October 31, 2018

Todd Kennedy
MOFFATT NICHOL
4700 Falls of Neuse Road, Suite 300
Raleigh, NC 27609

Re: Estimated Pile Capacities & Recommended Heavy Duty Pavement Section
Bucksport Marina
Bucksport, South Carolina
GeoTechnologies Project No. 1-18-0747-EA

Dear Mr. Kennedy:

GeoTechnologies, Inc. completed a subsurface investigation for the referenced project in October 2011, and provided recommendations for a test pile program in November of 2011. These reports are included in Appendix A. Subsequently, we have been asked to update our pile capacity estimates and to provide a recommended heavy duty pavement section for the facility, minus the travel lift. Specifically, we have been asked to provide plots of axial pile capacity versus depth for the following:

1. Water side boat lift and floating dock guide using 14-18 inch square precast concrete piles with H-pile stingers to facilitate driving through cemented marl layers at depth. For the purposes of these analyses, we have assumed that HP14x73 or 117 stingers will be used; however, smaller sections will be needed if the smaller sized concrete sections are used. Test boring WB-3 was used for our analyses.
2. Water side pipeline protection pier using 16-18-inch diameter pipe piles. Test boring WB-3 was used for our analyses.
3. Landside boat lift using HP12 or 14 piles. Test borings LB-1 and LB-2 were used for our analyses.

Water Side Boat Lift/Floating Dock Guide. The attached Figure 1A presents estimated ultimate skin friction versus depth for a series of square concrete piles ranging from 14 to 18 inches in width with a 40 foot long HP14x73 or 117 stinger to facilitate driving. Again, a smaller H-section will be needed with the smaller sized concrete piles. With the described geometry and soil profile associated with boring WB-3, the width of the concrete pile has only a negligible impact on skin friction as the majority of the pile capacity is achieved from the H-section in the marl layer. As such, to achieve greater estimated capacities, the length and/or section of the H-pile should be modified.

Water Side Pipeline Protection Pier. The attached Figure 1B presents estimated ultimate skin friction versus depth for 16 and 18 inch diameter pipe piles for the pipe line protection pier.

Landside Boat Lift. The attached Figure 1C presents estimated ultimate skin friction versus depth for HP 12 and 14 sections. We have assumed that as a minimum 12x53 or 14x73 sections will be used.

TEST PILE PROGRAM

Ultimately, the design capacities for the various pile configurations must be based on the results of a test pile program and not on preliminary static estimates. This particular site has uncertainties associated with a variable near surface profile, pile drivability, and shear strength associated with the marl layer at depth. Our concern related to pile drivability is achieving the depth necessary to develop the design capacity without damaging the pile, and possible debris above the mudline. Our concern related to the marl is that this stratum contains alternating layers of very hard cemented soils and relatively soft sands, silts, and clays. Based on the above, it is our opinion that attempting pile installation without a test pile program could result in A) over design and/or excessive contractor bids associated with unresolved uncertainties, and B) schedule overruns, change orders, and liquidated damage claims in the event that pile sections cannot be driven without damage, or if the sections do not achieve design capacity.

Water Side Boat Lift/Floating Dock Guide. Static calculations (Figure 1A) indicate that a 14 to 18-inch square concrete pile will achieve an allowable (safety factor of 2) capacity of about 80 tons in skin friction when driven to depths of about 70 to 75 feet with a 40 to 45-foot-long HP-14 section stinger. The actual capacity in compression will be higher than estimated due to the end bearing component; however, we recommend using caution when accounting for the end bearing component due to the composition of the marl layer. The presence of the cemented soils in a matrix with much softer materials will alternatingly give high and low values of end bearing, and the results of the recommended PDA testing will need to be closely evaluated if the end bearing contribution to capacity will be included.

For the purposes of the test pile program, we recommend driving 60 foot and 80 foot piles, with stinger lengths of 30 and 50 feet. The piles should be driven in the proximity of our test borings WB-3 through WB-5 as WB-1 and WB-2 were somewhat more favorable near surface. The different pile lengths are intended to help estimate the marl shear strength.

The contractors driving submittal should include a WEAP analysis to demonstrate that the piles can be driven safely to the desired depth without damage. The piles should be driven either to the maximum depth which allows for restriking, or until the design capacity is achieved in the marl layer using the minimum energy possible to limit damage potential. The piles should be monitored during driving with a PDA, and should then be allowed to set-up for 72 hours before PDA monitored restrikes are performed. Design capacities should be based on the restrike data and not static estimates. Subsequently, the test piles should be removed a with a crane and vibratory extractor to inspect for damage. If the piles are damaged, driving shoes or other provisions will be necessary, and additional test piles will need to be driven.

Water Side Pipeline Protection Pier. Static calculations (Figure 1B) indicate that 16 to 18-inch pipe piles will achieve an allowable (safety factor of 2) capacity of about 70 to 80 tons in skin friction when driven to a depth of about 75 feet. The actual capacity in compression will be higher than estimated due to the end bearing component; however, we recommend using caution when accounting for the end bearing component due to the composition of the marl layer. The presence of the cemented soils in a matrix with much softer materials will alternatingly give high and low values of end bearing, and the results of the recommended PDA testing will need to be closely evaluated if the end bearing contribution to capacity will be included.

For the purposes of the test pile program, we recommend driving 60 foot and 80 foot piles. The piles should be driven in the proximity of our test borings WB-3 through WB-5 as WB-1 and WB-2 were somewhat more favorable near surface. The different pile lengths are intended to help estimate the marl shear strength.

The contractors driving submittal should include a WEAP analysis to demonstrate that the piles can be driven safely to the desired depth without damage. The piles should be driven either to the maximum depth which allows for restriking, or until the design capacity is achieved in the marl layer using the minimum energy possible to limit damage potential. The piles should be monitored during driving with a PDA, and should then be allowed to set-up for 72 hours before PDA monitored restrikes are performed. Design capacities should be based on the restrike data and not static estimates. Subsequently, the test piles should be removed a with a crane and vibratory extractor to inspect for damage. If the piles are damaged, driving shoes/points or other provisions such as spudding will be necessary, and additional test piles will need to be driven.

Landside Boat Lift. Static calculations (Figure 1C) indicate that HP 12 or 14 piles will achieve an allowable (safety factor of 2) capacity of about 75 tons in skin friction when driven to depths of about 55 to 65 feet. The actual capacity in compression will be higher than estimated due to the end bearing component; however, we recommend using caution when accounting for the end bearing component due to the composition of the marl layer. The presence of the cemented soils in a matrix with much softer materials will alternatingly give high and low values of end bearing, and the results of the recommended PDA testing will need to be closely evaluated if the end bearing contribution to capacity will be included.

For the purposes of the test pile program, we recommend driving 50 foot and 70 foot piles. The piles should be driven in the proximity of our test boring LB-2. The different pile lengths are intended to help estimate the marl shear strength.

The contractors driving submittal should include a WEAP analysis to demonstrate that the piles can be driven safely to the desired depth without damage. The piles should be driven either to the maximum depth which allows for restriking, or until the design capacity is achieved in the marl layer using the minimum energy possible to limit damage potential. The piles should be monitored during driving with a PDA, and should then be allowed to set-up for 72 hours before PDA monitored restrikes are performed. Design capacities should be based on the restrike data and not static estimates. Subsequently, the test piles should be removed a with a crane and vibratory extractor to inspect for damage. If the piles are damaged, driving shoes or other provisions will be necessary, and additional test piles will need to be driven.

Additional Considerations. Lateral analyses can be performed by GeoTechnologies once section details and pile locations are available.

Pavement Design. All pavement subgrades should be moisture conditioned and recompacted to not less than 98% of the standard Proctor maximum dry density immediately prior to placement of base course stone. The subgrades should also be proofrolled for stability. The only landside test borings performed by GeoTechnologies were LB-1 and LB-2 which indicated the presence of organic fill below some cleaner surface fills. If similar conditions exist in pavement areas, the owner should be aware that the subgrade may proofroll stable but that settlement associated with the organic (and soft) soils may occur over time. As such, the owner should consider performing additional exploration to evaluate the extent of these soils below settlement sensitive areas.

For the purposes of this report, we have assumed that the subgrade soils will be suitable for a design subgrade CBR value on the order of 5%, and that materials which do not meet this criteria will be removed and replaced, or chemically stabilized. Appropriate laboratory testing should be performed to verify this assumption at the time of site development.

Since traffic data was not available, we have assumed that the site may be visited by up to 50 heavy (2.43 ESAL) trucks per week, resulting in about 125,000 ESAL's accruing over a 20 year period.

Based on the assumed design CBR value and traffic outlined above, a preliminary recommended pavement section in heavy duty areas consists of 4 inches of asphalt over 8 inches of CABC stone. A light duty section for car parking may consist of 2 inches of asphalt over 8 inches of stone. These sections should only be used for preliminary purposes, and must be updated once actual soil support conditions, traffic loads, and traffic volumes are known.

GeoTechnologies, Inc. appreciates the opportunity to be of service on this phase of the project. Please contact us if you have any questions concerning this letter or if we may be of additional service on this or other projects.

