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Geotechnical Exploration and Evaluation Report

Beverly Hills West Septic Tank Phase-Out
Jacksonville, Florida

CSI Geo Project No.: 71-18-329-09
Client Project No.: 09302-054-01
JEA Contract No.: 177312
Purchase Order: 178195

Prepared by

CSI Geo, Inc.
2394 St. Johns Bluff Road S., Suite 200
Jacksonville, FL 32246
Tel: (904) 641-1993
Fax: (904) 641-0057

Prepared for

Jones Edmunds & Associates, Inc.

June 19, 2019

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June 19, 2019

Dr. Harold Bridges, Ph.D, P.E.
Jones Edmunds & Associates, Inc.
8657 Baypine Road, Suite 300
Jacksonville, FL 32256-8634

RE: Beverly Hills West Septic Tank Phase-Out
Jacksonville, Florida

Subject: Geotechnical Exploration and Evaluation Report
CSI Geo Project No.: 71-18-329-09
Client Project No.: JEA 09302-054-01
JEA Contract No.: 177312
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Dear Dr. Bridges:

CSI Geo, Inc. has performed the authorized geotechnical exploration and laboratory testing program for the proposed Beverly Hills West Septic Tank Phase-Out project in Jacksonville, Florida. This report presents our understanding of the subsurface conditions along with our engineering evaluation and recommendations for the new manholes and lift station construction.

We have enjoyed working with you on this project and look forward to working with you on future projects. If you have any questions concerning this report, please contact our office.

Sincerely,

CSI Geo, Inc.

Nader Amer, Ph.D
Geotechnical Engineer

Digitally signed by Bruce B Khosrozadeh
Date: 2019.06.19 15:45:25-04'00'

BRUCE KHOSROZADEH
LICENSE
No 45273
STATE OF FLORIDA
PROFESSIONAL ENGINEER

Bruce Khosrozadeh, P.E.
Senior Geotechnical and
Materials Engineer
Registered, Florida No. 45273

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1.0 PROJECT INFORMATION

1.1 General Project Information

The purpose of this geotechnical exploration program was to develop information concerning the subsurface conditions in order to evaluate the site with respect to the proposed Beverly Hills West Septic Tank Phase-Out Project in Jacksonville, Florida.

This report describes the field and laboratory testing activities performed and presents the findings. The subsurface soil and groundwater conditions are presented in this report along with site preparation and construction recommendations.

Information regarding this project was provided to CSI Geo, Inc. (CSI Geo) by Dr. Harold Bridges, Ph.D., P.E, and Mr. Kenneth A. Fraser, P.E. of Jones Edmunds & Associates, Inc. (Jones Edmunds). The following documents have been furnished regarding this project:

- Beverly Hills West Septic Tank Phase-out Plan and Profile sheets (30% Phase)
  Prepared by: Jones Edmunds
  Provided: April 12, 2019

1.2 Existing Conditions and Project Description

Currently the Beverly Hills west area sewer system consists of septic tanks and drain fields located in residential lots. These septic tanks were identified to be failing and could eventually cause negative environmental and health impacts. Therefore, the Beverly Hills sewer west improvements include the construction of a central wastewater collection system with manholes that will connect to the existing JEA collection system grid and will eventually result in phasing out the existing septic tank system. The project also consists of the construction of a new lift station.

2.0 GEOTECHNICAL EXPLORATION

2.1 Field Exploration

The areas of the new manholes were explored by means of a total of sixty-four (64) Standard Penetration Test (SPT) borings B-1 through B-64. The approximate locations of the soil borings are shown on the Field Exploration Plan sheets included in the **Appendix**. All borings were drilled to depths ranging from 10 to 25 feet below the existing grades. In addition, one test boring (LS-1) was performed for the proposed lift station and was drilled to a depth of 25 feet below the existing grades.

The boring locations were located in the field by personnel from CSI Geo and placed as close as possible to the proposed manhole locations. Soil samples collected were visually classified in the field and then transported to our laboratory for re-classification and testing. Representative soil samples obtained during our field exploration program for the manholes were visually classified using the American Association of State Highway and Transportation Officials (AASHTO) Soil Classification System. The soil samples obtained from the lift station exploration program were visually classified using the Unified Soil Classification System (USCS).

2.2 Laboratory Testing

Quantitative laboratory testing was performed on representative soil samples to better define their composition. Laboratory tests performed were percent fines, natural moisture content, organic content, and Atterberg limits. A Summary of Laboratory Test Results, and Field and Laboratory Test Procedures are included in the **Appendix**.

3.0 GENERAL SUBSURFACE CONDITIONS

3.1 General

An illustrated representation of the subsurface conditions encountered in the proposed construction areas are shown on the Report of SPT Borings sheets presented in the **Appendix**. The Report of SPT Borings and the soil conditions outlined below highlight the major subsurface stratification. The Report of SPT Borings in the **Appendix** should be consulted for a detailed description of the subsurface conditions encountered at each boring location. When reviewing the Report of SPT Borings, it should be understood that soil conditions may vary outside of the explored area.

3.2 Soil Conditions

3.2.1 Manholes

Review of SPT borings B-1 through B-64 indicates that the existing pavement system is generally underlain by very loose to medium dense sands and slightly silty sands (A-3, AASHTO), silty sands (A-2-4), soft to very stiff clayey sands (A-2-6), sandy clays (A-6) and clays with sand (A-7-6) until the boring termination depths of 10 to 25 feet below the existing grades. Isolated and intermittent deposits of peat and highly organic sands (A-8) were encountered in the upper 2.5 feet in test boring B-3 and between the depths of 4 and 6 feet in test boring B-63.

3.2.2 Lift Station

Review of SPT boring LS-1 indicates that the area of the lift station is generally underlain by loose to medium dense sands (SP, USCS), slightly silty sands (SP-SM), and firm clayey sands (SC) until the boring termination depth of 25 feet below the existing grade.

3.3 Groundwater Conditions

The groundwater level was measured and recorded as encountered at the time of drilling. The depths of the groundwater level and estimated seasonal high water level at the test locations are marked on the Report of SPT Borings sheets presented in the **Appendix**. The depth of groundwater level measured at the time of drilling ranged from 1.5 to 8.5 feet below the existing grades. The estimated seasonal high groundwater table for the borings performed ranged from 1.0 to 7.0 feet below the existing grades.

Determination of the estimated seasonal high groundwater table was made using the methodology described by the United States Department of Agriculture (USDA) Soil Conservation Service (SCS). In sandy soils the method involves examining soil cuttings from an auger boring for subtle changes in root content, soil coloration, and the presence of a polychromatic matrix. These subtle changes are indicators of the highest level the groundwater level has been for a prolonged period. In heavily developed commercial areas, the method may produce misleading results, since man-induced drainage modifications are not taken into consideration.

Fluctuations of the groundwater level should be anticipated as a result of the close proximity to the St. Johns River, urbanization/development, topographic changes, seasonal climatic variations, surface water runoff patterns, fluctuations of adjacent water bodies, construction activities, and other factors. During seasonal high precipitation, groundwater levels can be expected to rise. Therefore, design drawings and specifications should account for the possibility of groundwater level variations, and construction planning should be based on the assumption that such variations will occur.

3.4 Existing Pavement System Thickness

Pavement cores were performed at each of the test boring locations to determine the thickness of the existing pavement system. Generally, the existing pavement system was found to consist of 2 to 12 inches of asphalt over 1 to 4 inches of limerock base course. Limerock base was not encountered in many locations. The results of the pavement cores are included in the **Appendix**.

4.0 DESIGN RECOMMENDATIONS

4.1 General

Our geotechnical evaluation of the site and the subsurface conditions is based on our understanding of the proposed project, our observations, and results of field and laboratory testing. The recommendations provided in this report present construction methods and techniques that are appropriate for the proposed construction. If the project location or installation method is changed or if field conditions encountered during construction are different from those presented in this report, the information should be provided to CSI Geo for evaluation. We also recommend that CSI Geo be given the opportunity to review the design plans and specifications to ensure that our recommendations have been properly included and implemented.

4.2 Evaluation of Manholes

In general, we consider the subsurface soil conditions at the site to be favorable for support of the proposed manhole structures over a properly prepared and compacted subgrade, provided that the site preparation and earthwork construction recommendations in this report are performed. Based on our subsurface findings and our understanding of the proposed construction, it is our opinion that the site and subsurface conditions are adequate for support of the proposed manhole foundations with a maximum allowable soil bearing capacity of 2,000 pounds per square foot (psf).

Review of test borings B-1 through B-64 indicates that the proposed manhole locations are generally underlain by sands and slightly silty sands (A-3), silty sands (A-2-4), clayey sands (A-2-6), sandy clays (A-6) and clays with sand (A-7-6). Isolated peat and highly organic sands (A-8) encountered in test borings B-3 and B-63.

The A-3 type soils are considered select material. Silty sands (A-2-4) can be treated as select material, however, they may contain excess moisture and may be difficult to dry and to compact. Clayey sands (A-2-6) and sandy clays (A-6/A-7) are considered plastic and highly plastic materials. Unsuitable organic and highly organic sands (A-8) should be considered as muck and not suitable for use as backfill.

The presence of A-2-6 and A-6/A-7 type soils should be expected throughout the site at varying depths below the existing grades. Depending on the depth of excavation, it is very likely that the excavated suitable soils may get mixed with plastic soils during excavation activities. Therefore, we recommend that allowances be made for possible overruns in quantities of subsoil removal and replacement with select backfill.

