXAS3, etc. For this project, the slope stability analyses were performed using XSTABL marketed by Interactive Software Designs, Inc. This program evolved from PCSTABL by Purdue University. The program is capable of searching for the minimum safety factor with an easy to use interface. The Bishop method of analysis was used for this project. The accepted measure of a slope's stability is its "safety factor," as defined above. Typical acceptable safety factors common in practice are:

Low Water Condition:	1.3 - 1.5
Rapid Drawdown Condition:	1.0 - 1.1

The rapid drawdown case is not applicable for this project due to the nature of the tidal conditions at the proposed structures.

3.2.2 Cases Analysed. The borings (BJW-1 through BJW-4) were made along a contemplated route for the levee. However, the water depths at these borings ranged from 7 to 9 feet. This infers a relatively great *total* height of the levee; for instance, achieving a 2 foot freeboard requires a total height of some 9 to 11 feet. Stability for such heights on these low-strength soils will require quite flat side slopes, especially for levees with over 2 feet of freeboard. Therefore, analyses were made for varying water depths so that design can evaluate alternative locations and water depths. In all cases, the levee portion was assumed to have a crest width of 20 feet and the area fill to be placed on one side of the levee. The cases analysed are summarized below:

Initial Freeboard:	1, 2, 3, and 4 feet above water
Water Depths:	0, 2, 4, 6, 8, and 10 feet
Side Slopes:	1(V):6(H), 1(V):8(H), and 1(V):10(H)
Levee Material:	Silty Sand/Silt and Clay

3.2.3 Results. The results are summarized in Table 3.2-1, below. In each case, the minimum safety factor obtained for the two levee material types is assumed to govern and is presented in Table 3.2-1.

TABLE 3.2-1
LEVEE SAFETY FACTORS

Initial Freeboard (feet)	Water Depth (feet)	Safety Factor for Side Slope			
		1(V):4(H)	1(V):6(H)	1(V):8(H)	1(V):10(H)
1	0	4.6	4.7	4.7	4.8
	2	2.5	2.7	2.8	3.0
	4	1.9	2.4	2.6	2.8
	6	1.9	2.1	2.3	2.7
	8	1.7	2.0	2.3	2.4
	10	1.6	1.9	2.0	2.1
2	0	2.3	2.4	2.5	2.6
	2	1.6	1.8	2.0	2.1
	4	1.5	1.8	2.0	2.1
	6	1.5	1.7	1.9	2.1
	8	1.4	1.6	1.9	2.0
	10	1.4	1.6	1.7	1.8
3	0	1.5	1.6	1.7	1.8
	2	1.3	1.4	1.6	1.8
	4	**1.2**	1.5	1.6	1.7
	6	**1.2**	1.4	1.6	1.8
	8	**1.2**	1.4	1.6	1.7
	10	**1.2**	1.4	1.5	1.6
4	0	**1.2**	1.3	1.4	1.5
	2	**1.0**	1.2	1.4	1.5
	4	**1.1**	1.3	1.4	1.5
	6	**1.1**	1.3	1.4	1.6
	8	**1.1**	1.3	1.4	1.5
	10	**1.1**	1.3	1.3	1.4

Bold: SF less than desired 1.3

In some cases, the safety factor appears to increase with increasing water depth. This occurs because the deeper water will have a smaller thickness of the weakest soils beneath the levee.

3.2.4 Summary of Stability Results. Slopes of around 1(V):4(H) or flatter have adequate safety factors for levees with freeboards of 2 feet or less, and for levees with 3 feet of freeboard if the water depth is 2 feet or less. Slopes of 1(V):6(H) have adequate safety factors for all conditions analysed, but the 1(V):8(H) slopes are recommended for the levees with 4 feet of freeboard. Examples of the stability analyses illustrating the soil conditions and geometric configurations, plus the critical failure surfaces, are given on Figures 5 through 10. There may be some areas where the soils are somewhat weaker than those used for the analyses. Some sloughing may occur in such areas, but the soils should slough back only enough to be stable.

3.2.5 Bearing Capacity. While this soil characteristic was requested, it is not truly applicable to the case of these levees. This is because of the theoretical assumptions made in deriving the bearing capacity equations.

- Bearing Capacity assumes a zero strength for the material applying the load. This is far too conservative for the situation of a levee or fill edge.

- Bearing Capacity assumes a vertical side slope for the material applying the load. The actual stability depends on the side slopes, which will be on the order of 1(V):4(H) to 1(V):8(H), not the bearing capacity value of 1(V):0(H). Again, this assumption is far too conservative for the situation of a levee or fill edge.

As an example, consider the case of a levee having two feet of freeboard set in 4 feet of water. The applied bearing pressure is about 350 lb./sq.ft. The bearing capacity is as low as around 250-300 lb./sq./ft., i.e., the safety factor in "bearing capacity" is less than 1.0 (indicates failure). However, reference to Table 3.2-1 shows that this levee with a 1(V):8(H) slope has a real safety factor of around 2.0.

3.2.6 *Strengthening.* It is clear that designs with freeboards exceeding about 3 feet and those in water over 2 feet deep will have safety factors less than desired if 1(V):4(H) slopes are used. Geotextile reinforcement is indicated for such conditions. The geotextile should have an allowable tensile strength of at least 1,000 pounds per lineal foot (measured perpendicular to the levee centerline). It should extend under the full width of the levee. It is recommended that this office be contacted for details once the levee configuration (especially water depth and freeboard) is fixed.

3.2.7 *Levee Borrow.* It is often desirable to obtain the borrow material for the levees from near the levee. Excavating this material from too close to the levee toe can affect the stability of the levee adversely. It is therefore recommended that the edge of the borrow pit not be closer to the levee toe than about 30 feet plus twice the depth of the borrow excavation.

3.3 Levee Settlement Analyses. The assignments relative to levee settlements were given in Section 3.1. They require calculating both the total amounts of settlement which will occur after a very long time, and the time-rates at which these movements will occur. Levee settlement is composed of three parts:

- Settlement in the foundation soils due to the weight of the levee and adjacent area fill,
- Settlement within the levee itself due to self-weight consolidation (minor), and
- Geological Subsidence. This rate was furnished by LDNR as about 0.025 feet per year.

3.3.1 *Analyses - Total Amount of Settlement due to Levee and Fill Weight.* The total amount of settlement depends on the geometry and intensity of the applied load and on the compressibilities of the underlying soil strata. The area fill will extend up to the levees, and its effect on settlement of the levees was included. It must be noted that, as settlement progresses, the net intensity of the applied load decreases. This is especially true for levees built in water.

The maximum possible settlement is that calculated without taking this phenomenon into account, and forms the basis for calculations which do use load intensity decrease. Note that this decrease occurs if the levees are not periodically rebuilt to their initial elevations.

The actual settlement calculations were performed using the computer code VSTRESS, originally developed by the Corps of Engineers, and SETOFF as developed by Ensoft, Inc. These programs calculate one-dimensional settlement based on either Boussinesq or Westergaard stress distributions. The Boussinesq stress distribution was used for these analyses. Actual consolidation curves from this and adjacent projects were used in the calculations to evaluate material response to loading. Consolidation tests from complex project were also used for this project. The consolidation curves from that project are also attached to this report.

3.3.2 *Cases Analysed.* Although there were some differences between the soils at the four boreholes, they were all of relatively compressible Clays (CH, CL). The major variable here is the applied load, which depends on the total height (water depth plus freeboard) of the levee/fill system. The side slope was assumed as 1(V):8(H), to produce the maximum settlement. The cases analysed included:

- Levee Freeboards: 1, 2, 3, and 4 feet above water
- Water Depth: 0, 2, 4, 6, 8, and 10 feet
- Locations: Toe of Levee on the water side (Point "A"), crest of Levee (Point "B"), and 30 feet into the area fill (Point "C")

The soil conditions below the bottom levels of the boreholes (40 feet below water level) were assumed to be similar to those encountered in the nearby Pass Chaland and Pelican Headland area (Project BA-38).

3.3.3 *Effect of Settlement on Further Settlement.* These levees will probably be constructed in water. The levees are originally built to some level above water, and produce a stress level which includes the *total* weight of levee material above water. As settlement proceeds, some of the material which was originally above water becomes submerged. That material now exerts pressure due to its *buoyant* weight, which is less than its total weight. The net result is that the pressure decreases and the real settlement is less than would be predicted using the total weight. There are two other factors which must be considered:

- If the levee heights are rebuilt, settlement will tend to reach the "total weight" movements.

- At most locations, the levee will be underlain by some granular soils, which consolidate rapidly. The total movements due to the granular soils, and part of that from the other soils will probably occur during construction.

3.3.4 *Total Levee Settlements due to Levee/Fill Weight.* The starting point is the "raw" settlement computed using the total unit weight for above-water fill. For the observable post-construction settlements, these must be reduced by the movements occurring in the granular layers during construction, termed "adjusted" settlement. Then, the effect of the remaining long-term settlement on the applied pressure must be considered (See Section 3.3.3). This is termed the "net" settlement.

The results of these analyses are presented in Table 3.2-2, below. Point "A" refers to the toe of the levee (away from the fill side), Point "B" to the crest of the levee, and Point "C" to a location about 30 feet towards the fill from the levee crest. The weight effect of the adjacent area fill is included.

TABLE 3.2-2
LONG-TERM WEIGHT-INDUCED SETTLEMENTS

Geometry Situation		Settlement (feet) at								
		Point "A"			Point "B"			Point "C"		
Water (feet)	Freeboard (feet)	Raw	Adj.	Net	Raw	Adj.	Net	Raw	Adj.	Net
0	1	0.11	0.10	0.09	0.20	0.16	0.15	0.23	0.19	0.17
	2	0.18	0.16	0.15	0.41	0.35	0.31	0.44	0.37	0.32
	3	0.24	0.21	0.20	0.63	0.54	0.48	0.65	0.55	0.49
	4	0.28	0.25	0.24	0.85	0.73	0.65	0.86	0.74	0.66
2	1	0.11	0.10	0.09	0.35	0.30	0.28	0.31	0.27	0.25
	2	0.15	0.13	0.12	0.57	0.50	0.45	0.53	0.47	0.41
	3	0.18	0.16	0.15	0.79	0.69	0.62	0.74	0.65	0.58
	4	0.20	0.18	0.17	1.02	0.89	0.78	0.95	0.83	0.74
4	1	0.12	0.11	0.10	0.52	0.47	0.42	0.40	0.36	0.33
	2	0.14	0.12	0.11	0.74	0.65	0.59	0.62	0.55	0.49
	3	0.16	0.14	0.13	0.96	0.84	0.76	0.83	0.74	0.66
	4	0.17	0.15	0.14	1.19	1.05	0.92	1.03	0.92	0.82
6	1	0.12	0.11	0.10	0.70	0.61	0.55	0.49	0.44	0.41
	2	0.14	0.13	0.12	0.92	0.80	0.72	0.70	0.62	0.56
	3	0.15	0.14	0.13	1.14	1.00	0.88	0.91	0.81	0.73
	4	0.16	0.15	0.14	1.36	1.18	1.05	1.12	0.99	0.88
8	1	0.12	0.11	0.10	0.88	0.76	0.68	0.57	0.50	0.45
	2	0.13	0.12	0.11	1.10	0.96	0.86	0.78	0.69	0.62
	3	0.14	0.13	0.12	1.32	1.16	1.05	0.99	0.87	0.79
	4	0.14	0.13	0.12	1.53	1.38	1.22	1.20	1.08	0.97
10	1	0.12	0.11	0.10	1.06	0.93	0.83	0.65	0.58	0.52
	2	0.13	0.12	0.11	1.27	1.12	1.00	0.86	0.77	0.70
	3	0.13	0.12	0.11	1.49	1.31	1.17	1.07	0.95	0.86
	4	0.14	0.13	0.12	1.70	1.50	1.33	1.27	1.13	1.03

The "Raw" settlements at Point "C" are often less than those at Point "B" because the area fill should have a lighter unit weight than the levee fill, and thus produce less pressure.

In the above Table 3.2-2 the "Net" settlements should be used to evaluate long-term performance unless the levees are raised by additional fill at some time in the future. In the latter case, the "Adjusted" settlements would be more appropriate.

3.3.5 *Geological Subsidence.* The only other significant source of levee settlement will be that due to geological subsidence. This rate was furnished by LDNR as about 0.025 feet per year for the nearby Complex project. The same rate is assumed for this project. This is of little consequence for the first few years, but becomes significant over long periods. For example, the movement due to geological subsidence is estimated as 0.1 feet in the first 4 years, but increases to 0.5 feet at the end of the estimated project life at 20 years after construction.

3.3.6 *Analyses - Time Rate of Settlement due to Levee Weight.* The time-rate of settlement as observed at the ground surface depends on several factors, as discussed below:

- Soil Rate Parameter (c_v). This is intrinsic to each soil type, but varies with the total vertical pressure in the soil layer. In general, settlement within granular soils (ML, SC, SM) will occur virtually during construction. While a small amount of settlement will occur during construction, most of the settlement in the more cohesive soils (CH, CL, OH) proceeds at a much slower rate.

- Drainage Path Length (L). Consolidation is a process of squeezing water out of the soil voids. The water has to go somewhere, and that is to either the surface or a relatively permeable layer (such as a silt layer in a clay mass).

- Vertical Distribution of the Total Settlement. The time rate applies to each layer; the contribution of each layer is its own ultimate settlement multiplied by its degree of consolidation at a particular time.

Like other problems in time-dependent flow in soils, the analysis for the time-rate of consolidation is inherently inaccurate. Normally, settlement occurs faster than the prediction.

Calculations were made at the locations cited in Section 3.3.2. The results were normalized by dividing the "Net" settlements at various times by the ultimate (long-term) "Net" settlement. This approach accounts directly for the settlements which occur during construction. Settlement rates were analyzed for various levee heights and averaged. The percentages of settlement given in Table 3.2-3 should be applied to the "Net" total settlements given in Table 3.2-2.

TABLE 3.2-3
NET RATES OF WEIGHT-INDUCED SETTLEMENT

Time (years)[d]	Percentage of Weight-Induced Net Settlement Complete			
	Point A	Point B	Point C	Average
0.0	0	0	0	0
0.5	44	66	59	56
1.0	55	76	69	67
2.0	67	84	82	77
5.0	79	91	90	86
10.0	87	95	95	92
20.0	94	98	98	97

[d]: After construction is complete

3.3.7 Time to Reach Marsh Elevation. The average desired marsh elevation is assumed to be approximately +1.0 feet NAVD, equal to about one foot above water level at the site. It is assumed that the initial levee top elevation will be up to 4 feet above the site water level, or +4 feet NAVD. The "Net" settlement data from Section 3.3.4 (no rebuilding) and the time-rate information in Section 3.3.6 was analyzed. The analysis showed that none of the levees analysed will settle below Elevation 0 feet NAVD unless geologic subsidence is included. Levees with crest elevations of 2 feet or more will not settle below Elevation +1 foot NAVD only if the water depth exceeds 2 feet. The times to settle below +1.0 feet NAVD can be approximated by the following relationship:

$$t(yr) = 40\,[(FB\text{-}Net) - 1.0]$$

Where t = Required time (years)
FB = Initial Crest Elevation (feet)
Net = Net Long-term Settlement (feet) from Table 3.2-2

3.4 Suitability of Borrow Soil. Granular borrow is preferable if available. The soils encountered in the levee area borings were predominately fine-grained materials. However, they were mainly CH and CL soils; any Peat (PT) or Organic Clay (OH) materials should be used as area fill and excluded from the levees.

3.5 Dewatering Time for Area Fill. When soil particles are in suspensions with low concentration of solids, particles settle as individual entities, and there is no significant interaction with neighboring particles (Type I settling). With increasing solids concentration, the particles coalesce or flocculate. By coalescing, the particles increase in mass and settle at a faster rate (Type II settling). With further increase in concentration, the interparticle forces are sufficient to hinder the neighboring particles (Type III settling). Finally, the soil particles settle to form a structure (Type IV settling). The dredging operation typically creates a soil suspension with 5 to 10 percent solids. At this concentration range, the clayey portion of the soils settles at a rate close to Type III. Types I and II settling are applicable for the more granular fills on this project. These two types are typically used for sediment transport modeling.

Type IV settling is typically simulated with the diffusion equation using either Terzaghi or Gibson consolidation theory.

The dewatering time varies with type of soils and salinity of the water and is often determined using a column test. If the bulk of the borrow material is relatively granular soil from offsite, the settling velocities can be calculated using Stokes' Law (see, e.g., ASTM D422). Velocities for various particle size groups were calculated following D422 and are given in Table 3.2-4.

TABLE 3.2-4
PARTICLE SETTLING VELOCITIES

Particulate Group	Size Range (mm)	Settling Velocity (ft./day)
Medium Sand	0.2-0.4	>1000
Fine Sand	0.07-0.2	>1000
Coarse Silt	0.02-0.07	330
Fine Silt	0.005-0.02	24
Clay	<0.005	1-6

The time-sedimentation rate for more cohesive materials was computed from a column test performed for the Barataria Landbridge Project (BA-36) and the above data. Some 90% of the pumped fill should decant its water (i.e., sediment out) in a period of 1 to 2 months or less. For the coarser material planned at this site, we anticipate the dewatering time be reduced to within two weeks of the completion of the fill.

3.6 Settlements Induced by Area Fill. The area fill within the deposition area will induce two types of settlements:

- Settlements within the deposition areas
- Additional settlements at the perimeter levees. These were included in the values cited on Table 3.2-2.

In addition, the surface of the Area Fill will exhibit settlement from two other sources: consolidation within the fill itself and geologic subsidence.

The settlements induced by weight of the Area Fill are described below.

3.6.1 Method of Analyses. The calculations were made in the manner outlined in Section 3.3.1. A ± 25% accuracy is commonly achieved in settlement analyses. The computations were performed for the same soil conditions as in the stability analyses (Section 3.2) to evaluate the additional levee settlements.

3.6.2 Settlements within Deposition Areas. The major consideration here is the loading which will occur as the soil grains settle out of suspension in the introduced water. It has been assumed that sediment-laden water will be added periodically until the sediment surface is approximately 0.5 feet below the design long-term levee crests. Various sediment heights were also checked for completeness. The applied loading will be the resulting sediment thickness multiplied by the unit weight of the sediment. The latter was taken from earlier Column Tests and the densities observed for the shallow sediments in the boreholes; the design value was 70 lb.cu.ft. Net settlements adjusted for the effects of settlement on applied loading as described in Section 3.3.4 were used. The resulting settlement values are presented in Table 3.2-5 below:

TABLE 3.2-5
LONG-TERM AREA SETTLEMENTS DUE TO WEIGHT OF GENERAL FILL

Water Depth (feet)	Net Settlement (feet) for Freeboard			
	1 foot	2 feet	3 feet	4 feet
0	0.18	0.33	0.48	0.62
2	0.20	0.36	0.53	0.68
4	0.24	0.39	0.55	0.71
6	0.29	0.45	0.60	0.75
8	0.34	0.49	0.63	0.78
10	0.36	0.51	0.65	0.80

[†]: Design top of sediment (ft. NAVD); min. 0.5 feet freeboard.
Settlements calculated for 50+ feet from levee

The values tabulated above are valid for relatively uniformly loaded areas at least 50 feet away from the toes of the levees. In the zones closer to the levees, the settlements can be approximated (if necessary) by interpolating between the values given above and those for levee settlements at Points "B" and "C" given in Section 3.3.4.

As described in Section 3.5, Type I and Type II settling will dominate even at the solids concentration expected from the dredging operation. However, the time to complete the two processes will be relatively short as compared to Type III settling process. The next step is dewatering (Type III), during which the unit weight of the sediments change from the buoyant to the total state. Given these complications, a time-rate analysis is only an approximation. However, it is estimated that the settlement rates will be approximately as follows. Upon completion of final filling (5 year estimate), most of the tabulated within-fill settlement values will be completed (See Table 3.2-6). The rates of settlement were estimated including the dewatering (consolidation) within the fill itself, and also including the effect of geologic subsidence. They were computed for fill thicknesses of 1, 2, 3, and 4 feet and water depths of 0, 2, 4, 6, 8, and 10 feet. This data is presented in tabular form on Table 3.2-6.

3.6.3 Overall Settlement Rate. The overall settlement rate is the combination of the effects from the three major sources:

- Settlement due to weight of Area Fill,
- Self-Weight Consolidation within the fill, and,
- Geologic Subsidence (0.025 feet per year).

These are combined in Table 3.2-6.Due to its size this table is presented separately at the end of the main text of this report.

The total, overall rate is presented graphically for the various initial Area Fill and bottom elevations on Figures 11 through 16. These figures show the anticipated fill top elevations over time, both for the case of no geologic subsidence and considering geologic subsidence.

3.6.4 Settlement Effects on Levees. The weight of the new sediments adjacent to the levees will cause additional settlements of these levees. These movements were calculated and are included in the levee settlement analyses results presented in Section 3.3.4. These movements will occur at approximately the rates given for the Depositional Areas in Section 3.6.2.

3.7 Cut/Fill Ratios. Two cases should be considered here. The first is the amount of cut necessary to create a given amount of levee fill. The levee fill is assumed to be placed mechanically (i.e., with draglines or similar equipment), not hydraulically. The general fill will be placed hydraulically, and will therefore have a cut/fill ratio different from that applicable to mechanically placed fill. Both cases are described below.

3.7.1 Levee Fill. Reference is made to the descriptions of the soil conditions given in Section 2.2, and to the material use recommendations in Section 3.4. Some of the cut material may be Peat (Pt), which is not recommended for levee construction. The shrinkage of the more suitable SM, ML, CL, and CH soils from pit to levee will depend primarily on transport losses and loss of water content. The former (transport) is best obtained from experienced contractors, but is expected to be on the order of 25%. The water loss shrinkage is estimated as 10% to 15% of the pit volume. Overall, then, preliminary estimate can be based on about 1.5 to 1.8 cubic yards of suitable cut to produce 1.0 cubic yard of levee fill.

3.7.2 Area Fill. It is very difficult to determine the cut/fill ratios for the hydraulically placed area fill. As discussed in Section 3.5, sedimentation or settlement occurs in stages, thereby the volume of fill changes. A reasonable assumption of the initial fill height is when the density of the fill reaches the end of Type III settling (fast) or beginning of the Type IV settling (slow). The fill soil volume then can be related to the density and cut/fill ratio determined. The soils from the boreholes are similar to but dryer than those for which a Column Test was performed on the Barataria Landbridge Project (BA-36). Based on that test, and adjusting for the soils' water contents, the cut/fill ratio is expected to be about 1.8, i.e., 1.8 cubic yards of suitable cut should produce 1.0 cubic yard of in-place area fill. Note that the borrow soils for Project BA-36 contain substantially higher organic content. The cut to fill ratio was adjusted empirically. If granular borrow is available, this ratio should drop to around 1.2 to 1.5 cubic yards of borrow for 1.0 cubic yards of fill.

3.8 **Erosion Protection.** A large portion of the levees may consist of ML, SC, and SM soils, which are highly erodible. Erosion protection will be required, especially on the gulf (south) side of the levee. It must be flexible to withstand the anticipated settlements; rip-rap is indicated. A filter fabric is recommended against the fill to prevent washing of the fines. It should be a non-woven geotextile having a weight of at least 8 ounces per square yard (ASTM D3776) an Equivalent Opening Size around 0.05 mm as determined by ASTM D4751, and a grab strength of at least 125 lb. by ASTM D4632. The 6 to 12 inch layer of riprap adjacent to the fabric should be 6 inch maximum store. The remainder of the riprap should be sized according to the appropriate methods for the wave action anticipated.

3.9 **Alternate Containment System.** The other forms of fill containment system considered for this project include a steel sheetpile retaining wall and a concrete soldier pile-lagging system. Due to corrosive environment, the steel retaining system will require significant corrosion protection and may not be suitable for this project. Concrete has a high corrosion resistance and should be considered. The most important aspect of the concrete soldier pile-lagging system is lateral resistance. The following table shows the soldier pile penetration requirements based on various fill heights. The soldier piles were assumed to be 12-inch square concrete installed at 10-foot spacing. It should be noted that some soldier pile penetration depths required exceed the boring depth. In those cases, we assumed that the soil conditions are similar to those found at the upper 40 feet.

TABLE 3.9-1
SOLDIER PILE PENETRATION REQUIREMENT

Water Depth (feet)	Soldier Pile Penetration Depth (feet) for Freeboard	
	2 feet	4 feet
0	12	40
2	30	75
4	75	100+

4.0 SPECIAL TESTING

4.1 **Sampling and Test Types.** One group of soil samples tested under this program was obtained by SJB and transported to STE's laboratory under Chain-of-Custody Control. The samples were tested for water content (ASTM D2216), Particle Size Distribution (ASTM D422 - Sieves Only), and Organic Content (ASTM D2974). A second group consisted of nine (9) grab samples collected by STE. Similar tests were performed on these samples. Descriptions of the test methodologies are given in Appendix A.

4.2 **Results - SJB Group Samples.** The results of these tests are summarized on Table 4.2-1 below. Particle Size Analysis Curves are attached in Appendix A.

TABLE 4.2-1
TEST RESULTS - SPECIAL SJB SAMPLES

Area	Sample	Water Content (%)	Particle Size Distribution Gravel (%)	Sand (%)	Fines (%)	d_{60} (mm)	d_{30} (mm)	d_{10} (mm)	Organic Content (%)
Dune	DN-1	16.5	0.1	99.9	0.0	0.23	0.17	0.13	0.7
	DN-2	23.0	0.0	99.8	0.2	0.24	0.19	0.16	0.7
	DN-3	2.5	0.0	99.9	0.1	0.24	0.19	0.17	0.95
	DN-4	25.2	0.1	99.9	0.0	0.25	0.19	0.17	0.95
	DN-5	1.6	0.0	99.9	0.1	0.24	0.19	0.15	0.9
	DN-6	0.5	0.6	99.1	0.3	0.24	0.19	0.16	0.8
	Mean	11.6	0.1	99.8	0.1	0.24	0.19	0.16	0.8
	S.D.	11.4	0.2	0.3	0.1	0.01	0.01	0.02	0.1
Near Shore	NN-1	25.2	0.6	98.9	0.5	0.25	0.19	0.16	0.8
	NN-2	25.1	0.5	99.0	0.5	0.24	0.18	0.14	0.6
	NN-3	29.5	0.2	99.3	0.5	0.22	0.16	0.11	1.4
	NN-4	24.2	0.0	98.7	1.3	0.21	0.16	0.10	1.4
	NN-5	30.7	0.3	98.6	1.1	0.18	0.12	0.085	1.0
	NN-6	29.5	0.1	99.2	0.7	0.20	0.14	0.092	0.9
	Mean	27.4	0.3	99.0	0.7	0.22	0.16	0.11	1.0
	S.D.	2.8	0.2	0.3	0.4	0.03	0.03	0.03	0.3
Beach	BN-1	23.2	0.6	99.2	0.2	0.26	0.19	0.16	1.3
	BN-2	27.7	0.3	99.5	0.2	0.24	0.18	0.14	1.6
	BN-3	22.7	0.3	96.8	2.9	0.24	0.18	0.13	1.1
	BN-4	24.7	0.1	99.3	0.6	0.23	0.17	0.13	1.3
	BN-5	26.7	6.7	92.8	0.5	0.27	0.19	0.14	1.3
	BN-6	28.1	0.8	97.7	1.5	0.24	0.17	0.14	1.3
	Mean	25.5	1.5	97.5	1.0	0.25	0.18	0.14	1.3
	S.D.	2.3	2.6	2.6	1.1	0.02	0.01	0.01	0.2

Gravel: Equivalent Diameter >4.76mm (#4 Sieve): Shell Fragments
Sand: 4.76mm >Equivalent Diameter >0.074mm (#200 Sieve)
Fines: 0.074mm >Equivalent Diameter
d_{60}: Equivalent Diameter at which 60% of Sample is smaller
d_{30}: Equivalent Diameter at which 30% of Sample is smaller
d_{10}: Equivalent Diameter at which 10% of Sample is smaller

Mean: Average
S.D.: Standard Deviation

4.3 Results - STE Grab Samples. The results of these tests are summarized on Table 4.3-1, below. They are also presented on the Grain Size Analysis Curves attached in Appendix A.

