

**Geotechnical Investigation,
Arawak Cay Extension
Nassau Harbour Port Improvement Project
The Bahamas**

Prepared for:

Ministry of Works and Transport
Government of the Bahamas

Trow International Ltd.

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1. Summary

Trow International Ltd. was authorized to proceed with the geotechnical investigation for the proposed extension to the west end of Arawak Cay and for harbour facilities in the Bay Street area as an addition to the work on the Nassau Harbour Improvement Project. This report addresses the extension to Arawak Cay. It is understood that this will be constructed primarily to provide storage space for materials from the dredging for the harbour improvements. It will be a steel sheet pile wall tied back to anchor blocks. In the future, the area in front of the wall will be dredged to accommodate shipping.

The fieldwork for this investigation consisted of drilling and sampling 4 boreholes to between El. -40 and -65 feet MLWS on the proposed line of the face of the cay. The overburden and the upper levels of the bedrock were sampled by carrying out Standard Penetration Tests and obtaining split barrel samples. Where possible, the bedrock was cored. After providing surficial samples for environmental testing, soil samples were retained for moisture content testing. The bedrock core was logged and samples were selected for strength testing by means of unconfined compression and point load tests.

The point load test results were correlated with the unconfined compression test results on the same length of core in the laboratory in the as received condition. Based on this data and the data previously obtained for the geotechnical investigation for the dredging for the harbour improvements, it was determined that the correlation multiplication factor is 9 times the point load test results for them to be equivalent to the unconfined compressive strength.

The site is located in the Bahamas Archipelago, which is a group of islands, discontinuous sand bars and coral reefs. The upper sediments consist of oolitic sands, aragonite sands, eroded coral and a relatively porous calcareous limestone. The upper portions of the limestone consist of fairly thin layers, strata and lenses of debris. This debris exists in the form of broken coral, flinty chert inclusions, distinct calcite or aragonite crystals or nodule-like inclusions of other limestone formations. Additionally, there are fossils of small marine animals and distinct shell inclusions.

On the alignment of the dock wall, the surficial stratum, which was encountered at Els. -2 to -12 feet, is 4 to 6 feet of very loose calcareous sand. Immediately under the sand, there is calcareous limestone bedrock. Its texture ranges from calcarenite to calcilutite and is tan in colour. It is predominantly vuggy where it was cored.

In the eastern portions of the extension, the upper levels of the calcareous limestone are equivalent to loose sand to between Els. -16 to -18 feet. Below these levels and from the upper surface of the bedrock in the westerly portion of the extension, it is equivalent to a compact to very dense sand down to Els. -16 to -19 feet. Below these levels, the bedrock was corable and the upper 10 to 21 feet is weak to very weak calcareous limestone with occasional medium strong zones. The unconfined compressive strengths of intact core from

this layer range from 16 to 330 tsf. Below Els. -27 to -40 feet, the bedrock is very weak to weak with extremely weak zones, which sometimes predominate. This rock extends to below the bottoms of the boreholes at Els. -40 to -65 feet on the south side of the extension. On the north side, it is underlain by weak to medium strong bedrock at Els. -51 to -57 feet.



The overburden has been assigned an angle of internal friction of 20° and a bulk density of 90 pcf. These properties increase in the upper levels of the bedrock to 25° to 35° and 120 to 125 pcf. Below Els. -17 to -21 feet, the bedrock has design parameters equivalent to an average cohesion of approximately 25 tsf on the active side of the sheeting and an angle of internal friction of 45° on the passive side with a bulk density of 135 pcf. Below Els. -27 to -40 feet, the strength of the bedrock decreases to an equivalent angle of internal friction of 40° with a bulk density of 135 pcf. The estimated geotechnical parameters of material, which has been dredged for the harbour improvements, ranges from an angle of internal friction of 25° with a bulk density of 100 pcf in the uncompacted condition to 40° at 125 pcf when compacted to 95% of the Modified Proctor Maximum Dry Density. The friction on steel, smooth concrete and rough concrete surfaces from these materials is respectively 0.7, 0.8 and 1.0 times the tangent of the angle of internal friction of the material adjacent to them times the applied effective pressure normal to the surface.

The calcareous limestone bedrock for 10 to 21 feet below Els. -17 to -21 feet contains occasional zones with unconfined compressive strengths of up to 330 tsf (4,500 psi), although it is generally much weaker. Consequently, the piling should be driven with a pile driving hammer capable of developing 40 ksi impact stress in the steel at practical refusal. A pile driving hammer with a rated energy of over 75 foot kips is likely suitable to drive pairs of the proposed Skylinesteel AZ 19-700 and AZ 14-770 sheeting or equivalent but this should be verified for the actual hammer by wave equation analysis. Even so, it is possible that some of the piling will meet refusal above their design bottom level. If this occurs above five feet below the design dredge grades, a trench will have to be created into which to drive the piles by means such as line drilling and possibly blasting. Below this level, the piles that are terminated higher than the design levels should have adequate toe support to resist the active pressures on the sheeting.

2. Introduction

On April 17, 2008, Trow International Ltd. was authorized to proceed with the geotechnical investigation for the Consulting Engineering Services for the Nassau Harbour Port Improvement Project for the Government of the Bahamas, Ministry of Works & Transport by means of receipt of a mobilization payment. Formal written authorization, dated July 29, 2008, was subsequently received. The investigatory work was subsequently extended to examine the geotechnical conditions for the extension of Arawak Cay to the west and for harbour facilities in the Bay Street area. This report addresses the proposed Arawak Cay Extension (the Extension).

The purpose of the investigation was to provide geotechnical data about the stratigraphy at the Extension in the location shown in Drawing 1, Borehole Location Plan and to provide geotechnical design parameters for steel sheet pile wall construction.

It is understood that the Extension is to be constructed to provide additional storage space for the dredgings for the Nassau Harbour Port Improvement Project. There will not be any dredging in the vicinity of the Extension at this time.