We anticipate that the buried pipe lines connecting the manholes will exert little downward pressure on the subgrade soils. In areas where the surrounding groundwater level is above the pipe invert elevation, the line should be designed to resist lateral earth pressures and hydrostatic uplift pressures appropriate to its depth below the existing grade and the seasonal high water level.

4.3 Evaluation of Lift Station

Review of test boring LS-1 indicates that the area is generally underlain by loose to medium dense sands and slightly silty sands (A-3) and firm clayey sands (A-2-6) until the boring termination depth. Based on our subsurface findings and our understanding of the proposed construction, it is our opinion that the site and subsurface conditions are capable of providing adequate support for the proposed lift station founded on conventional shallow foundation systems proportioned for a maximum allowable soil bearing pressure of 2,000 pounds per square foot (psf). Using a 2,000 psf bearing pressure, we estimate that total settlement of the structure will be on the order of 1 inch or less.

The bearing level soils, should be compacted to densities equivalent to 95% of the Modified Proctor maximum dry density (ASTM D1557). The foundation bearing level soils should also be inspected and tested by an engineering technician, acting under the direction and supervision of the geotechnical engineer, in order to evaluate the density and acceptability of the foundation bearing material prior to placement of reinforcing steel and footing construction.

As a general rule, below grade structures should be designed to resist hydrostatic uplift pressures appropriate for their depth below the wet seasonal groundwater table. The water table for hydrostatic uplift design purposes should be assumed to be 1.0 foot below prevailing grade. The lift station should be designed to withstand lateral earth pressures as well as hydrostatic pressures on the wall. Soil parameters which can be used for the design are presented as follows:

RECOMMNEDED SOIL PARAMETERS FOR LIFT STATION DESIGN

Boring LS-1

Soil Classification	Loose to Medium Dense Sands	Loose Clayey Sands	Medium Dense Sands	Loose Sands	Medium Dense Sands
Depth (ft)	0.0 to 6.0	6.0 to 12.0	12.0 to 17.0	17.0 to 22.0	22.0 to 25.0
Saturated unit weight (pcf)	110	100	115	100	110
Effective unit weight for input purposes (pcf)	48	38	53	38	48
Estimated friction angle ϕ (degrees)	32	---	33	28	32
Friction angle between soil and wall δ (degrees)	21	---	22	19	21
Cohesion C (psf)	---	900	---	---	---
Adhesion C_A (psf)	---	450	---	---	---
At-Rest Earth Pressure Coefficient K_o	0.47	0.55	0.46	0.53	0.47
Active Earth Pressure Coefficient K_a	0.31	0.38	0.29	0.36	0.31
Passive Earth Pressure Coefficient K_p	3.25	2.66	3.39	2.77	3.25
Modulus of Subgrade Reaction K (pci)	150	150	200	150	200

5.0 SITE PREPARATION & EARTHWORK RECOMMENDATIONS

5.1 Existing Utilities

The locations of existing utilities should be established prior to construction. Provisions should be made to relocate utilities interfering with the proposed alignments and construction, as needed. Underground pipes that are not operational should be either removed or grouted in place otherwise they may become conduits for subsurface erosion and cause settlements.

5.2 Dewatering for Lift Station and Manhole Construction

Groundwater level was encountered at the time of drilling at a depth ranging from 1.5 to 8.5 feet below the existing grades. Lowering the groundwater level at this site by means of an extensive wellpoint system will be needed. Supplemental pumping from sump pumps will also be necessary to remove any water not removed by the wellpoints. Ideally, the water table should be lowered to a level at least one foot below the bottom of any excavations made during construction and at least two feet below the level of any vibratory compaction operations. We recommend that the dewatering system be kept operational until the lift station and manhole are constructed and that the dewatering system should not be turned off unless approved by the Engineer. It is also recommended that the groundwater level should be monitored on a regular basis to ensure that the groundwater level stays below the bottom of the lift station elevation during construction.

5.3 Excavation Protection

All excavations should meet OSHA Excavation Standard Subpart P regulations for Type C soils. If needed, trench box or braced sheet pile structures may be used where deep installation is required. The soil support system should be designed by a Florida registered Professional Engineer.

5.4 Backfill and Compaction of Backfill

If the excavated suitable soils get mixed with unsuitable soils during construction, then the excavated material should be regarded as unsuitable for backfill purposes. We recommend that allowance be made for overruns in quantities of subsoil removal and replacement with select backfill.

Backfill should be placed in layers of not more than 12 loose inches and mechanically tamped to 95% Standard Proctor Density. All foundation bases should be set level on bedding consisting of 12 inches (at a minimum) of granular material (57 stone) in accordance with the JEA Water and Wastewater Standard Specifications.

Silts, clays, and unsuitable organic and highly organic soils (AASHTO Class A-4, A-6, A-7, & A-8 and USCS Class ML, CL, OL, MH, CH, OH & PT) should be over-excavated an additional 24 inches (at a minimum) and backfilled with AASHTO Class A-3 soil and compacted to 98% (ASTM D1557), or over-excavated an additional 12 inches (at a minimum) and backfilled with granular backfill (57 stone). All materials should be provided in accordance with the JEA Water and Wastewater Standard Specifications.

For pipelines connecting the manholes installed by means of open cut excavation, the backfill material within the excavation should be placed in thin loose lifts not exceeding 12 inches in thickness as required by JEA. The backfill material should be compacted by the use of hand-operated equipment. The backfill material should be granular (A-3) fill with less than 10 percent material passing the no. 200 mesh sieve and containing less than 3 percent organic matter. The backfill material should be compacted to a minimum density of 98% or 95% of maximum dry density obtained from the Modified Proctor compaction test (ASTM D1557), as required by JEA. The moisture content during compaction should be maintained within ± 3 percent of the optimum moisture content as obtained from the Modified Proctor compaction test. Hand held compaction equipment should be used for the backfill placed around pipelines and to a height of 2 feet above the pipe. Heavier equipment may be used on the remaining backfill lifts placed above the 2 feet above the pipe. However, care should be taken not to damage the pipe below. The pipe should be designed to withstand the anticipated dead (overburden) and live loads.

6.0 REPORT LIMITATIONS

The subsurface exploration program including our evaluation and recommendations was performed in general accordance of accepted geotechnical engineering principles and standard practices. CSI Geo is not responsible for any independent conclusions, opinions, or interpretations made by others based on the data presented in this report.

This report does not reflect any variations that may occur adjacent or between soil borings. The discovery of any site or subsurface condition during construction that deviates from the findings and data as presented in this report should be reported to CSI Geo for evaluation. If the locations of the proposed features are changed, our office should be contacted so our recommendations can be re-evaluated. We recommend that CSI Geo be given the opportunity to review the final design drawings and specifications to ensure that our recommendations are properly included and implemented.

APPENDIX

Site Location Map

Field Exploration Plan

Report of SPT Borings

Summary of Laboratory Testing Results

Existing Pavement System Thickness

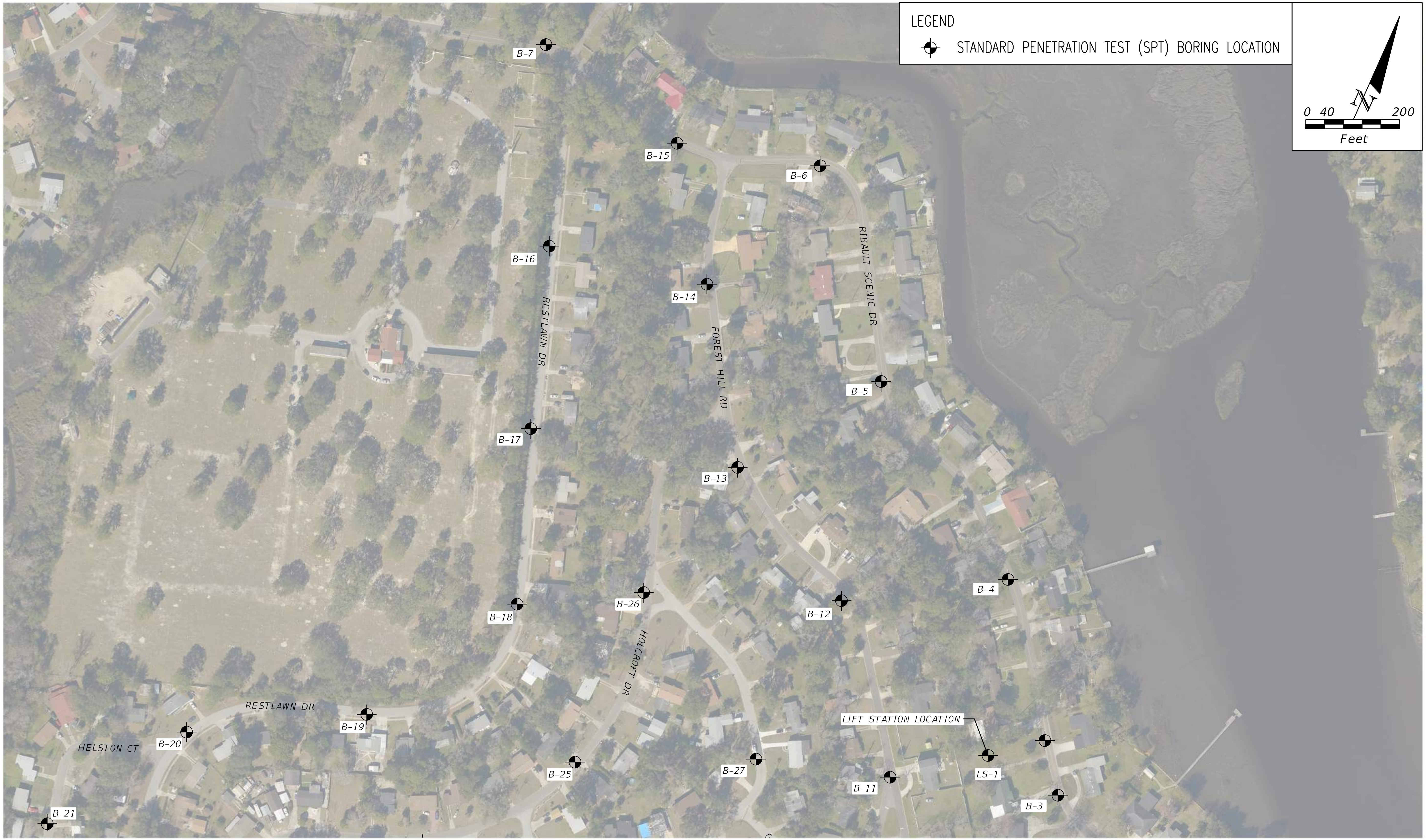
Key to Soil Classification

Field and Laboratory Test Procedures

Site Location Map

CSI GEO, INC. 2394 ST. JOHNS BLUFF ROAD S., SUITE 200 JACKSONVILLE, FLORIDA 32246	SITE LOCATION MAP BEVERLY HILLS WEST SEPTIC TANK PHASE-OUT JACKSONVILLE, FLORIDA