TABLE 4.3-1
TEST RESULTS - SPECIAL STE SAMPLES

Sample ID	Water Content (%)	Particle Size Distribution							Organic Content (%)
		Gravel (%)	Sand (%)	Silt (%)	Clay (%)	d_{60} (mm)	d_{30} (mm)	d_{10} (mm)	
Beach Sand	20.8	0.0	99.7	(0.3)	NT	0.18	0.12	0.086	NT
Dune Sand	0.4	0.0	99.8	(0.2)	NT	0.20	0.15	0.097	NT
Marsh	NT	0.0	3.1	50.5	46.4	0.0094	0.0022	<0.0015	NT
SMSS-01	147.3	0.6	6.5	37.0	55.9	0.0070	<0.0021	<0.0021	11.3
SMSS-01A	180.1	1.1	8.9	32.5	57.5	0.0062	<0.0021	<0.0021	11.2
SMSS-02	99.1	0.8	19.3	41.1	38.8	0.036	0.0020	<0.0020	11.9
SMSS-03	65.5	0.0	25.1	46.9	28.0	0.055	0.0064	<0.0023	5.6
SMSS-04	73.9	0.0	7.2	50.7	42.1	0.026	<0.0023	<0.0023	6.2

Notes: See Table 4.2-1.
Silt: 0.074mm > Equivalent Diameter > 0.005mm — ( ): Total Silt + Clay
Clay: 0.005 mm > Equivalent Diameter — NT: Not Tested

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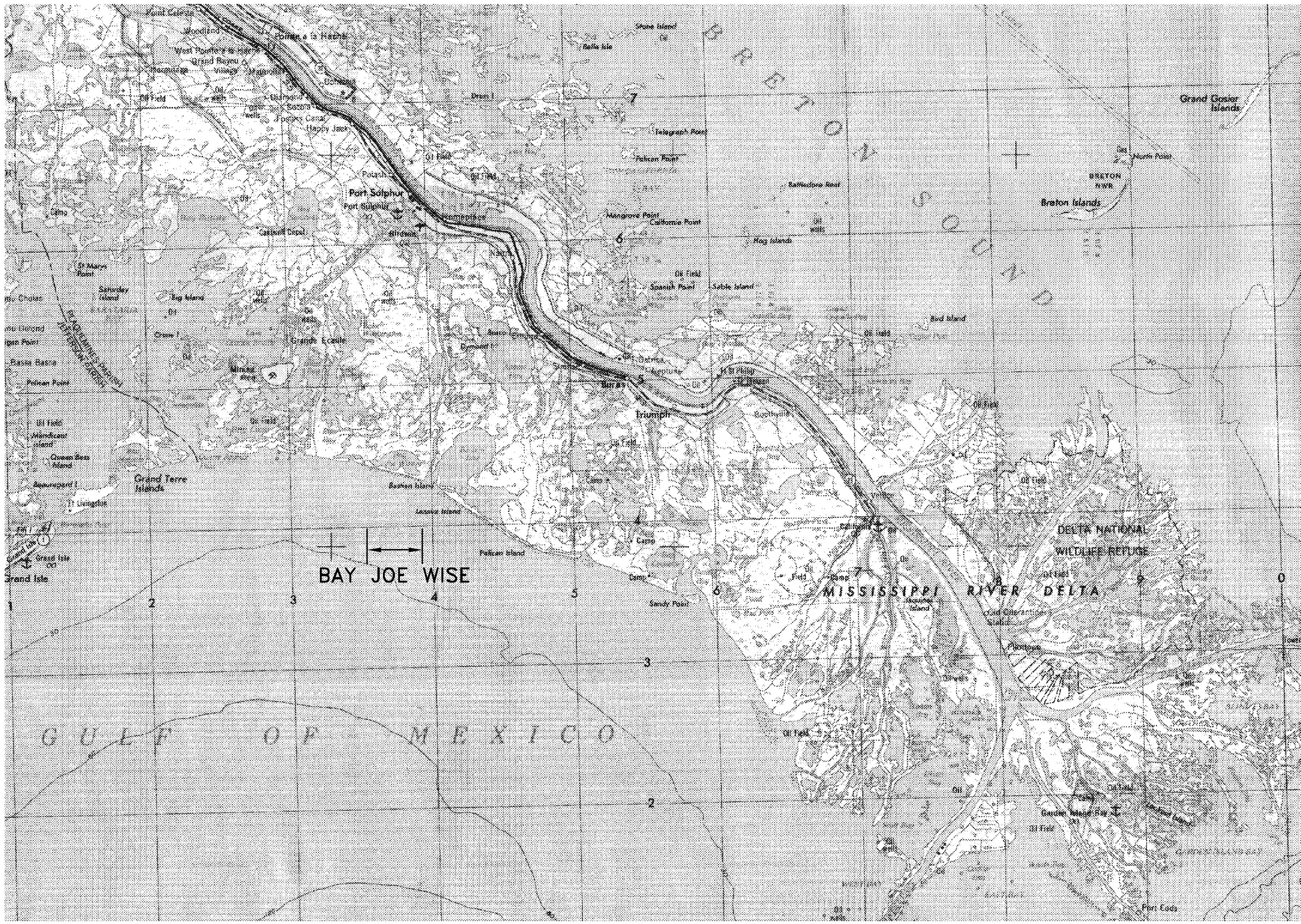

0 (miles) 10±

SCALE

REFERENCE:
USGS NH 16–7 of Breton Sound, LA,
1957 edition, revised 1973.

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PASS CHALAND TO GRAND BAYOU PASS
BARRIER SHORELINE RESTORATION
(BAY JOE WISE)
PROJECT BA-35
PLAQUEMINES PARISH, LOUISIANA

for

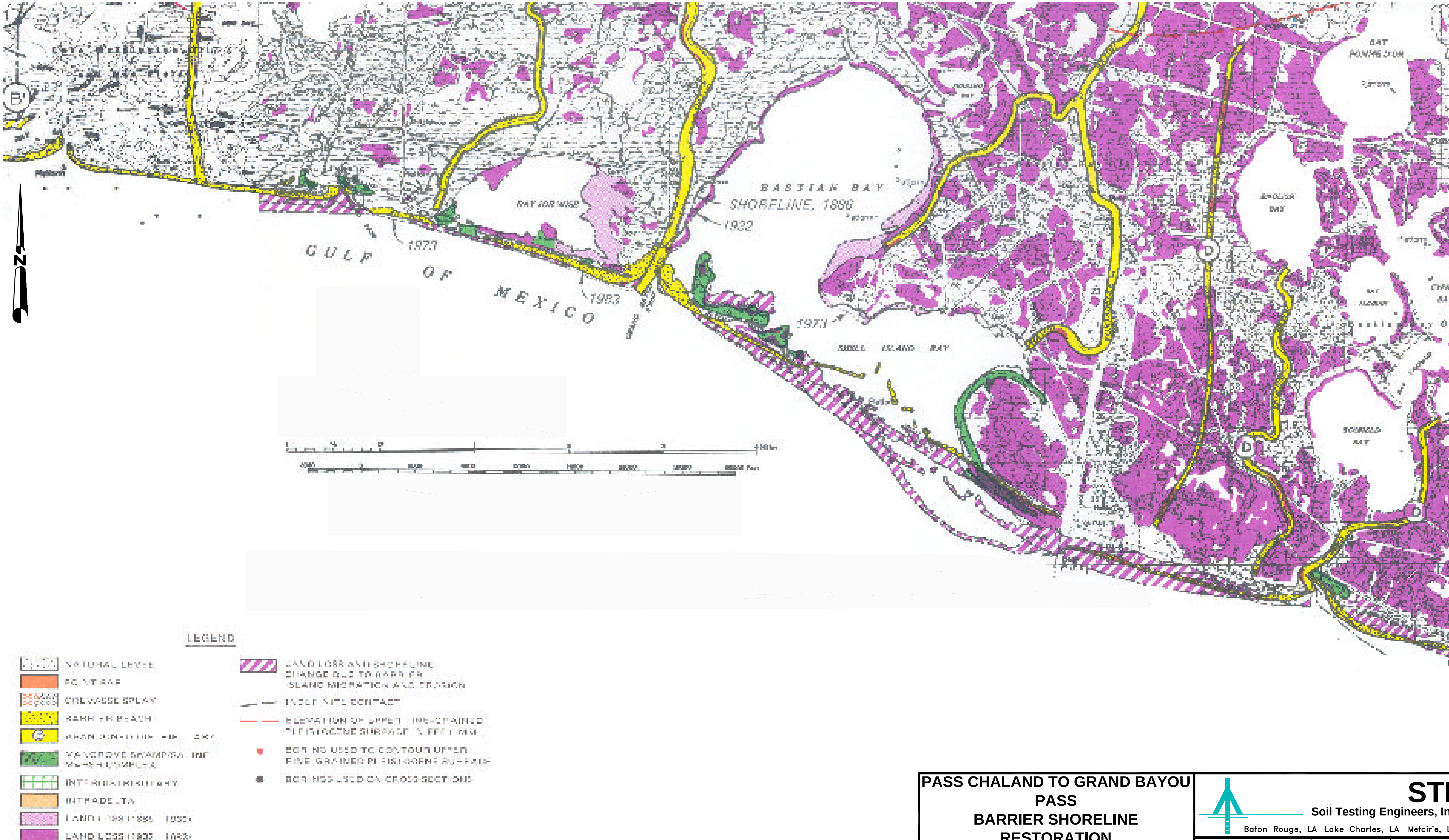
LOUISIANA DEPARTMENT OF NATURAL RESOURCES
BATON ROUGE, LOUISIANA

SJB GROUP, INC.
BATON ROUGE, LOUISIANA

STE
Soil Testing Engineers, Inc.
Baton Rouge, LA Lake Charles, LA Metairie, LA

Project Engineer:	Drawn by:	Checked by:
G.P. Boutwell	DMS	
File No.: 03–1033	Date: 10–21–03	Figure No.: 1

Title:
VICINITY MAP



REFERENCE:
USA/COE "Distribution of Deltaic and Marine Deposits," of Empire, LA (1987) and Ft. Livingston, LA (1990).

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Baton Rouge, LA Lake Charles, LA Metairie, LA

Project Engineer: G.P. Boutwell	Drawn by: DMS	Checked by:
File No.: 03-1033	Date: 10-14-03	Figure No.: 2

Title: **GELOGIC MAPPING**

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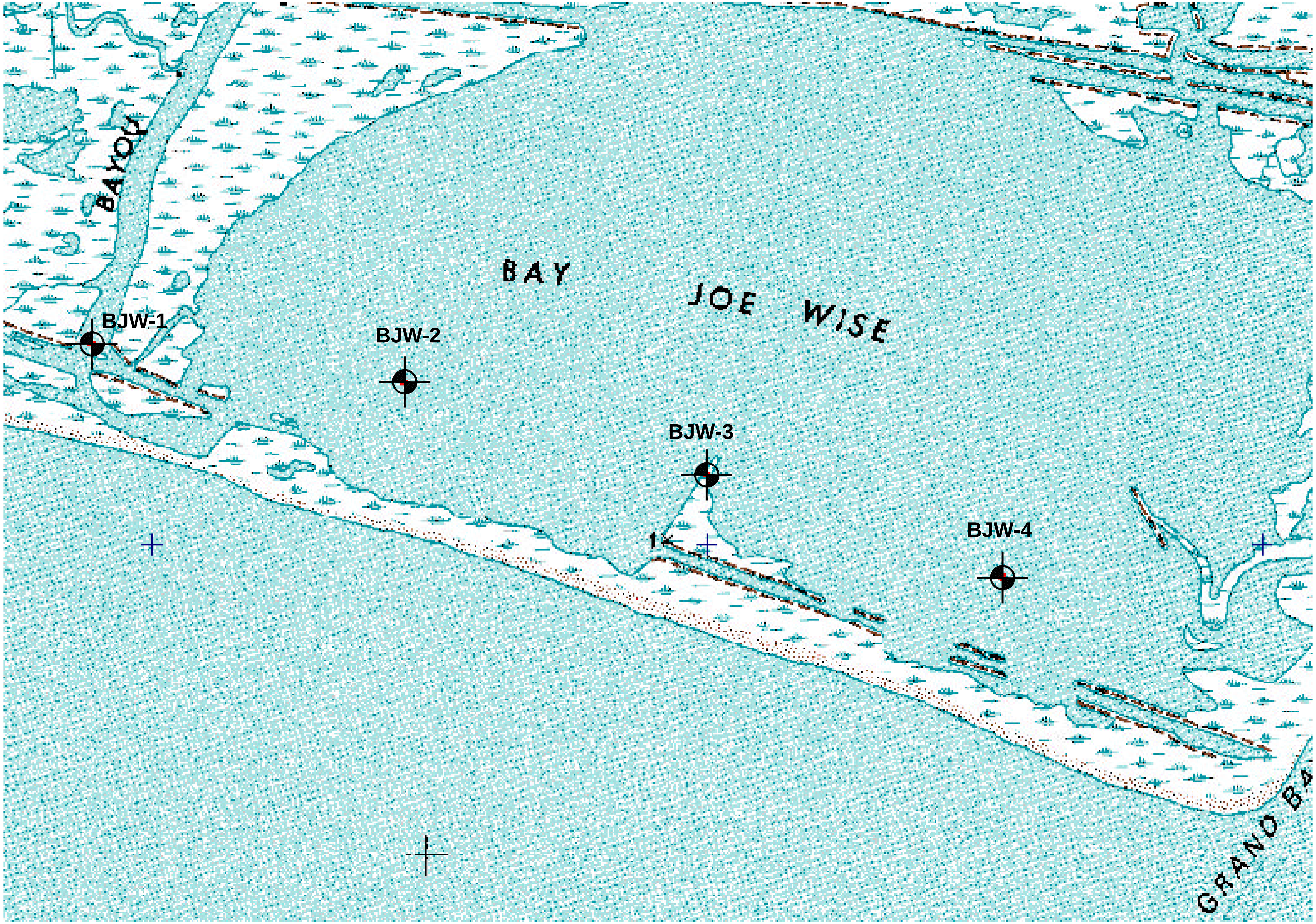

REFERENCE:
USGS Quadrangle Map of Bastian Bay, LA, 1993 edition.

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Project Engineer: G.P. Boutwell	Drawn by: DMS	Checked by:
File No.: 03-1033	Date: 10-14-03	Figure No.: 3

Title:
BORING PLAN

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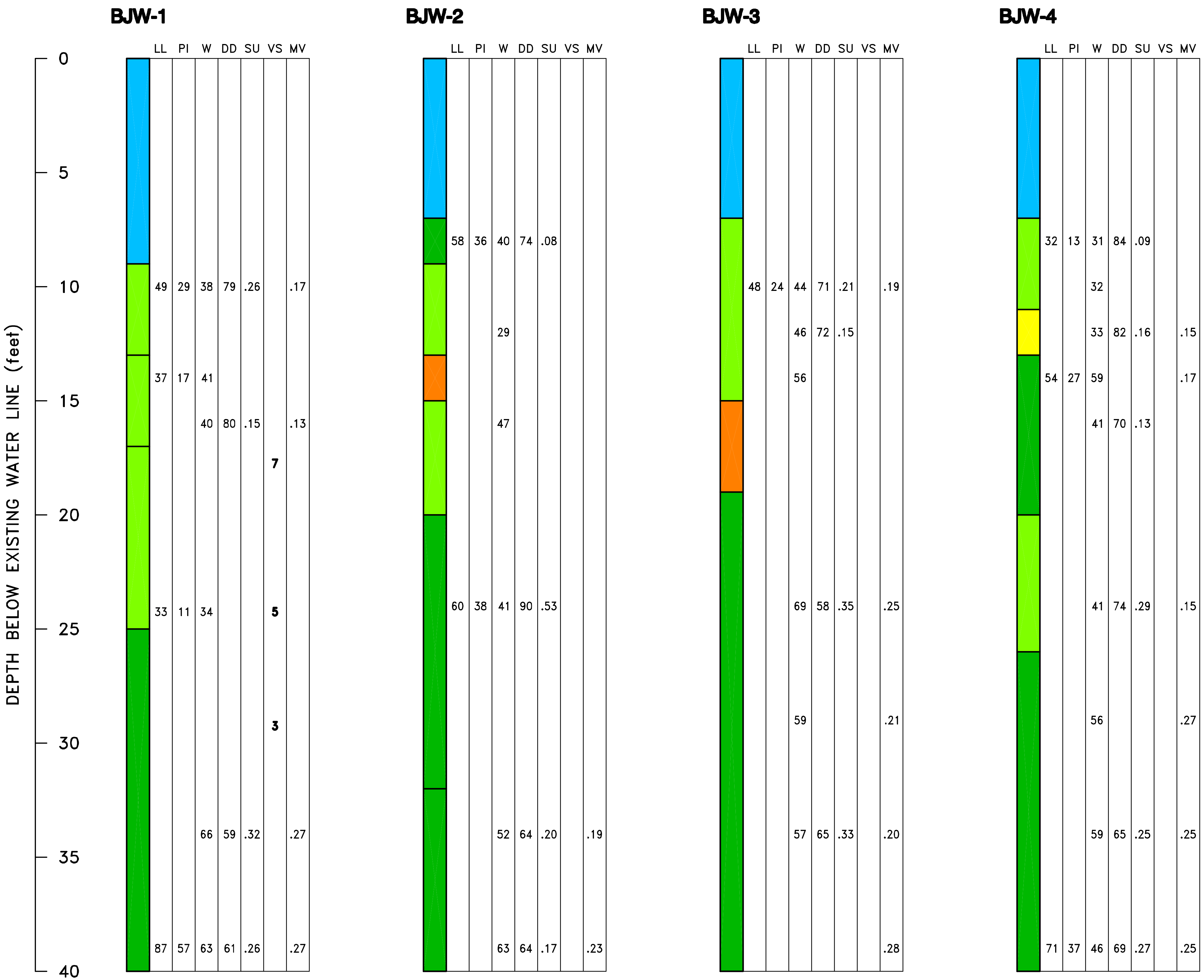
Nov 14, 2003 - 10:09am



SYMBOL	SOIL TYPE
	WATER
	PEAT
	ORGANIC CLAY (OH)
	CLAY (CH)
	SILTY CLAY or SANDY CLAY (CL)
	CLAYEY SAND (SC)
	SANDY SILT, CLAYEY SILT (ML)
	SILTY SAND (SM)

LEGEND:

LL	LIQUID LIMIT (%)
PI	PLASTICITY INDEX (%)
W	WATER CONTENT (%)
DD	DRY DENSITY (pcf)
SU	UNDRAINED SHEAR STRENGTH (ksf)
VS	FIELD VANE SHEAR (ksf)
	NOTE: **BOLD** = Standard Penetration Resistance (blows/foot)
MV	MINIATURE LAB VANE SHEAR (ksf)

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Project Engineer: G.P. Boutwell	Drawn by: DMS	Checked by:
File No.: 03-1033	Date: 10-14-03	Figure No.: 4

Title: **SOIL PROFILE**

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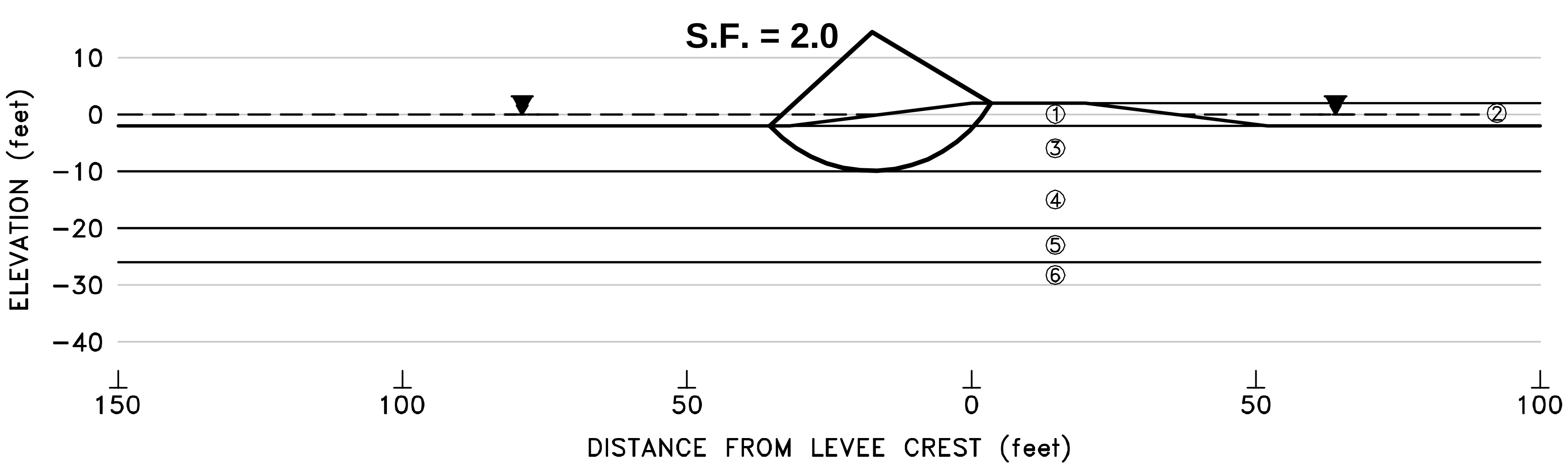
1(V):6(H) SIDE SLOPES



1(V):8(H) SIDE SLOPES

SOIL DATA

LAYER	SHEAR STRENGTH (psf)	FRICTION ANGLE (degrees)	UNIT WEIGHT (pcf)
1	100	0	100
1a	50	22	110
2	50	0	70
3	85	0	104
4	130	0	100
5	290	0	104
6	180	0	100

a: Fill. Analyzed for both types. Lowest Safety Factor governs and is illustrated.

PASS CHALAND TO GRAND BAYOU PASS
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Soil Testing Engineers, Inc.
Baton Rouge, LA Lake Charles, LA Metairie, LA

Project Engineer: G.P. Boutwell	Drawn by: DMS	Checked by:
File No.: 03−1033	Date: 11−10−03	Figure No.: 5

Title: **STABILITY ANALYSES**
2 FEET WATER, 2 FEET FREEBOARD

Nov 14, 2003 − 10:09am

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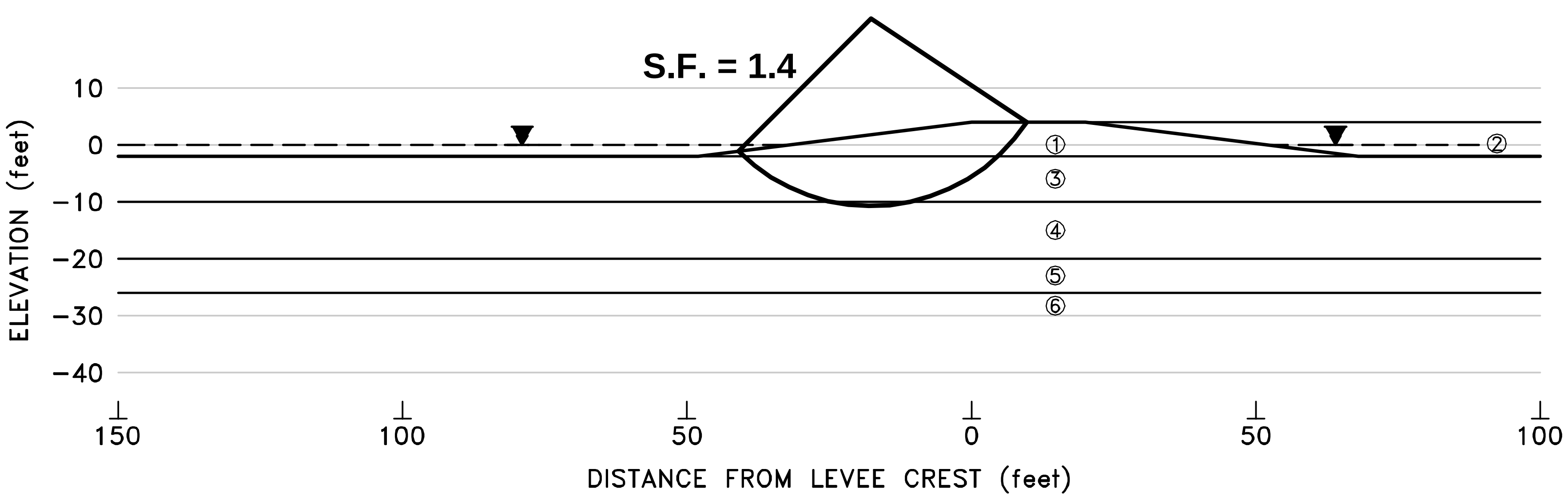
1(V):4(H) SIDE SLOPES

1(V):6(H) SIDE SLOPES



1(V):8(H) SIDE SLOPES

SOIL DATA

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1a	50	22	110
2	50	0	70
3	85	0	104
4	130	0	100
5	290	0	104
6	180	0	100

a: Fill. Analyzed for both types. Lowest Safety Factor governs and is illustrated.

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Project Engineer: G.P. Boutwell	Drawn by: DMS	Checked by:
File No.: 03−1033	Date: 11−10−03	Figure No.: 6

Title: **STABILITY ANALYSES**
2 FEET WATER, 4 FEET FREEBOARD

Nov 14, 2003 – 10:09am

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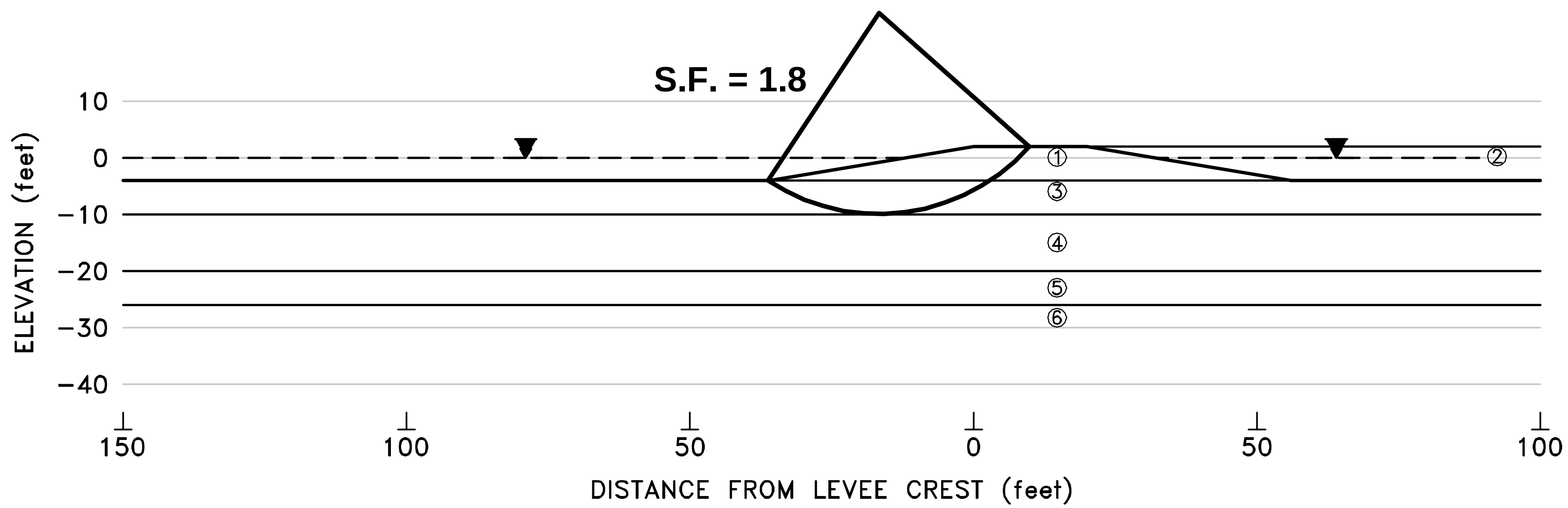
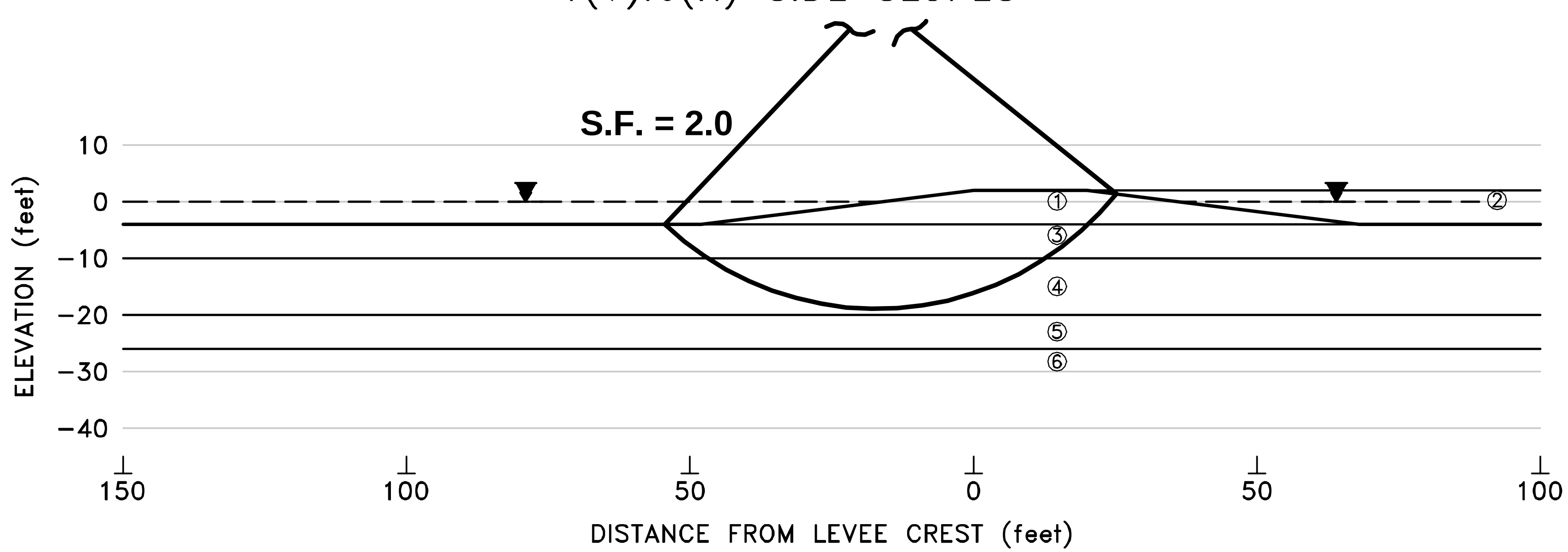
1(V):4(H) SIDE SLOPES



1(V):6(H) SIDE SLOPES



1(V):8(H) SIDE SLOPES

SOIL DATA

LAYER	SHEAR STRENGTH (psf)	FRICTION ANGLE (degrees)	UNIT WEIGHT (pcf)
1	100	0	100
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2	50	0	70
3	85	0	104
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6	180	0	100

a: Fill. Analyzed for both types. Lowest Safety Factor governs and is illustrated.