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The proposed Extension is to be constructed of steel sheet piles tied back to anchor blocks. It will then be backfilled with dredgings, which may subsequently be compacted by means of vertical vibro densification techniques possibly using a top vibrator on a pipe pile.

As well as the data obtained during this investigation, information presented in “Geotechnical Investigation, Part 1, Dredging, Nassau Harbour Port Improvement Project” by Trow International Ltd., dated August 8, 2008 (the Dredging Report), has been used to assess the geotechnical conditions in the area.

The comments given in this report are intended only for the guidance of the design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. would be much greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results to draw their own conclusions as to how the subsurface conditions may affect them.

3. Terms of Reference

The terms of reference were to:

- Drill and sample 3 boreholes to El. -65 feet and one borehole (BH A3) to El. -40 feet at the locations shown on Drawing 1. Location of the boreholes was to be by GPS methods. The elevations were to be established by using the top of the north pier of Prince George Wharf as a bench mark at El. +7.5 feet MLWS.
- Soil samples were to be obtained at 2.5 feet intervals with a split barrel sampler undertaking Standard Penetration Tests (SPT). The bedrock was to be cored using HX size core barrels.
- The 2.5 to 4 feet depth samples were to be taken by procedures acceptable to the BEST Commission & Environmental Agencies and delivered to Blue Engineering for environmental testing.
- Soil Samples were to be stored in airtight containers.
- Geotechnical laboratory testing would consist of: moisture contents on all soil samples and unconfined compression tests on two sections of bedrock per borehole with point load tests where the intact bedrock was too short for unconfined compression testing.
- A report was to be produced that: included borehole logs showing SPT results; moisture contents; % core recovery and rock quality designation (RQD) of the bedrock cores; and soil and bedrock descriptions. It would also include: tabulation of the unconfined compression and point load test results; and provision of geotechnical engineering parameters for the design of the sheet pile walls.
- The report was to be distributed to members of the consortium and the Government of the Bahamas, Ministry of Works & Transport for review and comment and would be finalized thereafter.

This report is provided on the basis of the terms of reference and on the assumption that the design will be in accordance with applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning geotechnical aspects of the codes and standards, this office should be contacted to review the design. It may then be necessary to carry out additional borings and reporting before the recommendations of this office may be relied upon.

The scope of services described above is based upon a limited number of soil samples obtained from widely spaced subsurface explorations. The nature and extent of variations

between these explorations may not become evident until construction. If variations or other latent conditions do become evident, it may be necessary to reevaluate the scope of this report.

Consideration of the environmental conditions at the site was not part of the terms of reference for this investigation and is not commented on in the report.

4. Procedure

The fieldwork involved drilling and sampling 3 boreholes (BHs A1, A2 and A4) to approximately El. -65 feet MLWS and one boreholes (BH A3) to approximate El. -40 feet MLWS. The drilling and sampling was carried out by Toney Drilling Supplies, Inc. under the direction of Trow International Ltd. between July 29 and 31, 2008. The drilling was undertaken in hollow stem augers from a jack up barge using a CME 55 drill rig.

The sampling consisted of taking split barrel samples of the overburden and the weaker bedrock while performing Standard Penetration Tests (SPTs). This sampling was carried out continuously in all the boreholes. Once it was considered that core could be recovered, the boreholes were advanced by coring with HQ3 wire line triple tube core barrels.

The boreholes were located using Garmin Global Positioning Systems (GPS) equipment. The boreholes were located on the grid for UTM Zone 18R. The elevations of the boreholes were established by measuring the depth of water at each borehole. The depth of the water surface below the top of the cope wall for the north pier at Prince George Wharf was measured as soon as possible before and after the measurement of the water depth at the borehole. The elevation of the top of the cope wall was assumed to be +7.5 feet MLWS based on information provided by Cox & SHAL. All elevations in this report are referenced to Mean Low Water Spring tide datum. It is considered that the locations are accurate to approximately 5 m (16.4 feet) and the elevations to 0.2 m (0.7 feet).

On completion of the drilling and sampling, a geotechnical engineer from Trow carried out a detailed examination of the soil samples and bedrock cores to develop borehole logs for reporting purposes. Selected soil samples and bedrock cores were then transported to Trow's laboratory in Brampton, Ontario, Canada for further review and testing. On completion of this process, the borehole logs were finalized for this report (Drawings 2 to 5). The bedrock descriptions were carried out in general conformity with those suggested by Clark and Walker (1977), except that the bedrock strength descriptions conform to the International Society of Rock Mechanics (ISRM) System. The Clark and Walker classification system is derived from "A Proposed Scheme for the Classification and Nomenclature for use in the Engineering Description of Middle Eastern Sedimentary Rocks", published in *Geotechnique*, Volume 27, Pages 93 to 99, 1977. It is shown in Table 1 and the Classification of Rock with Regard to Strength is in Table 2. The Classification of Rock with Regard to Spacing of Discontinuities and the Classification of Rock with Regard to RQD-Values are shown on Tables 3 and 4 respectively. These classifications are derived from internationally accepted practice.

The laboratory testing involved moisture contents on all but one of the soil samples and unconfined compression and point load tests on the bedrock cores. The soil samples were small, because portions of them had been given to Blue Engineering for environmental

testing and it is possible that the samples had dried out somewhat, even though they were stored in airtight bags.

A total of 7 unconfined compression tests were performed on bedrock core, as received from the field. A total of 14 point load tests were undertaken on the cores both axially (in the vertical direction) and laterally (in the horizontal direction) as received in the laboratory. The point load test results were calibrated for unconfined compression strength by adding all those, on which unconfined compression tests had also been undertaken, to the correlation carried out for the Dredging Report. The as received unit weight of the bedrock cores was measured for 8 samples. The results of the testing are presented in Table 5.