Field Exploration Plan



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FIELD EXPLORATION PLAN

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FIELD EXPLORATION PLAN

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Report of SPT Borings

LEGEND

ASPHALT	FINE SAND; SLIGHTLY SILTY FINE SAND (A-3)
LIMEROCK	WOOD PIECES AND ROOTS (A-8)
SILTY FINE SAND (A-2-4)	CLAYEY FINE SAND (A-2-6)
SANDY CLAY (A-6)	LIMESTONE FRAGMENTS

(A-3) AASHTO SOIL CLASSIFICATION SYSTEM

ESTIMATED SEASONAL HIGH GROUND WATER LEVEL

GROUND WATER LEVEL AT TIME OF DRILLING

W NATURAL MOISTURE CONTENT (%)
-200 FINES PASSING NO. 200 SIEVE (%)
OC% ORGANIC CONTENT
LL LIQUID LIMIT
PI PLASTICITY INDEX

B.T. STANDARD PENETRATION TEST BORING TERMINATION

N STANDARD PENETRATION RESISTANCE IN BLOWS PER FT UNLESS OTHERWISE NOTED, NUMBERS TO THE LEFT OF BORING INDICATE N-VALUES.

H.A. HAND AUGER

W.O.H. WEIGHT OF HAMMER

GRANULAR MATERIALS

RELATIVE DENSITY	AUTOMATIC HAMMER SPT N-VALUE (BLOWS/FT)
VERY LOOSE	LESS THAN 3
LOOSE	3-8
MEDIUM DENSE	8-24
DENSE	24-40
VERY DENSE	GREATER THAN 40

SILTS AND CLAYS

CONSISTENCY	AUTOMATIC HAMMER SPT N-VALUE (BLOWS/FT)
VERY SOFT	LESS THAN 1
SOFT	1-3
FIRM	3-6
STIFF	6-12
VERY STIFF	12-24
HARD	GREATER THAN 24

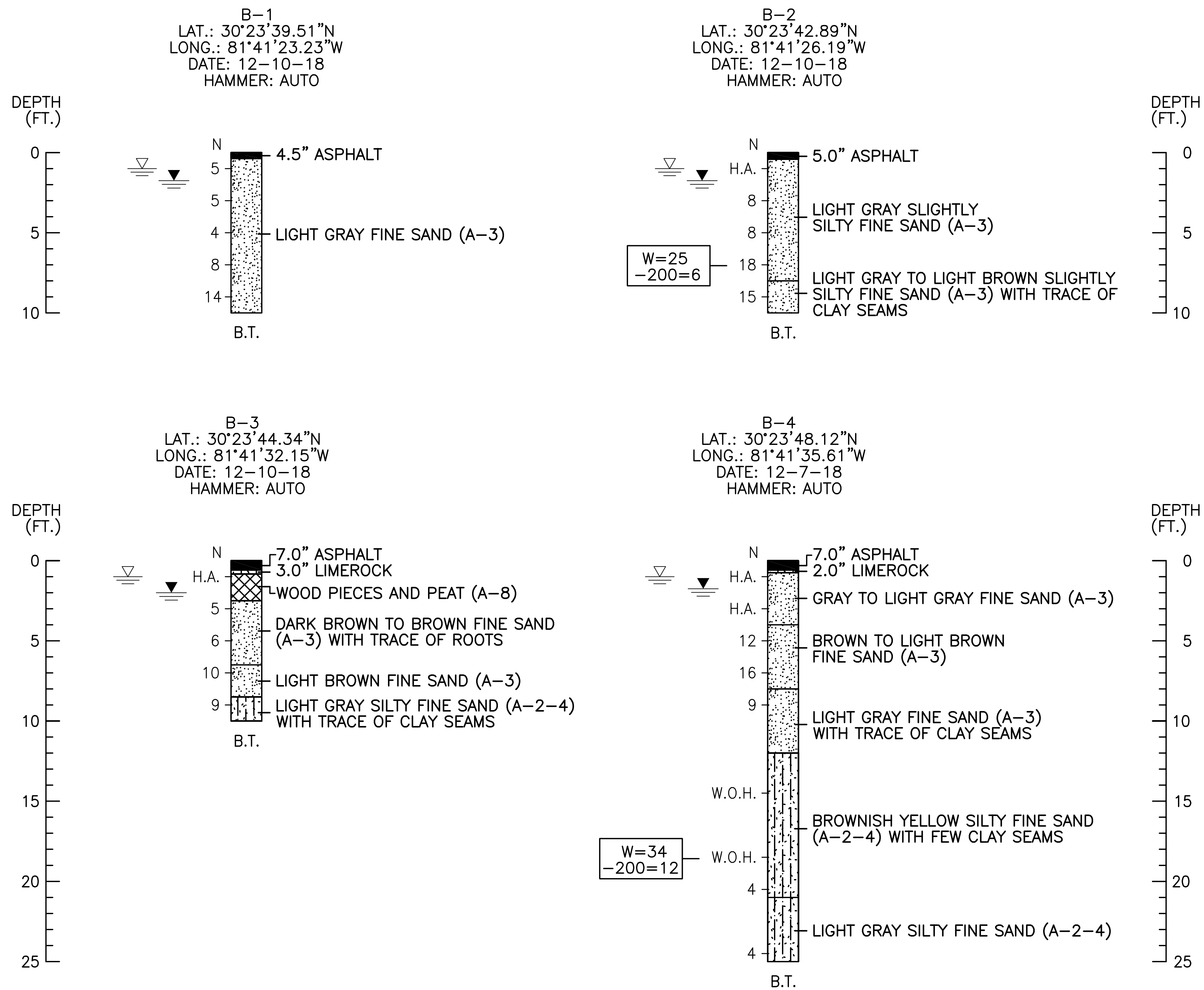
STANDARD PENETRATION TEST DATA

SPOON INSIDE DIA.	1.44 INCHES
SPOON OUTSIDE DIA.	2.0 INCHES
ASTM STANDARD DROP HAMMER	
AVG. HAMMER DROP	30.0 INCHES
HAMMER WEIGHT	140.0 LBS

NOTES:

1) DRILL AND PENETRATION TESTING WAS PERFORMED IN ACCORDANCE WITH ASTM D-1586.

2) LAYER BOUNDARIES ARE APPROXIMATE AND MAY VARY BETWEEN OR AWAY FROM BORING LOCATIONS.



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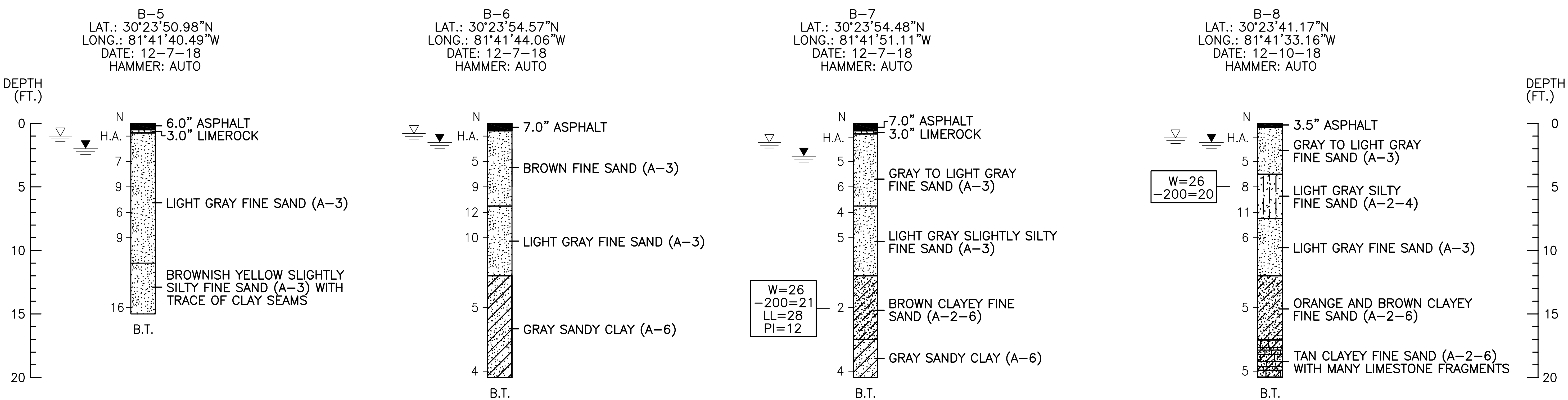