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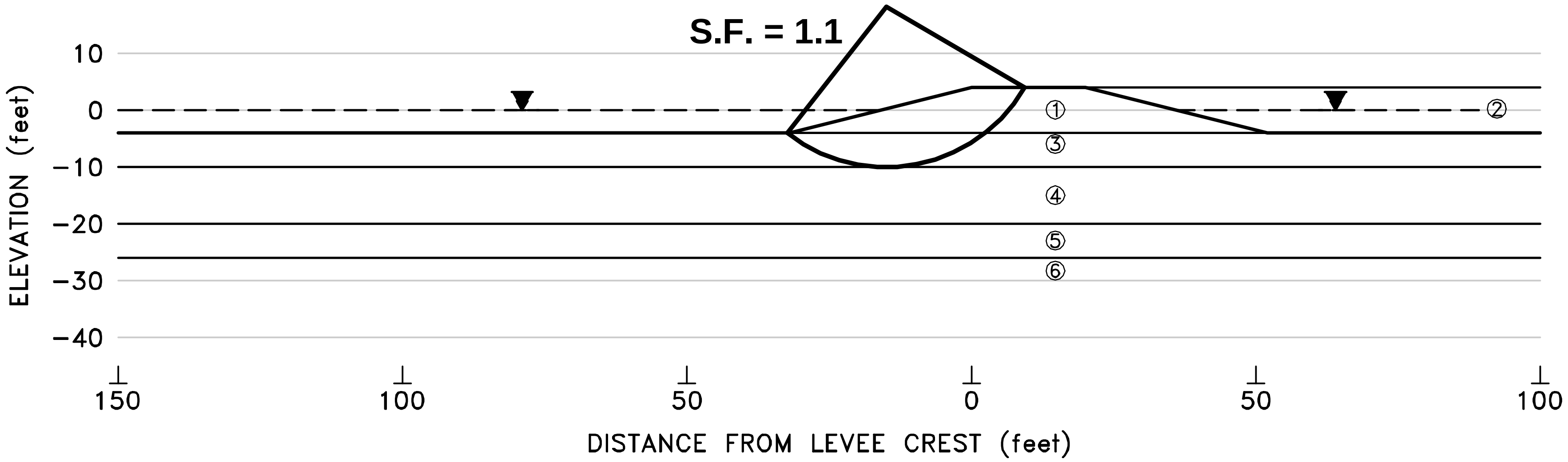
for
LOUISIANA DEPARTMENT OF NATURAL RESOURCES
BATON ROUGE, LOUISIANA
SJB GROUP, INC.
BATON ROUGE, LOUISIANA

STE
Soil Testing Engineers, Inc.
Baton Rouge, LA Lake Charles, LA Metairie, LA

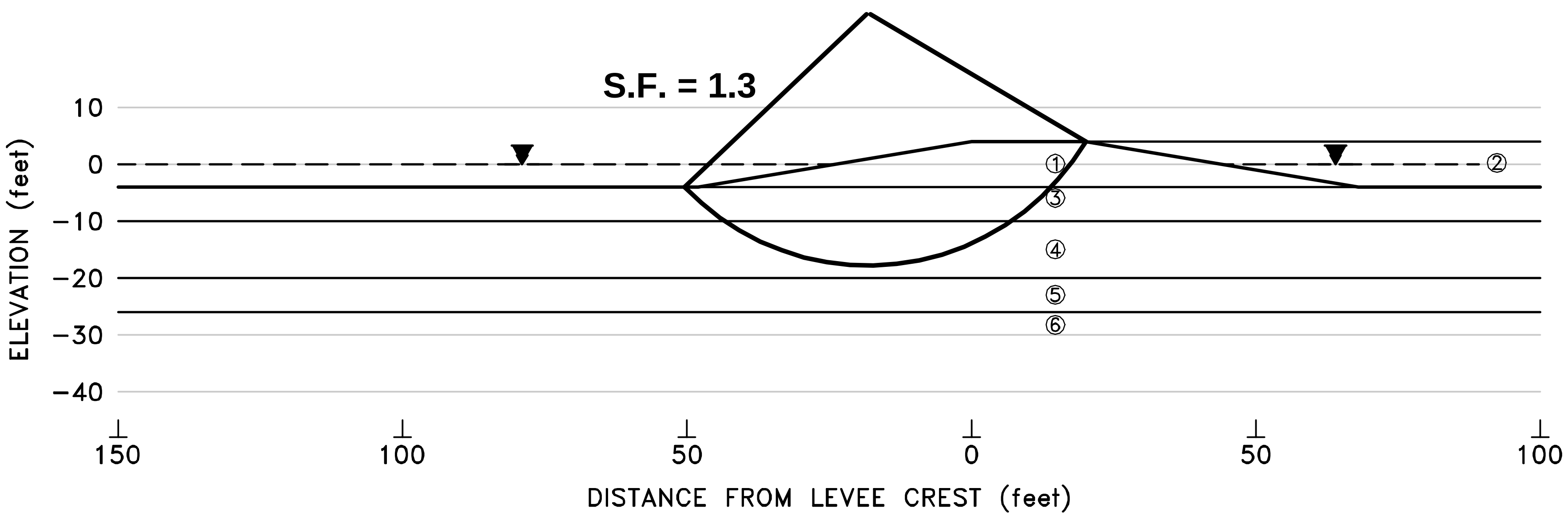
Project Engineer:	Drawn by:	Checked by:
G.P. Boutwell	DMS	
File No.: 03–1033	Date: 11–10–03	Figure No.: 7

Title: **STABILITY ANALYSES**
4 FEET WATER, 2 FEET FREEBOARD

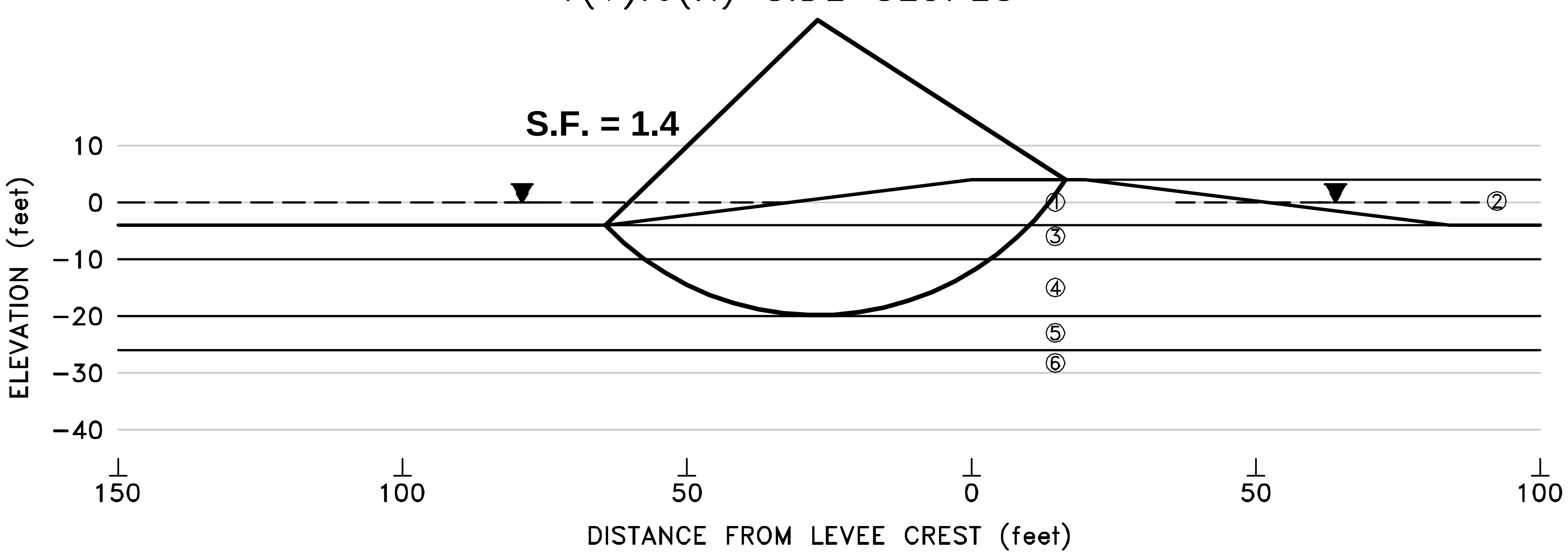
Nov 14, 2003 - 10:10am



1(V):4(H) SIDE SLOPES



1(V):6(H) SIDE SLOPES



1(V):8(H) SIDE SLOPES

SOIL DATA

LAYER	SHEAR STRENGTH (psf)	FRICTION ANGLE (degrees)	UNIT WEIGHT (pcf)
1	100	0	100
1a	50	22	110
2	50	0	70
3	85	0	104
4	130	0	100
5	290	0	104
6	180	0	100

a: Fill. Analyzed for both types. Lowest Safety Factor governs and is illustrated.

PASS CHALAND TO GRAND BAYOU PASS
BARRIER SHORELINE RESTORATION
(BAY JOE WISE)
PROJECT BA-35
PLAQUEMINES PARISH, LOUISIANA
for
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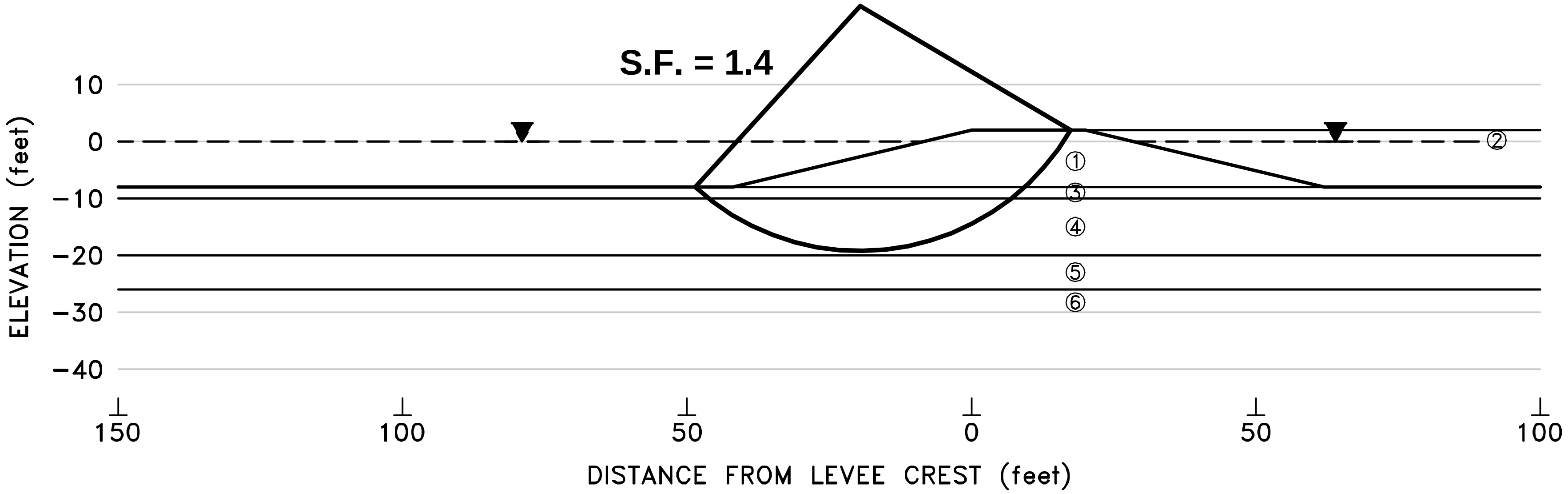
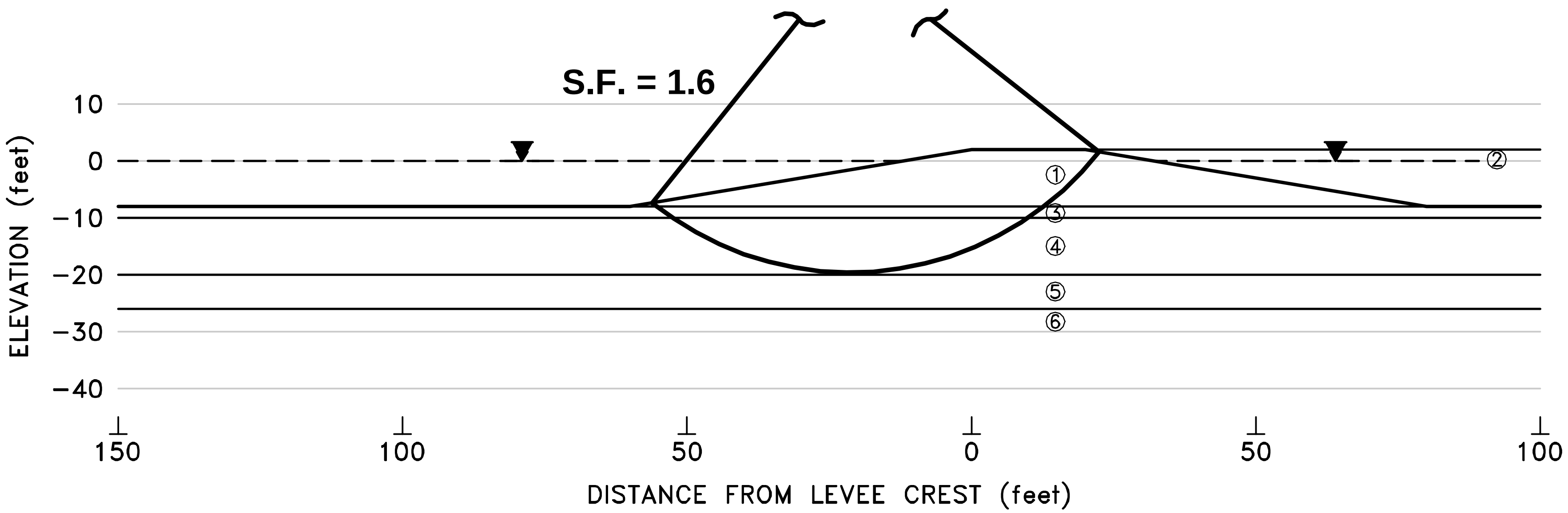
Project Engineer: G.P. Boutwell	Drawn by: DMS	Checked by:
File No.: 03−1033	Date: 11−10−03	Figure No.: 8

Title: **STABILITY ANALYSES**
4 FEET WATER, 4 FEET FREEBOARD

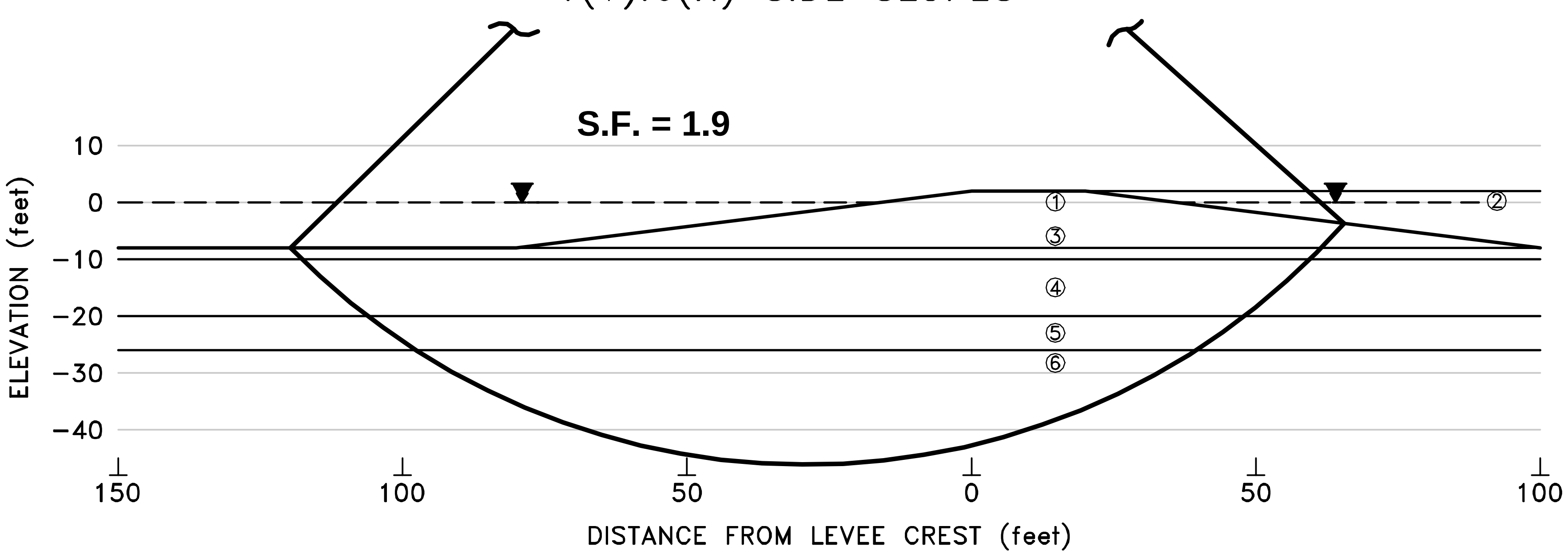
P:\2003\03−1033\CADD\031033−8 STABILITY 4−4.DWG



1(V):4(H) SIDE SLOPES



1(V):6(H) SIDE SLOPES



1(V):8(H) SIDE SLOPES

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a: Fill. Analyzed for both types. Lowest Safety Factor governs and is illustrated.

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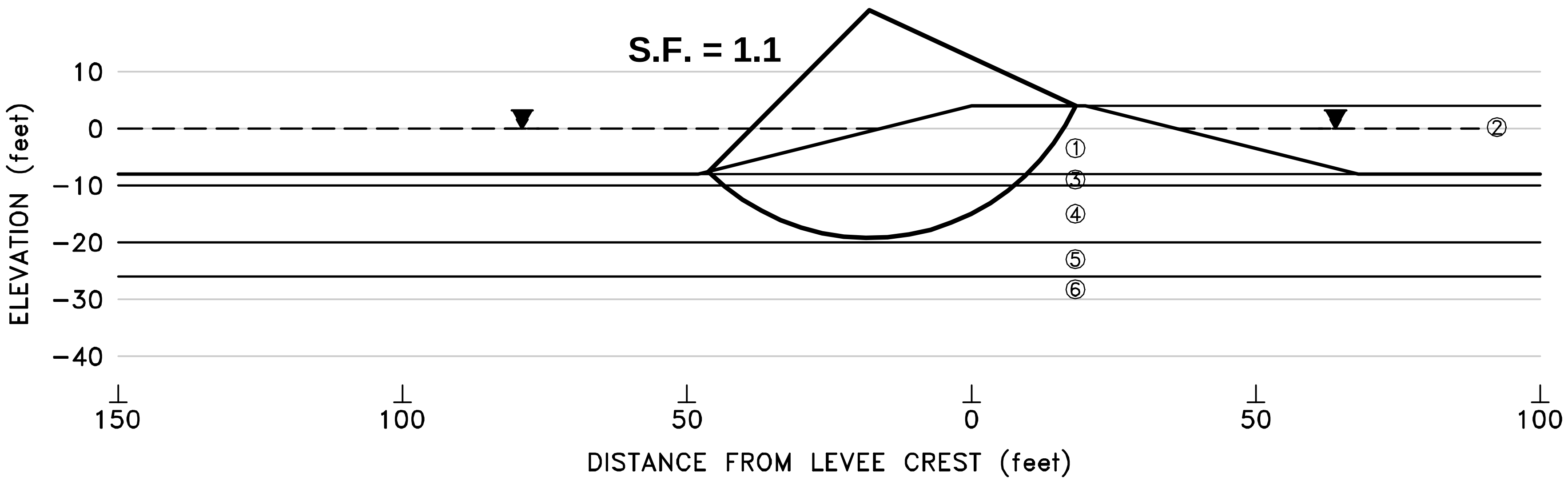
Project Engineer: G.P. Boutwell	Drawn by: DMS	Checked by:
File No.: 03-1033	Date: 11-10-03	Figure No.: 9

Title: **STABILITY ANALYSES**
8 FEET WATER, 2 FEET FREEBOARD

Nov 14, 2003 - 10:10am

P:\2003\03-1033\CADD\031033-9 STABILITY 8-2.DWG



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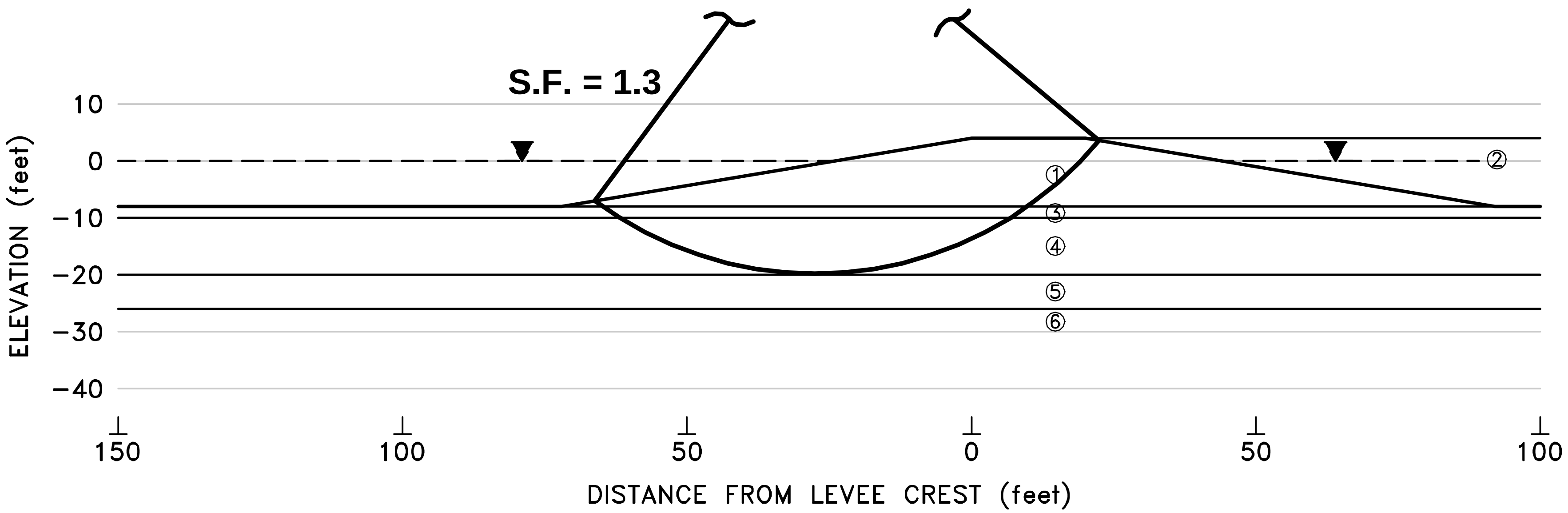
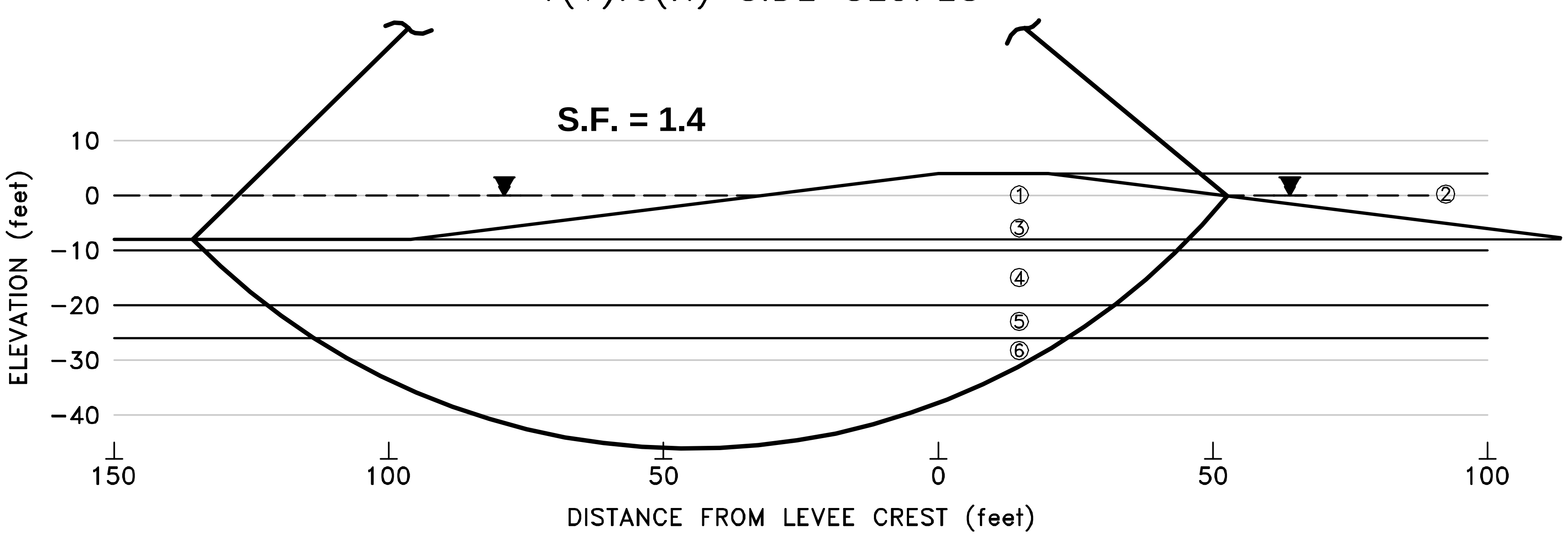


1(V):6(H) SIDE SLOPES



1(V):8(H) SIDE SLOPES

SOIL DATA

LAYER	SHEAR STRENGTH (psf)	FRICTION ANGLE (degrees)	UNIT WEIGHT (pcf)
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1a	50	22	110
2	50	0	70
3	85	0	104
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a: Fill. Analyzed for both types. Lowest Safety Factor governs and is illustrated.

PASS CHALAND TO GRAND BAYOU PASS BARRIER SHORELINE RESTORATION

(BAY JOE WISE)

PROJECT BA-35

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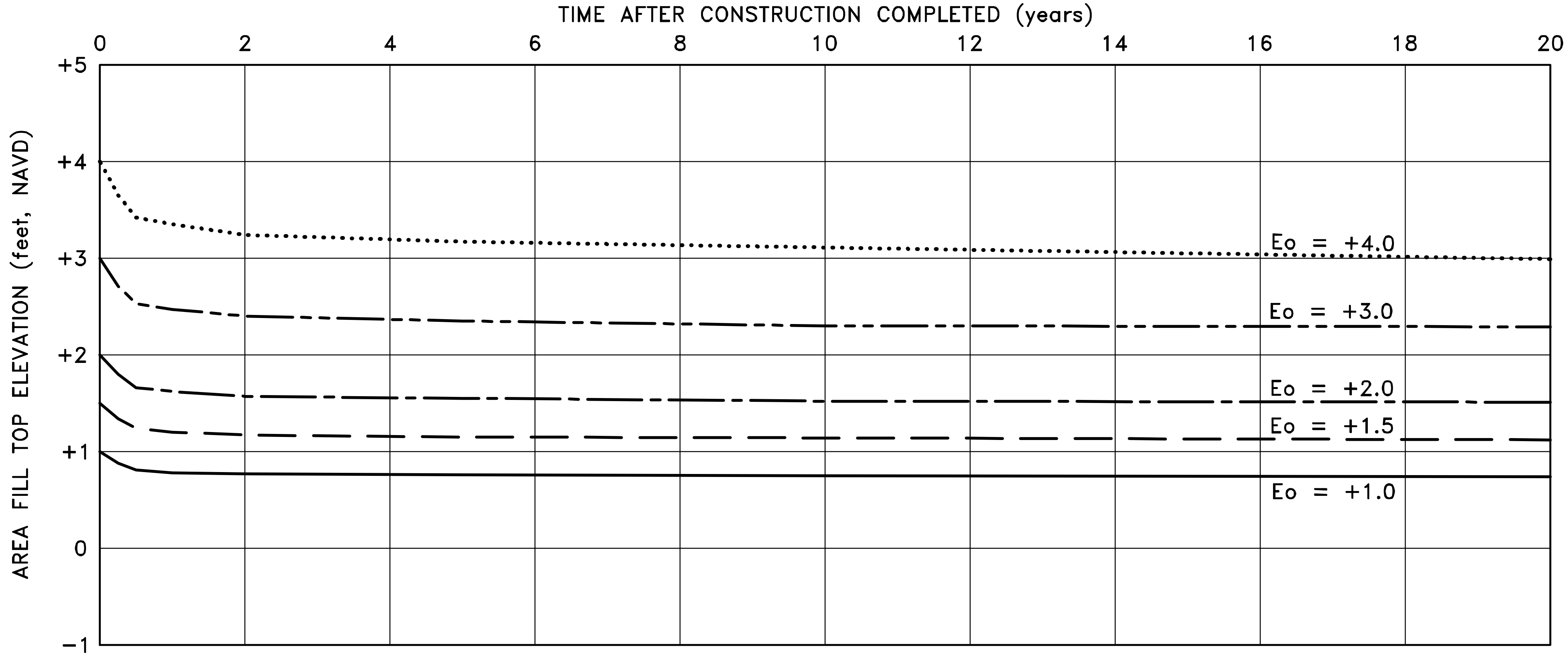
Project Engineer: G.P. Boutwell	Drawn by: DMS	Checked by:
File No.: 03–1033	Date: 11–10–03	Figure No.: 10

Title: **STABILITY ANALYSES**

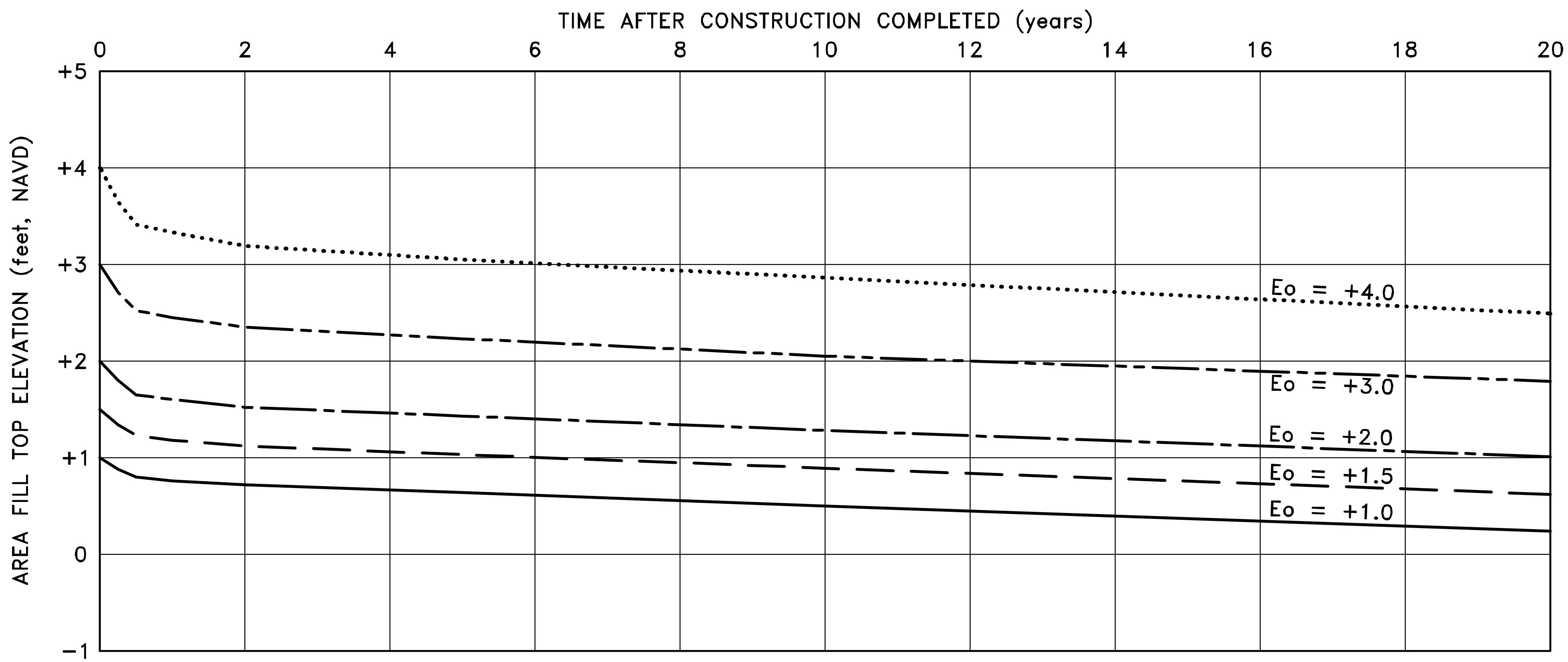
8 FEET WATER, 4 FEET FREEBOARD

Nov 14, 2003 – 10:10am

P:\2003\03–1033\CADD\031033–10 STABILITY 8–4.DWG



WITHOUT GEOLOGIC SUBSIDENCE

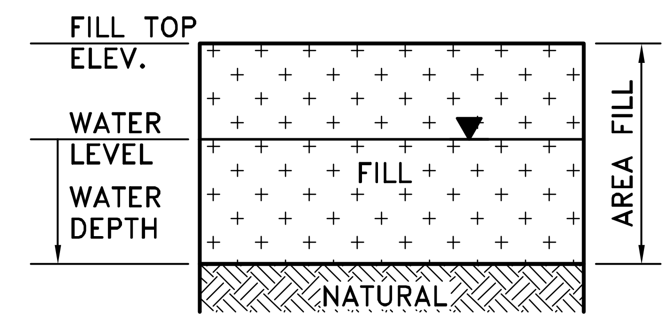


WITH GEOLOGIC SUBSIDENCE

NOTES:

1. Settlements include both self-weight consolidation and weight-induced settlements in foundation soils.

2. Geologic subsidence rate furnished by LDNR = 0.5 ft./20 yr.



PASS CHALAND TO GRAND BAYOU PASS
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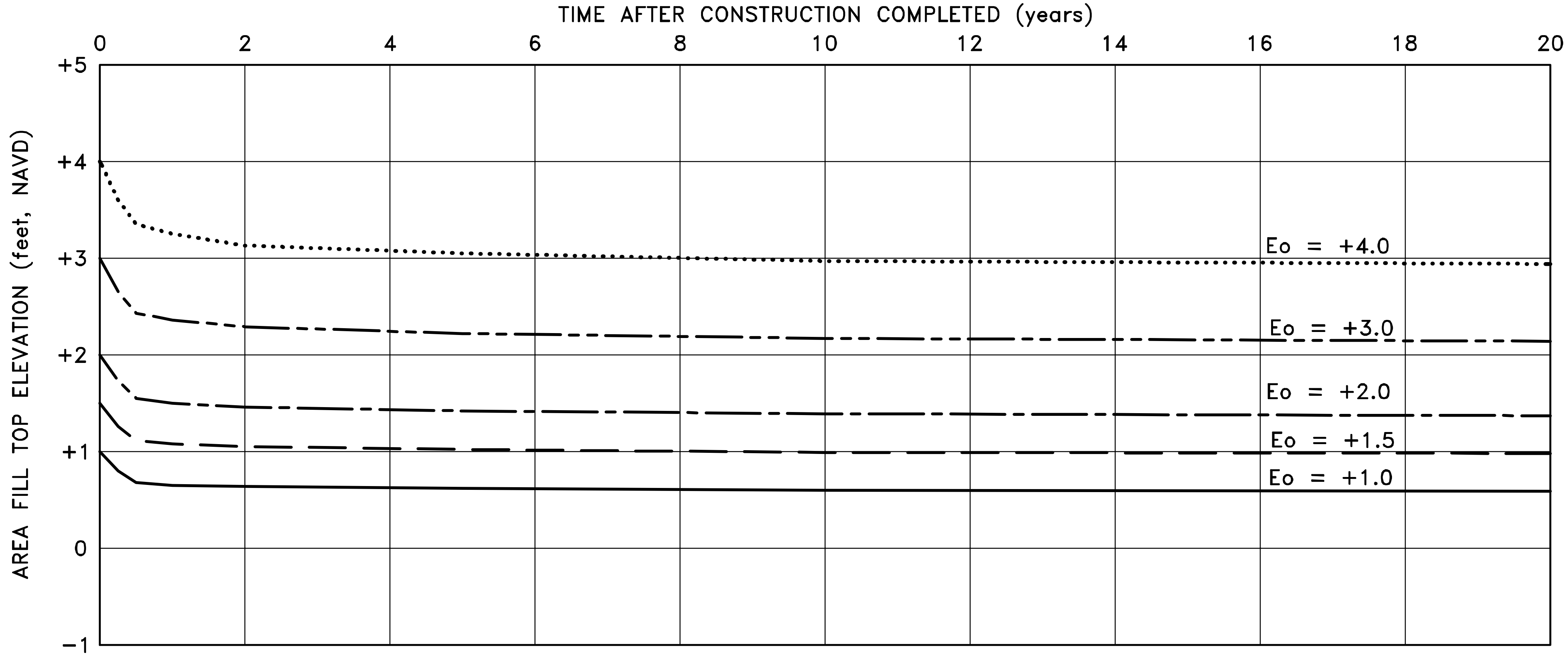
G.P. Boutwell	DMS	
File No.: 03-1033	Date: 11-10-03	Figure No.: 11

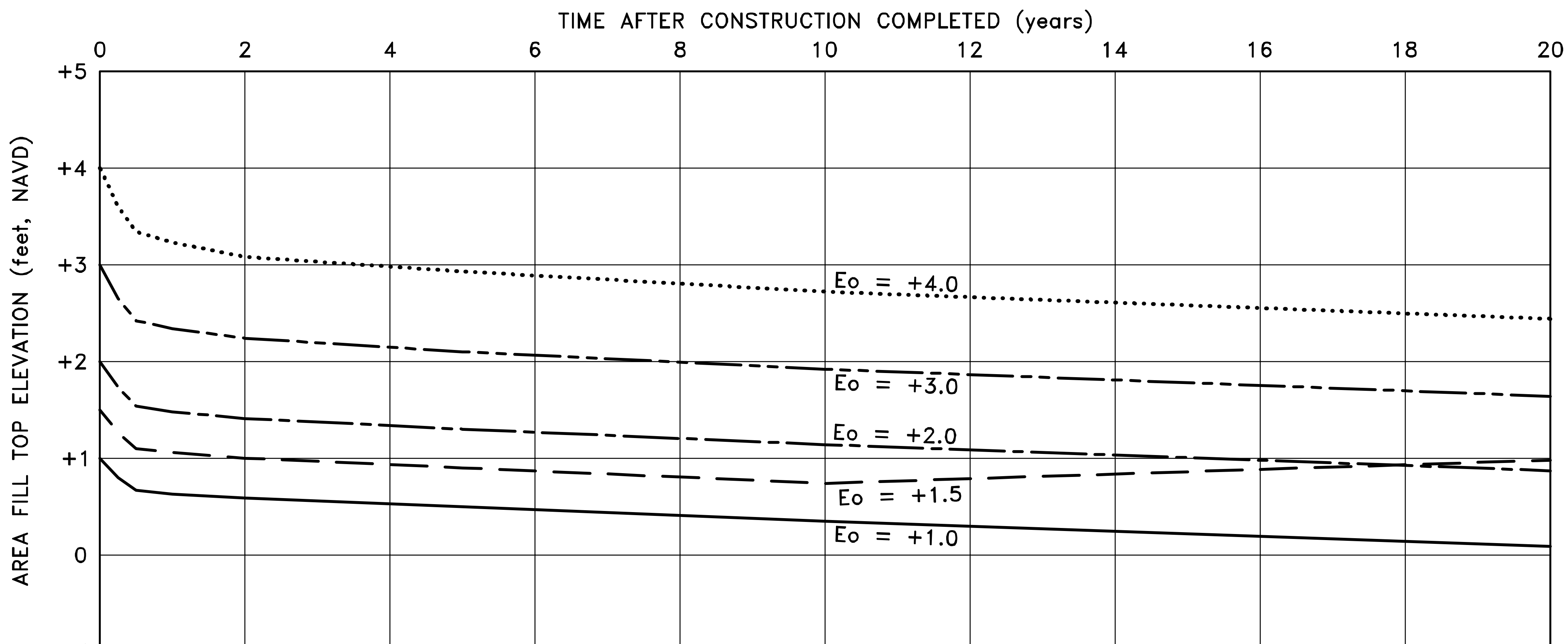
Title: **TIME-SETTLEMENT CURVES AREA FILL: WATER DEPTH 0 FT.**

Nov 14, 2003 - 10:11am

P:\2003\03-1033\CADD\031033-11 SETTLEMENT 0FT.DWG



WITHOUT GEOLOGIC SUBSIDENCE

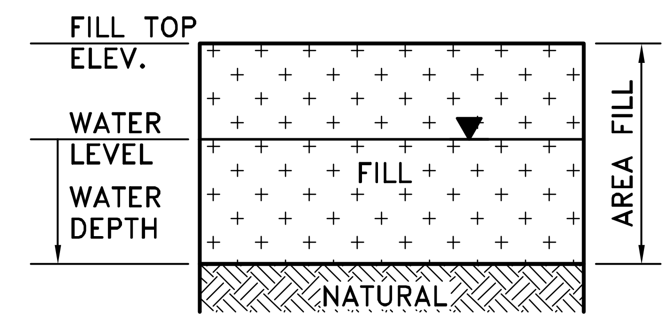


WITH GEOLOGIC SUBSIDENCE

NOTES:

1. Settlements include both self-weight consolidation and weight-induced settlements in foundation soils.

2. Geologic subsidence rate furnished by LDNR = 0.5 ft./20 yr.



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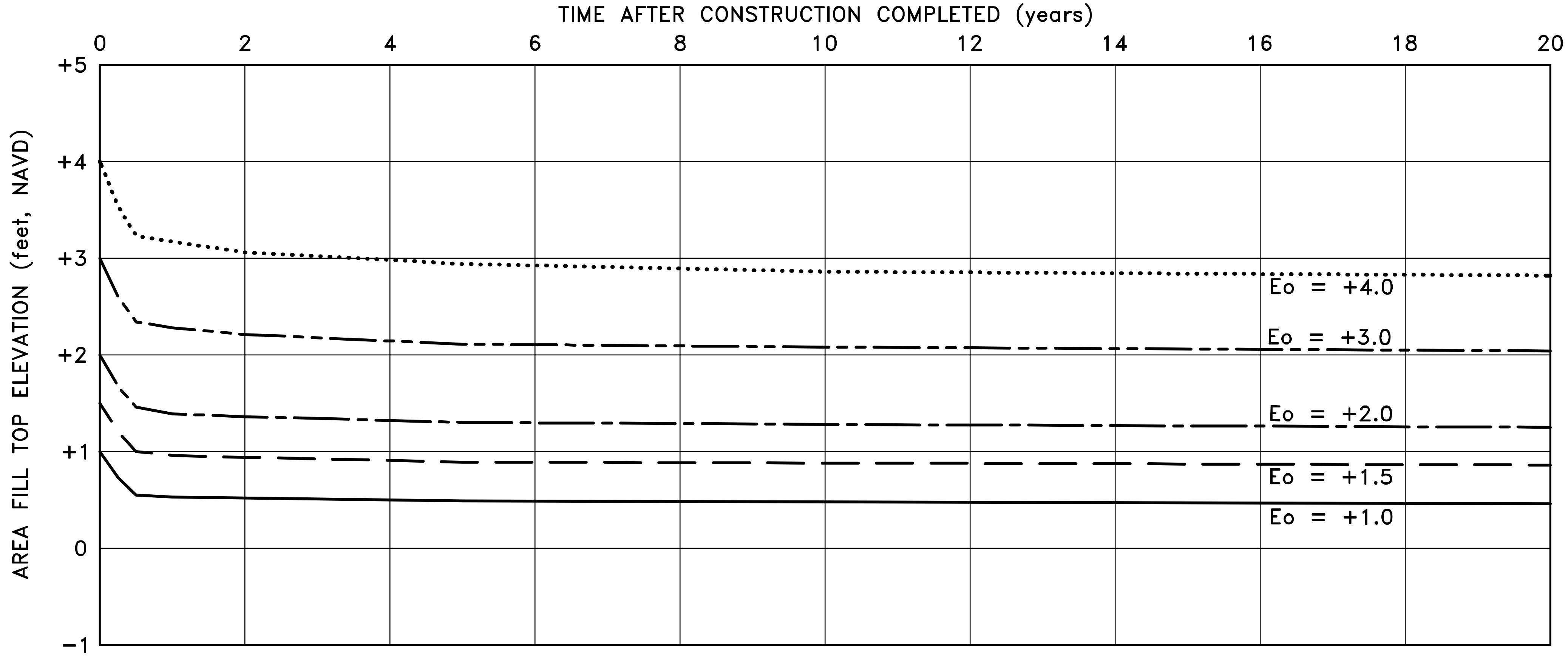
Project Engineer:	Drawn by:	Checked by:
G.P. Boutwell	DMS	
File No.: 03-1033	Date: 11-10-03	Figure No.: **12**

Title: **TIME-SETTLEMENT CURVES AREA FILL: WATER DEPTH 2 FT.**

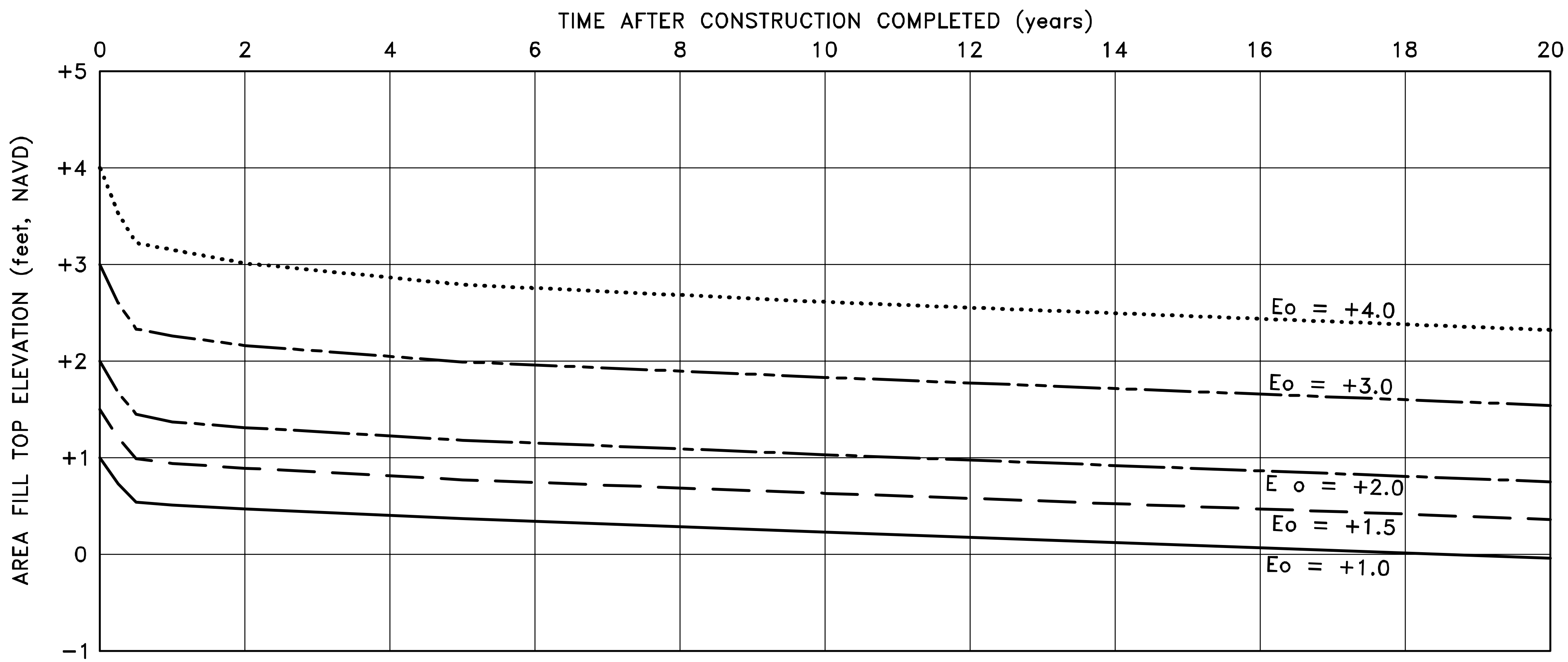
Nov 14, 2003 – 10:11am

P:\2003\03-1033\CADD\031033-12 SETTLEMENT 2FT.DWG

Nov 14, 2003 - 10:11am



WITHOUT GEOLOGIC SUBSIDENCE

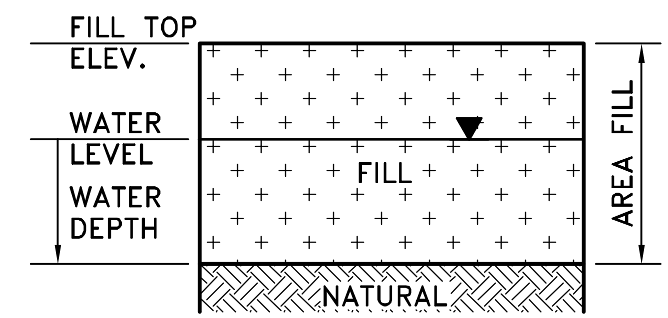


WITH GEOLOGIC SUBSIDENCE

NOTES:

1. Settlements include both self−weight consolidation and weight−induced settlements in foundation soils.

2. Geologic subsidence rate furnished by LDNR = 0.5 ft./20 yr.



PASS CHALAND TO GRAND BAYOU PASS
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(BAY JOE WISE)
PROJECT BA-35
PLAQUEMINES PARISH, LOUISIANA

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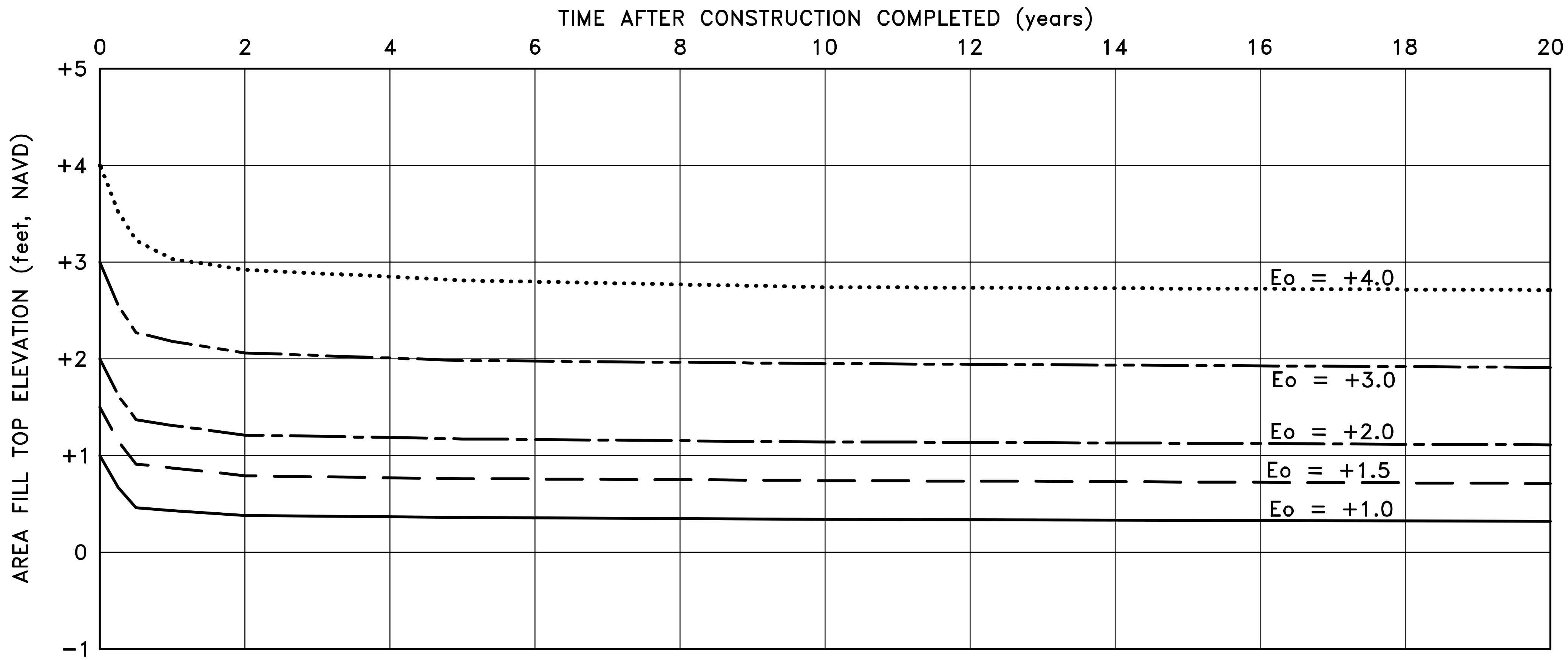
Project Engineer:	Drawn by:	Checked by:
G.P. Boutwell	DMS	
File No.:	Date:	Figure No.:
03−1033	11−10−03	13

Title: **TIME-SETTLEMENT CURVES AREA FILL: WATER DEPTH 4 FT.**

P:\2003\03−1033\CADD\031033−13 SETTLEMENT 4FT.DWG

Nov 14, 2003 - 10:11am

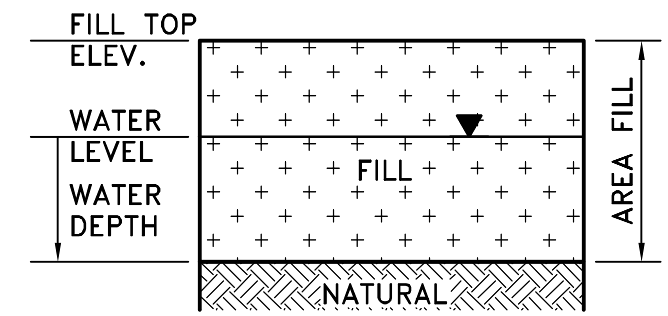


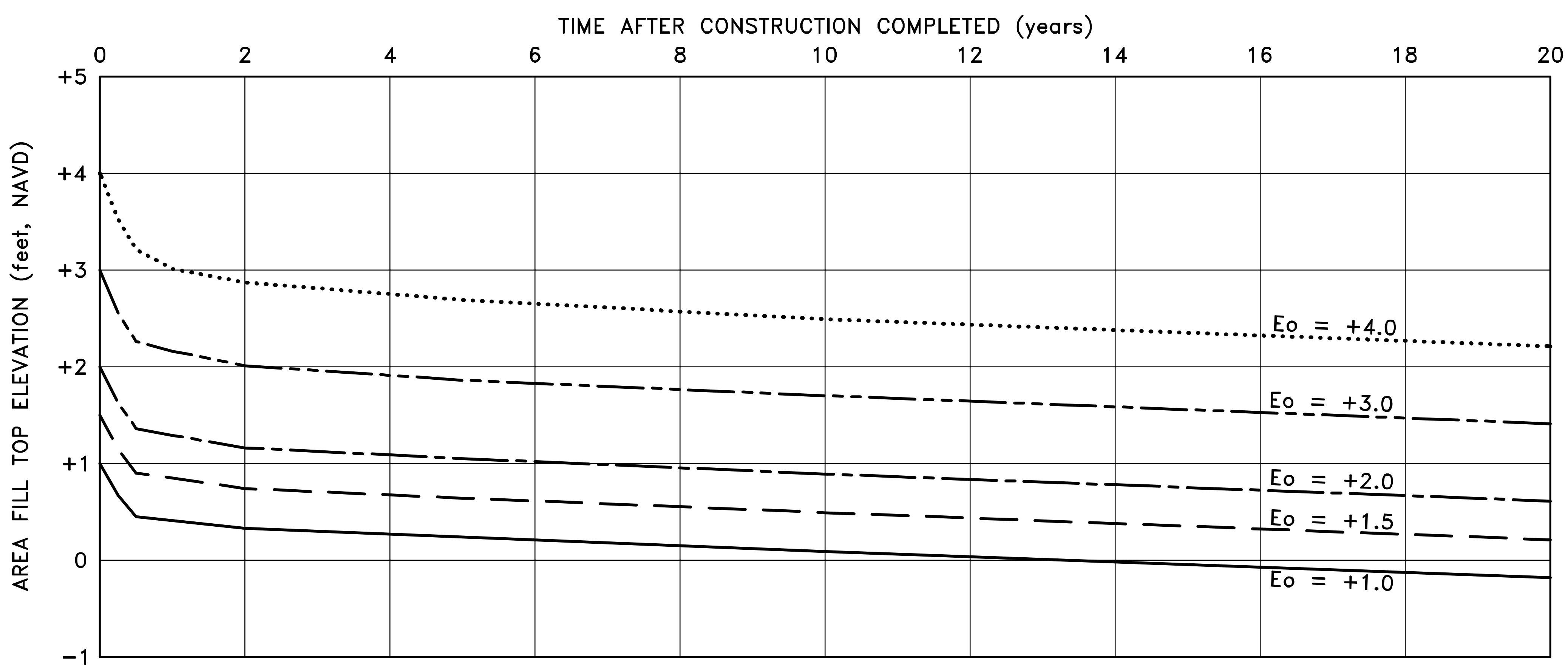
WITHOUT GEOLOGIC SUBSIDENCE

NOTES:

1. Settlements include both self-weight consolidation and weight-induced settlements in foundation soils.

2. Geologic subsidence rate furnished by LDNR = 0.5 ft./20 yr.





WITH GEOLOGIC SUBSIDENCE

P:\2003\03-1033\CADD\031033-14 SETTLEMENT 6FT.DWG

PASS CHALAND TO GRAND BAYOU PASS
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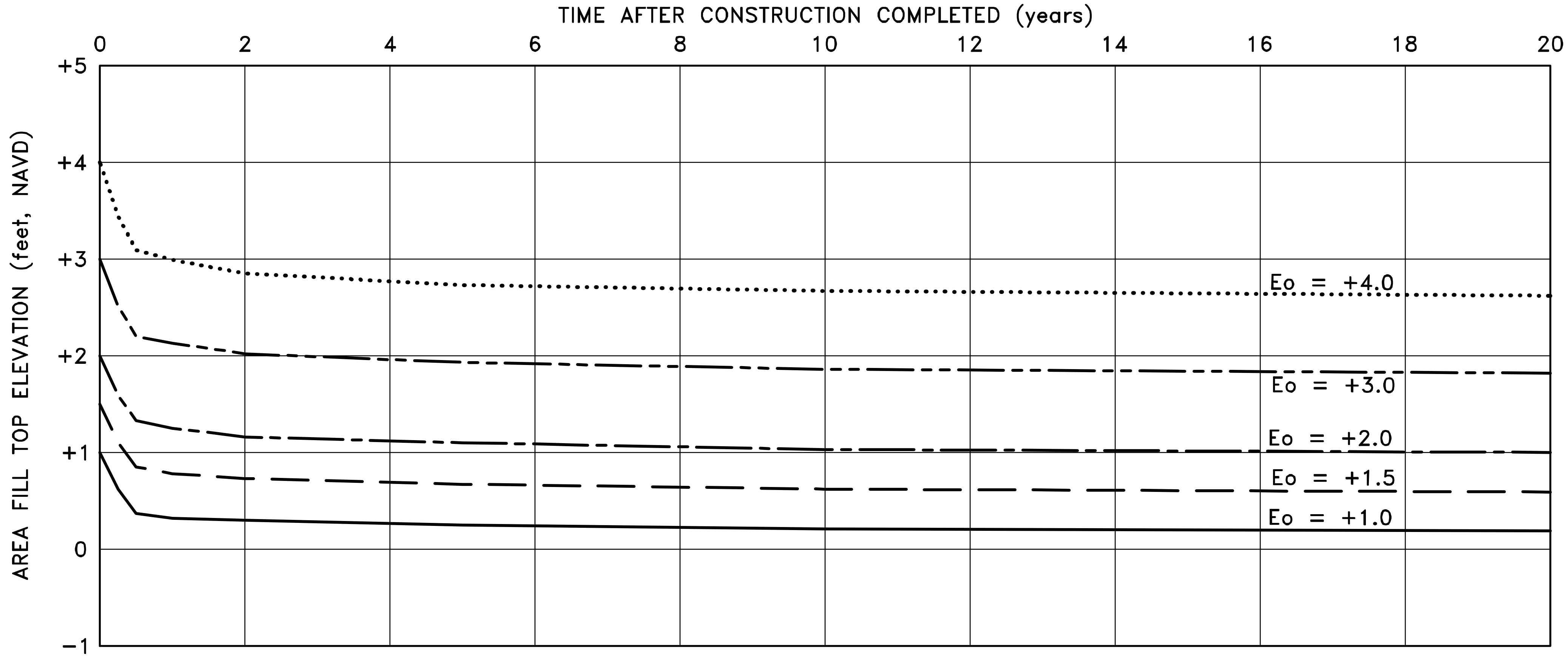
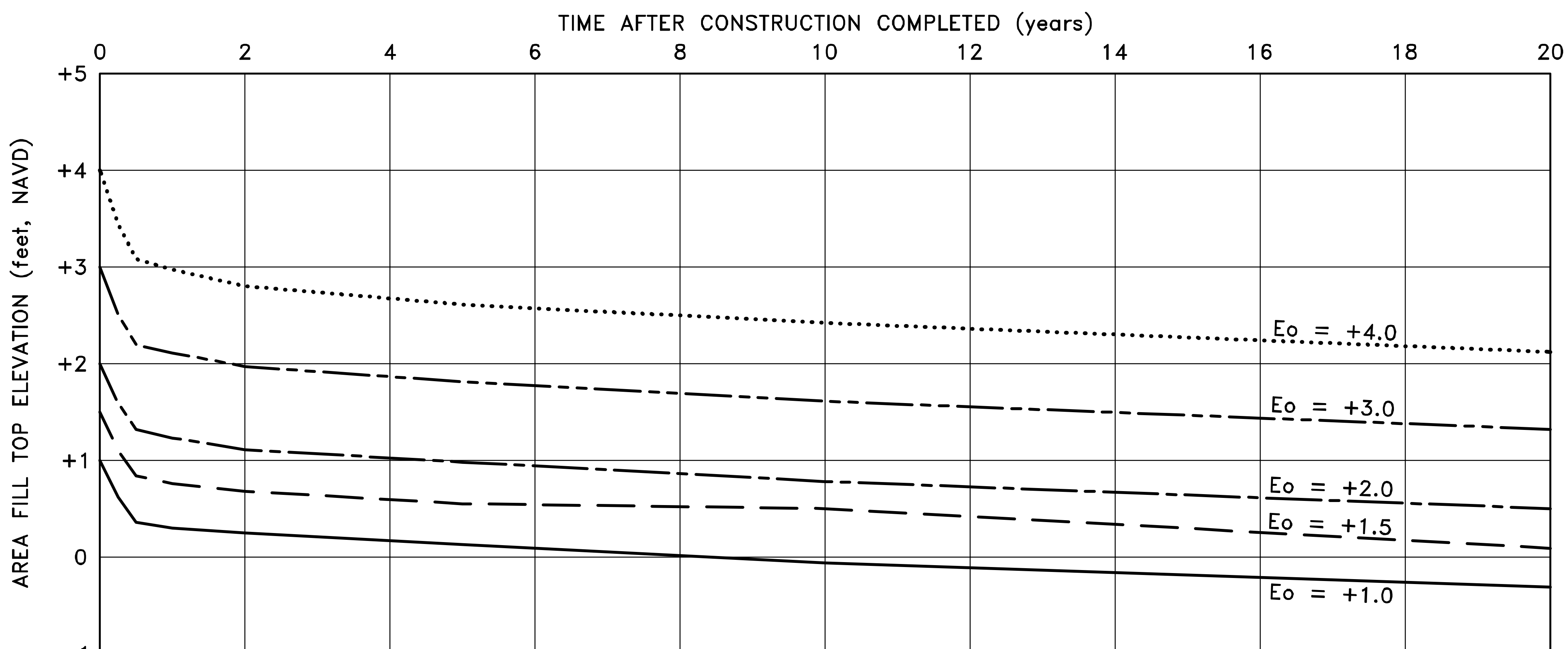
G.P. Boutwell	DMS	
File No.: 03-1033	Date: 11-10-03	Figure No.: 14