5. Interpretation of Factual Information

The factual information obtained during this investigation and/or the investigation for the Dredging Report has been subjected to interpretation to: correlate point load testing with unconfined compression strengths; assess whether strengths obtained from tests on bedrock cores, which had air dried before receipt in the laboratory, differ significantly from the wet condition; and aid in the understanding of the descriptions of the bedrock in the borehole logs.

5.1 Correlation of Point Load Test Results with the Unconfined Compression Strength

Only the bedrock cores, which were obtained above El. -58 feet and tested in both unconfined compression (U/C) and by point load tests (PLT), were used for correlation purposes. This had been noted in the investigation for the Dredging Report to be the level, at which the correlation changed from a multiplier of 9 (shallow) to 12 (deep). This resulted in 10 test results being available for correlation purposes. Because of the scatter of results, it was considered that it was more appropriate to include these with the correlation for the Dredging Report to determine whether there is a significant difference. It was decided that the correlation was similar, although it could be a slightly lower multiplier of 8 on the basis of the test results for the current investigation only. This was considered not to be significant and a correlation multiplier of 9 was used to estimate the U/C from the PLT results.

5.2 Impact on Strength of Soaking Bedrock Cores

The impact of soaking the bedrock cores was not assessed for this investigation. It has been assessed for the Dredging Report and during previous investigations in the Bahamas Archipelago and it has been found that the scatter of test results was so great as to mask any impacts from the preparation procedures.

5.3 Bedrock Descriptions in Borehole Logs

Data is provided in the borehole logs about the quality of the bedrock by means of the Rock Quality Designations (RQD), which is the total length of core over 4 inches in length between joints as a percentage of the length of the core run carried out, as per the classification in Table 4. Comments about the joint spacing of the corable bedrock are made for each core run in relation to the classification, which is shown in Table 3. Throughout the descriptions, the non-recovered core is treated as joints in the bedrock.

The strength testing was performed on bedrock cores, which were generally of sufficient length and quality to allow them to be tested in unconfined compression. Consequently, even though representative samples of suitable core were tested, the tests were carried out on the

stronger bedrock at the site. The strength descriptions in Section 6 and the borehole logs were developed by correlation to the laboratory strengths by means of the methods presented in Table 2 for core that was recovered. Based on the field observations of the drilling, it was concluded that the material, which was not recovered in the coring operation, is predominantly extremely weak bedrock rather than open joints. The descriptions in the borehole logs reflect these assumptions for each core run.

6. Geology and Subsurface Conditions

The geotechnical investigation for the Extension consisted of four widely spaced boreholes. The geological stratification in the Bahamas Archipelago is known to be predominantly calcareous limestone under a thin veneer of overburden. There is considerable variability of the engineering properties of the shallow bedrock both vertically and horizontally, such that interpolation of stronger and weaker layers between even closely spaced boreholes can be difficult. Consequently, the boreholes have been undertaken to assess ranges of engineering properties that can be expected rather than the precise stratification at the site. The data from them should be interpreted and analyzed on this basis.

6.1 Geology of the Bahamas

The Bahamas Archipelago consists of a group of low islands, discontinuous sand bars and coral reefs. The group stretches approximately 600 miles long by 150 miles wide paralleling the southern coast of Florida and the northeastern coast of Cuba. The existing land forms were created by sedimentary deposition and erosion.

The upper sediments consist of oolitic sands, aragonite sands, eroded and weathered coral and a fairly porous limestone. The upper portions of the porous limestone consist of fairly thin layers, strata and lenses of debris. This debris exists in the form of broken coral, flinty chert inclusions, distinct calcite or aragonite crystals or nodule-like inclusions of other limestone formations. Additionally, there are fossils of small marine animals and distinct shell inclusions.

While the islands were formed in a similar manner, there are some distinct differences in the limestone depending on the location. The western islands have a stronger, indurated limestone formation. The softer and correspondingly younger limestone formations of New Providence and the eastern islands primarily consist of oolitic sands with inclusions of coral and shells. Recrystallization occurs to some extent in most of the limestone formations. The upper levels of limestone are thinly bedded and weathered resting on thicker strata of older limestone.

Immediately adjacent to several of the islands are extremely deep canyons cutting several miles into the limestone formations. These canyons have steep sides thus indicating a stable limestone crust beneath the existing banks of weathered limestone, oolitic sands and coral.

The surface of the weathered limestone is often erratic in elevation and texture. This is fairly common for karst topography, in which limestone formations are continually changed due to current erosion and solution weathering and sedimentation. Solution cavities are common in this subterrain and thus contribute to the changes in the subterranean surfaces.

The limestone of these islands is formed by accumulations of calcium carbonate in several different forms. The calcium carbonate may exist as calcite or aragonite, a relatively unstable form of calcium carbonate. The aragonite tends to break down and form calcite over a period of time.

Oolites form around discrete particles such as silica. The oolites are rounded or subrounded particles, which can exist as individual particles or can be cemented by calcareous silts. Consolidation by overburden pressure followed by cementation transforms this material into a fairly consistent, granular form of limestone.

The in-situ limestone may thus be formed by an accumulation of oolitic particles, shell particles, particle fragments, chert inclusions and other sedimentary constituents bound together in a cemented matrix. The matrix can further lithify by recrystallization or other physical changes to produce a hardened mass.

6.2 Subsurface Conditions in the Vicinity of the Extension

The location of the Extension and of the boreholes drilled to investigate the geotechnical conditions at it are shown in Drawing 1.

The seabed at the boreholes ranges in elevation from -1.8 feet at BH A3 to -12.2 feet at BH A2. The surficial soil stratum is grey fine to coarse grained calcareous sand. It contains shell fragments. It is 4 to 6 feet thick and extends down to between El. -5.8 feet at BH A3 and -17.2 feet at BH A2. It is loose to very loose with Standard Penetration Resistances (SPRs) of zero to 6 blows/foot. The moisture content of the sand ranges from 22 to 43%.