Sincerely,

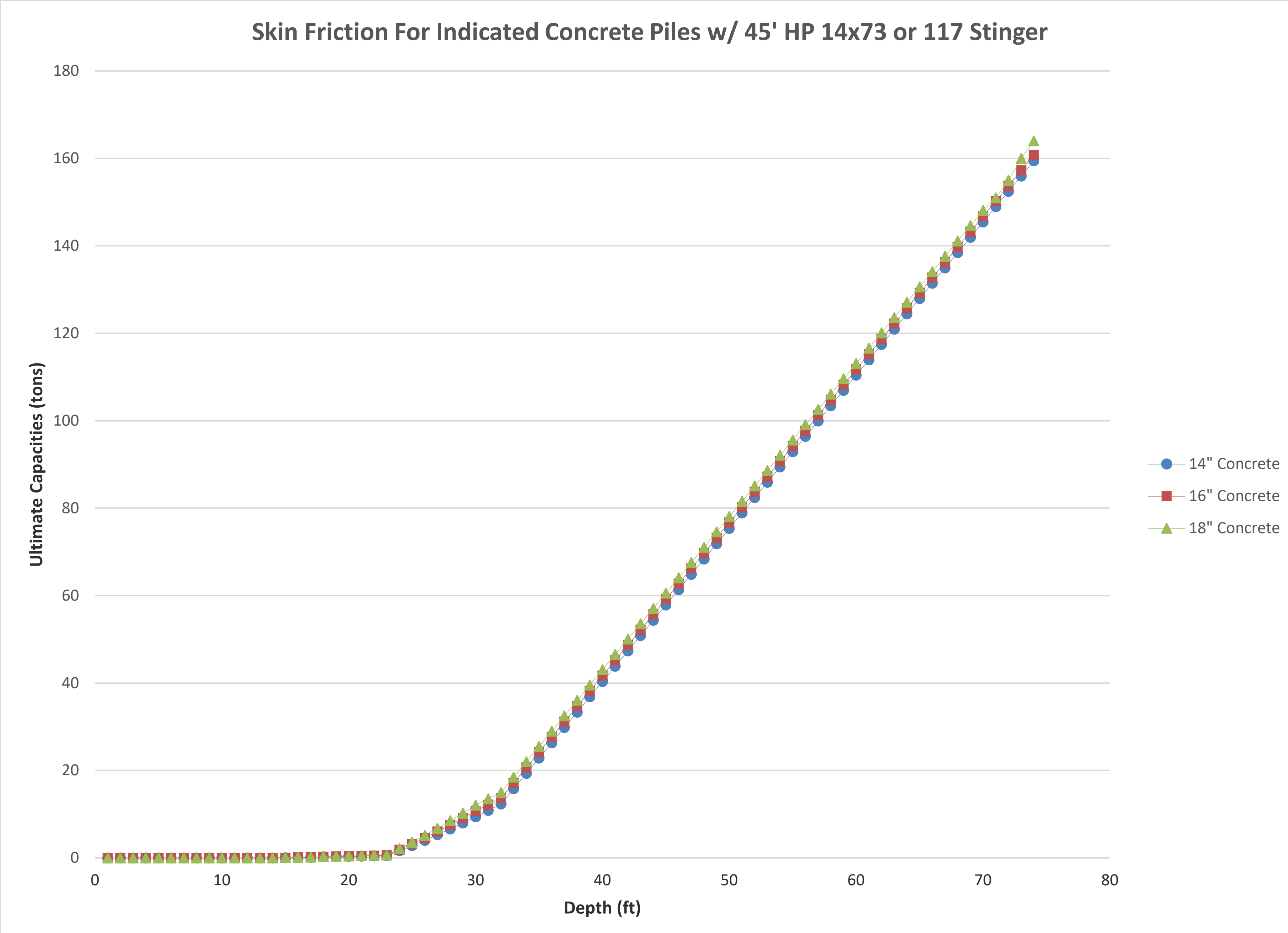
GeoTechnologies, Inc.

Ernest L. Stitzinger, P.E.
Senior Engineer

David L. Israel, P.E.
SC Registration No. 13473

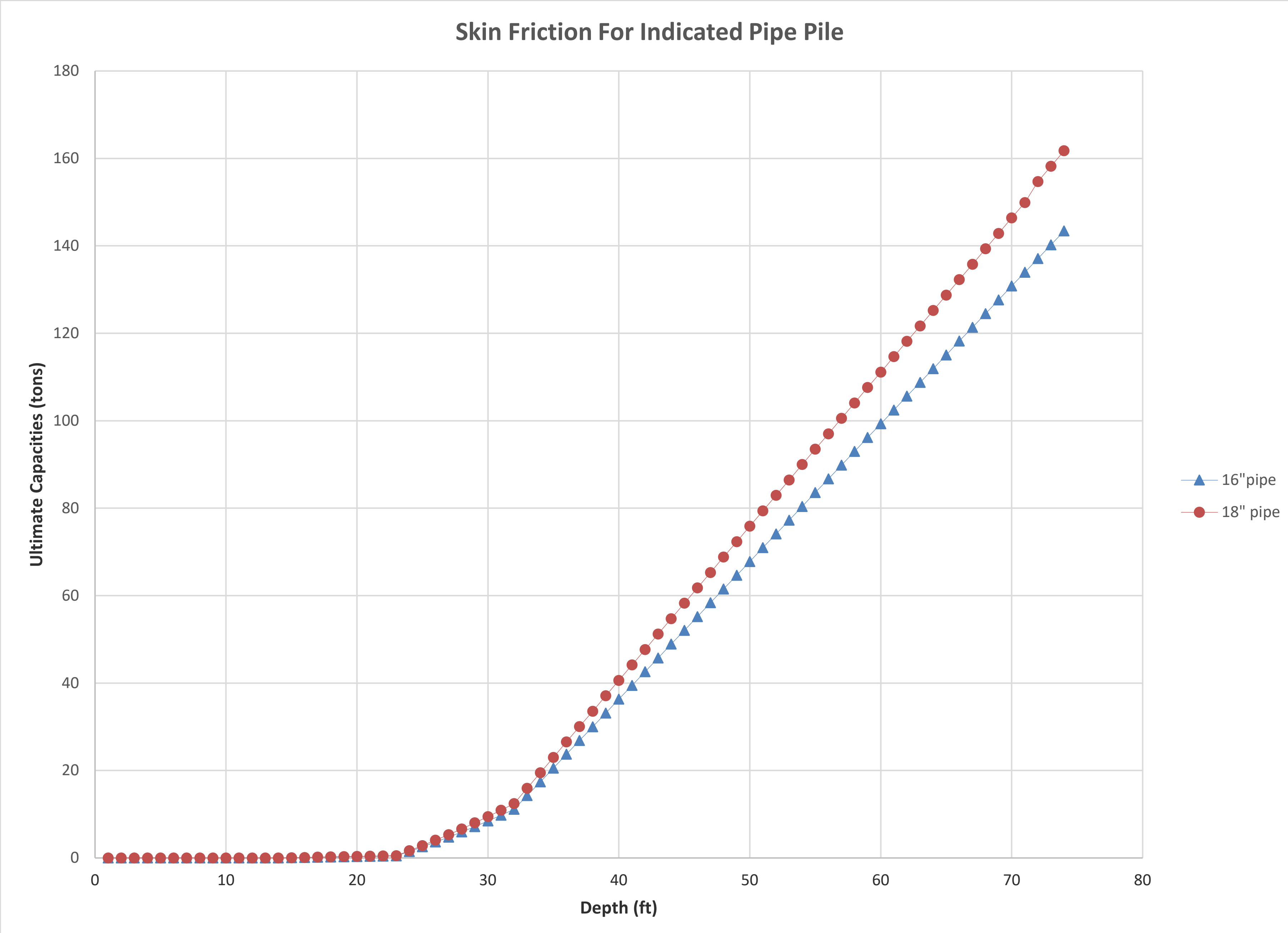
10/31/2018

Figure 1A



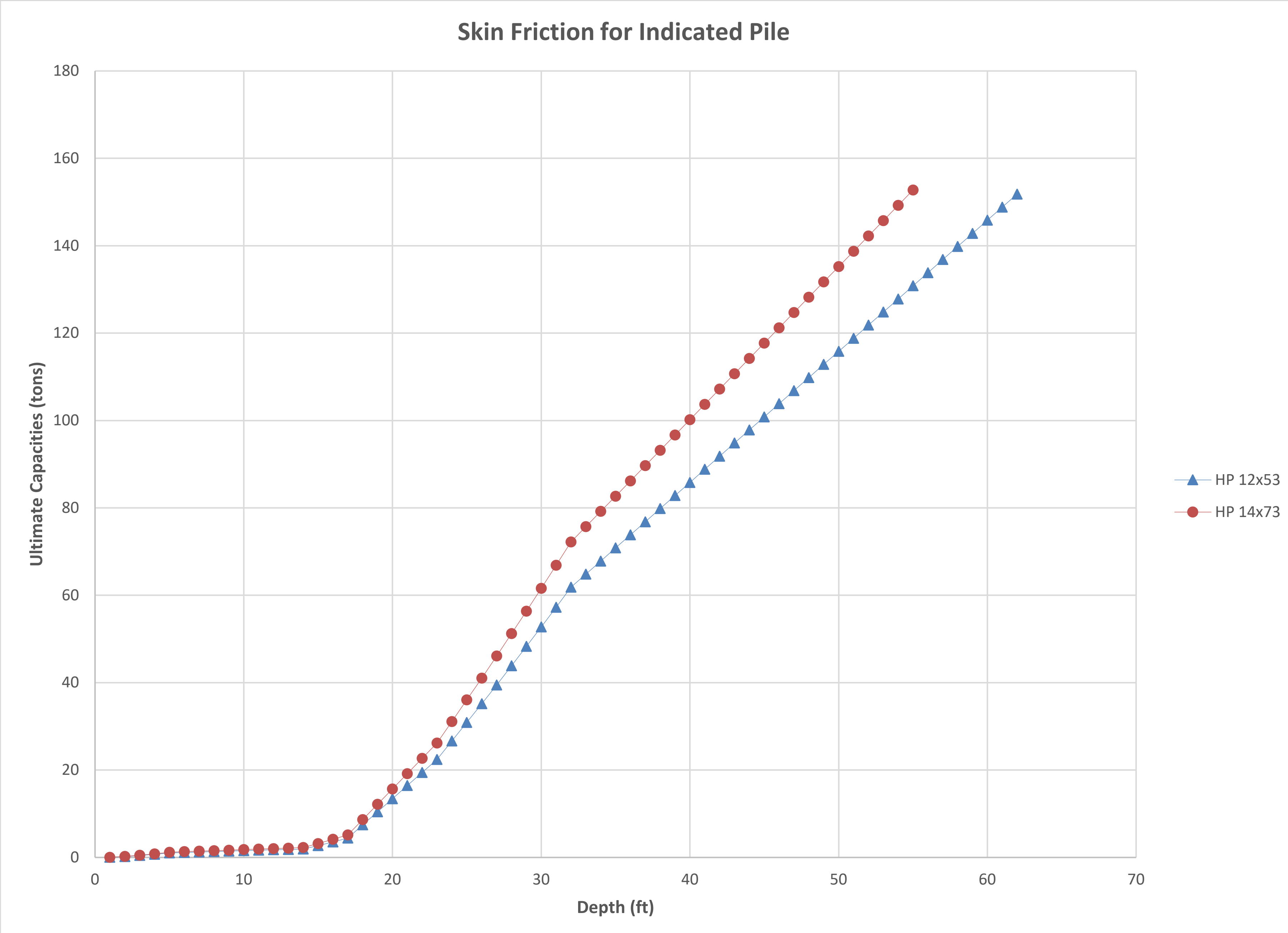
Safety factor should be applied. Note that values below about 55 feet were extrapolated. WB-3 boring used.

Figure 1B



Safety factor should be applied. Note that values below about 55 feet were extrapolated. WB-3 boring used.

Figure 1C



Safety factor should be applied. Note that values below about 40 feet were extrapolated. WB-3 boring used.