Title: **TIME-SETTLEMENT CURVES AREA FILL: WATER DEPTH 6 FT.**



WITHOUT GEOLOGIC SUBSIDENCE

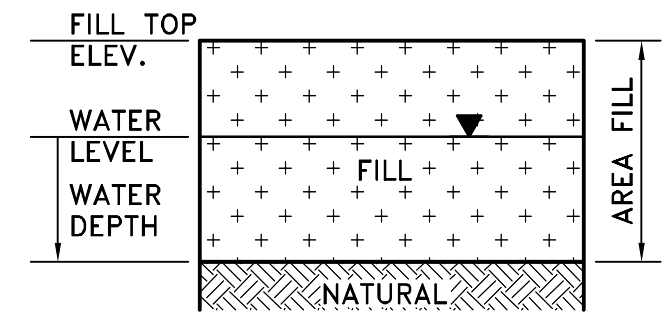


WITH GEOLOGIC SUBSIDENCE

NOTES:

1. Settlements include both self-weight consolidation and weight-induced settlements in foundation soils.

2. Geologic subsidence rate furnished by LDNR = 0.5 ft./20 yr.



PASS CHALAND TO GRAND BAYOU PASS
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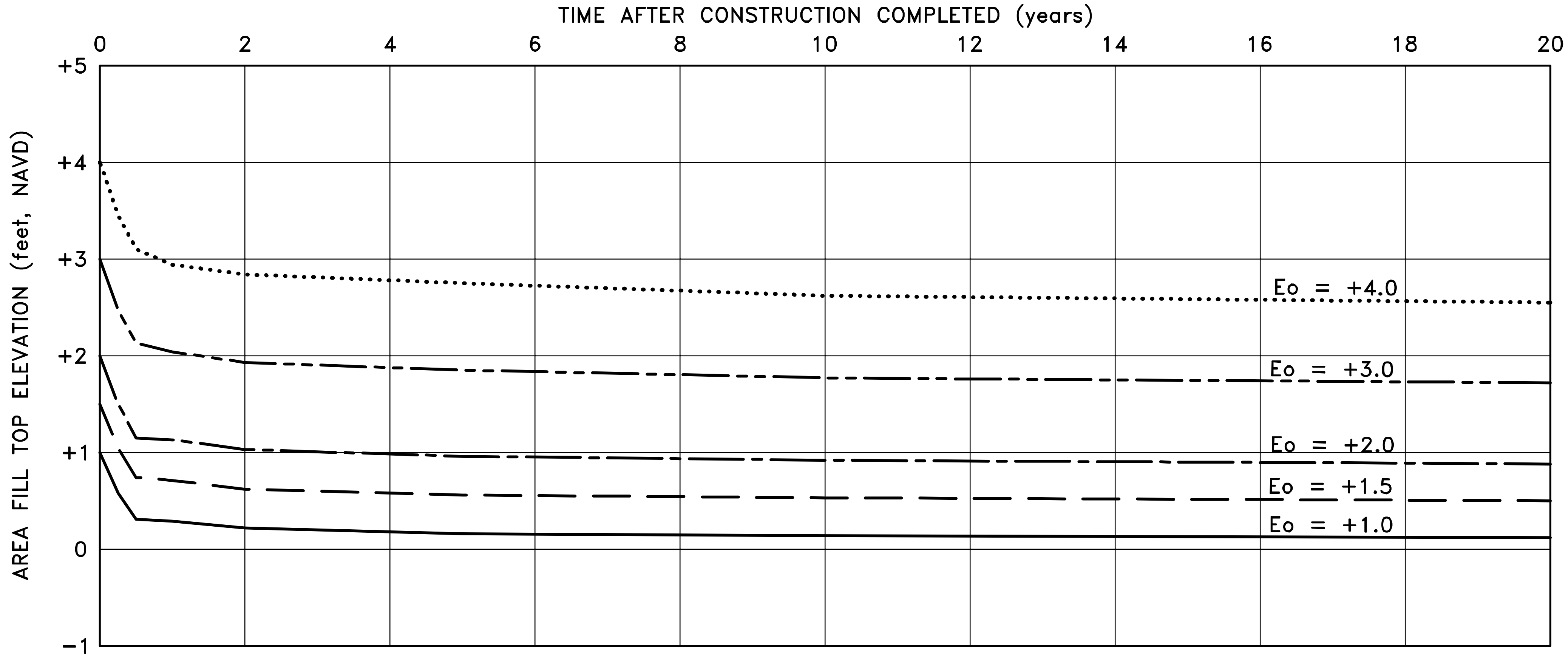
File No.: 03−1033	Date: 11−10−03	Figure No.: 15

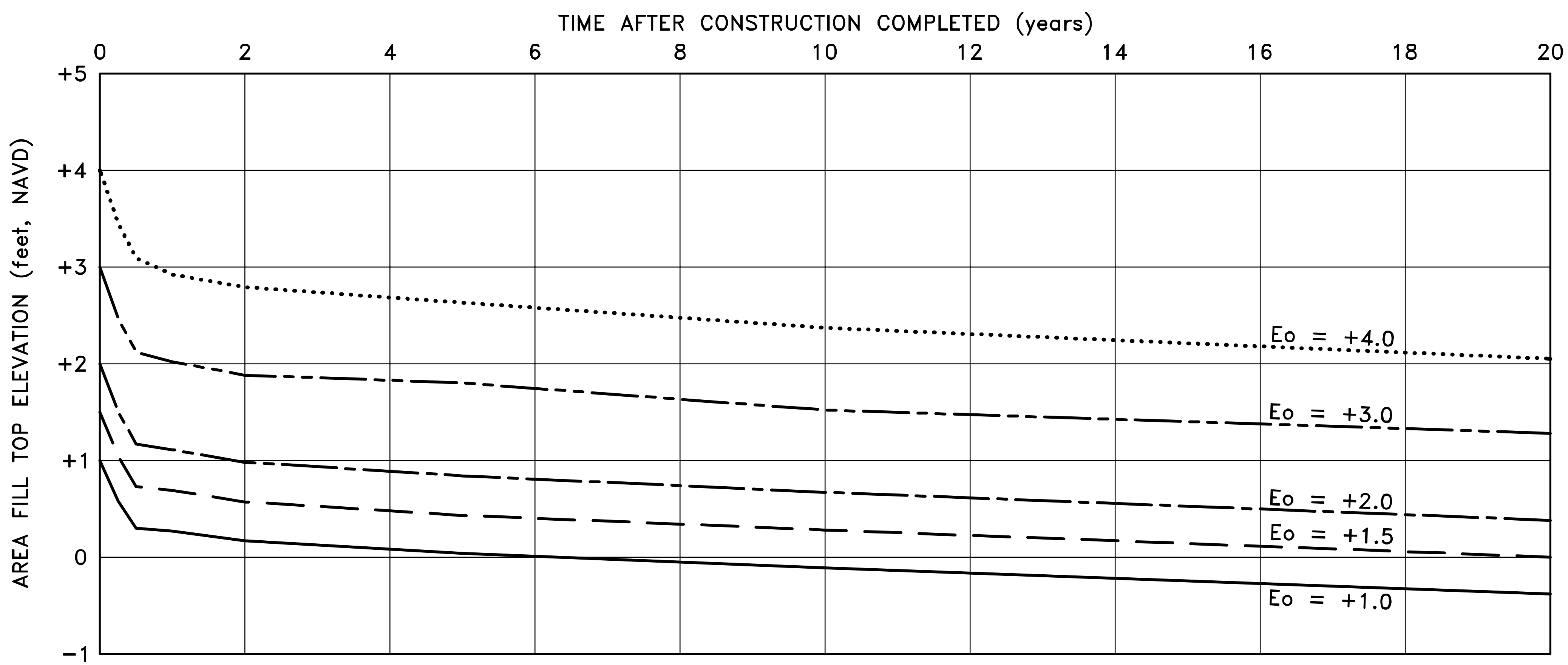
Title: **TIME-SETTLEMENT CURVES AREA FILL: WATER DEPTH 8 FT.**

Nov 14, 2003 − 10:12am

P:\2003\03−1033\CADD\031033−15 SETTLEMENT 8FT.DWG



WITHOUT GEOLOGIC SUBSIDENCE

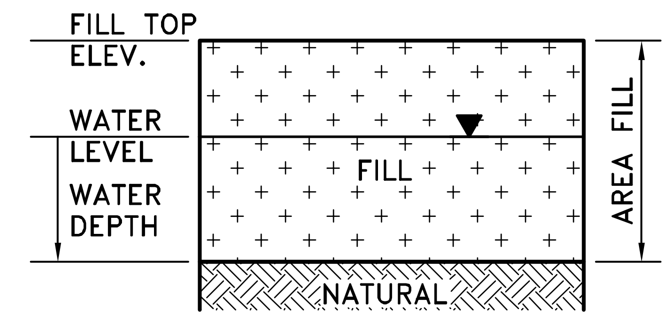


WITH GEOLOGIC SUBSIDENCE

NOTES:

1. Settlements include both self−weight consolidation and weight−induced settlements in foundation soils.

2. Geologic subsidence rate furnished by LDNR = 0.5 ft./20 yr.



PASS CHALAND TO GRAND BAYOU PASS
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G.P. Boutwell	DMS	
File No.: 03−1033	Date: 11−10−03	Figure No.: 16

Title: **TIME-SETTLEMENT CURVES AREA FILL: WATER DEPTH 10 FT.**

Nov 14, 2003 − 10:12am

P:\2003\03−1033\CADD\031033−16 SETTLEMENT 10FT.DWG

REPORT OF GEOTECHNICAL INVESTIGATION
PASS CHALAND TO GRAND BAYOU PASS
BARRIER SHORELINE RESTORATION
PROJECT BA-35
PLAQUEMINES PARISH, LOUISIANA

APPENDIX A

FIELD AND LABORATORY PROCEDURES

The following paragraphs describe the field and laboratory procedures used for this investigation Soil Boring Logs are included with this appendix. The boring logs, tables, and figures in this Appendix provide the field and laboratory data collected.

A.1 FIELD EXPLORATION

four soil borings were drilled for this project to a depth of 40 feet below water or ground surface. These borings were drilled on September 8, 2003. The approximate locations of the borings are shown on the Boring Plan, Figure 3. The locations were established and physically located by SJB Groups, Inc. and STE. The borings totaled 160 lineal feet, 40 feet of which were continuously sampled. Logs of the borings, corrected to reflect the laboratory test results, are attached to this Appendix.

A.1.1 Sampling Procedures - Undisturbed Samples. In these cohesive and semi-cohesive soils, relatively undisturbed samples were secured using a three-inch diameter, thin wall steel tube sampler, essentially following ASTM D1587. In this sampling procedure, the borehole is advanced to the desired level, and the tube is lowered to the bottom of the boring. It is then pushed about two feet into the undisturbed soil in one continuous stroke. The sample and tube are retrieved from the borehole and detached from the drill string. The tube is then sealed to minimize disturbance and moisture loss, and protected for transportation to the laboratory.

After any laboratory vane shear tests are performed, the samples are extruded in the laboratory by a hydraulic piston onto a rigid sample catcher to minimize disturbance. The sample is then visually classified. The classification includes description of soil color, strength estimates, identification of structural conditions (layering, seams, etc.) and variations (organics, oxide inclusions, etc.). A pocket penetrometer strength test is performed.

A.1.2 Sampling Procedures - Standard Penetration Tests. In the less cohesive materials, standard penetration tests were performed; these tests provide a measure of the in situ characteristics of the soil and secure a disturbed sample. In this test, a 2 inch OD, 1.37ID, heavy-walled "split Spoon" sampler is driven into the undisturbed soil at the bottom of the borehole with a drop hammer weighing 140 pounds and having a stroke of 30 inches. It is first seated 6 inches, then driven an additional two, 6-inch increments. The "Penetration Resistance" is the number of such blows required to drive the spoon the final 12 inches. It is recorded on the boring log in the following manner:

4 b/f
2-2-2

where the figures A-B-C indicate the number of blows required for each 6 inch increment.

A.1.3 Soil Classifications. The soil classifications are given on the attached logs. The materials' strength group, color, and material type are presented. The strength groups are in accordance with normal procedures as given in, e.g., *Mitchell (1993)*. The material type is based on the primary and secondary constituents (gravel, sand, silt, clay). The letters in parentheses represent the Unified Soil Classification (ASTM D2487 supplemented by ASTM D2488).

A.1.4 Grouting. Each borehole was grouted upon completion. The grout mixture was prepared in its own tub (not the mud tank used for drilling). The typical grout mix was 28 pounds of bentonite and 14 sacks of Portland cement per 100 gallons of water. After the grout was thoroughly mixed, it was pumped to the bottom of the borehole through the drill stem which was placed to the bottom of the hole. The grout mixture was circulated in the borehole to assure that the drilling fluid had been replaced with grout. After the circulation, the drill stem was withdrawn and grout fluid from the tub was used to replace the volume of the drill stem as it was withdrawn.

A.2 LABORATORY TESTING

The various types of laboratory testing performed on samples from the boring program are described below. The samples actually tested were selected by the Project Engineer to provide the information necessary for both evaluation of the soils and design.

A.2.1 Classification Testing - Atterberg Limits. These tests were necessary to determine the actual soil types more accurately than can be done by visual/manual methods. For cohesive soils, only Atterberg Limits Determinations were necessary. These parameters are used in classifying the semi-cohesive and cohesive materials, i.e., SC, ML, CL, CH, OL and OH under ASTM D2487. The actual procedure followed ASTM D4318; it consists of determining the water content corresponding to:

* Liquid Limit (LL) - Where the soil changes behavior from that of a plastic solid to that of a viscous liquid.

* Plastic Limit (PL) - Where the soil changes behavior from that of an elastic (rigid) solid to that of a plastic (deformable) solid.

* Plasticity Index (PI)- The difference between the above limits: PI = LL - PL.

Ten (10) of these tests were performed on the samples. The results are presented in the appropriate columns of the boring logs.

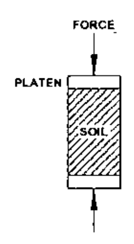
A.2.2 Classification - Particle Size Analyses. The information from these tests is used in classifying the less cohesive (more granular) soils such as PL and SM types. The test consists of two parts

- Sieve Analyses, where the sample is washed over progressively finer sieves, ending with the #200 (0.074 mm). The dry weight retained above each sieve is determined.

- Hydrometer Analysis, where the sample is suspended in water, and the particle sized are determined using the sedimentation rates and Stokes' Law.

This procedure is given in more detail in ASTM D422. There were two (2) such tests. Their results are summarized on the Logs of Borings, and presented graphically on Figures A-1 through A-6, which are attached to this Appendix A. In addition, there were twenty-four (24) such tests (Sieve only) on the samples obtained by SJB; their results are presented graphically on Figures A-1 through A-6. FiguresA-7 through A-14 presents the results of te partical size analyses on the grab samples taken by STE; these included six (6) tests with hydrometer analysis.

A.2.3 Strength Testing. The strength test program consisted of unconsolidated undrained (UU) triaxial compression tests. These tests provided data for slope stability analysis/design. In this test, a cylindrical sample (typically 3 inches in diameter and 6 inches high) is encased in a rubber membrane and then placed between two solid, flat end pieces ("platens"). Lateral pressure is applied to the sample by air pressure acting against the membrane. Stress is applied parallel to the long axis of the sample by advancing the end platens in a strain-controlled manner. Both the stress and corresponding axial strain are measured. The peak strength is the maximum axial stress measured before the axial strain reaches the commonly accepted value of 10%. These procedures conform essentially to ASTM D2850. A diagram for this test is given on the sketch below:



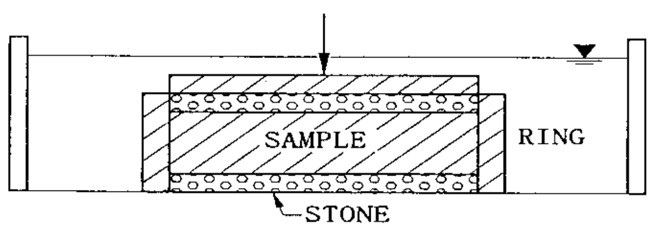
SKETCH A-1 - STRENGTH TESTING

Eighteen (18) of these tests were performed on the samples.

The results of these tests are presented in the appropriate columns of the attached boring logs.

A.2.4 Laboratory Vane Shear Testing. These tests were performed in the laboratory before the samples were extruded from their Shelby Tubes. The procedures are essentially those described for the field test in Section A.1.3, but a much smaller vane (1 inch blade diameter x 2 inches long) is used. This test conforms to the manufacture's recommendations and essentially to ASTM D4648. A total of 17 such tests was performed; their results are given on the attached Logs of Borings.

A.2.5 Consolidation Testing. These tests provide data on the compressibility and time-rate of settlement characteristics of the natural soils. In this test, a thin (0.8 inch) cylinder of the soil is trimmed into a 2.5 inch diameter, thick-walled ring. The sample and ring are submerged in water, and various one-dimensional vertical loads applied as illustrated in the sketch below.



SKETCH A-2 - CONSOLIDATION TESTING

Each load is maintained until 100% consolidation occurs under that load; the next load is then applied. This procedure conforms essentially to ASTM D2435. The results of the five (5) such tests performed on the samples are presented graphically on the Consolidation Test Figures A-17 through A-21, and are summarized on Table A-1. The consolidation tests from the Complex Project were also used in our analyses and are presented in Figures A-22 through A-35. In addition, Specific Gravity Determinations were made for all samples subjected to Consolidation Testing. The procedures conformed essentially to ASTM D854.

A.2.6 Water Content Determination. This test consists of determining the weight of a soil sample before and after it is dried some 24 hours in an oven maintained at 105 degrees C. The water content is the loss in weight divided by the dry weight of the sample. This procedure conforms essentially to ASTM D2216. There were twenty-six (26) such tests on samples from boreholes; their results are given on the attached Logs of Borings. In addition, twenty-four (24) water content determinations were made on the samples obtained by SJB Groups; see Table 4.2-1 of the main test for the results.

A.2.7 Organic Content Determination. This procedure consists of first drying the soils as utlined in Section A.2.6, then heating the dried soils to 440 degrees C. The weight loss in the second heating is divided by the final dry weight of soil to obtain the organic content. This procedure conforms essentially to ASTM D2974. There were twenty-four (24) such tests on samples obtained by SJB, and six (6) on grab samples collected by STE.

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PASS CHALAND TO GRAND BAYOU PASS
BARRIER SHORELINE RESTORATION
PROJECT BA-35
PLAQUEMINE PARISH, LA
DNR CONTRACT NO. 2511-03-09
TABLE A-1
SUMMARY OF CONSOLIDATION TESTS

BORING NO.	DEPTH (feet)	LL (%)	PI (%)	Wo (%)	DDo (pcf)	Pc (tsf)	C'c (decimal)	SOIL TYPE
BJW-1	9-11	49	29	37	81	0.56	0.11	CL-CH
BJW-1	38-40	87	57	54	65	1.10	0.23	CH
BJW-2	23-25	60	38	39	74	0.67	0.15	CH
BLW-4	11-13	-	NP	26	93	1.80	0.04	SM
BLW-4	38-40	71	37	46	74	1.10	0.11	CH

LL = Liquid Limit (D4318)
PI = Plas. Index (D4318)
W_o = Initial Water Content (D2216)
DD_o = Initial Dry Density (D2937)

P_c = Preconsolidation Pressure (Casagrande Method)
C'_c = Compression Index = $C_c/(1+e_o)$
NP = Non-Plastic

DESCRIPTION OF TERMS AND SYMBOLS USED ON SOIL BORING LOG

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FIELD DATA				LABORATORY DATA								
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	PL	PI	Other	Soil Type	DESCRIPTION

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40

Field test results shown: 3.5 (P); 13 b/f (3-7-6)

Ground Water Levels

Long-Term Depth
Depth to water after boring is completed (time noted).

Short-Term Depth
Depth to water after initial water encountered prior to proceeding with boring (time noted).

Initially Encountered
Depth where free water was initially encountered during augering.

Sampling/Field Data

Undisturbed
3" dia. Tube sample

Pocket Penetrometer (P)
Penetration resistance (tons/sq. ft.).

Torvane (T)
Shearing resistance (tons/sq. ft.)

Split Spoon
Std. penetration test

Std. Penetration
No. of blows per foot (blows per each six inch increments).

Auger
Disturbed (auger) collected in accordance with ASTM D-1452.

No Recovery
Sampling attempted but no sample retrieved.

Description

Classifications are based on visual observations by field & lab representatives as well as results of laboratory data (when available).

Laboratory Data

Compressive Strength
Value based on peak compressive strength. Determined by unconfined compression test unless otherwise noted.

Dry Unit Weight
As determined by method similar to ASTM D-2937.

Water Content
As determined by pertinent portions of ASTM D-2216.

Atterberg Limits
LL : Liquid Limit
PL : Plastic Limit
PI : Plasticity Index
(= Liquid Limit - Plastic Limit)

Other
Results of other tests such as consolidation, permeability, grain size or notes associated with testing program.

Soil Type

Graphical representation of soil type. In accordance with USCS Symbols.

Ground Water Level Data	Boring Advancement Method	Notes
	Boring Abandonment Method	

Form LOGTERMS

Strata Boundaries May Not Be Exact

Pass Chaland to Grand Bayou Pass
Barrier Shoreline Restoration
Project BA-35
Plaquemines Parish, LA

SJB Group, Inc.
Baton Rouge, LA

LOG OF SOIL BORING BJW-1

STE
Soil Testing Engineers, Inc.
Sheet 1 of 1
LELAP Certificate No. 02052

File: 03-1033
Date: 09/08/03
Logged by: K. Moody
Driller: MASA
Rig: Barge

Location: Lat. 29° 18' 18.1"
Long. 89° 43' 06.4"
Mudline at -5.0 NAVD
Surface Elevation: N/A (ft., NAVD)

Ground Water Level	Depth (feet)	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	LL	PL	PI	Mini Vane Shear (ksf) / Other	Description
		No (P)	0.26t1	38	79	49	20	29	0.17 CS,SG1	Soft gray SLIGHTLY SILTY CLAY (CL-CH) -- w/shell fragments at 9 to 11 ft.
		No (P)								
	5			41		37	20	17		Very soft gray SILTY CLAY (CL) -- w/shells at 13 to 15 ft.
		1.0 (P)	0.15t2	40	80				0.13	
		7 b/f 5-3-4								Medium gray SILTY CLAY (CL)
	10									
	15	5 b/f 5-3-2		34		33	22	11		-- w/shells at 23 to 25 ft.
										Soft gray CLAY (CH)
	20	3 b/f 1-1-2								
	25	0.75 (P)	0.32t3	66	59				0.27	
	30	0.5 (P)	0.42t4	63	61	87	30	57	0.27 CS,SG2	-- dark gray at 38 to 40 ft.
										Boring completed at 31 ft.
	35									
	40									

Ground Water Level Data

Deck to Water 5 ft.
Water to Mudline 9 ft.

Boring Advancement Method

4" Dia. Rotary Wash:
0 to 31 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
t1: Lateral Pressure = 10.5 psi
t2: Lateral Pressure = 14.7 psi
t3: Lateral Pressure = 27.3 psi
t4: Lateral Pressure = 30.8 psi
CS: See Consolidation Curve
SG: Specific Gravity
SG1 = 2.58
GS2 = 2.59

Strata Boundaries May Not Be Exact

03-1033 LOG01 PASS CHALAND LDNR 11-14-2003 031033R.GPJ LOG01.GDT 12/23/03

Pass Chaland to Grand Bayou Pass
Barrier Shoreline Restoration
Project BA-35
Plaquemines Parish, LA

SJB Group, Inc.
Baton Rouge, LA

LOG OF SOIL BORING BJW-2

STE
Soil Testing Engineers, Inc.

Sheet 1 of 1

LELAP Certificate No. 02052

File: 03-1033
Date: 09/08/03
Logged by: K. Moody
Driller: MASA
Rig: Barge

Location: Lat. 29° 18' 14.5"
Long. 89° 42' 32.6"
Mudline at -2.1 NAVD
Surface Elevation: N/A (ft., NAVD)

FIELD DATA				LABORATORY DATA								
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	PL	PI	Mini Vane Shear (ksf) / Other	Soil Type	Description
	0–2	X	No (P)	0.08t1	40	74	58	22	36			Very soft dark gray CLAY (CH) w/sand seams and mica flakes
	2–6 (5)	X	No (P)		29							Soft gray VERY SILTY CLAY (CL-ML) -- w/shells at 11 to 13 ft.
	6–8	X	No (P)									Loose gray SANDY SILT (ML)
	8–13 (10)	X	No (P)		47					GS		Very soft gray SILTY CLAY (CL)
	13–25 (15, 20)	X	1.25 (P)	0.53t2	41	90	60	22	38	CS,SG		Medium gray CLAY (CH) -- w/sand seams at 23 to 25 ft.
		X	1.0 (P)									-- w/silt seams at 28 to 30 ft.
	25	X	0.5 (P)	0.20t3	52	64				0.19		
	25–33 (30)	X	1.25 (P)	0.17t4	63	64				0.23		Very soft gray CLAY (CH) -- w/silt seams at 33 to 35 ft. -- w/1-inch sand layers at 38 to 40 ft.
	33–40 (35, 40)											Boring completed at 33 ft.

Ground Water Level Data

Deck to Water 5 ft.
Water to Mudline 7 ft.

Boring Advancement Method

4" Dia. Rotary Wash:
0 to 33 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
t1: Lateral Pressure = 9.1 psi
t2: Lateral Pressure = 20.3 psi
t3: Lateral Pressure = 27.3 psi
t4: Lateral Pressure = 30.8 psi
GS: Particle Size Analysis
Gravel = 0%, Sand = 3%, Silt = 54%, Clay = 43%
CS: See Consolidation Curve
SG: Specific Gravity = 2.60

Strata Boundaries May Not Be Exact

Pass Chaland to Grand Bayou Pass
Barrier Shoreline Restoration
Project BA-35
Plaquemines Parish, LA

SJB Group, Inc.
Baton Rouge, LA

LOG OF SOIL BORING BJW-3

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Soil Testing Engineers, Inc.

LELAP Certificate No. 02052

File: 03-1033
Date: 09/08/03
Logged by: K. Moody
Driller: MASA
Rig: Barge

Location: Lat. 29° 18' 05.7"
Long. 89° 41' 59.9"
Mudline at -1.5 NAVD
Surface Elevation: N/A (ft., NAVD)

FIELD DATA				LABORATORY DATA			Atterberg Limits				Soil Type	Description
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	LL	PL	PI	Mini Vane Shear (ksf) / Other		
											CL	Very soft dark gray SILTY CLAY (CL)
			0.5 (P)	0.21t1	44	71	48	24	24	0.19		
	5		No (P) 1.0 (P)	0.15t2	46	72						-- w/6-inch sand layer at 11.0 to 11.5 ft. -- w/silt seams at 12.0 to 12.5 ft.
			0.0 (P)		56						ML	Loose gray SANDY SILT (ML)
			No (P)									
	10											
											CH	Soft gray CLAY (CH)
	15											
			0.75 (P)	0.35t3	69	58				0.25		-- w/sand seams at 23 to 25 ft.
	20											
			0.5 (P)		59					0.21		-- w/silt seams at 28 to 30 ft.
	25											
			0.75 (P)	0.33t4	57	65				0.20		-- w/sand seams at 33 to 35 ft.
	30											
			0.5 (P)							0.28		
												Boring completed at 33 ft.
	35											
	40											

Ground Water Level Data

Deck to Water 5 ft.
Water to Mudline 7 ft.