Calcareous limestone bedrock underlies the sand and extends to the bottom of the boreholes at Els. -40.8 to -65.2 feet. Its texture ranges from calcarenite to calcilitite. It is tan in colour. It is predominantly vuggy where it was cored. It has shell inclusions.

The bedrock was sampled with split barrel samplers to between El. -15.8 feet (BH A3) and -20.2 feet (BH A2). The SPRs of this upper extremely weak bedrock ranged from 4 to 88 blows/foot, which, in equivalent soil consistency terms, would be firm to hard with estimated U/C values of 0.5 to 8 tsf or alternatively loose to very dense granular material.

The bedrock was corable from below Els. -15.8 to -20.2 feet. Below 1 to 2 feet of bedrock with lower core recovery and a RQD classification of very poor to fair, core with recoveries close to 100% were encountered for 10 to 11 feet in BHs A1 to A3 and for 21 feet at BH A4 to approximate Elevations -27 feet at BH A3 to -40 feet at BH A4. The joint spacing of this bedrock is wide at more than 24 inches to very close at less than 2.4 inches with occasional extremely close spacing of less than 0.8 inches. Its RQD classification ranges from poor to excellent. The strength classification of this upper layer of corable bedrock is generally very weak to weak with very occasional extremely weak zones. However, it contains occasional

medium strong zones. The unconfined compressive strength of the core from it, which could be tested, ranges from 16 to 330 tsf and is generally less than 100 tsf. This layer has widely variable strength characteristics both in the vertical and horizontal directions. However, it appears to be generally stronger on the north side of the Extension than on the south side.

The bedrock below the upper layer, which is described in the previous paragraph, has a joint spacing of extremely close (less than 0.8 inches) to moderately close (8 to 24 inches), generally being close or less, i.e., less than 8 inches. It has an RQD classification of very poor to fair and is generally very poor to poor. Its strength classification is interlayered very weak to weak rock with extremely weak rock and at some levels it is extremely weak throughout (U/C of less than 10 tsf). Core could not be readily obtained from this material for laboratory strength testing. This layer extends to below the bottom of BHs A3 and A4 at Els. -40.8 and -65.1 feet respectively. It extends to Els. -57 and -51 feet in BHs A1 and A2 respectively.

Below Els. -57 and -51 feet in BHs A1 and A2, the bedrock is more competent than the overlying bedrock. The joint spacing is generally wide with occasional zones of close to extremely close spacing. The RQD classification is fair to excellent. Its strength classification is weak to medium strong with very occasional extremely weak layers. The measured unconfined compression of this layer ranges from 100 to 550 tsf and is generally less than 200 tsf. This material extends to below the bottom of BHs A1 and A2 at El. -65 feet approximately.

7. Design of Sheet Pile Walls

The dock walls for the extension of Arawak Cay will be a steel sheet pile bulkhead tied back to anchor blocks.



There is similarity between the stratification at the four boreholes, which were drilled for this investigation. However, they were spaced too widely to be able to determine the extent of the dock wall that can be designed for on the basis of the stratigraphy at each borehole or even the borehole, the stratigraphy of which will result in the most critical design parameters. Consequently, it is suggested that the design calculations should be carried out for the estimated geotechnical parameters at all of BHs A1, A2 and A4 for the north and west walls and the most critical design should be used for construction. The same approach should be taken for the south wall for the geotechnical parameters at BHs A3 and A4. These parameters are presented in the table on Page 14, entitled Estimated Average Geotechnical Parameters, Steel Sheet Pile Walls, Arawak Cay.

A major purpose of the area being enclosed by the sheet pile walls is the storage of the material, which will be dredged for the harbour improvements. Dredgings backfill may be assumed to have the geotechnical parameters presented in the table on Page 15 entitled Estimated Geotechnical Parameters, Dredged Material.

The friction between the structural components and the soil and bedrock may be determined by multiplying the tangent of the angle of internal friction times the effective pressure applied normal to the surface of the structure by:

- 0.7 for steel.
- 0.8 for formed concrete
- 1.0 for concrete with a rough surface.

It is understood that the sheet pile walls will be tied back to anchor blocks with either horizontal tie rods or with the anchor blocks placed on the seabed. Both of these are considered to be geotechnically feasible, although the latter approach presents some challenges. The anchor blocks should be designed according accepted practice as presented in manuals, such as “Pile Buck Steel Sheet Piling Design Manual”, latest edition. The geotechnical parameters presented in the tables on Pages 14 and 15 may be employed for this purpose.

**Estimated Average Geotechnical Parameters
Steel Sheet Pile Walls
Arawak Cay**

Material Description	Elevation (feet)								Geotechnical Parameters		
	BH A1		BH A2		BH A3		BH A4		ϕ^1	c^2	γ^3
	from	to	from	to	from	to	from	to	($^\circ$)	(tsf)	(pcf)
Calcareous Sand, very loose	-6	-12	-12	-17	-2	-6	-6	-10	20 ⁴ 30 ⁵	0 ⁴ 0 ⁵	90 ⁴ 110 ⁵
Calcareous Limestone, loose	-12	-18	-	-	-	-	-10	-16	25	0	120
Calcareous Limestone, compact to very dense	-18	-21	-17	-21	-6	-17	-16	-19	35	0	125
Calcareous Limestone, very weak to weak	-21	-32	-21	-36	-17	-27	-19	-40	45 ⁶ 0 ⁷	0 ⁶ 25 ⁷	135
Calcareous Limestone, very dense sand and gravel equivalency	-32	-57	-36	-51	-27	<-41	-40	<-65	40	0	135
Calcareous Limestone, weak to medium strong	-57	<-65	-51	<-65	-	-	-	-	0	100	140

- Notes:
1. ϕ is the angle of internal friction.
 2. c is the cohesion.
 3. γ is the bulk density.
 4. Existing Conditions.
 5. After compaction by vertical vibro densification techniques.
 6. For use in calculating passive resistance.
 7. For use in calculating active pressure

**Estimated Geotechnical Parameters
 Dredged Material**

Description	ϕ ($^{\circ}$) ¹	c (tsf) ²	γ (pcf) ³
Uncompacted	25	0	100
Compacted by Vertical Vibro Densification Techniques	35	0	120
Compacted to 95% MPMDD ⁴ above sea level	40	0	125

- Notes:
1. ϕ is the angle of internal friction.
 2. c is the cohesion.
 3. γ is the bulk density.
 4. Modified Proctor Maximum Dry Density.