APPENDIX A

2011 REPORTS

October 3, 2011

Brad Woodall
MOFFATT & NICHOL
4975 Lacrosse Road, Suite 151
N. Charleston, SC 29406

Re: Report of Subsurface Investigation
Bucksport Marine Industrial Park
Bucksport, South Carolina
GeoTechnologies Project No. 1-11-0506-EA

Gentlemen:

GeoTechnologies, Inc. has completed the authorized investigation to evaluate subsurface soil conditions for elements of the Bucksport Marine Industrial Park located off the Waccamaw River in Bucksport, South Carolina. Subsurface conditions were investigated with seven borings performed off a barge in the river and two landside borings. The boring locations (Figure 1) were determined in the field by estimating or measuring distances from site landmarks, and should be considered approximate. The borings were extended to depths of about 23 to 54 feet using standard penetration test procedures at selected intervals to evaluate the consistency and density of the subsurface soils. This report presents the findings of our borings and design and construction recommendations.

PROPOSED PROJECT

Specific project details were limited at the time of our investigation; however, it is our understanding that site development will involve about 150 acres for a new marine industrial park. GeoTechnologies investigation was directed towards evaluating subsurface conditions for a new bulkhead and associated 100 to 300 ton travel lifts, floating or fixed docks, and two (2) 80 to 100 foot long piers along the northeast side of the project. Conditions for a T-Dock for the loading of bulk cargo were also evaluated about a half mile to the south. Although the bulkhead may eventually be longer, we understand that initial development will involve construction of the new 10 to 15 foot high bulkhead and travel lifts in the proximity of borings LB-1 and LB-2.

The existing marina includes timber pile supported piers and docks, with boating access to the north. Several buildings associated with the facility are located to the west of these structures. A 30 inch water line easement runs perpendicular to the river between borings WB-3 and WB-4. During the time the test borings were drilled, tide fluctuations were estimated at approximately 2 to 3 feet; however, we anticipate those fluctuations will be higher during certain times of the year.

SUBSURFACE CONDITIONS

Generalized subsurface profiles prepared from the test boring data are attached to this report as Figures 2A through 2C to graphically illustrate subsurface conditions encountered at this site. More detailed descriptions of the conditions encountered at the test boring locations are then presented on the attached test boring records in Appendix A.

<u>Landside Borings</u>. Near surface conditions in the landside borings (Figure 2A) were characterized by the presence of about 5 to 6 feet of sandy and clayey fill exhibiting penetration resistances in the range of 5 to 13 blows per foot (bpf). These soils were underlain by fill or possible fill soils classified as clays and sands with significant quantities of wood from 6 to 12 feet in boring LB-2, and some wood in boring LB-1 from 6 to 14 feet. Laboratory testing on a sample from LB-1 indicated an organic content on the order of 10%. Penetration resistances in these soils ranged from 1 to 4 bpf. Below depths of about 12 to 14 feet, these borings encountered a few feet of 10 to 12 bpf clean sands.

Beginning at a depth of about 17 feet, the landside borings encountered marl consisting of alternating layers of soil with much harder cemented layers. Excluding thin lenses, the harder cemented layers were typically 4 to 20 inches in thickness with drilling rates of less than a 1/2 inch of penetration per minute up to 2 inches of penetration per minute with a tricone bit. Where sampled, the cemented layers exhibited penetration resistances of 50 blows per 2 to 3 inches of penetration (see boring records). The intermediate soils exhibited penetration resistances in the range of 5 to 22 bpf.

The landside borings were terminated at depths in the range of about 23 to 40 feet, with boring LB-1 terminated at 23 feet due to roller cone bit refusal. Groundwater levels at 24 hours were measured at 2.5 to 3.5 feet in these borings.

<u>North Side Water Borings.</u> Mud line depths in the north side water borings (Figure 2B) ranged from about 3 to 21 feet. Below these depths, the borings typically encountered wood and silty to clayey soils containing varying amounts of organics. Penetration resistances in these soils ranged from the weight of the drilling rods (WOR) to 8 bpf, with the higher resistances associated with sampling on wood. These materials were present to depths of up to 28 feet below the water line, with the depth increasing from south to north. With increasing depth, borings WB-2 through WB-5 encountered clean sands exhibiting penetration resistances of 4 to 28 bpf. The thickness of the sand layer decreased from south to north, and the sands were not encountered in boring WB-1. The bottom depth of the sand layer ranged from about 30 to 34 feet. Based on the boring profiles and the proximity of existing structures, it appears likely that some previous dredging has been performed in the proximity of WB-1 to WB-3.

With depth, the borings encountered marl consisting of alternating layers of soil with much harder cemented layers. The initial cemented zone was encountered at 30 feet in boring WB-2, but was consistently present in all borings at about 35 feet. Excluding thin lenses, the harder cemented layers ranged from about 6 to 19 inches in thickness with drilling rates of about 2 to 6 inches of penetration per minute (see borings records). Where sampled, the cemented layers typically exhibited penetration resistances of 50 blows per 0 to 1 inch of penetration. The intermediate soils exhibited penetration resistances in the range of 9 to 22 bpf. Borings WB-1 through WB-5 were terminated at depths of about 47 to 54 feet.

<u>T-Dock Borings.</u> Mud line depths for the T-Dock borings (Figure 2C) ranged from about 11 to 12 feet. Below these depths, the borings typically encountered a thin layer of organic clay which was predominantly underlain by clean, silty, or clayey sands with some isolated clays. Penetration resistances in these soils ranged from 8 to 14 bpf.

Beginning at a depth of about 19 feet, the borings encountered marl consisting of alternating layers of soil with much harder cemented zones. Excluding thin lenses, the harder cemented layers ranged from about 4 to 22 inches in thickness with drilling rates of less than 1/2 inch of penetration per minute to more than 6 inches of penetration per minute (see boring records). Where sampled, the cemented layers typically exhibited penetration resistances of 50 blows per 0 to 5 inches of penetration. The intermediate soils exhibited penetration resistances in the range of 7 to 18 bpf. Borings WB-6 and WB-7 were terminated at depths of about 40 to 49 feet.

RECOMMENDED DESIGN PARAMETERS

Landside Borings. Fill or possible fill was encountered to depths of 12 to 14 feet in the landside borings where the initial bulkhead and travel lift construction will occur. These soils appear to have been placed without control and are quite variable, particularly below 5 to 6 feet. These soils are underlain by sands and marl including hard cemented zones below about 20 feet. The recommended soil parameters for these conditions are presented in Table 1. It is noted that the parameters below 17 feet are a weighted average based on the relative percentage of the cemented layers in the marl encountered in all of our test borings once this stratum was encountered.

Water Borings. Recommended soil design parameters for the water borings are indicated in Table 1 which groups together similar profiles. In general, clean sands are assigned a friction angle based on standard penetration resistances, while a design cohesion is assigned to the marl layer as discussed previously. The organic materials at the mud line are assigned a small value of cohesion.

CONSTRUCTION CONSIDERATIONS

Details regarding foundation types and loadings were limited at the time of this investigation; however, we are assuming that heavier compressive loads will be handled with a high capacity pile, such as a prestressed concrete section or a steel pile (H-pile or pipe pile) driven into the marl. The capacity of the selected section will depend on pile geometry and penetration depth into the marl. Based on static calculations, we estimate that an 18 inch concrete pile will achieve a design capacity (safety factor of 2) in the range of 60 to 80 tons when driven to depths of 50 to 55 feet in the area of the land borings and T-Dock borings, while profiles similar to borings WB-1 through WB-5 could require embedment depths in the range of 55 to 65 feet to achieve similar capacities. Design capacities for H-piles or other steel sections can be estimated by using the parameters in Table 1 and assuming that the pile will achieve capacity primarily through skin friction in the marl. GeoTechnologies can provide these estimates if provided with proposed pile geometries.

The fine grained lower penetration resistance portion of the marl is expected to go "quick" during initial driving and then set-up after pore pressures dissipate, resulting in higher capacities than will be initially estimated by empirical pile driving formulas. Additionally, it should be recognized that the estimated shear strength of the marl is approximate due to the variability of the material. As such, it is imperative that final pile design capacities be based on the results of a well planned test pile program incorporating a pile driving analyzer (PDA) to estimate capacities both at the end of initial driving and following restriking after a minimum 72 hour set-up period. These results can then be used to determine appropriate production pile driving criteria for a given capacity.

As discussed, the marl at depth includes very hard cemented zones which approach 2 feet in thickness in some areas. As such, the contractor should anticipate variable driving conditions in this stratum, including very difficult driving where cemented layers are encountered. We expect that the specified pile driving set may be met or approached upon encountering the cemented zones; however, as these zones are underlain by softer soils, we recommend that the selected driving criteria also include a minimum embedment into the marl based on the results of the test pile program and the required pile capacity. To help determine the minimum embedment depth, we recommend that the shear strength of the marl be specifically estimated by driving different length test piles into the profile with PDA testing to separate end bearing and skin friction. With this approach, the piles will primarily be designed as friction piles such that the ultimate uplift capacity will approach the compressive capacity.

The test pile program should also demonstrate that the selected pile section can be driven to achieve the design capacity and embedment depth without damaging the pile. We anticipate that concrete piles cannot be driven through the harder cemented zones in the marl, and that they will have to be outfitted with a steel "stinger" to help break-up the cemented zones and to facilitate installation. Alternatively, a steel pile, such as an HP 14 x 74 could be used. Due to the potential for driving difficulties associated with the cemented layers, we recommend that consideration be given to driving more than one pile type as part of the test pile program. If the test pile program demonstrates that the piles cannot be driven without damage to the required embedment, alternate installation measures such as spudding or predrilling will have to be considered. The contractor should be prepared for such contingencies during the test pile program.

Prior to driving test pile sections, we recommend that the contractor be given the responsibility for selecting an appropriate hammer and that he be requested to provide a WEAP analysis indicating drivability and potential damage impact to the piles before mobilizing to the project. The PDA testing can be used to verify that the piles are not being damaged during the driving process.

In summary, the subsurface profile at this site is such that penetration into the marl layer at depth will be required to develop a high capacity pile. The shear strength of the marl and pile drivability can be estimated upfront; however, final design capacity, embedment depth, pile type, and installation techniques must be based on the results of the test pile program discussed above.

We anticipate that certain elements of construction, such as the proposed boat docks and piers, may not require the use of high capacity piles to handle axial loads; however, some penetration into the marl may still be necessary to develop lateral resistance. Once design loadings for these structures are available, an appropriate pile can be selected and evaluated as part of the test pile program.