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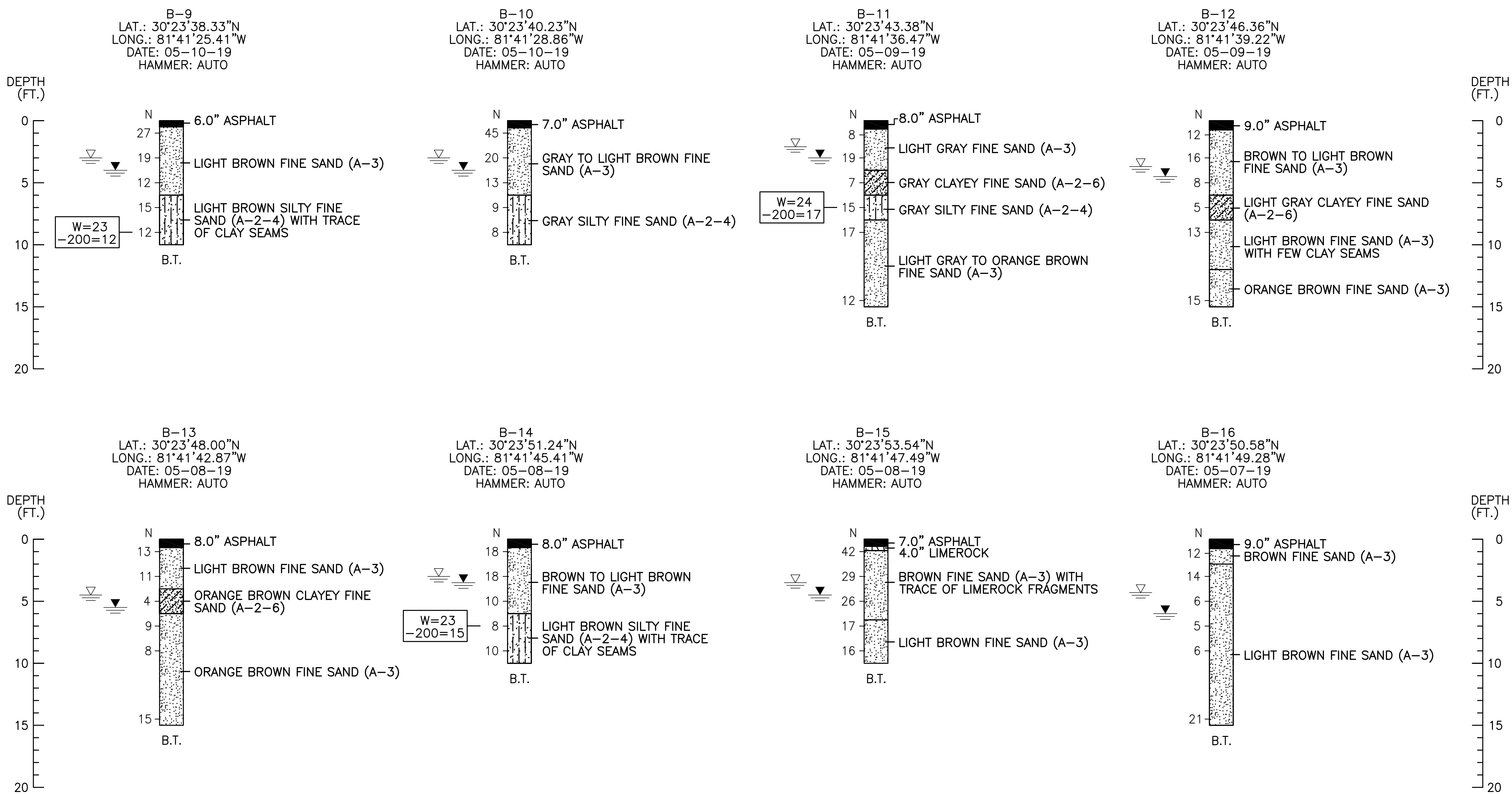

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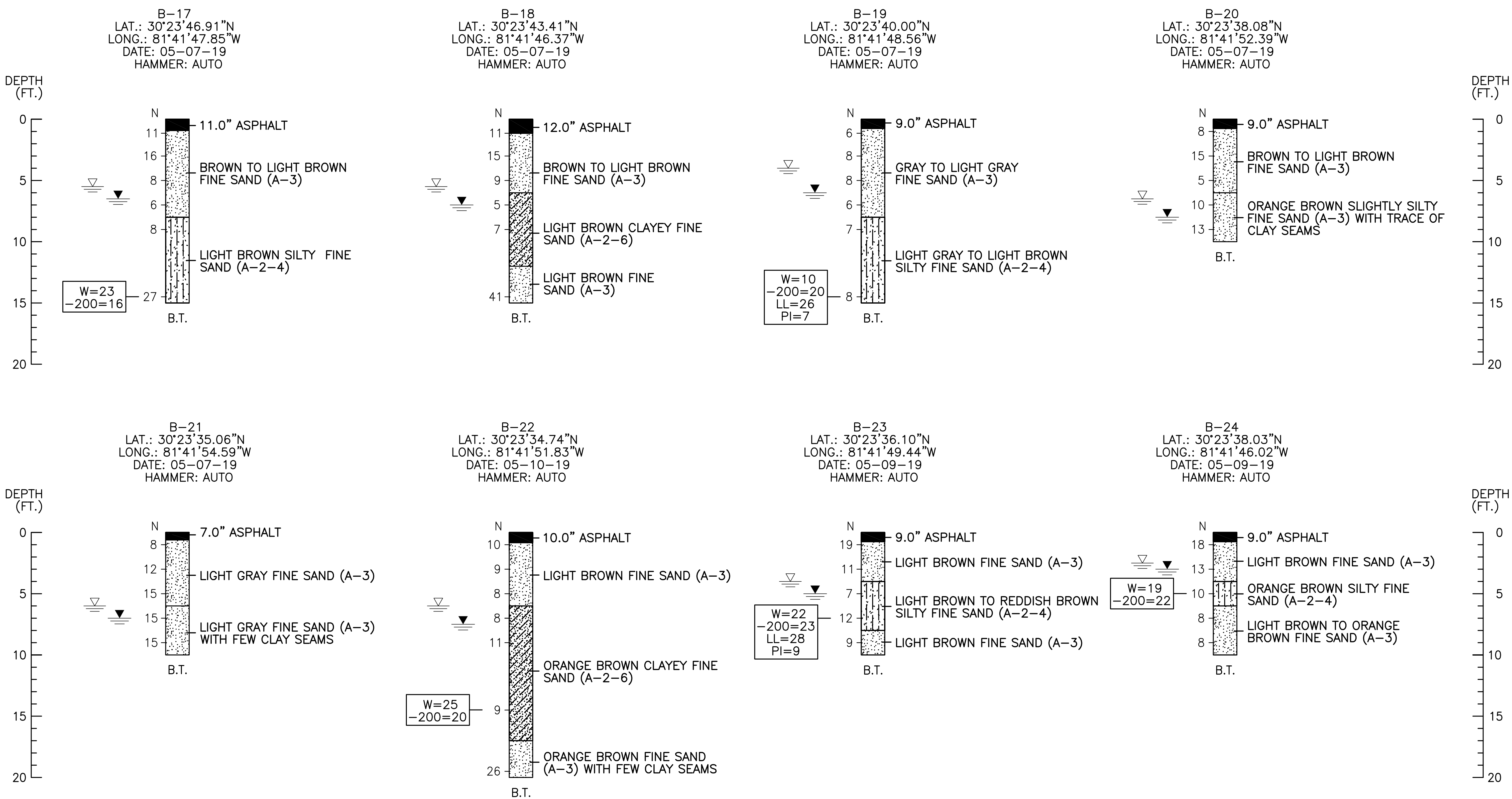