It is considered that the fill on both the active and passive sides of the anchor blocks should be well compacted before significant loads are applied to the sheet pile wall. Otherwise, the anchor blocks will tend to deflect significantly when loads are applied to them. The degree of compaction should, if possible, be 95% of the Modified Proctor Maximum Dry Density (MPMDD). This can be achieved fairly readily for anchor blocks with horizontal tie rods, as they will be above the sea level.

However, anchor blocks on the sea bed will be initially surrounded by loose fill. If the fill is left in a loose condition, the anchor blocks may move up to 6 to 10 inches after dredging to El. -32 feet, even though they may be theoretically stable. Consequently, the dredged fill should be placed and compacted carefully and in such a way as to minimize the loads on the steel sheet piling before the fill around the anchor blocks has been densified, as follows:

1. Place better quality dredged fill directly over the anchor blocks until a surface, which is capable of supporting the vertical vibro densification equipment is just provided;
2. Vibro densify first in the front of and then behind the anchor blocks;
3. Tighten the anchor rods to align the steel sheet piling as necessary;
4. Place better quality dredged fill between the anchor blocks and the steel sheet piling;
and
5. Vibro densify the fill between the anchor blocks and the steel sheet piling.

Installation of the steel sheet piling will be undertaken with the seabed at the current levels and dredging will be accomplished at a later date. Two potential approaches for pile installation are foreseen: driving the piles off floating facilities; and constructing a suitably

designed berm to above sea level and driving the piling from the top of it. Neither method is expected to cause driveability problems.

However, the more competent calcareous limestone layer encountered below Els. -17 to -21 feet contains some bedrock with U/Cs as great as 330 tsf or approximately 4,500 psi. Consequently, it is important the pile driving hammer have the capacity to drive the sheeting to the limits of the driving stresses that it can tolerate. It should be possible to drive the proposed Skylinesteel AZ 19-700 and AZ 14-770 sheet piles in pairs by using a pile driving hammer capable of developing 40 ksi impact stress at practical refusal, which is defined as a penetration resistance of 20 blows per inch. The pile driving hammer may be selected by undertaking wave equation analyses for the proposed piles assuming the geotechnical parameters presented in the tables on Pages 14 and 15 of this report. WEAP analysis for a number of Delmag hammers driving the AZ 19-700 sheet piles has indicated that a Delmag D30-32 with a rated energy of 75.4 foot kips per blow is suitable for this purpose. The steel area per pile of the AZ 14-770 sheet piles is effectively the same as the AZ 19-700. Consequently, the Delmag D30-32 is also likely to be suitable to develop 40 ksi impact stress in the AZ 14-770 sheet piles at practical refusal. If this or a similar hammer causes damage to the piles as they are driven, it may be throttled back and/or a softer cushion may be used in the helmet. The actual hammer should be selected by wave equation analysis techniques. The stresses developed in the piles may be verified by high strain dynamic testing during the pile installation, if it becomes necessary to drive to high stress levels to penetrate to the design toe levels.

The actual hammer used for pile driving should have a similar or greater rated energy than the Delmag D30-32, depending on the hammer's efficiency, which is defined as the delivered energy as a percentage of the rated energy, relative to that of an open ended diesel hammer. For instance, drop hammers generally have a lower efficiency than open ended diesel hammers. Consequently, a higher rated energy drop hammer would likely be required to deliver the same energy to the pile. Based on the analyses described above, it may be appropriate to specify that the rated energy of the pile driving hammer should be at least 75 foot kips.

Nevertheless, it is possible that piles will meet refusal above their design tip elevations in the upper competent bedrock. If this occurs above five feet below the design dredge grade, techniques such as line drilling and possibly blasting should be used to create a loosened rock trench, into which to drive the sheeting. If refusal occurs more than 5 feet below the dredge grade, it may be assumed that the piling has adequate lateral toe support to resist the active pressures on the sheeting.

Trow International Ltd. should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not accorded the privilege of making this review, Trow will assume no responsibility for interpretation of the recommendations in the report.

The recommended parameters have been calculated by Trow International Ltd. from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, more specific information is available with respect to conditions between boreholes, when pile installation is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field inspections provided by Trow International Ltd., to validate the information for use during the construction stage.

C. D. Thompson, P. Eng.
Technical Director
Geotechnical Engineering

S. E. Gonsalves, P. Eng.
President
Trow International Ltd.

Tables

Table 1: Clark and Walker Classification System

NOTES

- ① Non-carbonate constituents are likely to be siliceous apart from local concentrations of minerals such as feldspar and mixed heavy minerals (Emery 1956).
- ② In description the rough proportions of carbonate and non-carbonate constituents should be quoted and details of both the particle minerals and matrix minerals should be included.
- ③ The preferred lithological nomenclature has been shown in block capitals; alternatives have been given in brackets and these may be substituted in description if the need arises.
- ④ Calcareous is suggested as a general term to indicate the presence of unidentified carbonate. Where applicable, when mineral identification is possible calcareous referring to calcite or alternative adjectives such as dolomitic, aragonitic, sideritic etc. should be used.