Lateral load performance of the new construction may be evaluated using the soil parameters in Table 1. Lateral load tests can also be conducted on individual piles during the test pile program to verify lateral load resistance assumptions made in design for the piles. The design parameters included on the attached Table 1 may also be used for design of the bulkhead structure. The contractor probably will use a vibratory hammer to set the initial sheets and will then likely drive the sheets with an impact hammer if an attempt is made to penetrate into the underlying marl formation. If adequate penetration into the marl cannot be obtained with the sheet piles (expected), a king pile installation which employs a heavy "H" section can be used to supplement the sheet piles. Driving difficulties similar to those for high capacity axial pile installation should be expected where piles must be advanced into the marl.

MISCELLANEOUS CONSIDERATIONS

Construction of the new facilities will require installation through existing fill which has been placed on the land side of the project, and some wood debris in the water. During the course of drilling the test borings, no significant debris obstructions were encountered; however, if such materials are present intermediate of the boring locations, they may need to be removed to facilitate construction. Typically, we suggest backfilling any required excavations with clean sand.

We anticipate that the organic fill in LB-1 and LB-2 will be largely removed from the waterside of the new bulkhead in the area of the travel lifts, but the fill could be left in place beneath the land side. As indicated by the boring data, the deeper organic fill is soft and can be expected to settle if new loads (including new fill) are placed over the area in the future. Based on the nature of these materials, it is difficult to estimate potential settlements; however, it is likely to vary across the area resulting in differential movement. As such, consideration should be given to removing the old fill below settlement sensitive structures and surfaces unless they are pile supported. Alternatively, consideration can be given to preloading selected areas or to using structurally supported slabs on piles. Alternative improvement techniques including deep dynamic compaction and stone columns can also be considered.

Due to the proximity of this site to the Charleston area, there is a potential for liquefaction of some of the old fill as well as some of the loose to medium dense clean sands should an event similar to the 1886 Charleston earthquake occur. The probability of such an event is presently estimated at 2% in a 50 year period, or a return period of about 2,500 years on average. We anticipate that the owners probably will not want undertake a program of intentional densification of these soils to reduce the potential for liquefaction in the design earthquake for this site. If you do desire any recommendations with respect to improvement for liquefaction resistance, please notify us, and we will complete a liquefaction analysis and provide appropriate recommendations for mitigation if necessary.

ADDITIONAL ANALYSIS/INVESTIGATION

As discussed, details regarding the new construction were limited at the time of our investigation, and it is anticipated that our report will need to be modified as those details become available. We would appreciate being provided with development details when they become available so that appropriate modifications can be made. If the new bulkhead will ultimately extend to the north and south of the travel lifts along the shoreline, consideration should be given to having additional borings performed on the landside of the alignment in those areas to evaluate conditions in these areas, and to supplement the water side borings.

GeoTechnologies, Inc. appreciates the opportunity to provide you with our services during this phase of the project. Please contact us if you should have questions regarding this information or if we may be of further assistance.

Sincerely,

GeoTechnologies, Inc.

Ernest L. Stitzinger, P.E.
Senior Engineer

David L. Israel, P.E.
SC Registration No. 13473

SOUTH CAROLINA
REGISTERED PROFESSIONAL ENGINEER
No. 13473
DAVID L. ISRAEL

ELS/pr-ebh/dli
Attachments

TABLE 1

RECOMMENDED SOIL DESIGN PARAMETERS

LANDSIDE BORINGS (LB-1 & LB-2)

DEPTH (FT)	DESCRIPTION	N (bpf)	UNIT WEIGHT (pcf)*	STRENGTH PARAMETERS
0-5	Sandy/Clayey Fill	5-13	115	$\phi = 30^{O}$
5-14	Organic Fill	1-4	90	**c**= 50 psf
14-17	Clean Sand	10-12	125	$\phi = 31^{O}$
17+	Marl	5-100+	125	**c** = 1,500 psf

*Assume water and buoyant weight below 2.5 feet

WATER BORING WB-1

DEPTH (FT)	DESCRIPTION	N (bpf)	UNIT WEIGHT (pcf)*	STRENGTH PARAMETERS
0-21	Water	N/A		N/A
21-28	Organic Clay & Wood	13-19	80	**c** = 50 psf
28+	Marl	9-100+	125	**c** = 1,500 psf

*Use buoyant unit weight below water

WATER BORINGS WB-2 & WB-3

DEPTH (FT)	DESCRIPTION	N (bpf)	UNIT WEIGHT (pcf)*	STRENGTH PARAMETERS
0-14	Water	N/A		N/A
14-23	Wood/Soft Clay/Silt	WOR-8	80	**c** = 25 psf
23-32	Clean Sand	7-28	125	$\phi = 31^{O}$
32+	Marl	9-100+	125	**c** = 1,500 psf

*Use buoyant unit weight below water

WATER BORINGS WB-4 & WB-5

DEPTH (FT)	DESCRIPTION	N (bpf)	UNIT WEIGHT (pcf)*	STRENGTH PARAMETERS
0-3	Water	N/A		N/A
3-10	Organic Silt & Wood	WOR-4	80	**c** = 25 psf
10-30	Clean Sand	4-20	125	$\phi = 31^{O}$
30+	Marl	9-100+	125	**c** = 1,500 psf

*Use buoyant unit weight below water

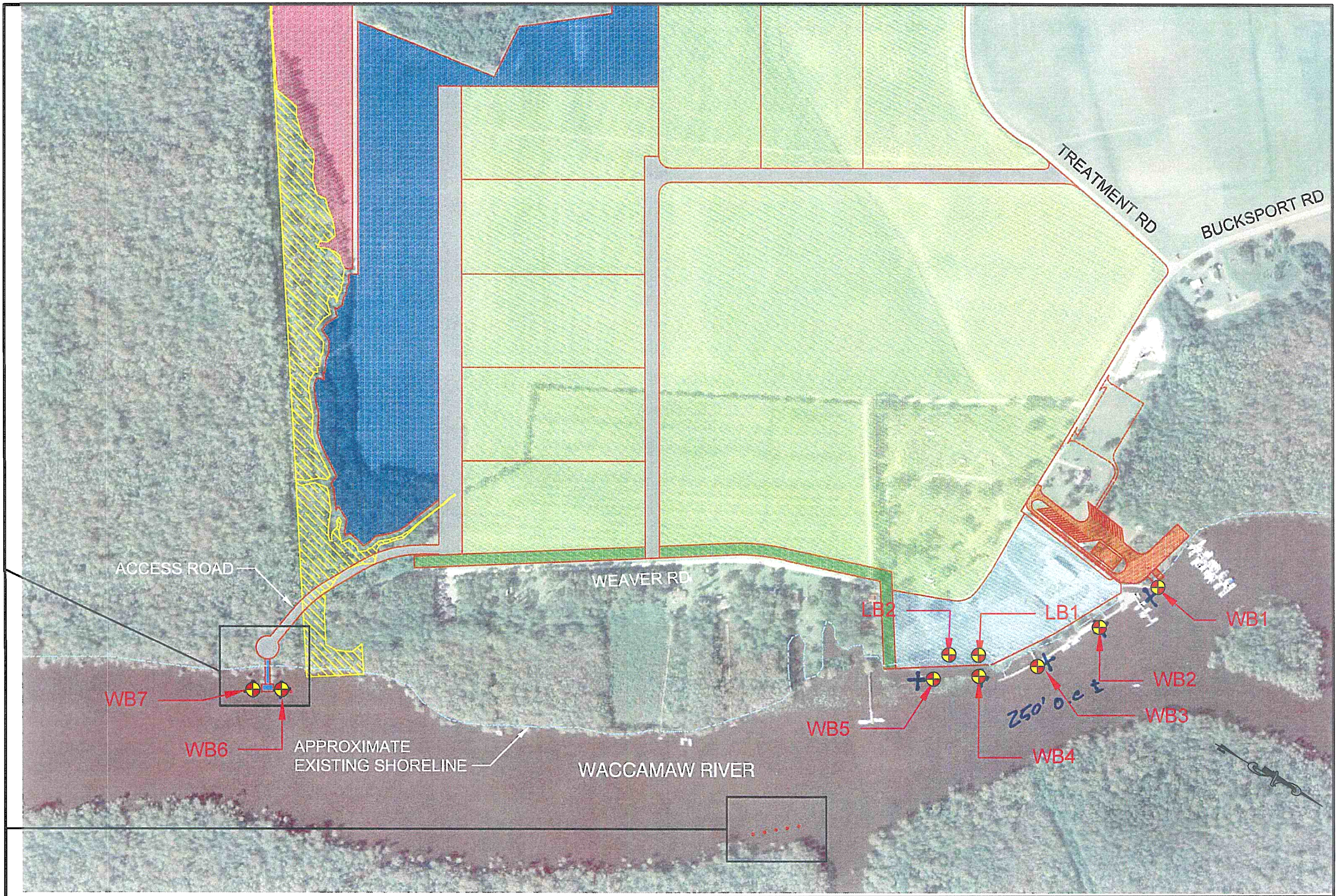
WATER BORINGS WB-6 & WB-7

DEPTH (FT)	DESCRIPTION	N (bpf)	UNIT WEIGHT (pcf)*	STRENGTH PARAMETERS
0-12	WATER	N/A		N/A
12-12.5	Organic Clay	N/A	90	50 psf
12.5-19	Sands	8-14	125	$\phi = 31^{\circ}$
19+	Marl	7-100+	125	**c** = 1,500 psf