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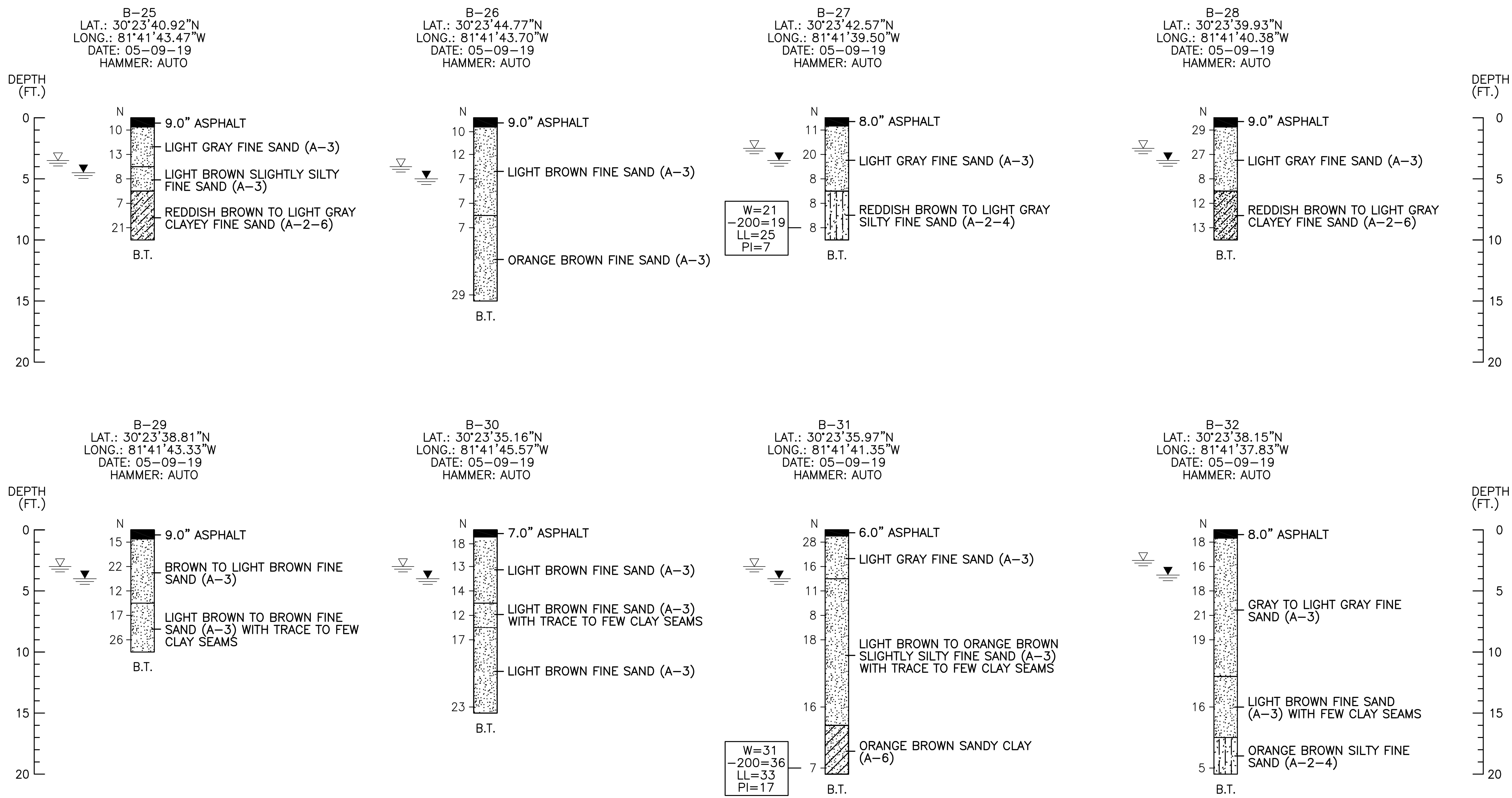

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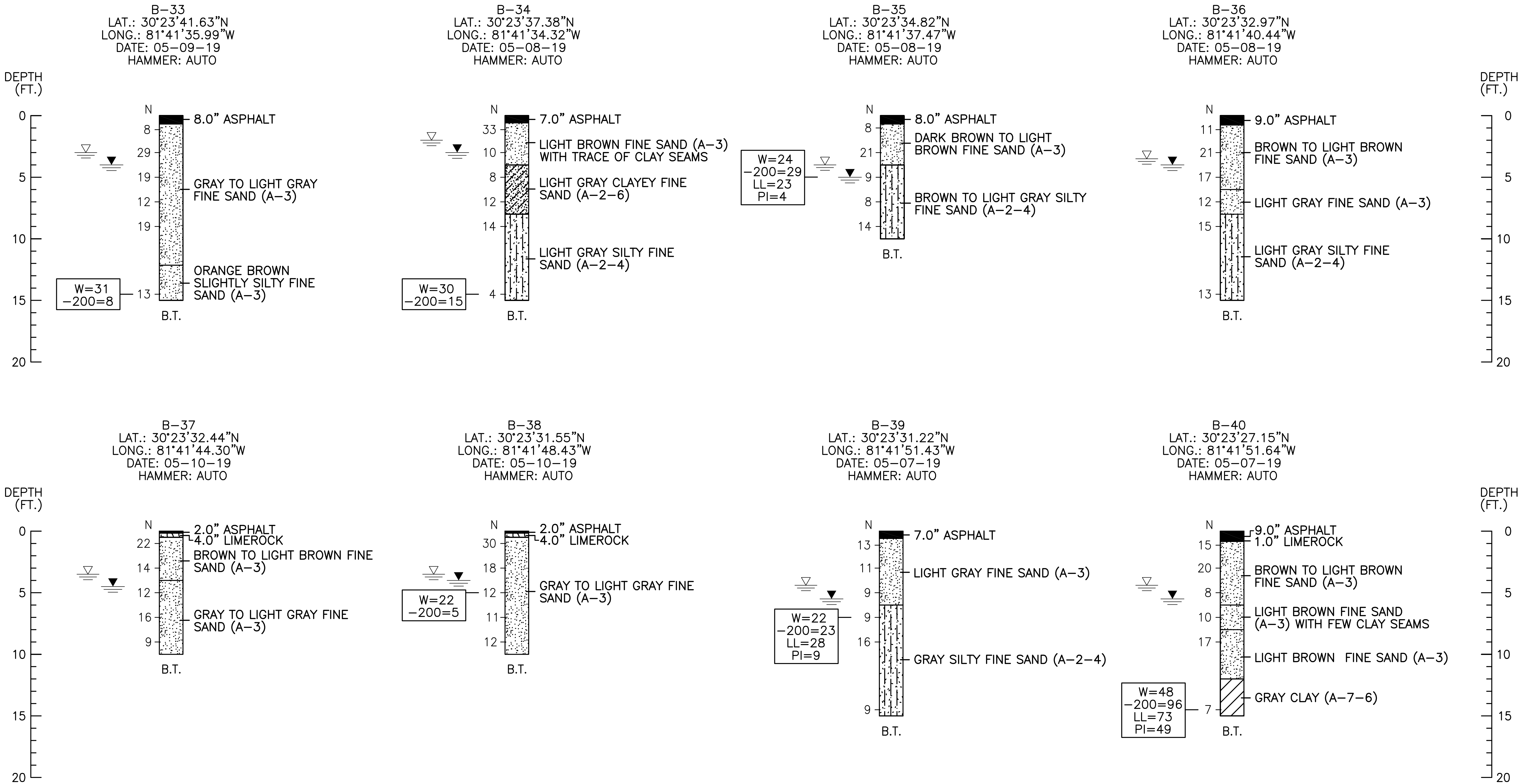

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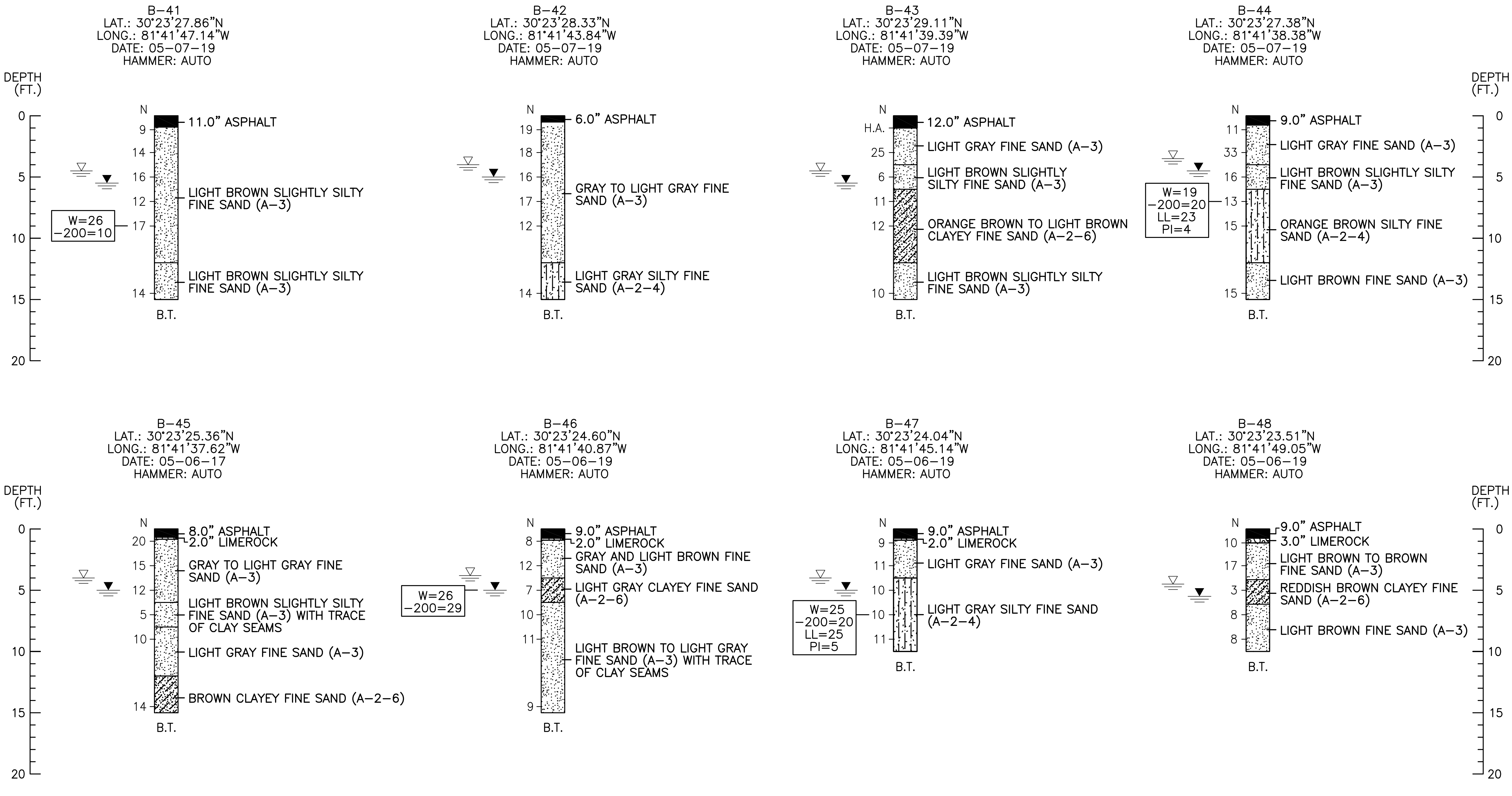