Degree of Induration	Approximate Unconfined Compressive Strength	ADDITIONAL DESCRIPTIVE TERMS BASED ON ORIGIN OF CONSTITUENT PARTICLES						TOTAL CARBONATE CONTENT % (constituent particles plus matrix)
		NOT DISCERNIBLE	BIOLASTIC (Organic)	OOLITE (Inorganic)	SHELL (Organic)	CORAL (Organic)	ALGAL (Organic)	
		INCREASING GRAIN SIZE OF PARTICULATE DEPOSITS →						
		0.002 mm	0.050 mm	2 mm				60 mm
Non-indurated	Very soft to hard (<36 to >300kN/m ²)	CARBONATE MUD	CARBONATE SILT	CARBONATE SAND	CARBONATE GRAVEL			90%
		Clayey CARBONATE MUD	Siliceous CARBONATE SILT ①	Siliceous CARBONATE SAND ①	Mixed carbonate and non-carbonate GRAVEL ②			50%
		Calcareous CLAY	Calcareous SILT ①	Calcareous silica SAND ①				10%
		CLAY	SILT	silica SAND	GRAVEL			90%
Slightly indurated	Hard to moderately weak (0.3 to 12.5 MN/m ²)	CALCILUTITE (carb. clayst.)	CALCISILTITE (carb. siltst.)	CALCARENITE (carb. sandst)	CALCIRUDITE (carb. conglom or breccia)			90%
		Clayey CALCILUTITE	Siliceous CALCILUTITE	Siliceous CALCARENITE	Conglomeratic CALCIRUDITE ②			50%
		Calcareous CLAYSTONE	Calcareous SILTSTONE	Calcareous SANDSTONE	Calcareous CONGLOMERATE			10%
		CLAYSTONE	SILTSTONE	SANDSTONE	CONGLOMERATE OR CRECCIA			90%
Moderately indurated	Moderately strong to strong (12.5 to 100 MN/m ²)	Fine-grained Limestone		Detrital Limestone	CONGLOMERATE Limestone			90%
		Fine-grained Argillaceous Limestone	Fine-grained Siliceous Limestone	Siliceous detrital Limestone	Conglomeratic Limestone ②			50%
		Calcareous CLAYSTONE	Calcareous SILTSTONE	Calcareous SANDSTONE	Calcareous CONGLOMERATE			10%
		CLAYSTONE	SILTSTONE	SANDSTONE	CONGLOMERATE OR BRECCIA			90%
Highly indurated	Strong to extremely strong (70 to >200 MN/m ²)	CRYSTALLINE LIMESTONE OR MARBLE (tends towards uniformity of grain size and loss of original texture)						50%
		Conventional metamorphic nomenclature applies in this section						

Table 2: Classification of Rock with Regard to Strength

STRENGTH		FIELD IDENTIFICATION METHOD	RANGE OF UNCONFINED COMPRESSIVE STRENGTH (MPa)	RANGE OF UNCONFINED COMPRESSIVE STRENGTH (tsf)
GRADE	CLASSIFICATION			
R0	Extremely weak	Indented by thumbnail	< 1	< 10
R1	Very weak	Crumbles under firm blows of geological hammer; can be peeled with a pocket knife	1 - 5	10 - 52
R2	Weak rock	Can be peeled by a pocket knife with difficulty; shallow indentations made by a firm blow with point of geological hammer	5 - 25	52 - 261
R3	Medium strong	Cannot be scraped or peeled with a pocket knife; specimen can be fractured with a single firm blow of geological hammer	25 - 50	261 - 522
R4	Strong	Specimen requires more than one blow of geological hammer to fracture	50 - 100	522 - 1044
R5	Very strong	Specimen requires many blows of geological hammer to fracture	100 - 250	1044 - 2611
R6	Extremely strong	Specimen can only be chipped by the geological hammer	> 250	> 2611

Table 3: Classification of Rock With Regard to Spacing of Discontinuities

SPACING CLASSIFICATION	SPACING WIDTH (m)	SPACING WIDTH (inch)
Extremely close	< 0.02	< 0.8
Very close	0.02 - 0.06	0.8 - 2.4
Close	0.06 - 0.20	2.4 - 8
Moderately close	0.2 - 0.6	8 - 24
Wide	0.6 - 2.0	24 - 80
Very wide	2.0 - 6.0	80 - 240
Extremely wide	> 6	> 240

**Table 4: Classification of Rock with
Regard to RQD-Value**

RQD CLASSIFICATION	RQD-VALUE (%)
Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent Quality	90 - 100

Table 5: Bedrock Core Test Results

Sample Number	Elevation	Unit Weight	Point Load Test Results (kPa)		Unconfined Compression Strength (U/C) (tsf) from		
	(ft)	(pcf)	U/A	U/L	U/A	U/L	U/C Test
BH A1-1	-24.6	144.6	1713	3501	161	329	125
BH A1-2	-60.4	139.1	2141	1172	201	110	119
BH A2-1	-30.8	128.0	942	873	89	82	46
BH A2-2	-64.9	141.5	5835		548		138
BH A3-1	-22.9	129.4	176	229	16.5	21.5	39
BH A3-2	-26.9	138.2	434		41		
BH A4-1	-25.8	136.6	176	263	16.5	24.7	65
BH A4-2	-35.7	130.5	1010	737	95	69	48