Boring Advancement Method

4" Dia. Rotary Wash:
0 to 33 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
t1: Lateral Pressure = 10.5 psi
t2: Lateral Pressure = 12.3 psi
t3: Lateral Pressure = 20.3 psi
t4: Lateral Pressure = 27.3 psi

Strata Boundaries May Not Be Exact

Pass Chaland to Grand Bayou Pass
Barrier Shoreline Restoration
Project BA-35
Plaquemines Parish, LA

SJB Group, Inc.
Baton Rouge, LA

LOG OF SOIL BORING BJW-4

STE
Soil Testing Engineers, Inc.

Sheet 1 of 1

LELAP Certificate No. 02052

File: 03-1033
Date: 09/08/03
Logged by: K. Moody
Driller: MASA
Rig: Barge

Location: Lat. 29° 17' 56.1"
Long. 89° 41' 27.9"
Mudline at -2.9 NAVD
Surface Elevation: N/A (ft., NAVD)

Depth (feet)	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	LL	PL	PI	Mini Vane Shear (ksf) / Other	Description
	0.0 (P)	0.09t1	31	84	32	19	13		Very soft gray SANDY CLAY (CL) w/shells
	0.0 (P)		32						
5	0.25 (P)	0.16t2	33	82				0.15 CS,SG1 GS	Loose gray SILTY SAND (SM) w/clay layers
	0.25 (P)		59		54	27	27	0.17	Very soft gray CLAY (CH)
		0.13t3	41	70					-- w/sand seams and organic matter at 15 to 17 ft.
10									
									Soft dark gray SILTY CLAY (CL)
15									
	0.5 (P)	0.29t4	41	74				0.15	
20									Soft gray CLAY (CH)
	0.25 (P)		56					0.27	-- w/sand pockets at 28 to 30 ft.
25									
	0.25 (P)	0.25t5	59	65				0.25	-- w/sand seams at 33 to 35 ft.
30									
	0.5 (P)	0.27t6	46	69	71	34	37	0.25 CS,SG2	
									Boring completed at 33 ft.
35									
40									

Ground Water Level Data

Deck to Water 5 ft.
Water to Mudline 7 ft.

Boring Advancement Method

4" Dia. Rotary Wash:
0 to 33 ft.

Boring Abandonment Method

Borehole grouted upon completion

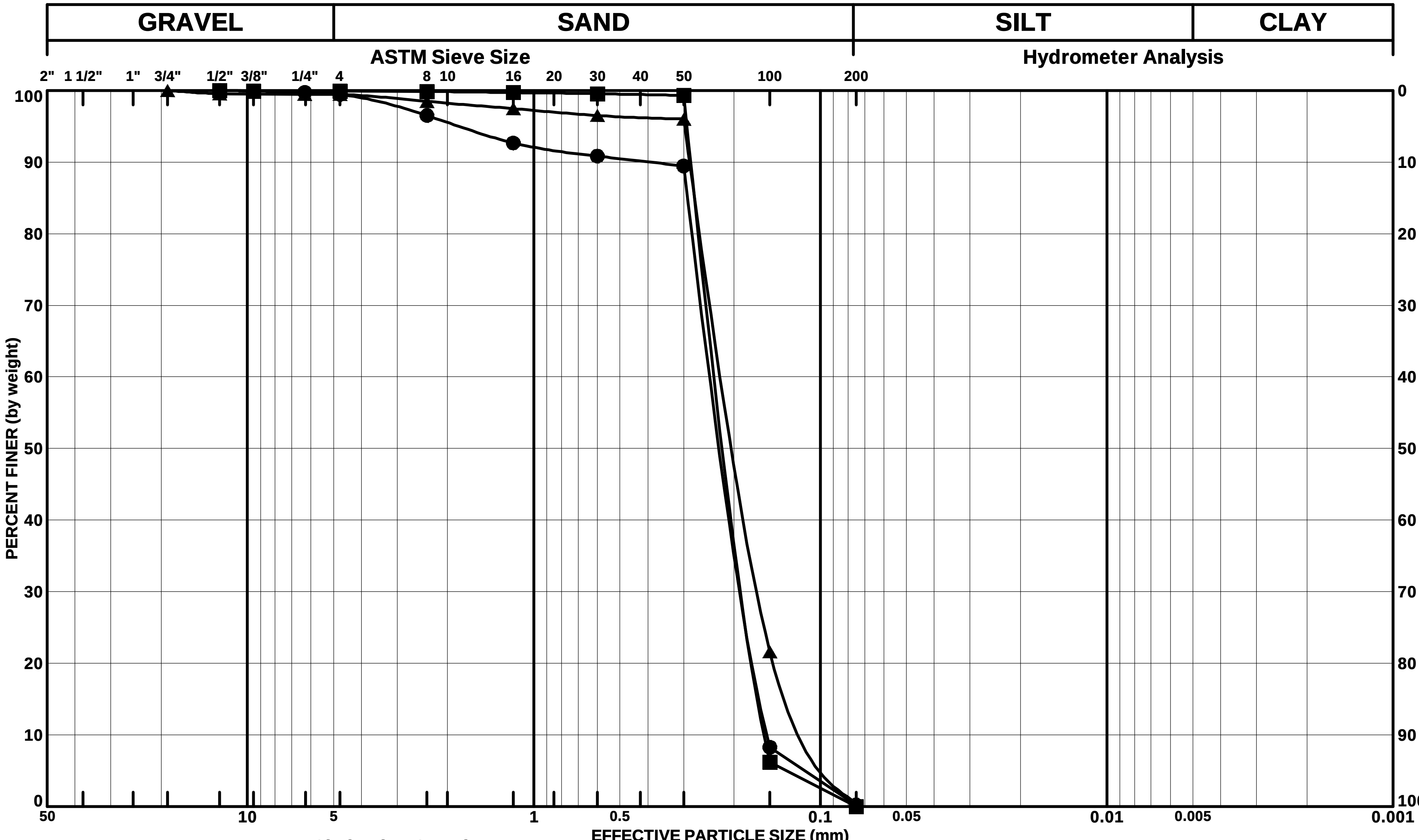
Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure
t1 = 9.1 psi t2 = 11.9 psi t3 = 14.7 psi
t4 = 20.3 psi t5 = 27.3 psi t6 = 30.8 psi
CS: See Consolidation Curve
SG: Specific Gravity
SG1 = 2.71
SG2 = 2.70
GS: Particle Size Analysis
Gravel = 0%, Sand = 77%, Silt = 13%, Clay = 10%

Strata Boundaries May Not Be Exact

LELAP Certificate No. 02052

Sample No.	Identification	Gravel (%)	Sand (%)
● N-1	Beach	0.6	99.2
■ N-1	Dune Sand	0.1	99.9
▲ N-1	Near Shore	0.6	98.9



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Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

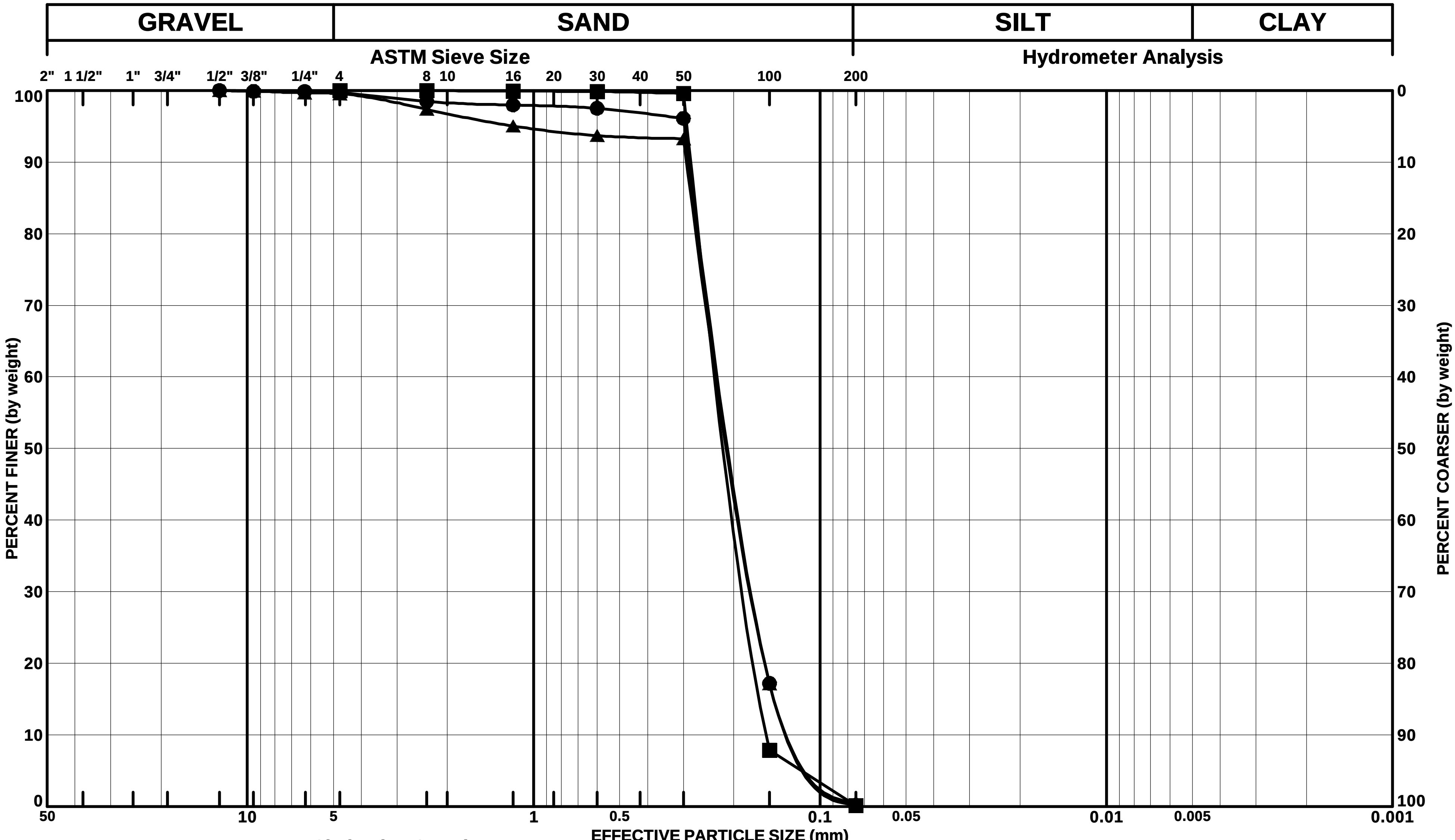
GRAIN SIZE ANALYSIS

Figure No.: A-1
File No.: 03-1033
Date: 10/09/2003

GRAINSZ01 031033R.GPJ GRAINSZ.GDT 11/14/03

Sample No.	Identification	Gravel (%)	Sand (%)
● N-2	Beach	0.3	99.5
■ N-2	Dune Sand	0.0	99.8
▲ N-2	Near Shore	0.5	99.0

LELAP Certificate No. 02052



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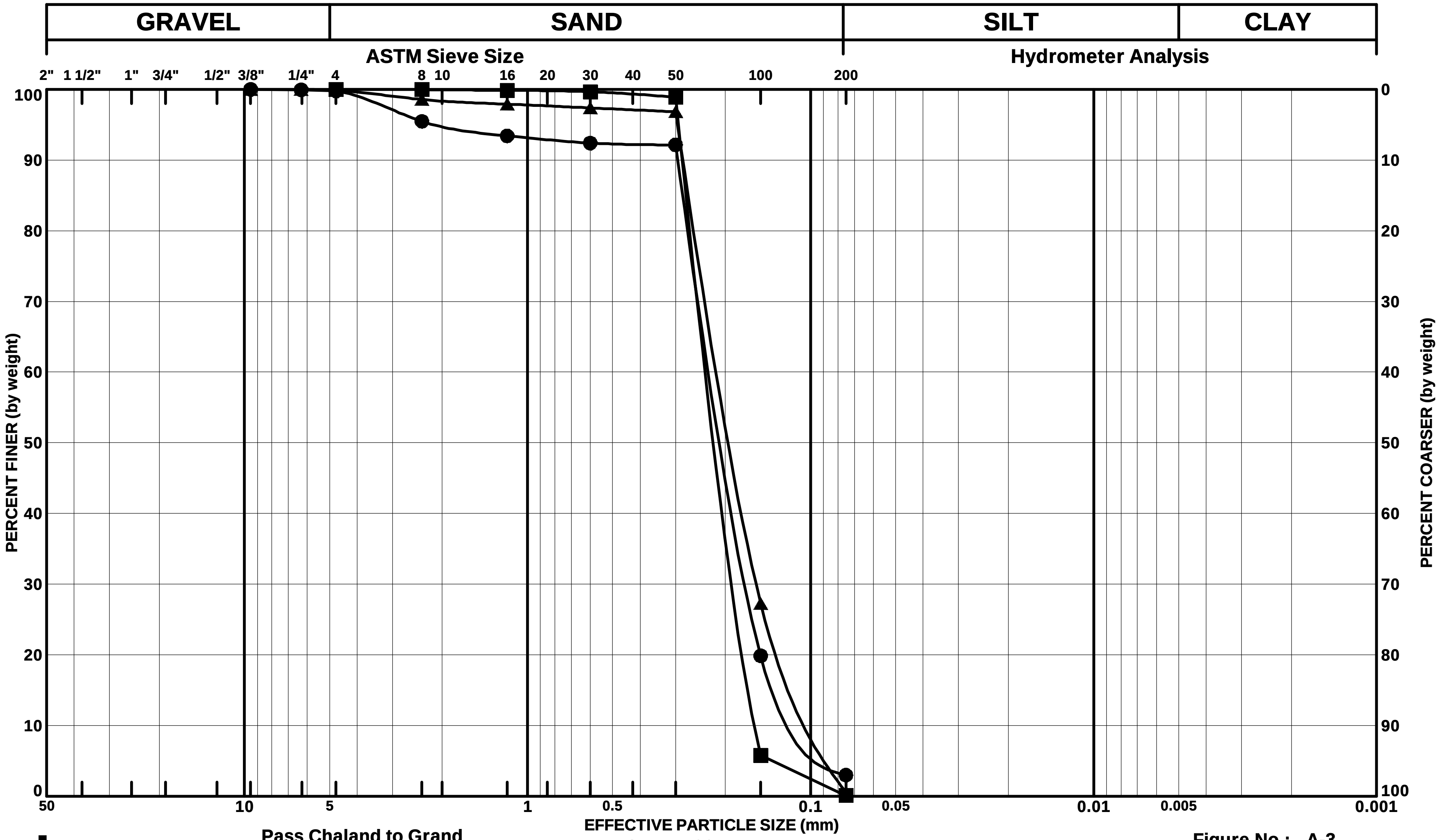
Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

GRAIN SIZE ANALYSIS

Figure No.: A-2
File No.: 03-1033
Date: 10/09/2003

LELAP Certificate No. 02052

Sample No.	Identification	Gravel (%)	Sand (%)
● N-3	Beach	0.3	96.8
■ N-3	Dune Sand	0.0	99.9
▲ N-3	Near Shore	0.2	99.3



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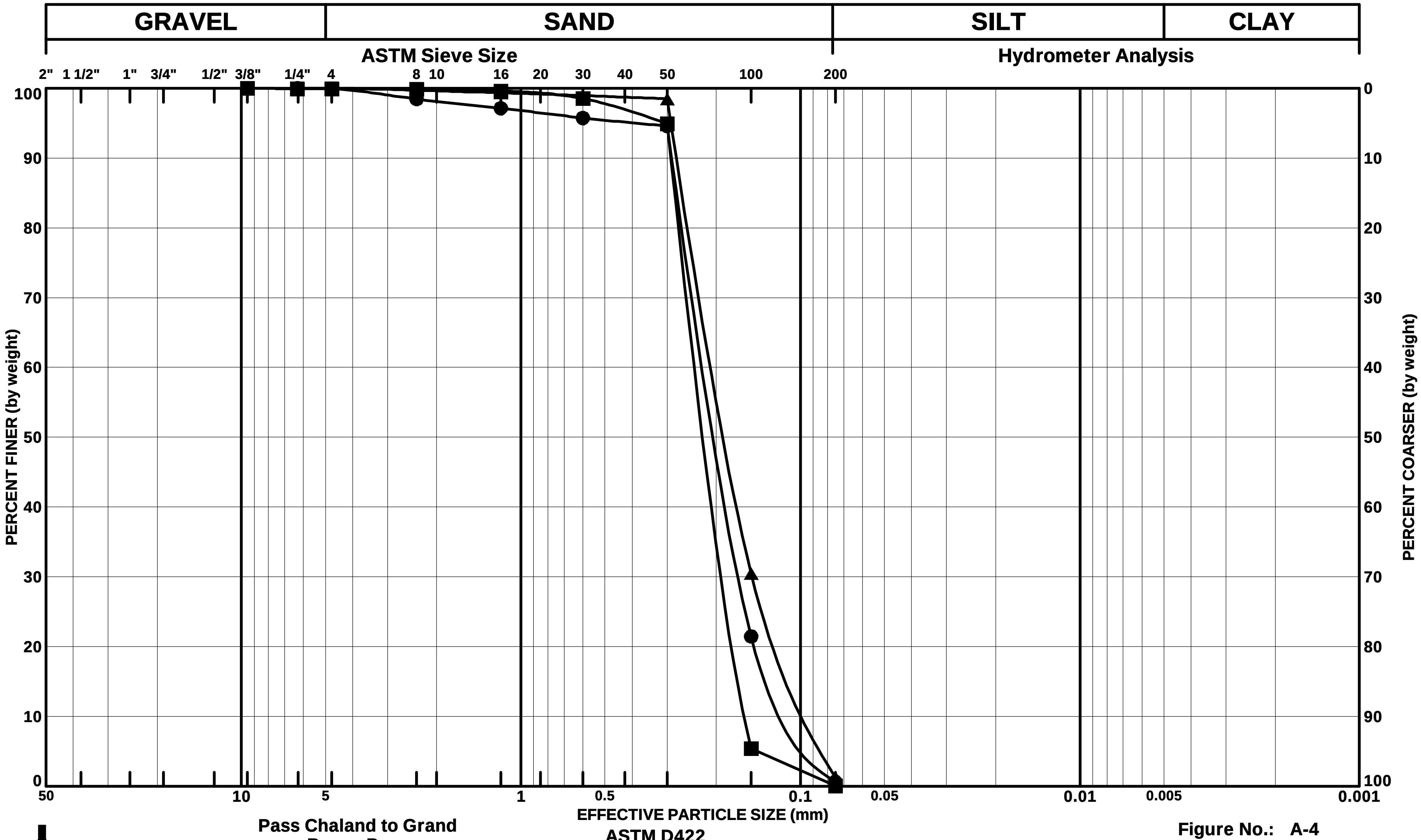
Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

GRAIN SIZE ANALYSIS

Figure No.: A-3
File No.: 03-1033
Date: 10/09/2003

LELAP Certificate No. 02052

Sample No.	Identification	Gravel (%)	Sand (%)
● N-4	Beach	0.1	99.3
■ N-4	Dune Sand	0.1	99.9
▲ N-4	Near Shore	0.0	98.7



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Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

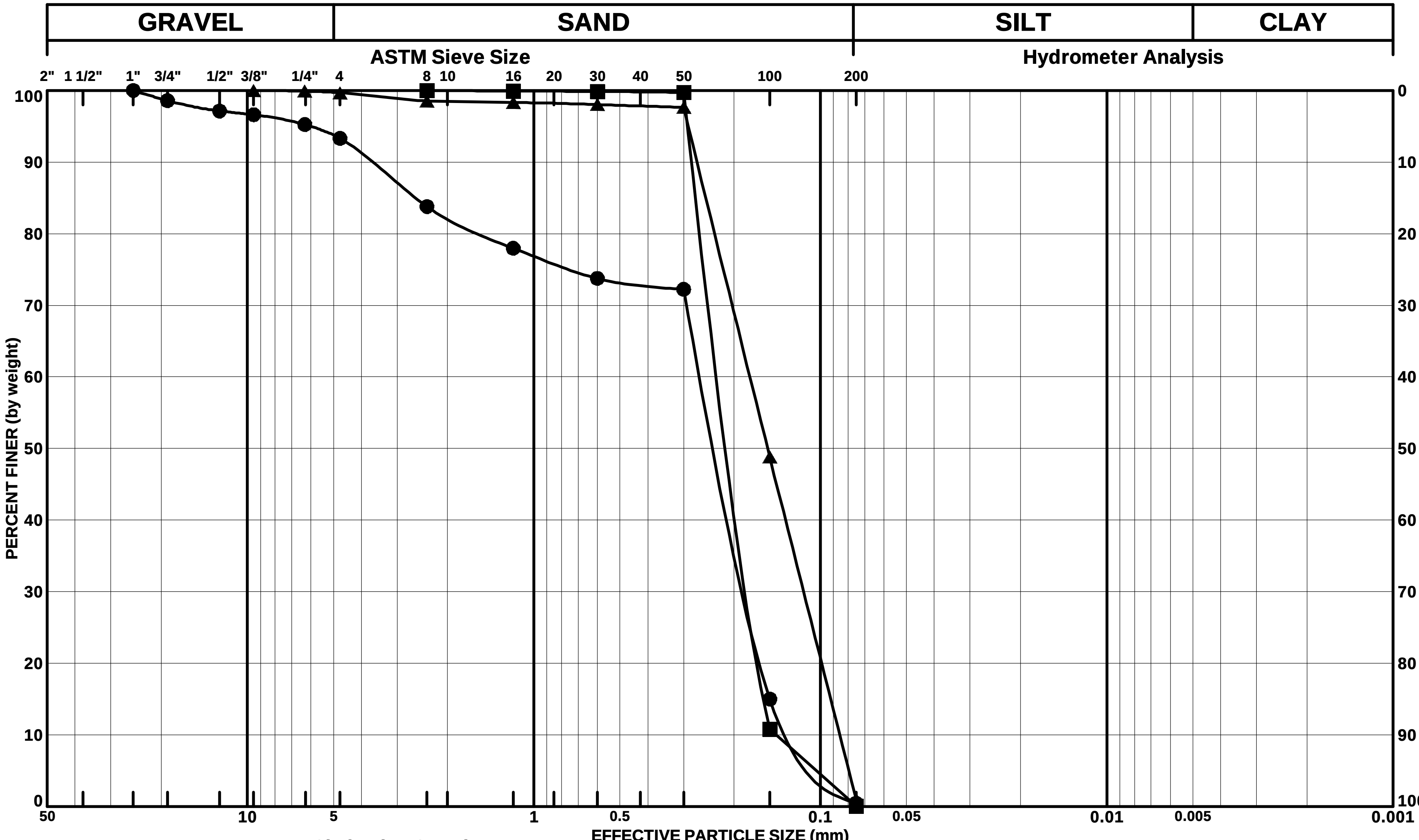
GRAIN SIZE ANALYSIS

Figure No.: A-4
File No.: 03-1033
Date: 10/09/2003

GRAINSZ01 031033R.GPJ GRAINSZ.GDT 11/14/03

Sample No.	Identification	Gravel (%)	Sand (%)
● N-5	Beach	6.7	92.8
■ N-5	Dune Sand	0.0	99.9
▲ N-5	Near Shore	0.3	98.6

LELAP Certificate No. 02052



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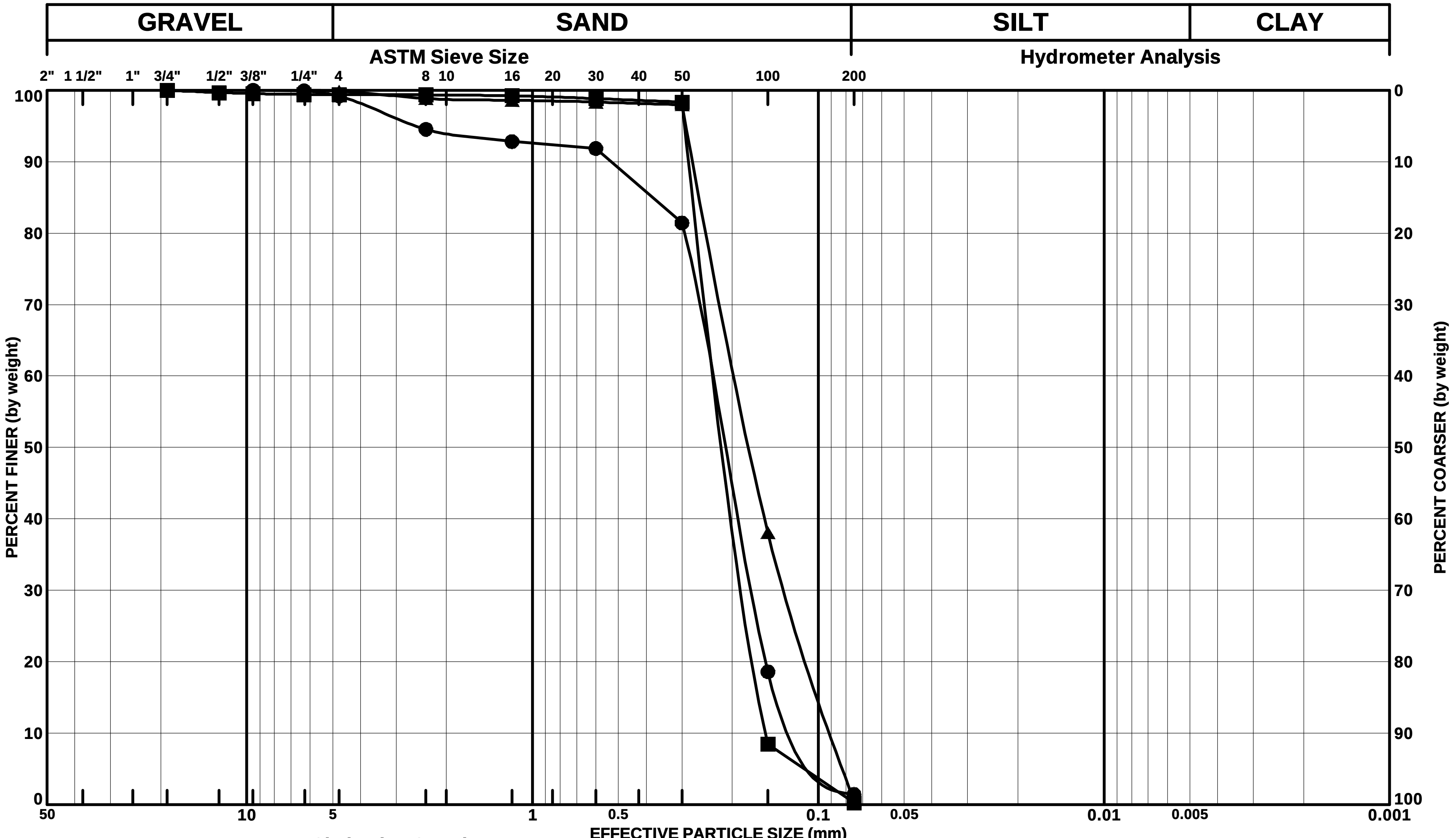
Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

GRAIN SIZE ANALYSIS

Figure No.: A-5
File No.: 03-1033
Date: 10/09/2003

Sample No.	Identification	Gravel (%)	Sand (%)
● N-6	Beach	0.8	97.7
■ N-6	Dune Sand	0.6	99.1
▲ N-6	Near Shore	0.1	99.2

LELAP Certificate No. 02052



ASTM D422

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Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

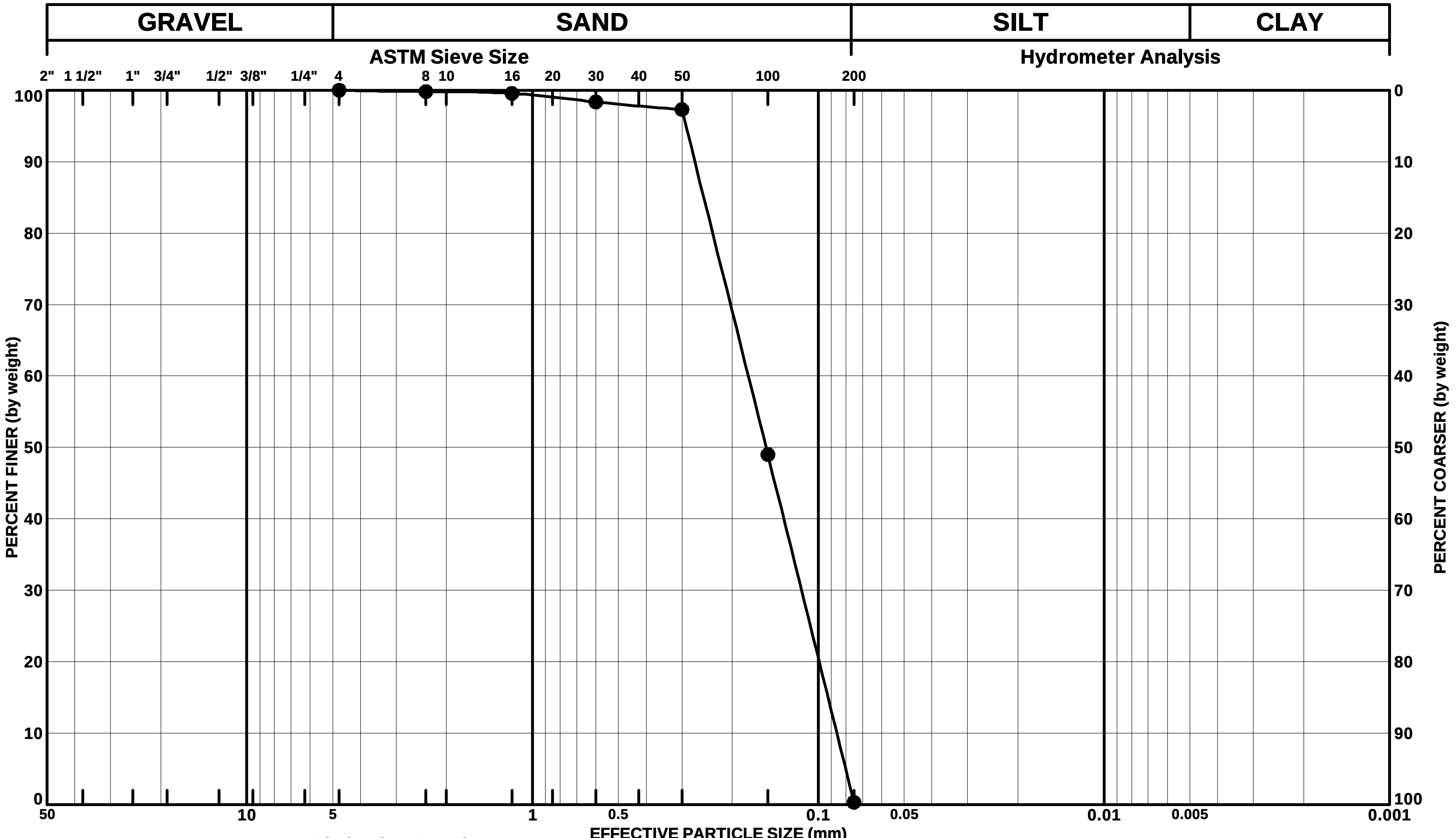
GRAIN SIZE ANALYSIS

Figure No.: A-6
File No.: 03-1033
Date: 10/09/2003

GRAINSZ01 031033R.GPJ GRAINSZ.GDT 11/14/03

Sample No.	Identification	Gravel (%)	Sand (%)
● Beach Sand	Silty Sand	0.0	99.7

LELAP Certificate No. 02052



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Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