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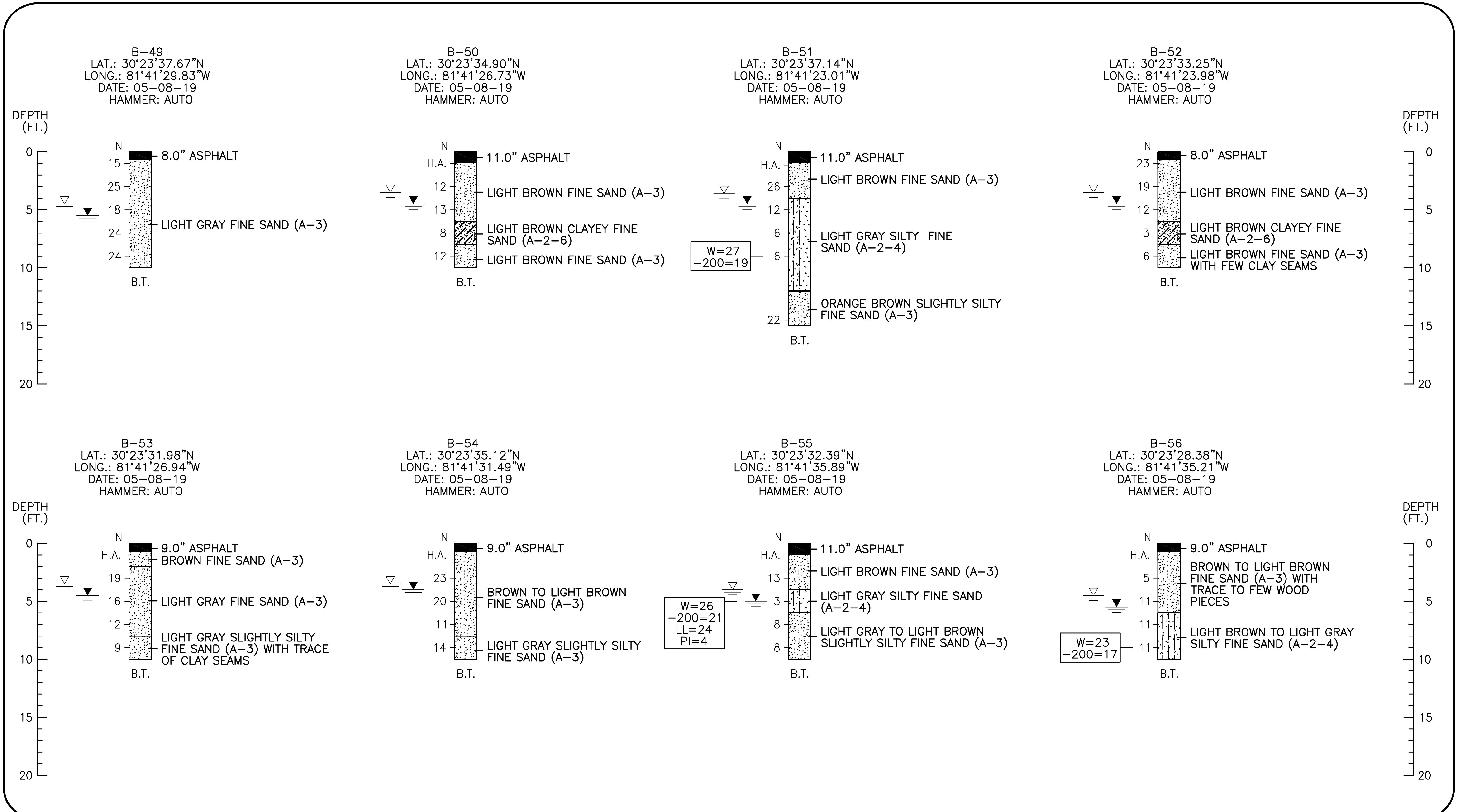

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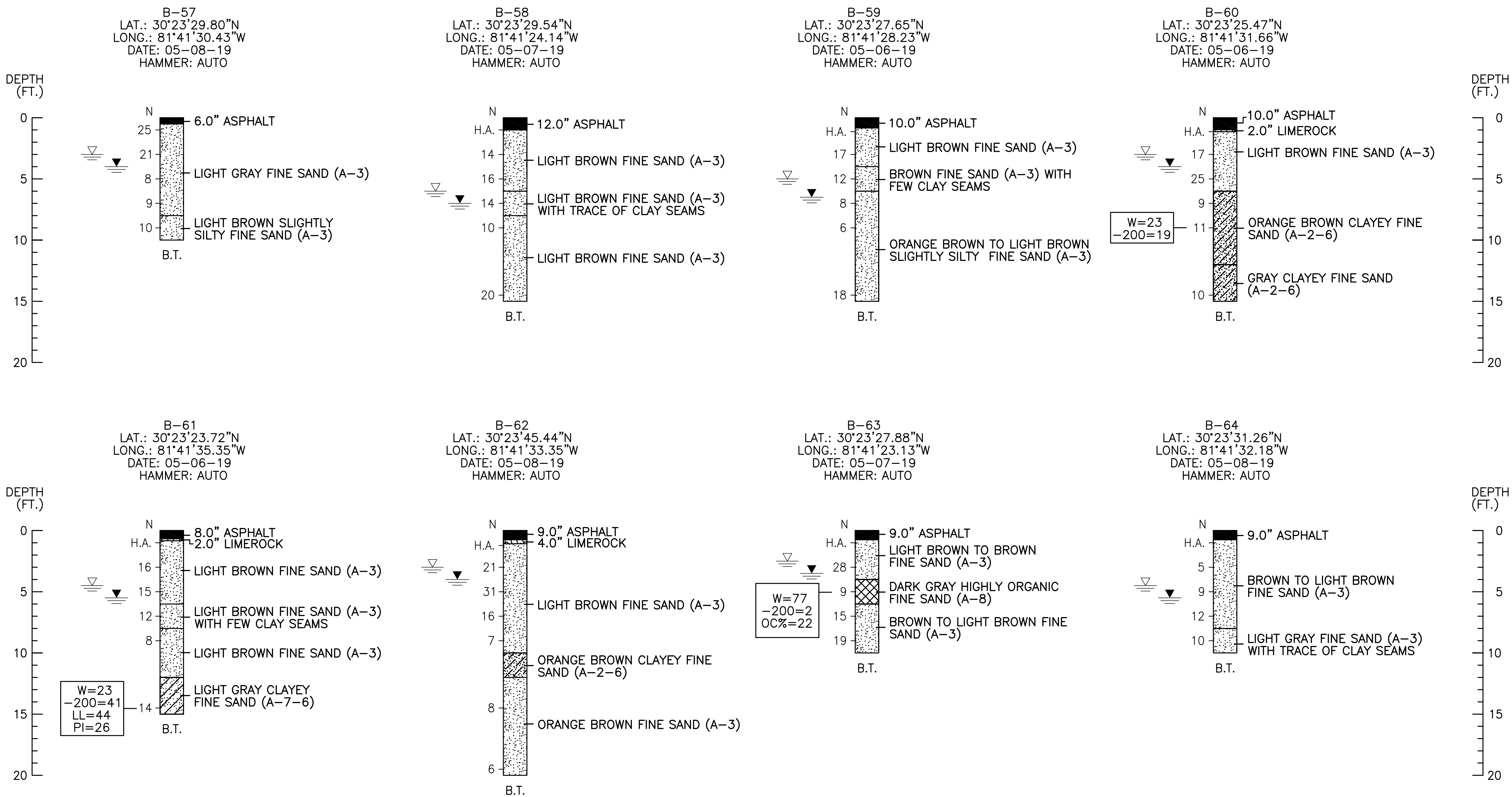

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LEGEND

FINE SAND; SLIGHTLY SILTY FINE SAND (SP)

CLAYEY FINE SAND (SC)