*Use buoyant unit weight below water

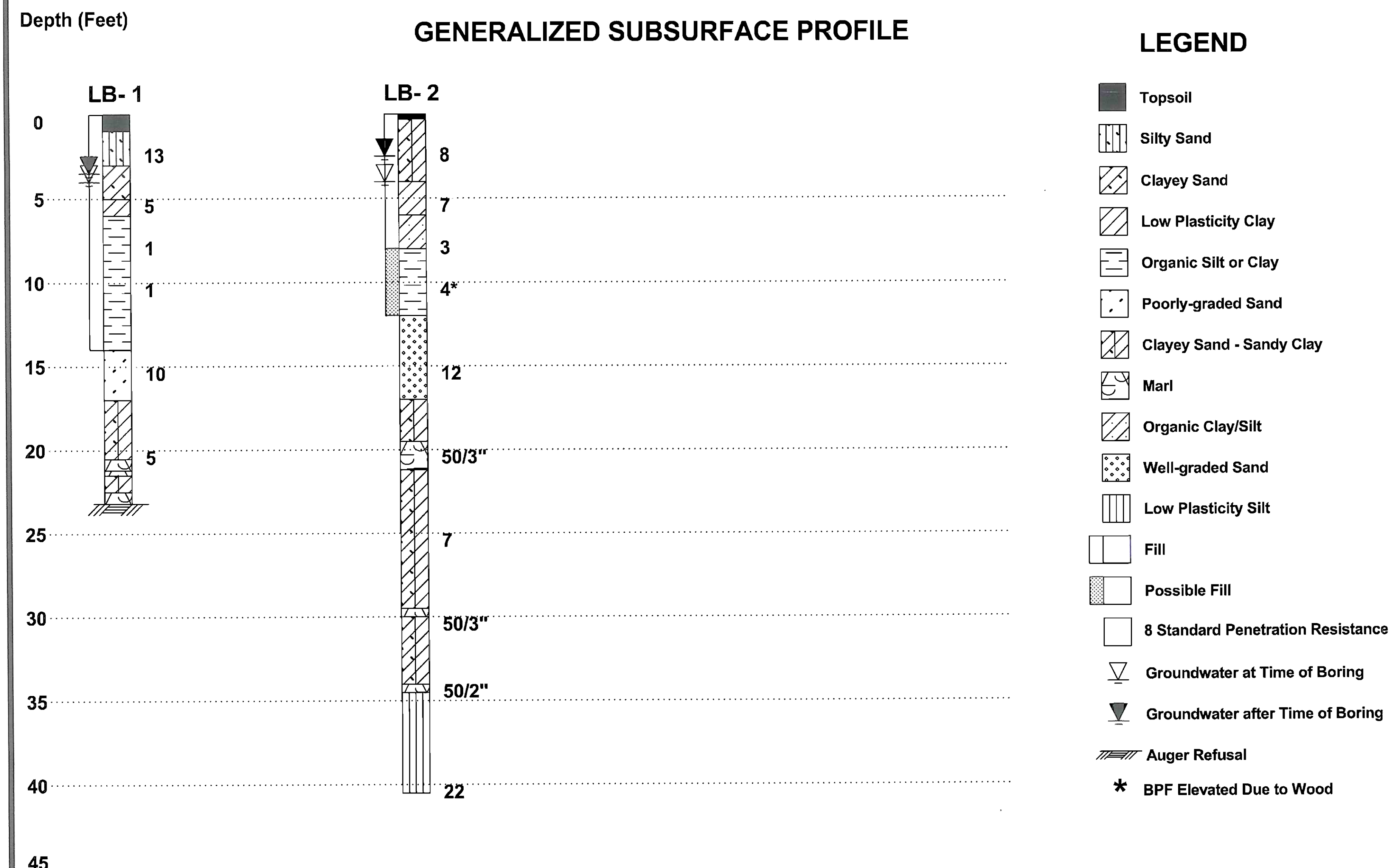
BORING NUMBER	DEPTH (FT)	% PASS #200 SIEVE	LL (%)	PL (%)	PI (%)
WB-1	29-30.5	58			
WB-1	32.5-34	59			
WB-4	31-32.5	65			
WB-4	36-37.5	60			
WB-4	41-41.7	52			
WB-5	31.5-33	68			
WB-5	36.5-38	68			
WB-6	24.5-26	30			
WB-6	44.5-46	67			
LB-2	39-40.5	67			
WB-3	36-37.5	62			
WB-3	46-47.5	83	72	27	45
WB-7	25.5-27	35			

TABLE 2: SUMMARY OF LABORATORY DATA



PROJECT: Bucksport Marina Bucksport, South Carolina	GeoTechnologies, Inc.	SCALE: Not To Scale JOB No: 1-11-0506-EA FIGURE No:1

GENERALIZED SUBSURFACE PROFILE



PROJECT:

Buckspport Marina
Bucksport, South Carolina

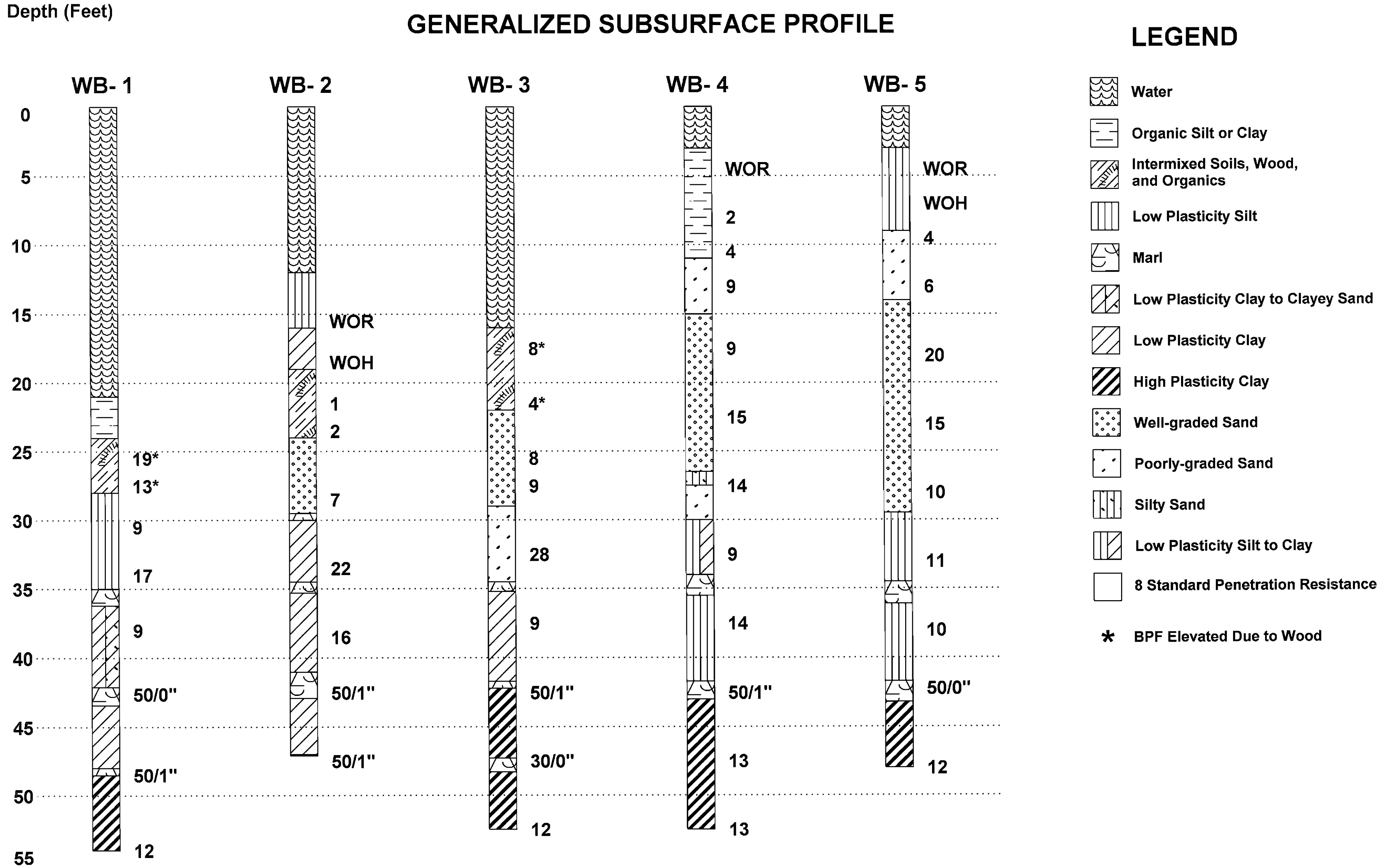
GeoTechnologies, Inc.

SCALE: As Shown

JOB No:1-11-0506-EA

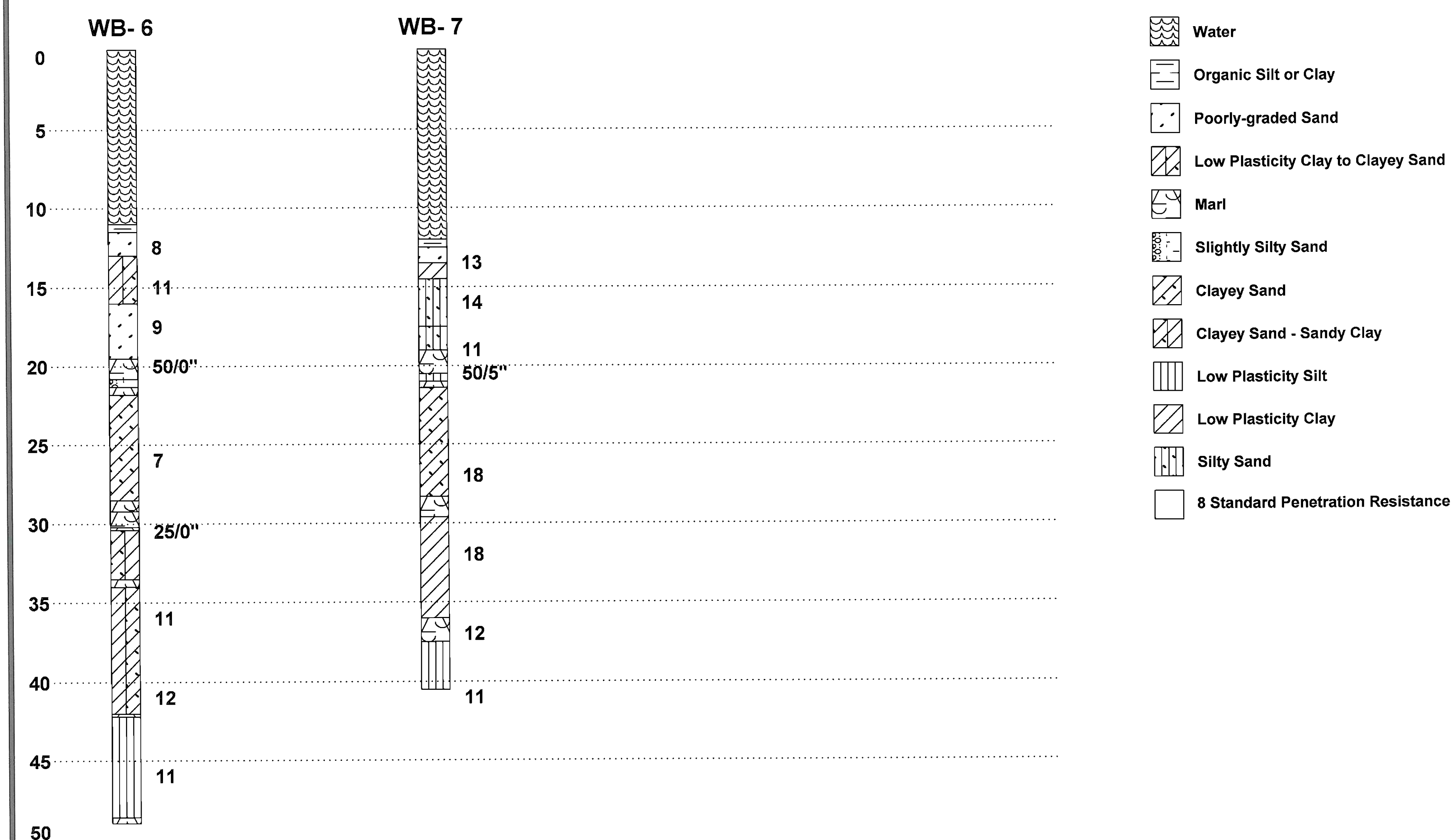
FIGURE No: 2A (Landside Borings)

GENERALIZED SUBSURFACE PROFILE



PROJECT: Bucksport Marina Bucksport, South Carolina	GeoTechnologies, Inc.	SCALE: As Shown JOB No:1-11-0506-EA FIGURE No: 2B (Bulkhead Borings)

GENERALIZED SUBSURFACE PROFILE



PROJECT:

Bucksport Marina
Bucksport, South Carolina

GeoTechnologies, Inc.