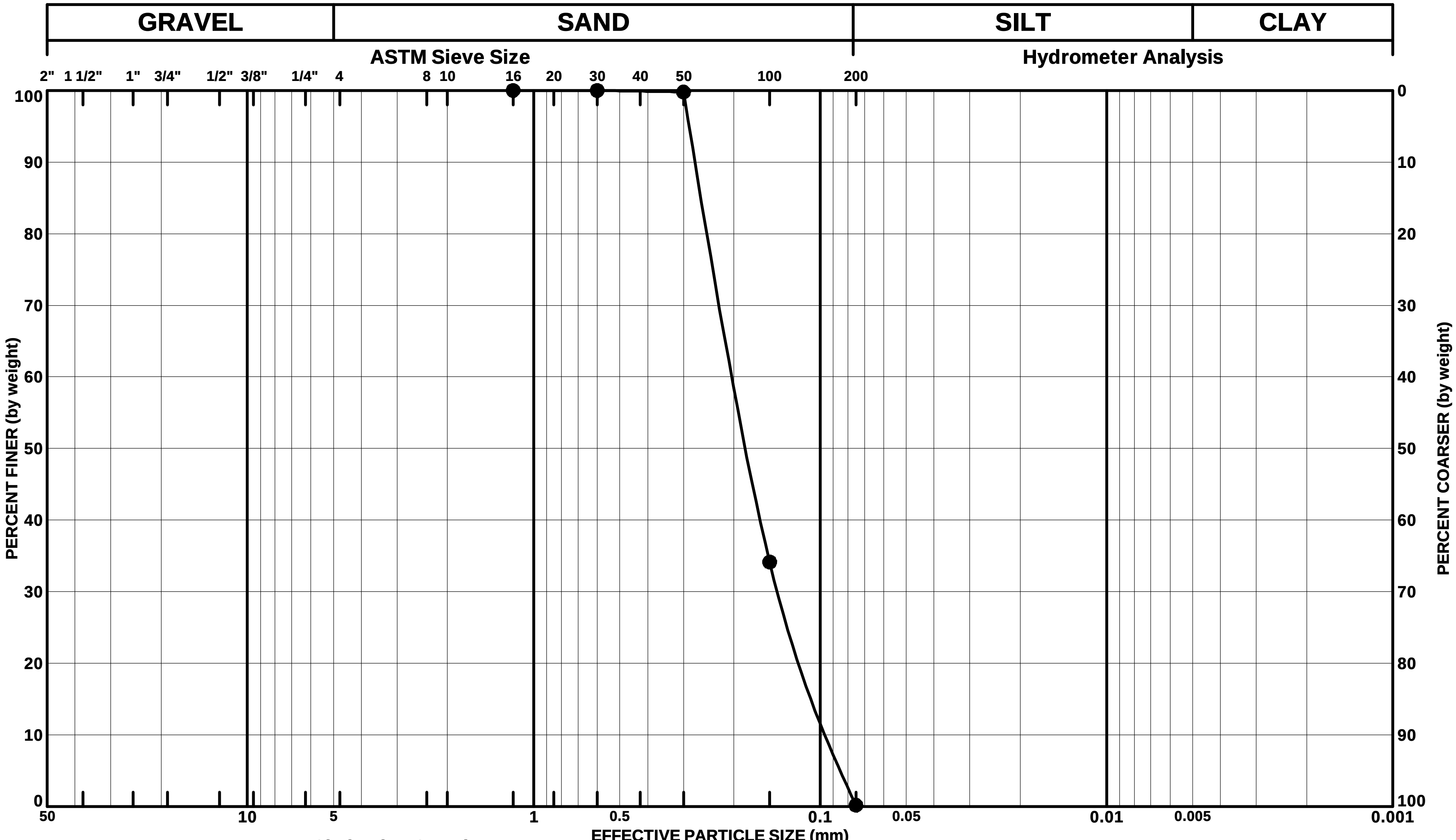
GRAIN SIZE ANALYSIS

Figure No.: A-7
File No.: 03-1033
Date: 08/03/2003

GRAINSZ01 031033R.GPJ GRAINSZ.GDT 11/14/03

LELAP Certificate No. 02052

Sample No.	Identification	Gravel (%)	Sand (%)
● Dune Sand	Sand	0.0	99.8



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Soil Testing Engineers, Inc.

Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

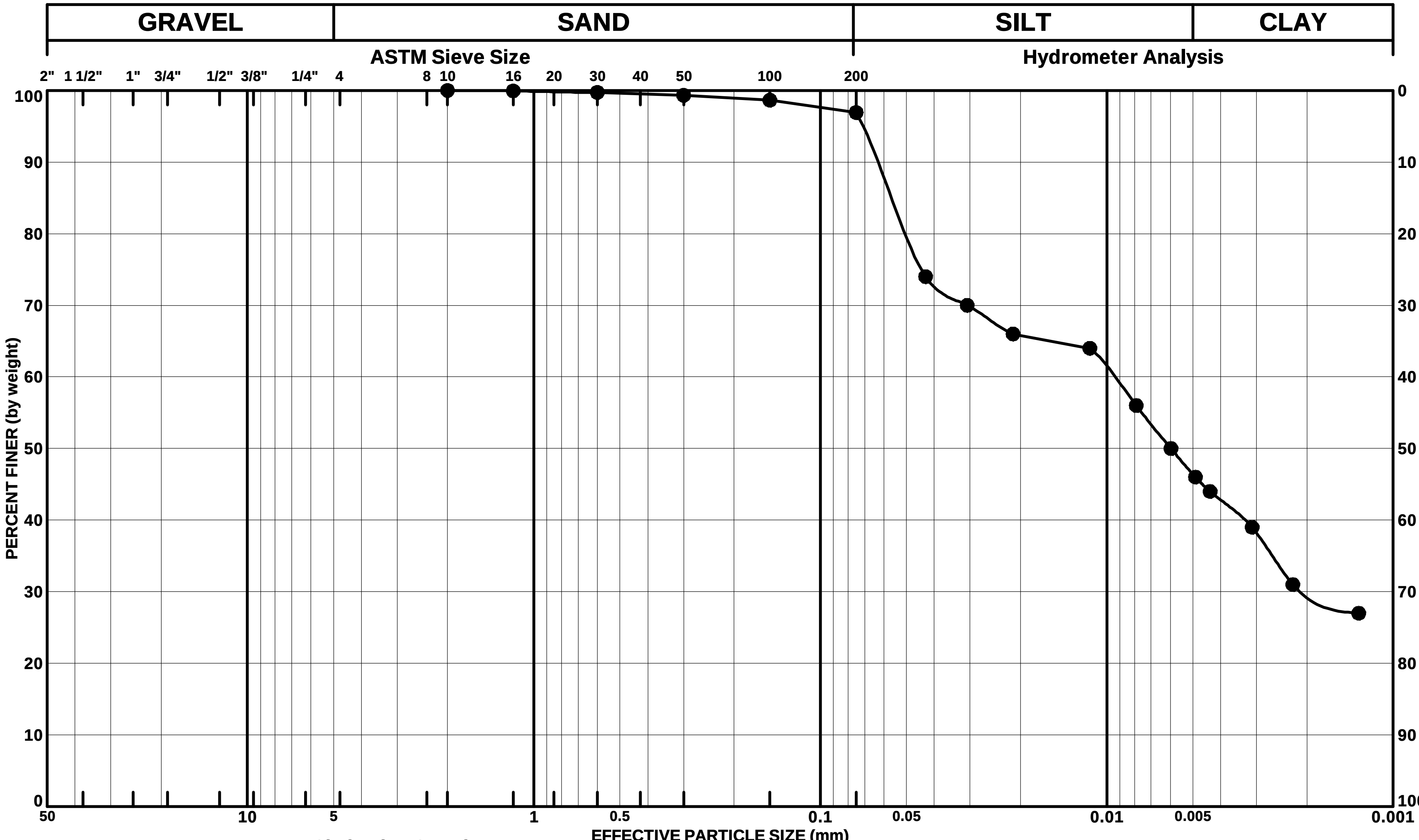
GRAIN SIZE ANALYSIS

Figure No.: A-8
File No.: 03-1033
Date: 08/03/2003

GRAINSZ01 031033R.GPJ GRAINSZ.GDT 11/14/03

LELAP Certificate No. 02052

Sample No.	Identification	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
● Marsh	Marsh	0.0	3.1	50.5	46.4



ASTM D422

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Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

GRAIN SIZE ANALYSIS

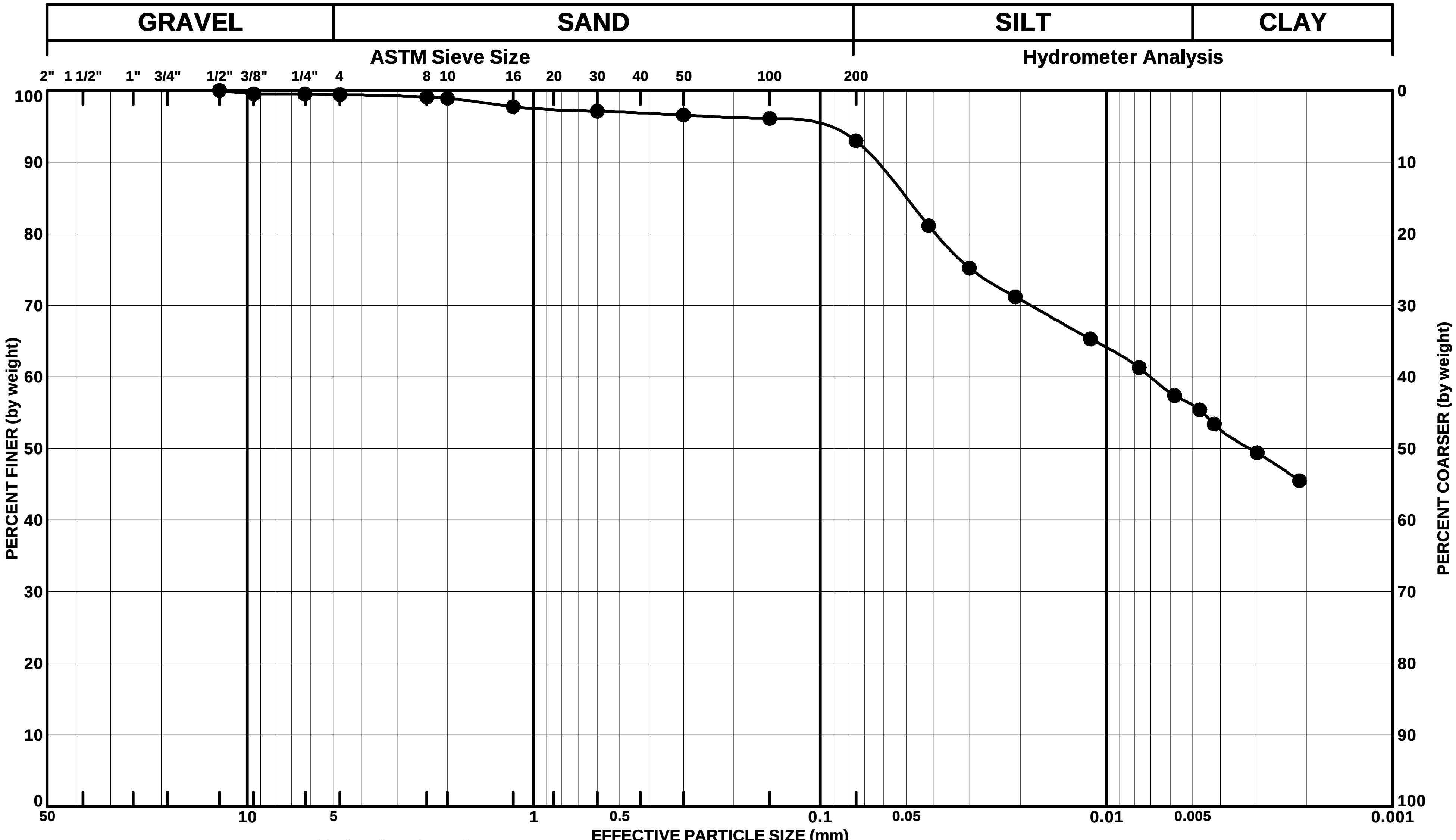
Figure No.: A-9
File No.: 03-1033
Date: 08/05/2003

GRAINSZ01 031033R.GPJ GRAINSZ.GDT 11/14/03

Sample No.	Identification	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
● SMSS-01		0.6	6.5	37.0	55.9

Organic Content = 11.3% at 0 to 1 ft.

LELAP Certificate No. 02052



ASTM D422

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Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

GRAIN SIZE ANALYSIS

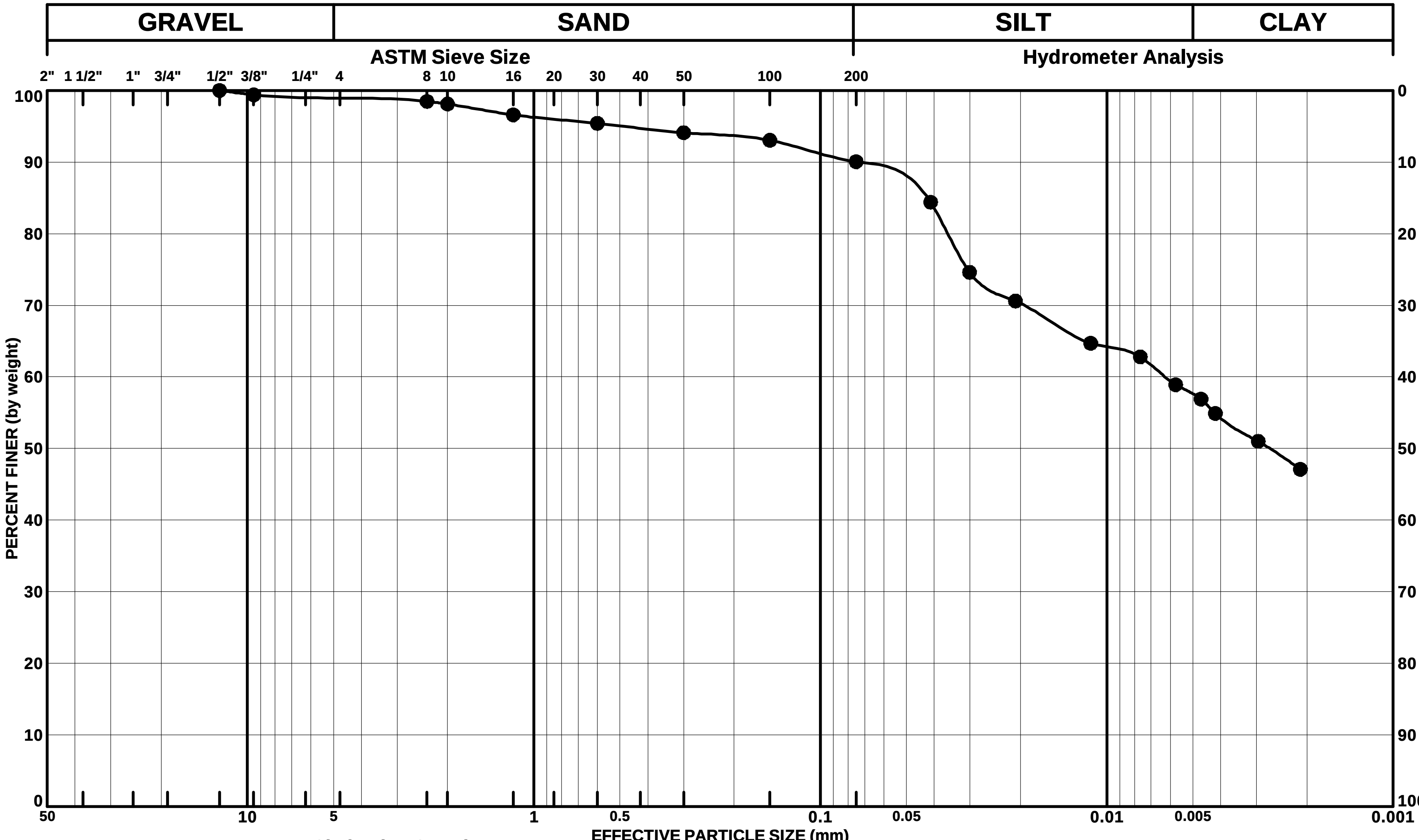
Figure No.: A-10
File No.: 03-1033
Date: 10/24/2003

GRAINSZ01 031033R.GPJ GRAINSZ.GDT 11/14/03

Sample No.	Identification	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
● SMSS-01		1.1	8.9	32.5	57.5

Organic Content = 11.3% at 0 to 1 ft.
Organic Content = 11.1% at 1 to 3 ft.

LELAP Certificate No. 02052



ASTM D422

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Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

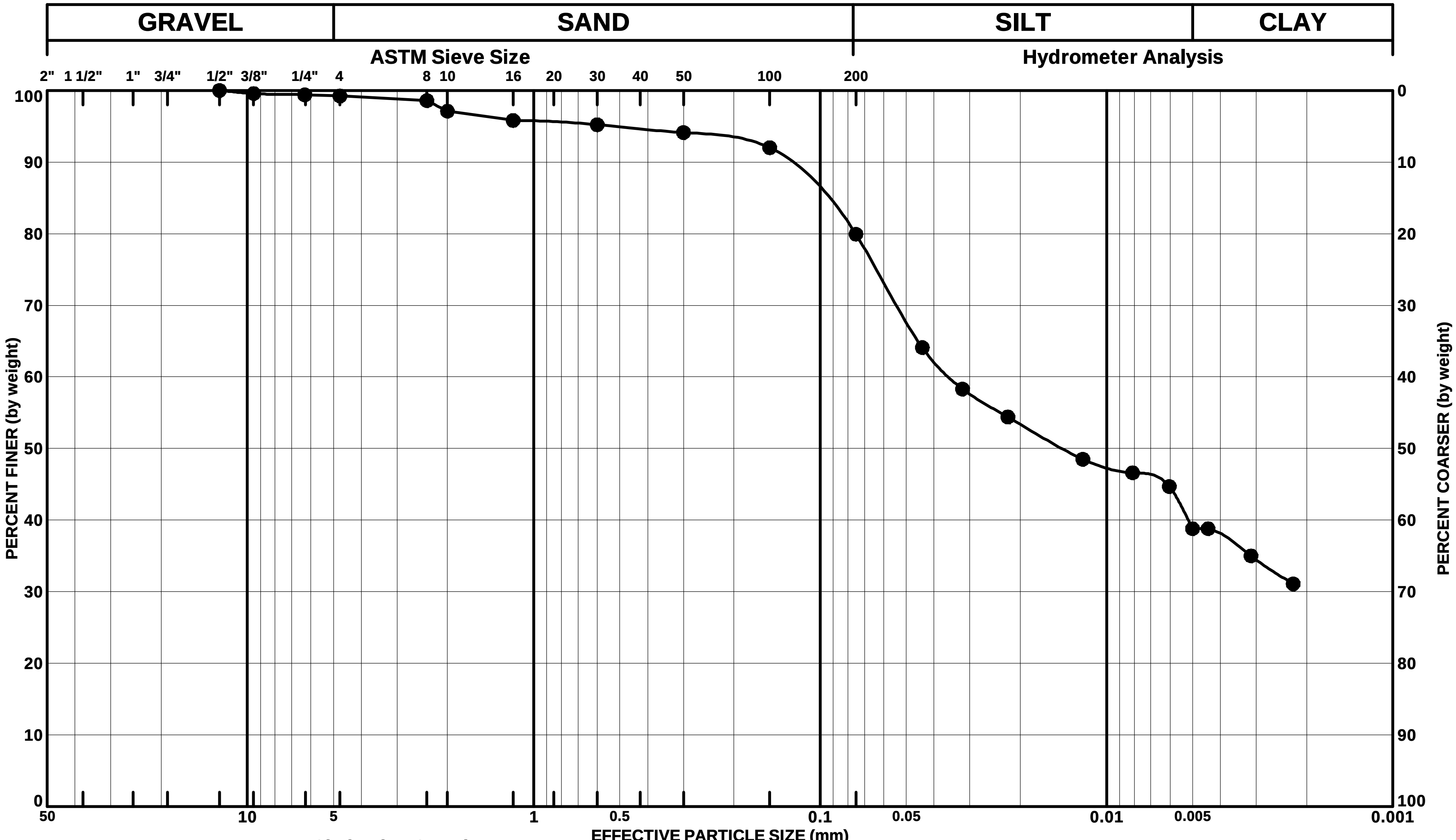
GRAIN SIZE ANALYSIS

Figure No.: A-11
File No.: 03-1033
Date: 10/24/2003

Sample No.	Identification	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
● SMSS-02		0.8	19.3	41.1	38.8

Organic Content = 11.9%

LELAP Certificate No. 02052



ASTM D422

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Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

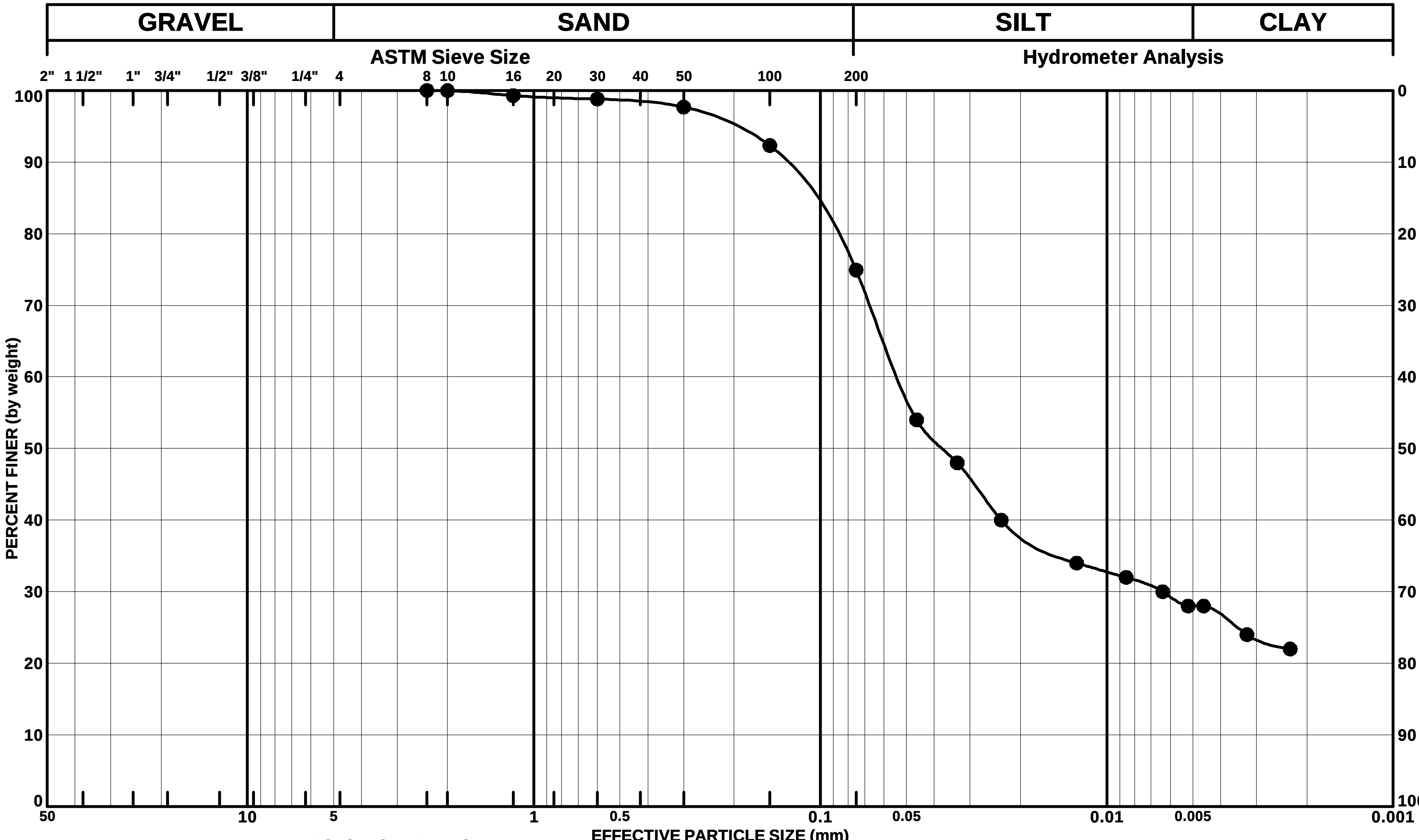
GRAIN SIZE ANALYSIS

Figure No.: A-12
File No.: 03-1033
Date: 10/24/2003

Sample No.	Identification	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
● SMSS-03		0.0	25.1	46.9	28.0

Organic Content = 5.6%

LELAP Certificate No. 02052



ASTM D422

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Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

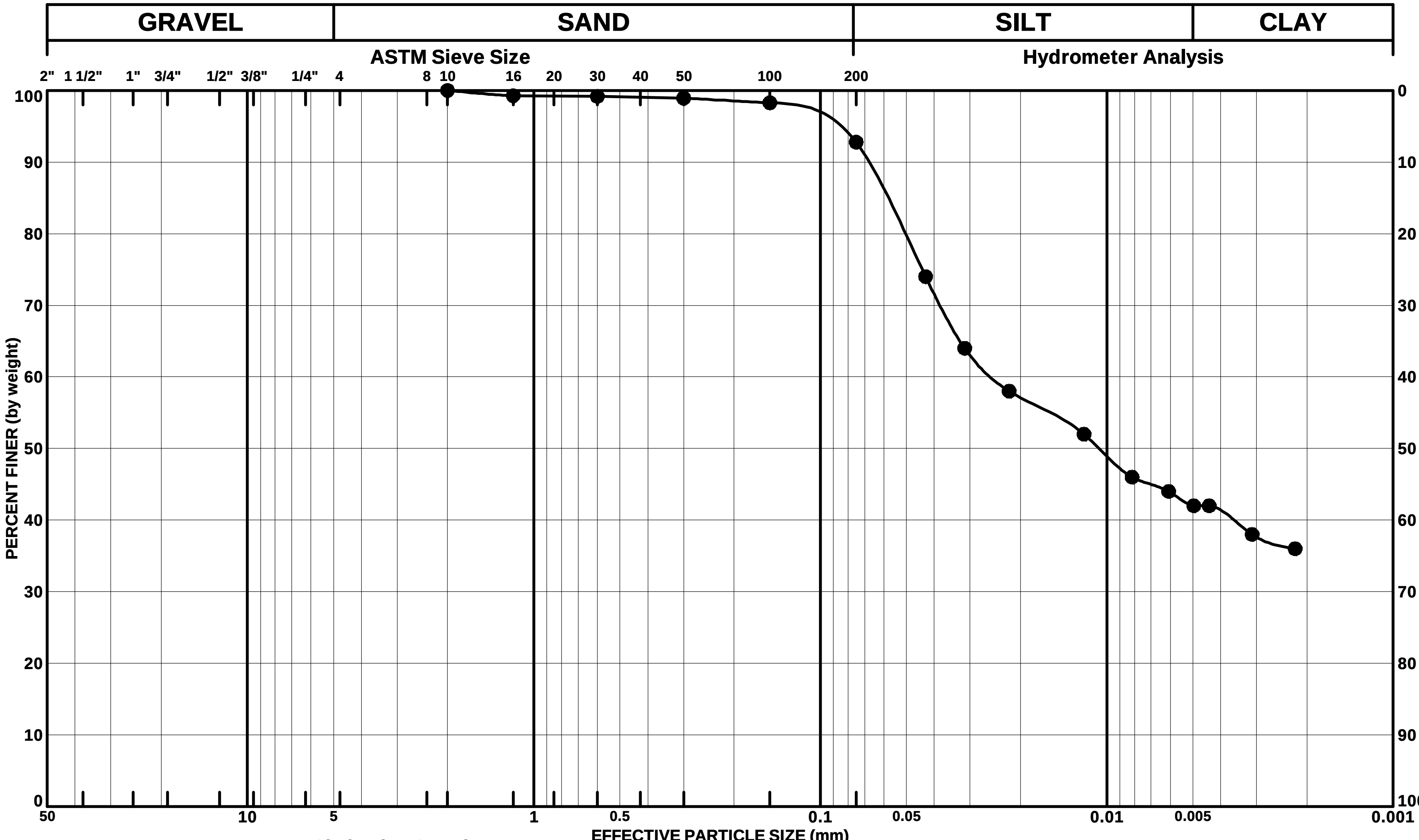
GRAIN SIZE ANALYSIS

Figure No.: A-13
File No.: 03-1033
Date: 10/24/2003

LELAP Certificate No. 02052

Organic Content = 6.2%

Sample No.	Identification	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
● SMSS-04		0.0	7.2	50.7	42.1



ASTM D422

Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

STE
Soil Testing Engineers, Inc.