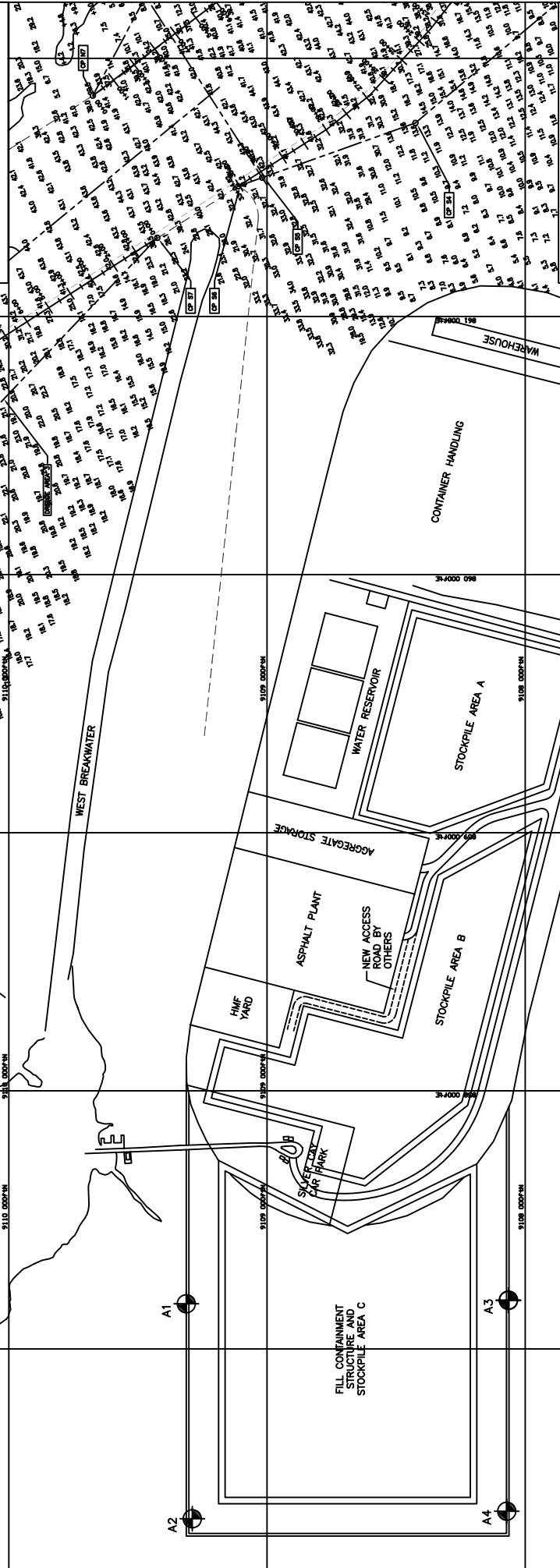
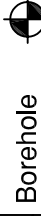
Notes:

- For the Point Load Test results, testing was carried out on bedrock cores, as follows:
 U/A Unsoaked/Axial (Vertical)
 U/L Unsoaked/Lateral (Horizontal)
- Point load test results multiplied by 9 to estimate Unconfined Compression Strength.
- Unconfined Compression Tests, Unsoaked Point Load Tests and Unit Weight determinations were carried out on the samples as received in the laboratory.

Drawings

UTM Zone 18R
 BH A1 0261323m north
 BH A2 0261069m north
 BH A3 0261327m north
 BH A4 0261078m north

Legend



Scale 1:100
 0 100 200 300 400 500



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Geotechnical Investigation
Arawak Cay
 Nassau, Bahamas
Borehole Location Plan