SCALE: As Shown

JOB No:1-11-0506-EA

FIGURE No: 2C (T-Dock Cargo Loading Borings)

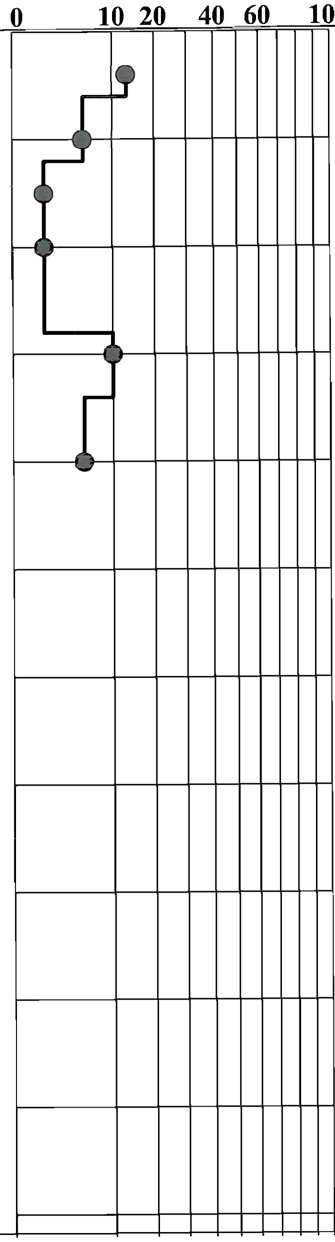
APPENDIX A

TEST BORING RECORDS

TEST BORING RECORD

DEPTH (FT.)	DESCRIPTION	ELEVATION (FT.)
0.0		
1.0	Fill - Topsoil/Organic Sand	
3.0	Fill - Medium Dense Tan Clayey Silty Fine to Medium SAND	SM
5.0	Fill - Loose Gray Black Clayey Fine to Medium SAND	SC
6.0	Fill - Soft Firm Tan Fine Sandy Silty CLAY	CL
14.0	Fill - Very Loose Black Brown Silty Clayey Fine to Medium SAND to Sandy Silty CLAY w/Wood (10% Organics by Weight)	OL
17.0	Loose Gray Slightly Silty Fine to Medium SAND	SP
20.5	Very Loose Gray Tan Clayey Fine to Coarse SAND to Olive Gray Fine Sandy Silty CLAY	SC CL
21.2	Marl (Soft)	
21.5	Marl (Hard - Cut 3-4" in 14 Minutes)	
22.5	Loose Gray Olive Silty Clayey Fine SAND to Sandy CLAY	SC CL
23.2	Marl (Hard - Last 1" = 4 Minutes)	
	Boring refusal at 23.2'	

PENETRATION (BLOWS/FT.)



BLOWS PER SIX INCHES

2-5-8
2-2-3
1-0-1
1-0-1
4-5-5
2-2-3

GTI_MAIN 110506.GPJ GTI.GDT 9/26/11

Groundwater encountered at 4' at time of boring and at 3.5' at 24 hours.

JOB NUMBER 1-11-0506-EA
BORING NUMBER LB- 1
DATE 8-15-11

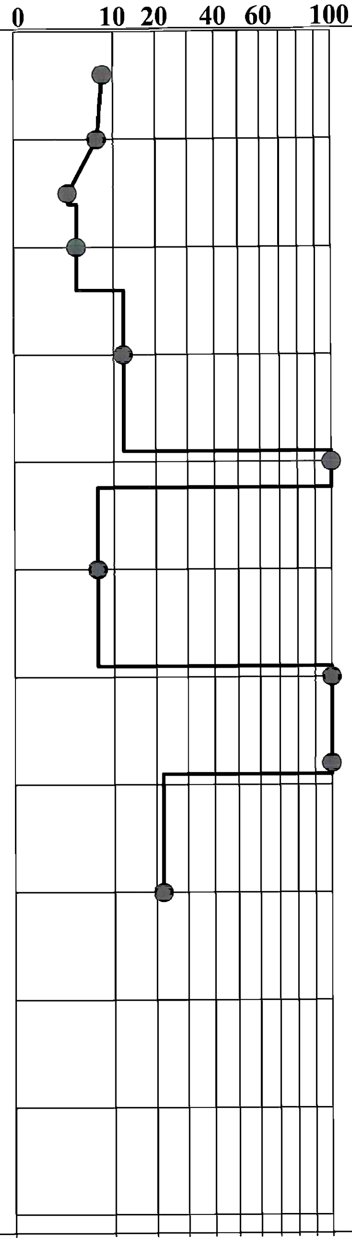
PAGE 1 OF 1

GeoTechnologies, Inc.
3200 Wellington Court, Suite 108
Raleigh, NC 27615

TEST BORING RECORD

DEPTH (FT.)	DESCRIPTION		ELEVATION (FT.)	BLOWS PER SIX INCHES
0.0				
0.3	Fill - Topsoil	SC CL		
4.0	Fill - Loose Tan Gray Silty Clayey Fine to Medium SAND to Sandy CLAY			3-4-4
6.0	Fill - Firm Brown & Tan Sandy Silty CLAY w/Few Organics	CL		1-4-3
8.0	Fill - Soft Black Gray Silty CLAY & Wood	CL OL		1-2-1
12.0	Possible Fill - Soft Black Gray Silty CLAY w/Lots of Wood (*Blow Counts Elevated Due to Wood)	OL		4-3-1*
17.0	Medium Dense Tan Medium SAND	SW		6-6-6
19.5	Loose Gray Olive Silty Clayey Fine to Medium SAND to Sandy CLAY w/Light Cementation	SC CL		
21.2	Marl (Cut 20" in 12 Minutes)			5-50/3"
29.5	Loose Gray Olive Silty Clayey Fine to Medium SAND to Sandy CLAY	SC CL		3-3-4
30.0	Marl (Cut 6" in 10 Minutes)			4-50/3"
34.0	Loose Gray Olive Silty Clayey Fine to Medium SAND to Sandy CLAY	SC CL		
34.5	Marl (Cut 6" in 3 Minutes)			50/2"
40.5	Very Stiff Gray Fine Sandy Clayey SILT w/Thin Harder Lenses	ML		13-14-8
	Boring terminated at 40.5'			

PENETRATION (BLOWS/FT.)



GTI_MAIN 110506.GPJ GTI.GDT 9/26/11

Groundwater encountered at 4' at time of boring and at 2.5' at 24 hours.

JOB NUMBER 1-11-0506-EA
BORING NUMBER LB- 2
DATE 8-15-11

GeoTechnologies, Inc.
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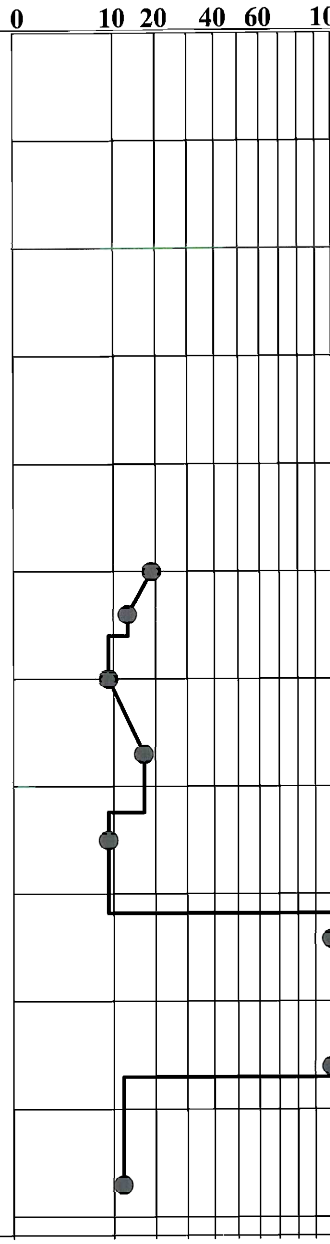
Raleigh, NC 27615

TEST BORING RECORD

DEPTH (FT.)	DESCRIPTION		ELEVATION (FT.)	PENETRATION (BLOWS/FT.)	BLOWS PER SIX INCHES
0.0	Water				
21.0	Very Soft Gray Black Organic Silty CLAY	OL			
24.0	Wood/Sand/Gravel (* Blow Counts Elevated Due to Wood)				23-15-4* 3-3-10*
28.0	Stiff to Very Stiff Olive Gray Fine Sandy Clayey SILT	ML			4-4-5 5-6-11
35.0	Marl (Cut 14" in 3 Minutes)				
36.2	Stiff Olive Gray Silty Fine Sandy CLAY to Clayey SAND	CL SC			4-4-5
42.1	Marl (Cut 15" in 6 Minutes)				50/0"
43.4	Stiff Gray Fine Sandy Silty CLAY w/Thin Harder Lenses	CL			
48.0	Marl (Cut 6" in 2 Minutes)				5-50/1"
48.5	Stiff Olive Gray Silty Fine Sandy CLAY	CH			4-5-7
54.0	Boring terminated at 54'				