(SP) UNIFIED SOIL CLASSIFICATION SYSTEM

GROUND WATER LEVEL AT TIME OF DRILLING

W — NATURAL MOISTURE CONTENT (%)
−200 — FINES PASSING NO. 200 SIEVE (%)
LL — LIQUID LIMIT
PI — PLASTICITY INDEX

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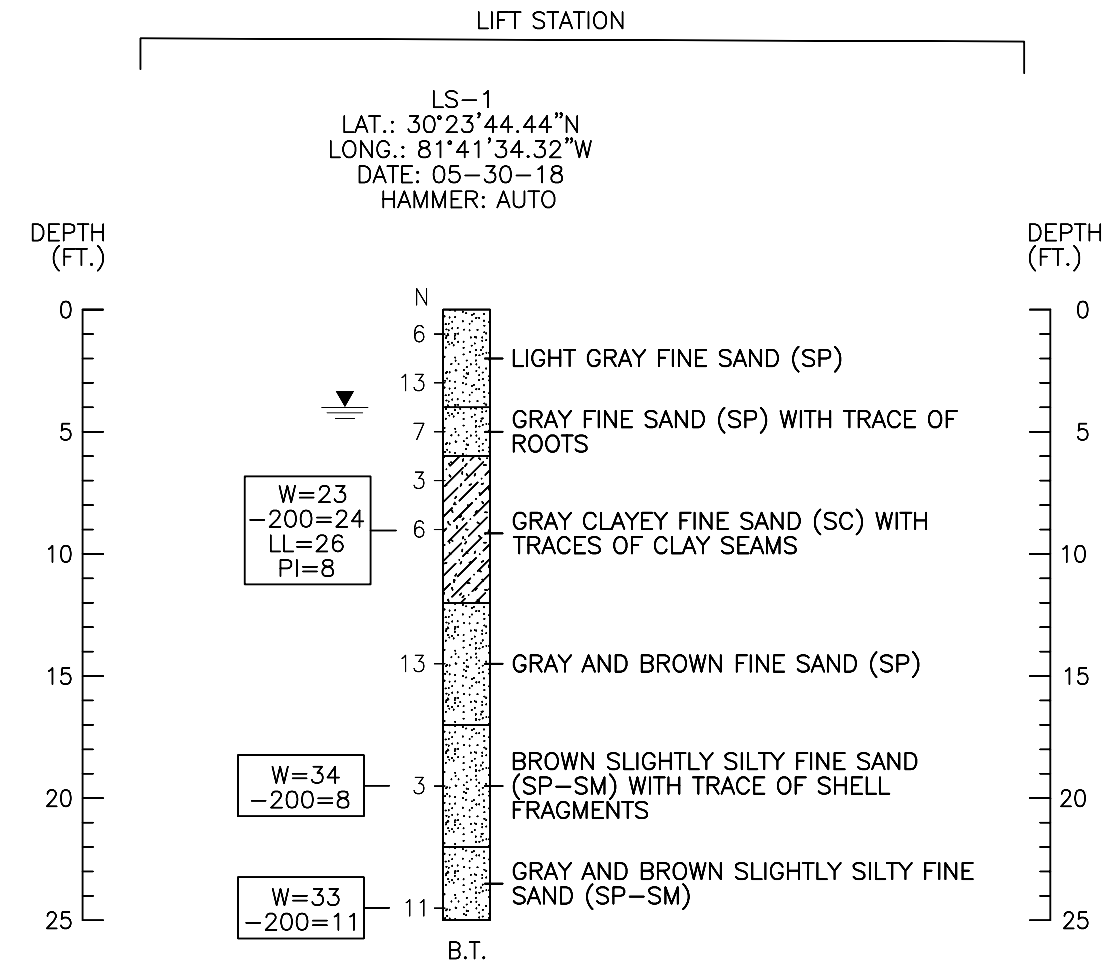
1) DRILL AND PENETRATION TESTING WAS PERFORMED IN ACCORDANCE WITH ASTM D−1586.

2) LAYER BOUNDARIES ARE APPROXIMATE AND MAY VARY BETWEEN OR AWAY FROM BORING LOCATIONS.



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JACKSONVILLE, FLORIDA 32246

GEOTECHNICAL ENGINEERING
CONSTRUCTION MATERIAL TESTING
CONSTRUCTION ENGINEERING INSPECTION

REPORT OF SPT BORINGS

BEVERLY HILLS WEST SEPTIC TANK PHASE−OUT
JACKSONVILLE, FLORIDA

Summary of Laboratory Testing Results

SUMMARY OF LABORATORY TEST RESULTS

Beverly Hills West Septic Tank Phase-Out
Jacksonville, Florida
Manholes

Boring No.	Sample No.	Approximate Depth (ft)	Natural Moisture Content (%)	Organic Content (%)	Percent Passing Sieve Size (%)						Atterberg Limits		AASHTO Soil Classification Symbol
					#4	#10	#40	#60	#100	#200	LL	PI	
B-2	4	6.0 - 8.0	25							6			A-3
B-4	7	15.0 - 17.0	34							12			A-2-4
B-7	6	13.5 - 15.0	26							21	28	12	A-2-6
B-8	3	5.0 - 6.0	26							20			A-2-4
B-9	5	8.0 - 10.0	23							12			A-2-4
B-11	4	6.0 - 8.0	24							17			A-2-4
B-14	4	6.0 - 8.0	23							15			A-2-4
B-17	5	8.0 - 10.0	23							16			A-2-4
B-19	5	8.0 - 10.0	10							20	26	7	A-2-4
B-22	6	13.5 - 15.0	25							20			A-2-6
B-23	4	6.0 - 8.0	22							23	28	9	A-2-4
B-24	3	4.0 - 6.0	19							22			A-2-4
B-27	5	8.0 - 10.0	21							19	25	7	A-2-4
B-31	7	18.5 - 20.0	31							36	33	17	A-6
B-33	6	13.5 - 15.0	31							8			A-3
B-34	6	13.5 - 15.0	30							15			A-2-4
B-35	3	4.0 - 6.0	24							29	23	4	A-2-4
B-38	3	4.0 - 6.0	22							5			A-3
B-39	4	6.0 - 8.0	22							23	28	9	A-2-4
B-40	5	13.5 - 15.0	48							96	73	49	A-7-6
B-41	5	13.5 - 15.0	26							10			A-3
B-44	4	6.0 - 8.0	19							20	23	4	A-2-4

SUMMARY OF LABORATORY TEST RESULTS

Beverly Hills West Septic Tank Phase-Out
Jacksonville, Florida
Manholes

Boring No.	Sample No.	Approximate Depth (ft)	Natural Moisture Content (%)	Organic Content (%)	Percent Passing Sieve Size (%)						Atterberg Limits		AASHTO Soil Classification Symbol
					#4	#10	#40	#60	#100	#200	LL	PI	
B-46	3	4.0 - 6.0	26							29			A-2-6
B-47	4	6.0 - 8.0	25							20	25	5	A-2-4
B-51	5	8.0 - 10.0	27							19			A-2-4
B-55	3	4.0 - 6.0	26							21	24	4	A-2-4
B-56	5	8.0 - 10.0	23							17			A-2-4
B-60	5	8.0 - 10.0	23							19			A-2-6
B-61	6	13.5 - 15.0	23							41	44	26	A-7-6
B-63	3	4.0 - 6.0	77	22						2			A-8

SUMMARY OF LABORATORY TEST RESULTS

Beverly Hills West Septic Tank Phase-Out
Jacksonville, Florida
Lift Station

Boring No.	Sample No.	Approximate Depth (ft)			Natural Moisture Content (%)	Organic Content (%)	Percent Passing Sieve Size (%)						Atterberg Limits		USCS Soil Classification Symbol
							#4	#10	#40	#60	#100	#200	LL	PI	
LS-1	5	8.0	-	10.0	23							24	26	8	SC
LS-1	7	15.0	-	20.0	34							8			SP-SM
LS-1	8	20.0	-	25.0	33							11			SP-SM