GRAIN SIZE ANALYSIS

Figure No.: A-14

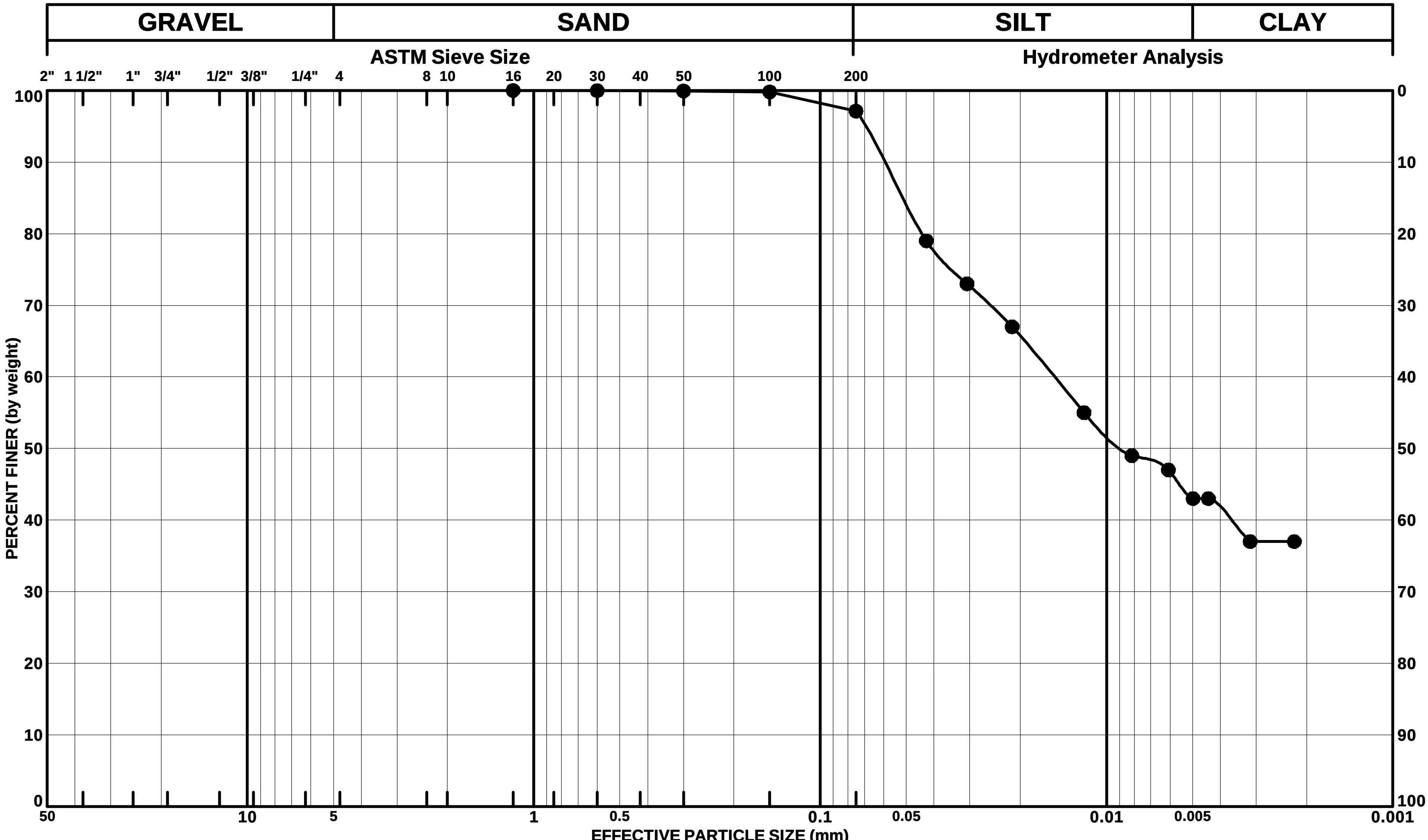
File No.: 03-1033

Date: 10/24/2003

GRAINSZ01 031033R.GPJ GRAINSZ.GDT 11/14/03

LELAP Certificate No. 02052

	Boring No.	Depth (ft.)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
●	BJW-2	15.0 - 17.0	0.0	2.9	54.1	43.0



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Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

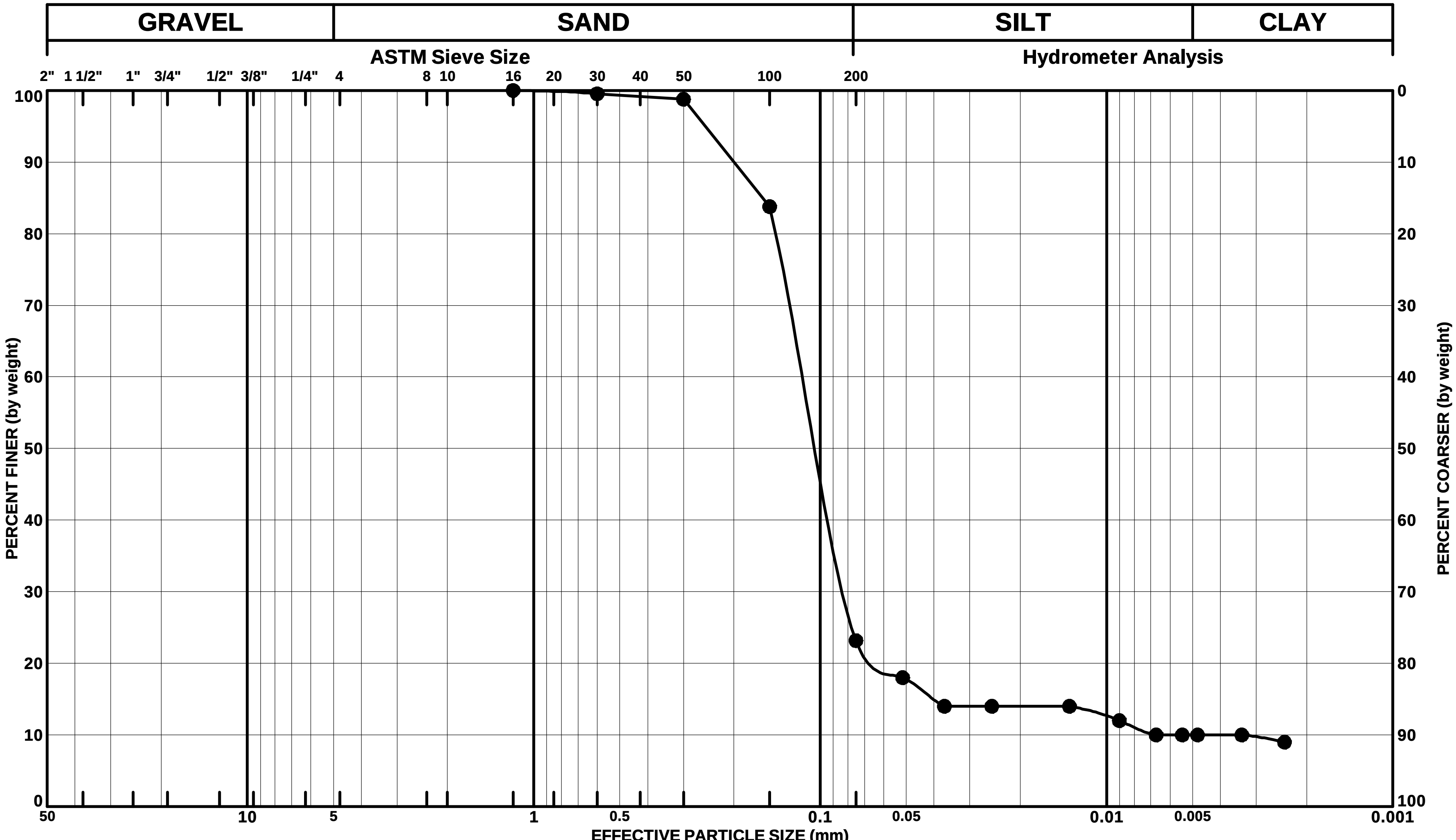
GRAIN SIZE ANALYSIS

Figure No.: A-15
File No.: 03-1033
Date: 09/12/2003

GRAINSZ 031033R.GPJ GRAINSZ.GDT 11/14/03

LELAP Certificate No. 02052

Boring No.	Depth (ft.)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
● BJW-4	11.0 - 13.0	0.0	76.8	13.2	10.0



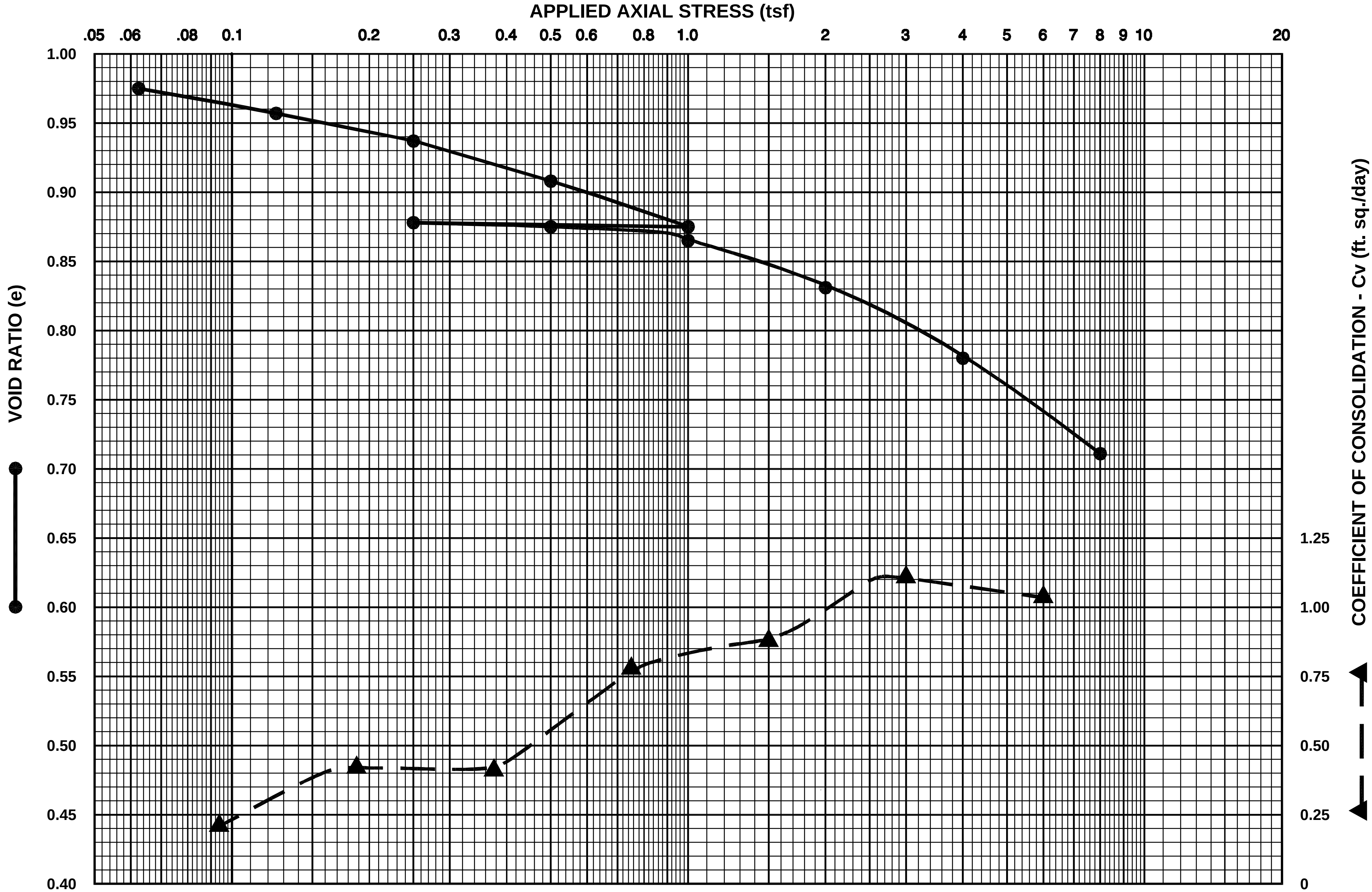
ASTM D422

GRAIN SIZE ANALYSIS

STE
Soil Testing Engineers, Inc.

Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration

Figure No.: A-16
File No.: 03-1033
Date: 09/15/2003



CONSOLIDATION TEST

SAMPLE IDENTIFICATION

BORING NO.: BJW-1
DEPTH (feet): 9-11
MATERIAL: Gray SILTY CLAY

FILE NO.: 03-1033

STE
Soil Testing Engineers, Inc.

CLASSIFICATION DATA

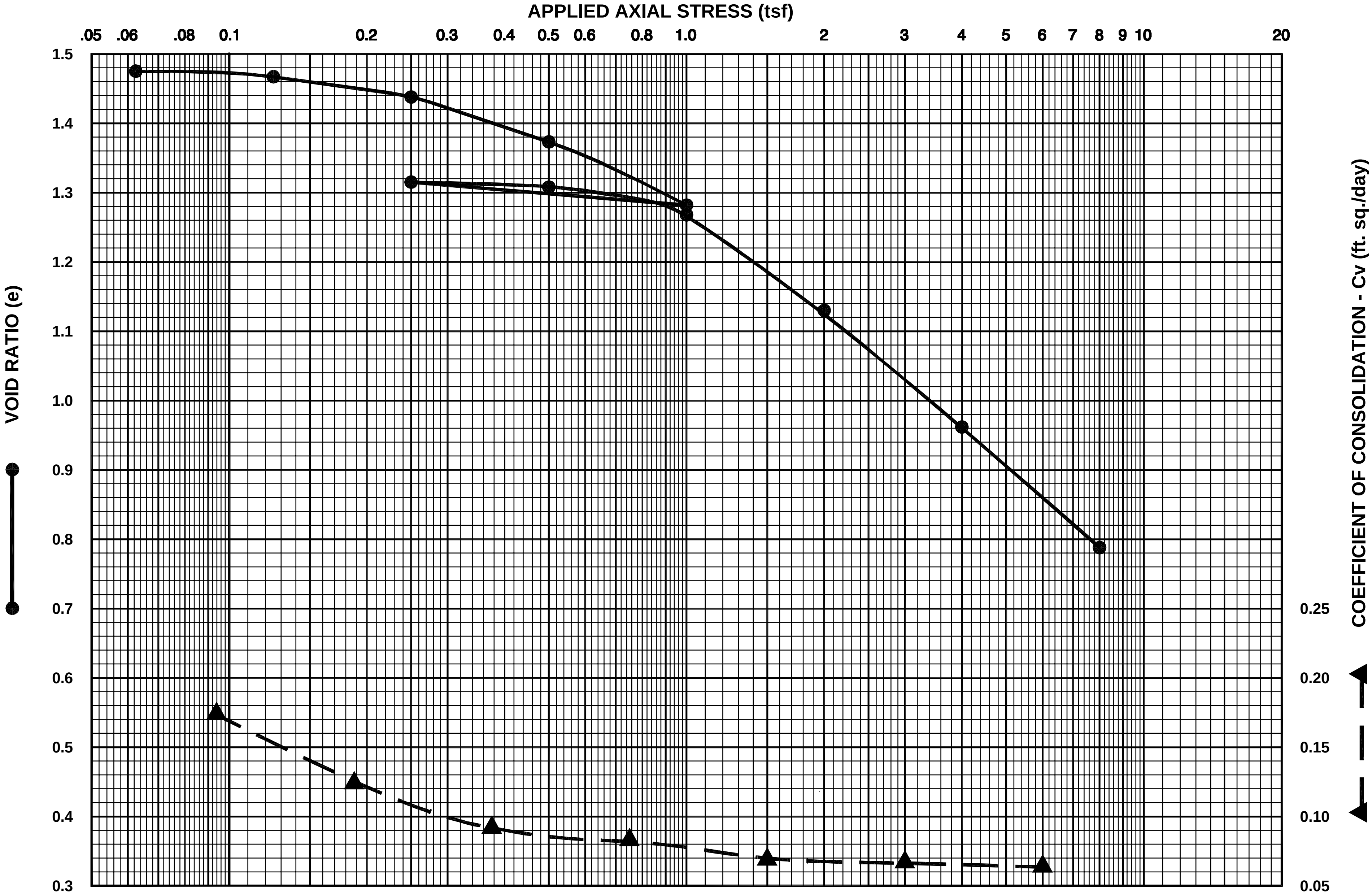
INITIAL MOISTURE CONTENT (%) =	36.7	LL = 49
INITIAL DRY DENSITY (lbs./cu.ft.) =	80.6	PL = 20
FINAL MOISTURE CONTENT (%) =	29.2	PI = 29

FIGURE NO.: A-17

Eo = 0.997 Gs = 2.58

P:\2003\03-1033\CADD\CS BJW1 38-40.DWG

Nov 14, 2003 - 10:04am



CONSOLIDATION TEST

SAMPLE IDENTIFICATION

BORING NO.: BJW-1
DEPTH (feet): 38-40
MATERIAL: Dark gray CLAY

FILE NO.: 03-1033

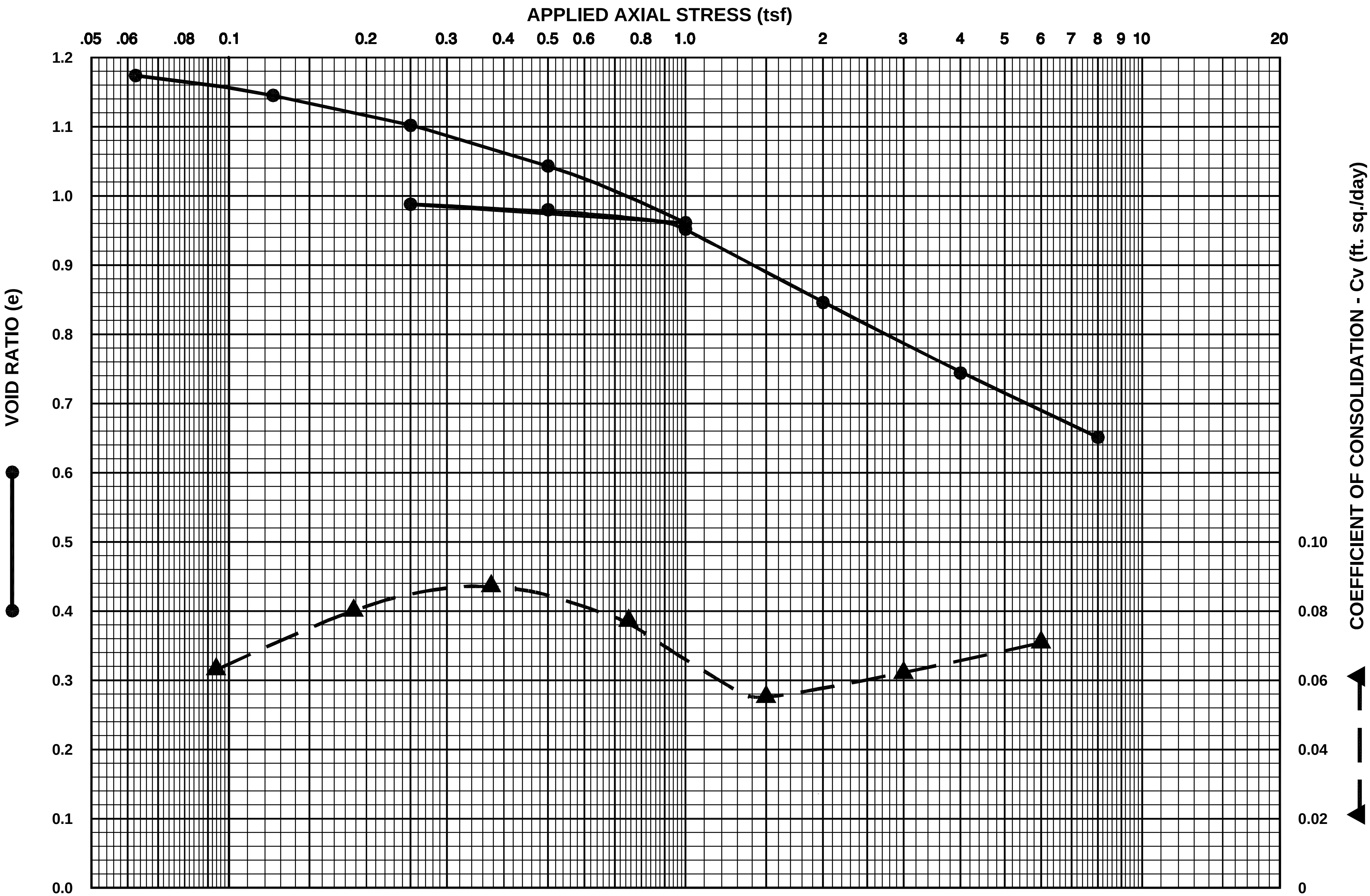
STE
Soil Testing Engineers, Inc.

CLASSIFICATION DATA

INITIAL MOISTURE CONTENT (%) =	54.4	LL = 87
INITIAL DRY DENSITY (lbs./cu.ft.) =	65.2	PL = 30
FINAL MOISTURE CONTENT (%) =	36.1	PI = 57

FIGURE NO.: A-18

Eo = 1.478 Gs = 2.59



CONSOLIDATION TEST

SAMPLE IDENTIFICATION

BORING NO.: BJW-2
DEPTH (feet): 23-25
MATERIAL: Gray CLAY w/sand seams
FILE NO.: 03-1033

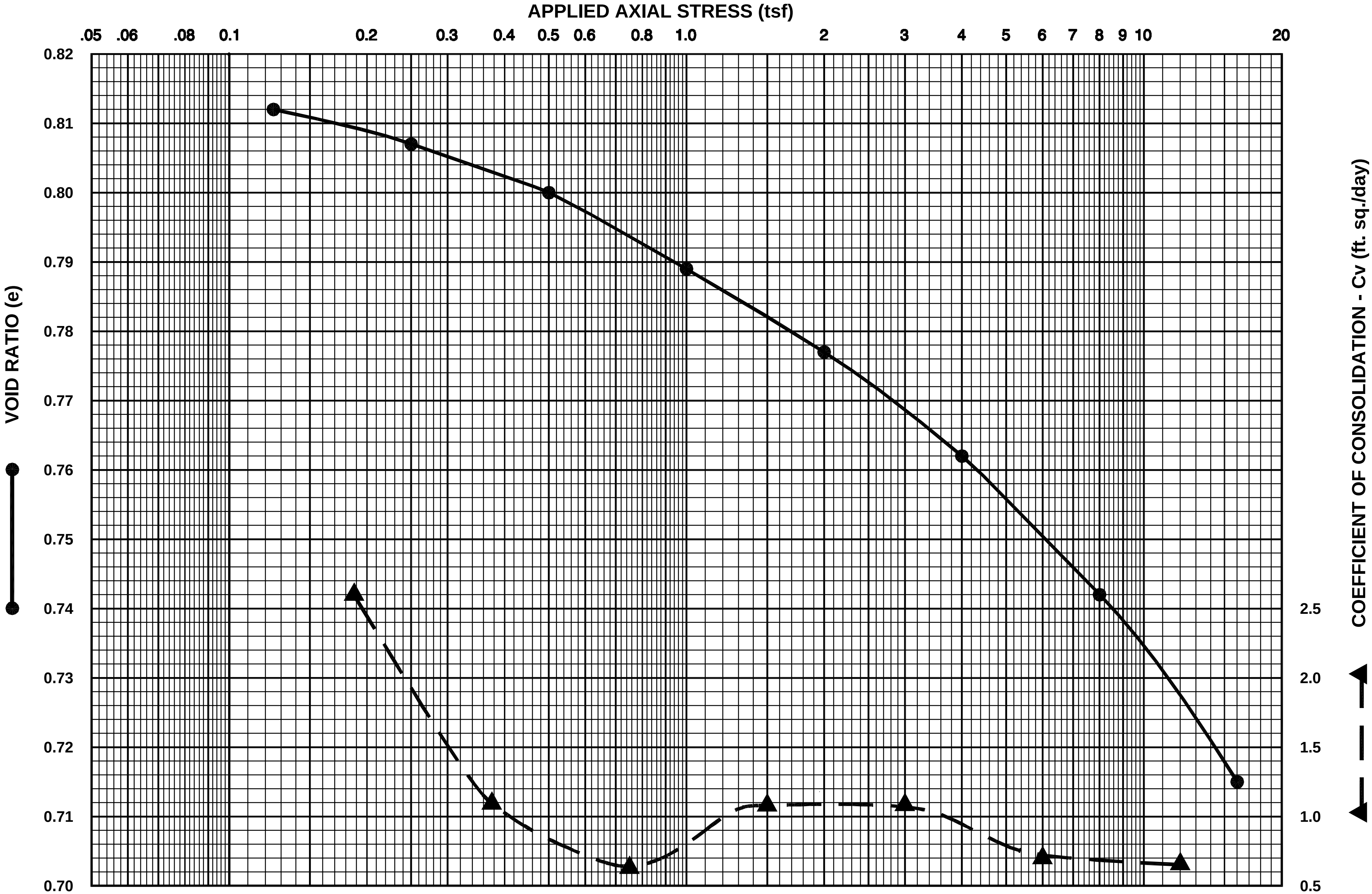
STE
Soil Testing Engineers, Inc.

CLASSIFICATION DATA

INITIAL MOISTURE CONTENT (%) =	38.7	LL = 60
INITIAL DRY DENSITY (lbs./cu.ft.) =	74.3	PL = 22
FINAL MOISTURE CONTENT (%) =	32.6	PI = 38

FIGURE NO.: A-19

Eo = 1.184 Gs = 2.60



CONSOLIDATION TEST

SAMPLE IDENTIFICATION

BORING NO.: BJW-4
DEPTH (feet): 11-13
MATERIAL: Gray SILTY SAND w/clay layers
FILE NO.: 03-1033

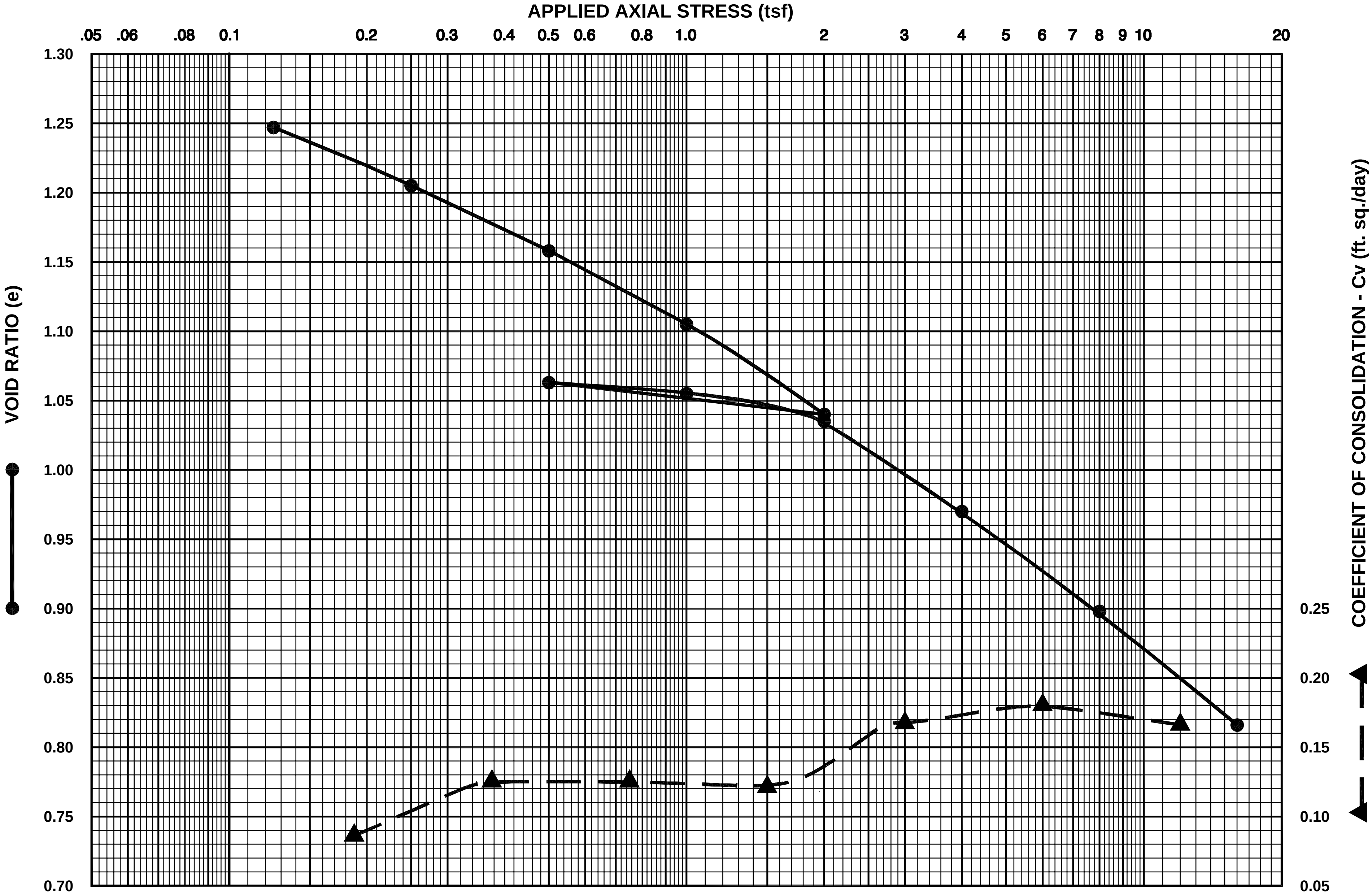
STE
Soil Testing Engineers, Inc.

CLASSIFICATION DATA

INITIAL MOISTURE CONTENT (%) = 25.5
INITIAL DRY DENSITY (lbs./cu.ft.) = 93.1
FINAL MOISTURE CONTENT (%) = 24.1

FIGURE NO.: A-20

Eo = 0.817 Gs = 2.71



CONSOLIDATION TEST

SAMPLE IDENTIFICATION

BORING NO.: BJW-4
DEPTH (feet): 38-40
MATERIAL: Gray CLAY

FILE NO.: 03-1033

STE
Soil Testing Engineers, Inc.

CLASSIFICATION DATA

INITIAL MOISTURE CONTENT (%) = 46.2 LL = 71
INITIAL DRY DENSITY (lbs./cu.ft.) = 74.3 PL = 34
FINAL MOISTURE CONTENT (%) = 25.3 PI = 37

FIGURE NO.: A-21

Eo = 1.268 Gs = 2.70