GTI_MAIN 110506.GPJ GTI.GDT 9/26/11

JOB NUMBER 1-11-0506-EA
BORING NUMBER WB- 1
DATE 8-17-11

GeoTechnologies, Inc.
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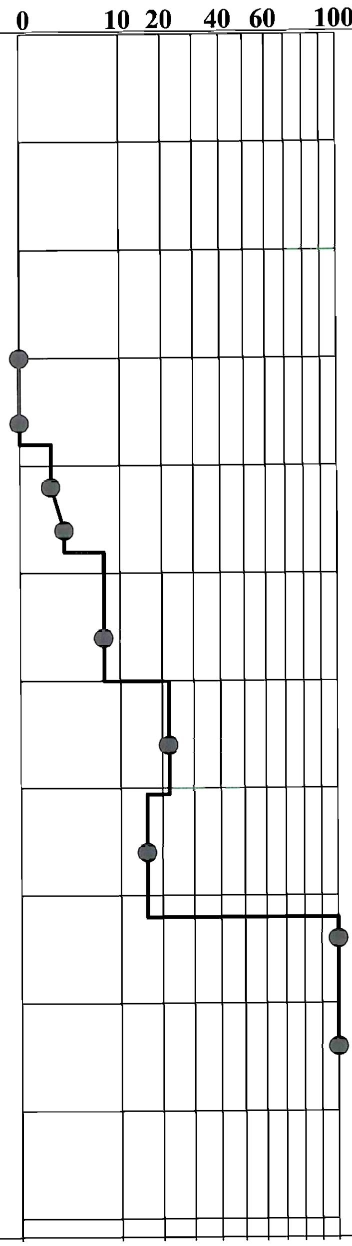
TEST BORING RECORD

DEPTH (FT.)	DESCRIPTION		ELEVATION (FT.)	BLOWS PER SIX INCHES
0.0	Water			
12.0	Very Soft Black Brown SILT	ML		WOR
16.0	Very Soft Black Slightly Fine Sandy Silty CLAY	CL		WOR-WOH
19.0	Wood and Sand			1-0-1
				3-1-1
24.0	Loose Tan Medium SAND	SW		4-3-4
29.5	Marl (Cut 6" in 1 Minute)			
30.0	Very Stiff Olive Gray Slightly Fine Sandy CLAY w/Thin Harder Lenses	CL		6-7-15
34.5	Marl (Cut 10" in 3 Minutes)			
35.3	Very Stiff Olive Gray Slightly Fine Sandy CLAY w/Thin Harder Lenses	CL		5-9-7
41.0	Marl (Cut 16" in 3 Minutes)			50/1"
42.9	Very Stiff Olive Gray Slightly Fine Sandy Sandy Silty CLAY	CL		50/1"
47.0	Marl			
47.1	Boring terminated at 47.1'			

PENETRATION (BLOWS/FT.)



GTI_MAIN 110506.GPJ GTI.GDT 9/26/11

JOB NUMBER 1-11-0506-EA
BORING NUMBER WB- 2
DATE 8-16-11

PAGE 1 OF 1

GeoTechnologies, Inc.
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TEST BORING RECORD

DEPTH (FT.)	DESCRIPTION		ELEVATION (FT.)	PENETRATION (BLOWS/FT.) 0 10 20 40 60 100	BLOWS PER SIX INCHES
0.0	Water				
16.0	Wood/Sand (*Blow Counts Elevated Due to Wood)				1-0-8*
					2-0-4*
22.0	Loose Tan Medium SAND	SW			2-4-4
					2-3-6
29.0	Medium Dense Gray Fine to Medium SAND	SP			9-14-14
34.5	Marl (Cut 8" in 4 Minutes)				
35.2	Stiff Olive Gray Fine Sandy Silty CLAY w/Thin Harder Lenses	CL			6-4-5
41.7	Marl (Cut 6" in 4 Minutes)				6-50/1"
42.2	Stiff Olive Gray Fine Sandy Silty CLAY w/Thin Harder Lenses	CH			4-5-30/0"
47.3	Marl (Cut 12" in 3 Minutes)				
48.3	Stiff Olive Gray Fine Sandy Silty CLAY	CH			4-6-6
52.5	Boring terminated at 52.5'				

GTI_MAIN 110506.GPJ GTI.GDT 9/26/11

JOB NUMBER 1-11-0506-EA
BORING NUMBER WB- 3
DATE 8-16-11

GeoTechnologies, Inc.
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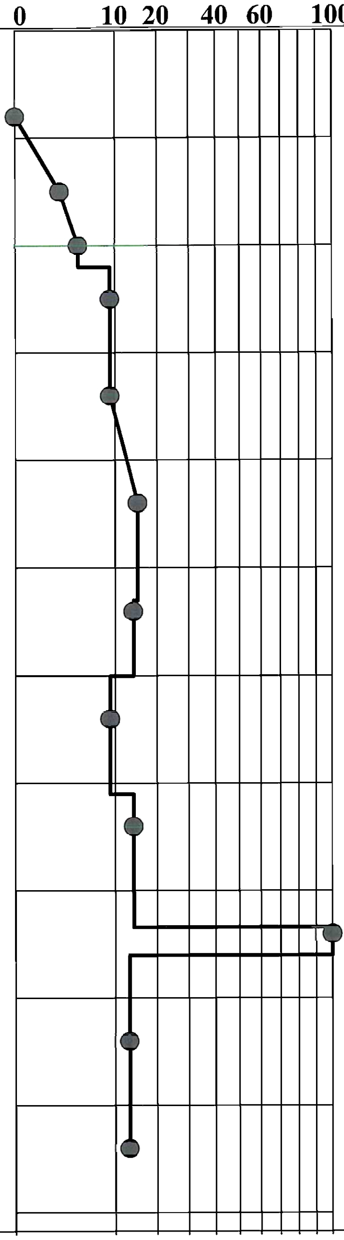
TEST BORING RECORD

DEPTH (FT.)	DESCRIPTION		ELEVATION (FT.)
0.0	Water		
3.0	Very Soft Black Organic SILT w/Lots of Wood	OL	
11.0	Loose Tan Slightly Silty Fine to Medium SAND w/Little Wood	SP	
15.0	Loose to Medium Dense Tan Gray Micaceous SAND	SW	
26.5	Very Loose to Loose Gray Black Silty Fine SAND w/Wood	SM	
27.5	Medium Dense Gray Fine to Medium SAND	SP	
30.0	Stiff Olive Gray Clayey Fine Sandy SILT	ML CL	
34.0	Marl (Cut 18" in 3 Minutes)		
35.5	Stiff Olive Gray Clayey Fine Sandy SILT	ML	
41.7	Marl (Cut 16" in 3 Minutes)		
43.0	Stiff Olive Gray Fine Sandy Silty CLAY w/Thin Harder Lenses	CH	
52.5	Boring terminated at 52.5'		

PENETRATION (BLOWS/FT.)



BLOWS PER SIX INCHES

WOR
1-1-1
1-2-2
3-4-5
2-4-5
7 7-8
1-4-10
3-4-5
9-7-7
6-50/1"
4-6-7
4-6-7

GTI_MAIN 110506.GPJ GTI.GDT 9/26/11

JOB NUMBER 1-11-0506-EA
BORING NUMBER WB- 4
DATE 8-16-11

PAGE 1 OF 1

GeoTechnologies, Inc.
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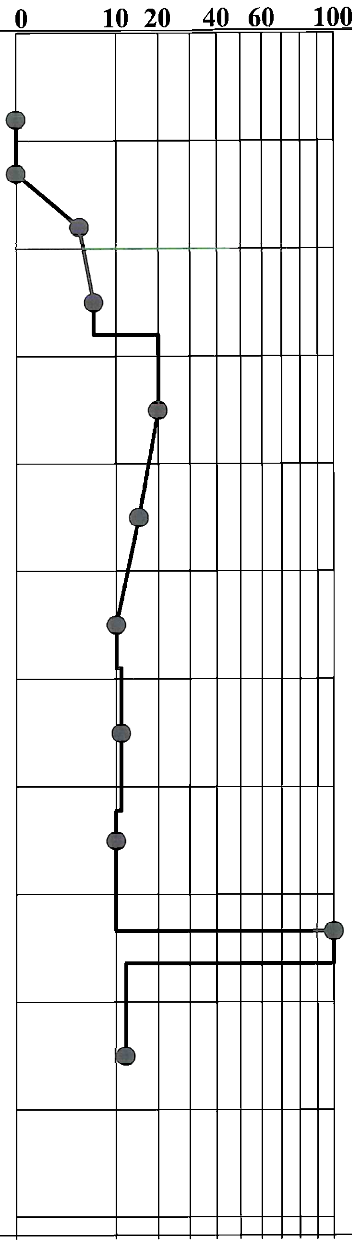
TEST BORING RECORD

DEPTH (FT.)	DESCRIPTION		ELEVATION (FT.)	BLOWS PER SIX INCHES
0.0	Water			
3.0	Very Soft Brown Silty Fine Sandy Clayey SILT w/Organics & Wood	ML OL		WOR
				WOR-WOH
9.0	Loose Gray Tan Fine to Medium SAND w/Little Wood	SP		WOR-1-3
				6-3-3
14.0	Medium Dense Tan Micaceous SAND w/Few Roots	SW		6-9-11
				3-6-9
				6-7-3
29.5	Stiff Olive Gray Fine Sandy Clayey SILT	ML		3-5-6
34.5	Marl (Cut 19" in 5 Minutes)			
36.1	Loose Olive Gray Clayey Fine Sandy SILT	ML		5-5-5
41.7	Marl (Cut 18" in 5 Minutes)			50/0"
43.2	Stiff Olive Gray Slightly Fine Sandy Silty CLAY w/Thin Harder Lenses	CH		5-6-6
48.0	Boring terminated at 48'			

PENETRATION (BLOWS/FT.)