Existing Pavement System Thickness

EXISTING PAVEMENT SYSTEM THICKNESS

Beverly Hills West Septic Tank Phase-Out
Jacksonville, Florida

Core No.	Location		Material Layer Thickness		Description & AASHTO Classification of Soil Beneath Pavement / Base
			Asphalt	Limerock	
	Lat.	Long.	(in)	(in)	
B-1	30°23'39.51"N	81°41'23.23"	4 1/2	-	Light Gray Fine SAND (A-3)
B-2	30°23'42.89"N	81°41'26.19"W	5	-	Light Gray Slightly Silty Fine SAND (A-3)
B-3	30°23'44.34"N	81°41'32.15"W	7	3	Wood Pieces and Peat (A-8)
B-4	30°23'48.12"N	81°41'35.61"W	7	2	Gray to Light Gray Fine SAND (A-3)
B-5	30°23'50.98"N	81°41'40.49"W	6	3	Light Gray Fine SAND (A-3)
B-6	30°23'54.57"N	81°41'44.06"W	7	-	Brown Fine SAND (A-3)
B-7	30°23'54.48"N	81°41'51.11"W	7	3	Gray to Light Gray Fine SAND (A-3)
B-8	30°23'41.17"N	81°41'33.16"W	3 1/2	-	Gray to Light Gray Fine SAND (A-3)
B-9	30°23'38.33"N	81°41'25.41"W	6	-	Light Brown Fine SAND (A-3)
B-10	30°23'40.23"N	81°41'28.86"W	7	-	Gray to Light Brown Fine SAND (A-3)
B-11	30°23'43.38"N	81°41'36.47"W	8	-	Light Gray Fine SAND (A-3)
B-12	30°23'46.36"N	81°41'39.22"W	9	-	Brown to Light Brown Fine SAND (A-3)
B-13	30°23'48.00"N	81°41'42.87"W	8	-	Light Brown Fine SAND (A-3)
B-14	30°23'51.24"N	81°41'45.41"W	8	-	Brown to Light Brown Fine SAND (A-3)
B-15	30°23'53.54"N	81°41'47.49"W	7	4	Brown Fine SAND (A-3)
B-16	30°23'50.58"N	81°41'49.28"W	9	-	Brown Fine SAND (A-3)
B-17	30°23'46.91"N	81°41'47.85"W	11	-	Brown to Light Brown Fine SAND (A-3)
B-18	30°23'43.41"N	81°41'46.37"W	12	-	Brown to Light Brown Fine SAND (A-3)
B-19	30°23'40.00"N	81°41'48.56"W	9	-	Gray to Light Gray Fine SAND (A-3)
B-20	30°23'38.08"N	81°41'52.39"W	9	-	Brown to Light Brown Fine SAND (A-3)
B-21	30°23'35.06"N	81°41'54.59"W	7	-	Light Gray Fine SAND (A-3)
B-22	30°23'34.74"N	81°41'51.83"W	10	-	Light Brown Fine SAND (A-3)
B-23	30°23'36.10"N	81°41'49.44"W	9	-	Light Brown Fine SAND (A-3)
B-24	30°23'38.03"N	81°41'46.02"W	9	-	Light Brown Fine SAND (A-3)
B-25	30°23'40.92"N	81°41'43.47"W	9	-	Light Gray Fine SAND (A-3)
B-26	30°23'44.77"N	81°41'43.70"W	9	-	Light Brown Fine SAND (A-3)
B-27	30°23'42.57"N	81°41'39.50"W	8	-	Light Gray Fine SAND (A-3)
B-28	30°23'39.93"N	81°41'40.38"W	9	-	Light Gray Fine SAND (A-3)
B-29	30°23'38.81"N	81°41'43.33"W	9	-	Brown to Light Brown Fine SAND (A-3)
B-30	30°23'35.16"N	81°41'45.57"W	7	-	Light Brown Fine SAND (A-3)
B-31	30°23'35.97"N	81°41'41.35"W	6	-	Light Gray Fine SAND (A-3)
B-32	30°23'38.15"N	81°41'37.83"W	8	-	Gray to Light Gray Fine SAND (A-3)
B-33	30°23'41.63"N	81°41'35.99"W	8	-	Gray to Light Gray Fine SAND (A-3)
B-34	30°23'37.38"N	81°41'34.32"W	7	-	Light Brown Fine SAND (A-3)
B-35	30°23'34.82"N	81°41'37.47"W	8	-	Dark Brown to Light Brown Fine SAND (A-3)
B-36	30°23'32.97"N	81°41'40.44"W	9	-	Brown to Light Brown Fine SAND (A-3)
B-37	30°23'32.44"N	81°41'44.30"W	2	4	Brown to Light Brown Fine SAND (A-3)
B-38	30°23'31.55"N	81°41'48.43"W	2	4	Gray to Light Gray Fine SAND (A-3)
B-39	30°23'31.22"N	81°41'51.43"W	7	-	Light Gray Fine SAND (A-3)
B-40	30°23'27.15"N	81°41'50.64"W	9	1	Brown to Light Brown Fine SAND (A-3)
B-41	30°23'27.86"N	81°41'47.14"W	11	-	Light Brown Slightly Silty Fine SAND (A-3)
B-42	30°23'28.33"N	81°41'43.84"W	6	-	Gray to Light Gray Fine SAND (A-3)
B-43	30°23'29.11"N	81°41'39.39"W	12	-	Light Gray Fine SAND (A-3)
B-44	30°23'27.38"N	81°41'38.38"W	9	-	Light Gray Fine SAND (A-3)

EXISTING PAVEMENT SYSTEM THICKNESS

Beverly Hills West Septic Tank Phase-Out
Jacksonville, Florida

Core No.	Location		Material Layer Thickness		Description & AASHTO Classification of Soil Beneath Pavement / Base
	Lat.	Long.	Asphalt (in)	Limerock (in)	
B-45	30°23'25.36"N	81°41'37.62"W	8	2	Gray to Light Gray Fine SAND (A-3)
B-46	30°23'24.60"N	81°41'40.87"W	9	2	Gray and Light Brown Fine SAND (A-3)
B-47	30°23'24.03"N	81°41'45.14"W	9	2	Light Gray Fine SAND (A-3)
B-48	30°23'23.51"N	81°41'49.05"W	9	3	Light Brown to Brown Fine SAND (A-3)
B-49	30°23'37.67"N	81°41'29.83"W	8	-	Light Gray Fine SAND (A-3)
B-50	30°23'34.90"N	81°41'26.73"W	11	-	Light Brown Fine SAND (A-3)
B-51	30°23'33.25"N	81°41'23.98"W	11	-	Light Brown Fine SAND (A-3)
B-52	30°23'33.25"N	81°41'23.98"W	8	-	Light Brown Fine SAND (A-3)
B-53	30°23'31.98"N	81°41'26.94"W	9	-	Brown Fine SAND (A-3)
B-54	30°23'35.12"N	81°41'31.49"W	9	-	Brown to Light Brown Fine SAND (A-3)
B-55	30°23'32.39"N	81°41'35.89"W	11	-	Light Brown Fine SAND (A-3)
B-56	30°23'28.38"N	81°41'35.21"W	9	-	Brown to Light Brown Fine SAND (A-3)
B-57	30°23'29.80"N	81°41'30.43"W	6	-	Light Gray Fine SAND (A-3)
B-58	30°23'29.54"N	81°41'24.14"W	12	-	Light Brown Fine SAND (A-3)
B-59	30°23'27.65"N	81°41'28.23"W	10	-	Light Brown Fine SAND (A-3)
B-60	30°23'25.47"N	81°41'31.66"W	10	2	Light Brown Fine SAND (A-3)
B-61	30°23'23.72"N	81°41'35.35"W	8	2	Light Brown Fine SAND (A-3)
B-62	30°23'45.44"N	81°41'33.35"W	9	4	Light Brown Fine SAND (A-3)
B-63	30°23'27.88"N	81°41'23.13"W	9	-	Light Brown to Brown Fine SAND (A-3)
B-64	30°23'31.26"N	81°41'32.18"W	9	-	Brown to Light Brown Fine SAND (A-3)

Key to Soil Classification

KEY TO SOIL CLASSIFICATION

Correlation of Penetration Resistance with Relative Density and Consistency

Granular Materials

Relative Density	**Auto Hammer SPT N-Value (Blows/foot)**
Very Loose	Less than 3
Loose	3 – 8
Medium Dense	8 - 24
Dense	24 - 40
Very Dense	Greater than 40

Silts and Clays

Consistency	**Auto Hammer SPT N-Value (Blows/foot)**
Very Soft	Less than 1
Soft	1 – 3
Firm	3 - 6
Stiff	6 - 12
Very Stiff	12 - 24
Hard	Greater than 24

Particle Size Identification (Unified Soil Classification System)

Boulders:	Diameter exceeds 8 inches
Cobbles:	3 to 8 inches diameter
Gravel:	Coarse - 3/4 to 3 inches in diameter
	Fine - 4.76 mm to 3/4 inch in diameter
Sand:	Coarse - 2.0 mm to 4.76 mm in diameter
	Medium - 0.42 mm to 2.0 mm in diameter
	Fine - 0.074 mm to 0.42 mm in diameter

Modifiers

These modifiers provide our estimate of the amount of fines (silt or clay size particles) in soil samples.

Approximate Fines Content	Modifiers
5% Fines 12%	Slightly silty or slightly clayey
12% Fines 30%	Silty or clayey
30% Fines 50%	Very silty or very clayey

These modifiers provide our estimate of shell, rock fragments, or roots in the soil sample.

Approximate Content, By Weight	Modifiers
< 5%	Trace
5% to 10%	Few
15% to 25%	Little
30% to 45%	Some
50% to 100%	Mostly

These modifiers provide our estimate of organic content in the soil sample.

Organic Content	Modifiers
1% to 3%	Trace
3% to 5%	Slightly Organic
5% to 20%	Organic
20% to 75%	Highly Organic (Muck)
> 75%	Peat

Field and Laboratory Test Procedures

FIELD AND LABORATORY TEST PROCEDURES

FIELD TEST PROCEDURES

Standard Penetration Test (SPT) Borings - The soil penetration test borings were made in general accordance with ASTM D-1586, "Penetration Test and Split-Barrel Sampling of Soils". The borings were advanced by continuously driving the split spoon sampler to a depth of 10 feet below the existing ground surface. At the sampling intervals, the drilling tools were removed and soil samples were obtained with a standard 1.4 inch I.D., 2.0 inch O.D., split-tube sampler. The sampler was first seated six inches and then driven an additional foot with blows of a 140 pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final foot is designated the "Penetration Resistance". The penetration resistance, when properly interpreted, is an index to the soil strength and density.

Representative portions of the soil samples, obtained from the sampler, were placed in glass jars and transported to our laboratory. The samples were then examined by a geotechnical engineer to confirm the field classifications.

LABORATORY TEST PROCEDURES

Natural Moisture Content

The water content is the ratio, expressed as a percentage, of the weight of water in a given mass of soil to the weight of the solid particles. This test was conducted in the general accordance with ASTM D2216.

Percent Fine Content

To determine the percentage of soils finer than No. 200 sieve, the dried samples were washed over a 200 mesh sieve. The material retained on the sieve was oven dried and then weighed and compared with the unwashed dry weight in order to determine the weight of the fines. The percentage of fines in the soil sample was then determined as the percentage of weight of fines in the sample to the weight of the unwashed sample. This test was conducted in accordance with ASTM D 1140.

Plasticity (Atterberg Limits) - The soil's Plastic Index (PI) is bracketed by the Liquid Limit (LL) and Plastic Limit (PL). The LL is the moisture content at which the soil flows as a heavy viscous fluid and is determined in general accordance with FM 1-T 089. The PL is the moisture content at which the soil begins to crumble when rolled into a small thread and is also determined in general accordance with FM 1-T 090. The water-plasticity ratio is computed from the above test data. This ratio is an expression comparing the relative natural state of soil with its liquid and plastic consolidation characteristics.

Percent Organic Content

This test is based on the percent of organics by weight of the total sample. This test was conducted in accordance with FM I - T 267.

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2394 St. Johns Bluff Road, Suite 200. • Jacksonville, FL 32246
(904) 641-1993 Phone • (904) 645-0057 Fax
www.csi-geo.com