GTI_MAIN 110506.GPJ GTI.GDT 9/26/11

JOB NUMBER 1-11-0506-EA
BORING NUMBER WB- 5
DATE 8-17-11

GeoTechnologies, Inc.
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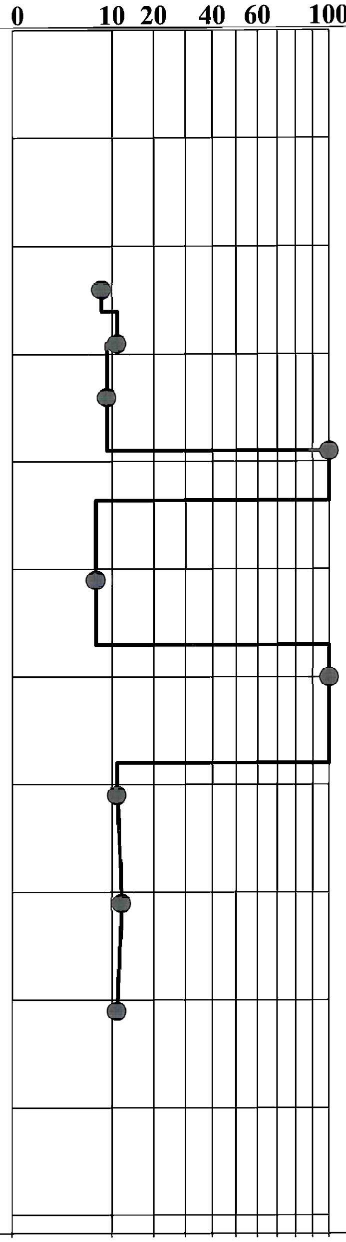
TEST BORING RECORD

DEPTH (FT.)	DESCRIPTION	ELEVATION (FT.)
0.0	Water	
11.0	Soft Organic Silty CLAY	OL
11.5	Loose Gray Slightly Silty Fine SAND	SP
13.0	Stiff Gray Slightly Fine Sandy CLAY to Clayey Fine SAND w/ Little Wood	CL SC
16.0	Loose Gray Silty Fine SAND	SP
19.5	Marl (Cut 15" in 14 Minutes)	
20.8	Loose Gray Silty Fine SAND	SM
21.3	Marl (Cut 6" in 6 Minutes)	
21.8	Loose Gray Olive Silty Clayey Fine SAND	SP SC
28.5	Marl (Cut 8" in 1 Minute)	
29.2	Marl (Cut 12" in 6 Minutes)	
30.2	Marl (Cut 2" in 10 Minutes)	
30.4	Loose Olive Gray Silty Clayey Fine to Medium SAND to Sandy CLAY w/Thin Harder Lenses	SC CL
33.5	Marl (Cut 6" in 1.5 Minutes)	
34.0	Stiff Olive Gray Silty Fine sandy CLAY to Clayey SAND w/Thin Harder Lenses	CL SC
42.0	Marl (Cut 14" in 6 Minutes)	
42.2	Stiff Olive Gray Fine Sandy Clayey SILT	ML
48.6	Marl (Cut 4" in 2 Minutes)	
49.0	Boring terminated at 49'	

PENETRATION (BLOWS/FT.)



BLOWS PER SIX INCHES

3-4-4
5-6-5
4-3-6
50/0"
3-3-4
25/0"
4-5-6
4-5-7
4-5-6

GTI_MAIN 110506.GPJ GTI.GDT 9/26/11

JOB NUMBER 1-11-0506-EA
BORING NUMBER WB- 6
DATE 8-17-11

PAGE 1 OF 1

GeoTechnologies, Inc.
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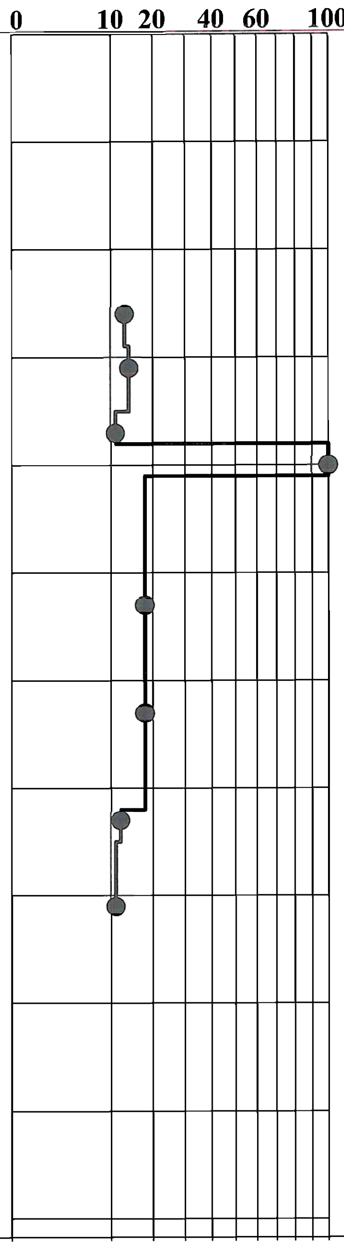
Raleigh, NC 27615

TEST BORING RECORD

DEPTH (FT.)	DESCRIPTION		ELEVATION (FT.)
0.0	Water		
12.0	Very Soft Black Brown Organic Silty CLAY	OL	
12.5	Medium Dense Gray Tan Silty Fine SAND	SP	
13.5	Stiff Gray Fine Sandy Silty CLAY	CL	
14.5	Medium Dense Gray Slightly Clayey Silty Fine to Medium SAND w/Little Wood	SM	
17.5	Medium Dense Olive Gray Slightly Clayey Silty Fine SAND	SM	
19.0	Marl (Cut 18" in 23 Minutes)		
20.5	Medium Dense Gray Clayey Silty Fine SAND (Cemented)	SM	
21.0	Marl (Cut 4" in 1 Minute)		
21.4	Medium Dense Gray Silty Clayey Fine SAND w/Thin Harder Lenses	SC	
28.3	Marl (Cut 15" in 7 Minutes)		
29.6	Stiff to Very Stiff Olive Gray Silty Fine Sandy CLAY	CL	
36.0	Marl (Cut 18" in 8 Minutes)		
37.5	Stiff Olive Gray Silty Fine Sandy Clayey SILT	ML	
40.5	Boring terminated at 40.5'		

PENETRATION (BLOWS/FT.)



BLOWS PER SIX INCHES
1-6-7
11-6-8
6-5-6
15-50/5"
2-3-15
6-8-10
4-5-7
5-5-6

GTI_MAIN 110506.GPJ GTI.GDT 9/26/11

JOB NUMBER 1-11-0506-EA
BORING NUMBER WB- 7
DATE 8-18-11

PAGE 1 OF 1

GeoTechnologies, Inc.
3200 Wellington Court, Suite 108
Raleigh, NC 27615

GeoTechnologies, Inc.

Geotechnical and Construction Materials Testing Services

November 15, 2011

Brad Woodall
MOFFATT & NICHOL
4975 Lacrosse Road, Suite 151
N. Charleston, SC 29406

Re: Test Pile Program
Bucksport Marine Industrial Park
Bucksport, South Carolina
GeoTechnologies Project No. 1-11-0506-EA

Gentlemen:

GeoTechnologies, Inc. completed a subsurface investigation for the referenced site dated October 3, 2011. Included within our report was the recommendation that an extensive test pile program be conducted in advance of production pile installation to resolve uncertainties related to pile drivability and capacities associated with the presence of a marl layer at depth. Our specific concerns related to pile driveability were a) could the selected pile sections be driven to the penetration depths necessary to achieve design capacity and b) could the piles be driven without damage. Our concerns related to pile capacity involved uncertainties with the shear strength of the marl as that stratum contains alternating layers of very hard cemented soils and relatively soft sands, silts, and clays. Based on our experience with similar subsurface conditions, it is our opinion that attempting pile installation without the test program could result in a) over design and/or excessive contractor bids associated with unresolved uncertainties, and b) schedule overruns, change orders, and liquated damage claims in the event that pile sections can not be driven without damage, or if the sections do not achieve design capacity.

It is our understanding that the marina facilities design primarily consists of a marine travel lift slip. The slip will be comprised of a pile supported concrete pier on one side and a steel bulkhead along the other side and at the head of the slip. As such, the design will consist of three different pile sections:

1. 18" square prestressed concrete piles with steel stingers designed for a working load of 80 to 100 tons.
2. Steel H-section (likely either HP 14x73 or HP 14x117) piles designed for a working load of 80 to 100 tons.
3. Standard Z-shape steel sheeting.

It is anticipated that the bulkhead will be an anchored design since the wall height will exceed 20 feet. Our site exploration also included borings to evaluate soil conditions for a T-Dock construction further south from the travel lift. We understand that the initial phase of development will not include this construction; however, we expect that the recommended test pile program and subsequent production pile installation for the travel lift can be used to evaluate future installations in the T-Dock area to eliminate the need for an additional test pile program at that location in the future.

To help minimize costs associated with the test pile program, we recommend that the test piles be driven on land in the area of planned construction. It should be recognized that the depth to the marl layer was generally 10 to 15 feet shallower in the land borings relative to nearby water borings, and as such, production piles driven in water may be somewhat deeper than those subsequently recommended for the test piles. However, the results of a landside test pile program can be used to evaluate pile lengths and capacities for all production piles.

Using the generated subsurface profiles in the area of the travel lift, we estimate that an HP 14x73 or HP 14x117 pile will achieve a design working load of 100 tons in skin frication at a depth of about 70 feet, or about 50 feet into the marl. The actual depths to achieve the design capacity will likely be lesser due to some contribution from end bearing. As such, we recommend that the test pile program include four HP sections as follows:

1. (1) 80' length 14x73 section
2. (1) 80' length 14x117 section
3. (1) 60' length 14x73 section
4. (1) 60' length 14x117 section

The different pile lengths and sections are intended to help estimate the marl shear strength, and evaluate driveability with respect to pile damage. Driving should be monitored with a PDA, and if pile damage is indicated, driving shoes should be used. The piles should be driven either to the maximum depth which allows for restriking, or until the design capacity is achieved using the minimum energy possible to limit damage potential. The pile should set-up for at least 72 hours prior to a PDA monitored restrike. Design capacities should be based on the restrike data. Subsequently, the pile should be removed with a crane and vibratory extractor to inspect for damage.

Static calculations indicate that an 18" composite concrete/steel pile with a 40' stinger will achieve a design capacity of 80 to 100 tons. In the area of the travel lift, we anticipate that the total pile length with a 40' stinger will be about 65', and therefore, we recommend that 75' test piles be driven. We recommend driving one pile with an HP 14x73 stinger and one with an HP 14x117 stinger to help assess pile driveability. Driving should be monitored with a PDA, and if pile damage is indicated, driving shoes should be used. The piles should be driven either to the maximum depth which allows for restriking, or until the design capacity is achieved using the minimum energy possible to limit damage potential. The pile should set-up for at least 72 hours prior to a PDA monitored restrike. Design capacities should be based on the restrike data. Subsequently, the pile should be removed with a crane and vibratory extractor to inspect for damage.

Attempts to install sheeting can be made with a vibratory hammer although the piles may have to be driven once the initial cemented zone is encountered. If the sheeting cannot be driven into the marl, or if the sheets are damaged, an attempt can be made to drive the sheeting with driving shoes. Successfully driven piles should be extracted and inspected for damage. If specified embedment cannot be achieved, king pile and/or tie-back installation may need to be modified by designer.

Please contact us if you have questions regarding this information or if we can be of any further assistance.

Sincerely,

GeoTechnologies, Inc.

Ernest L. Stitzinger, P.E.
Senior Engineer

David L. Israel, P.E.
SC Registration No. 13473

SOUTH CAROLINA
PROFESSIONAL ENGINEER
No. 13473
REGISTERED
DAVID L. ISRAEL
11/15/2011

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