Project No. INTL00302101A
 Date: September 2008
 Drawing No.: 1



LOG OF BOREHOLE A1

Project No. INTL00302101C






Drawing No. 2






Project: Arawak Cay - Nassau

Sheet No. 1 of 2

Location: Nassau Harbour UTM Zone 18R 0261323m east 2776506m north

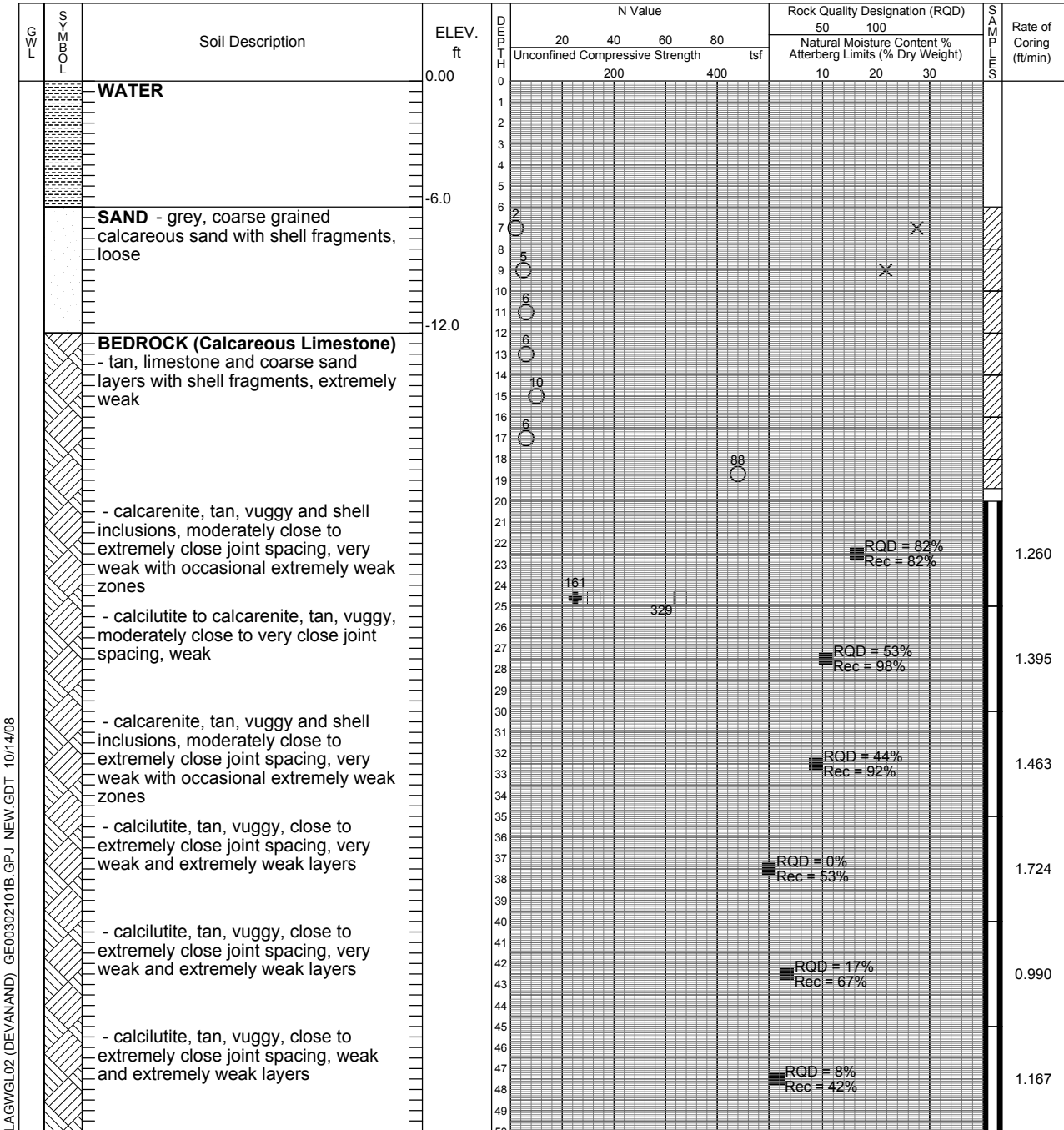
Date Drilled: July 31, 2008

Auger Sample 
 SPT (N) Value 
 Dynamic Cone Test 
 Point Load Test 
 Unconfined Compression 

Rock Quality Designation (RQD) 
 Natural Moisture 
 Plastic and Liquid Limit 
 Undrained Triaxial at % Strain at Failure 
 Penetrometer 

Drill Type: CME55 (Barge Mounted)

Datum: MLWS (ft)



Continued Next Page

LAGWGL02 (DEVANAND) GE00302101B.GPJ NEW.GDT 10/14/08



LOG OF BOREHOLE A2

Project No. INTL00302101C

Drawing No. 3

Project: Arawak Cay - Nassau

Sheet No. 1 of 2

Location: Nassau Harbour UTM Zone 18R 0261069m east 2776499m north

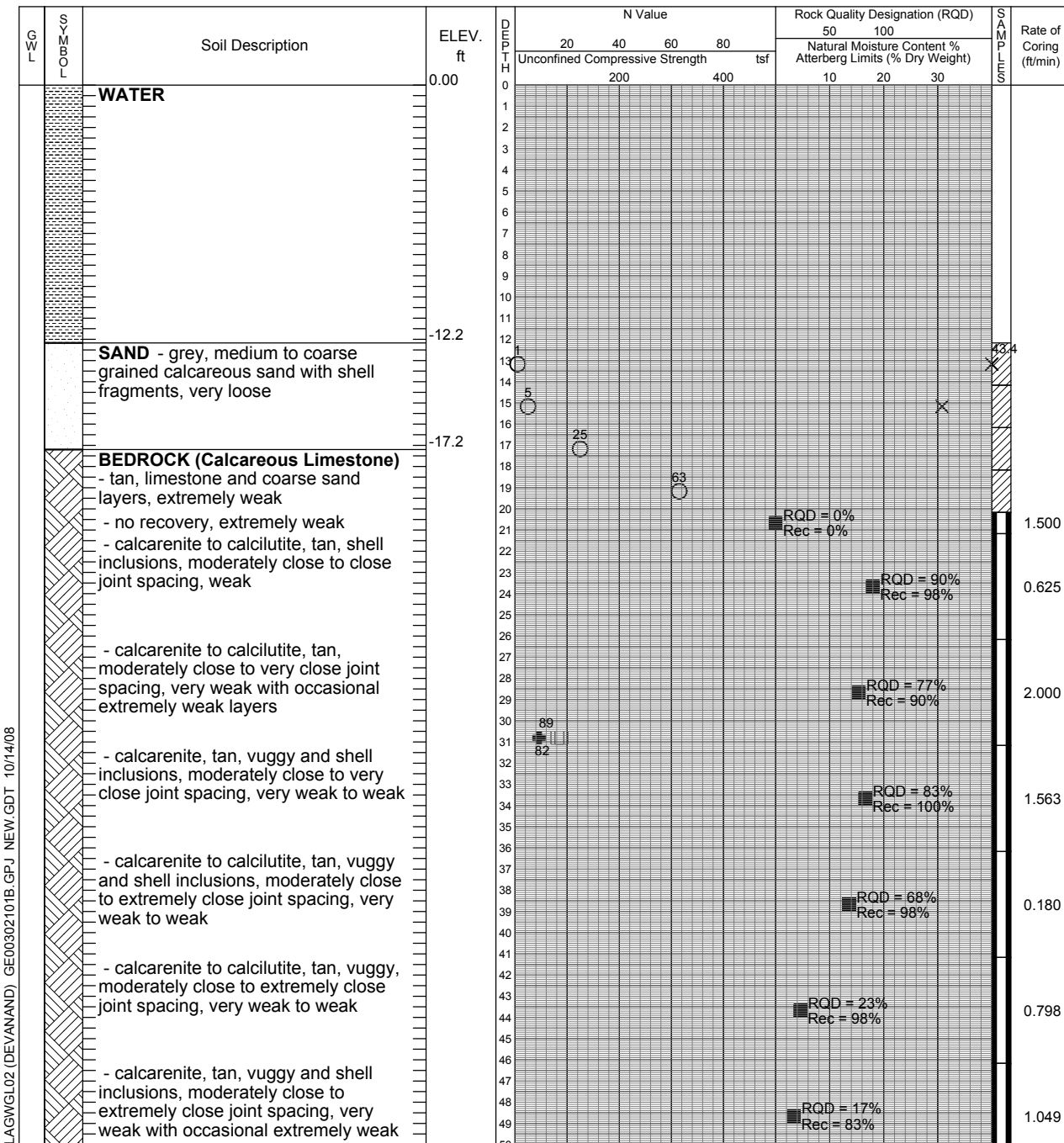
Date Drilled: July 29, 2008

- Auger Sample ☒
- SPT (N) Value ○
- Dynamic Cone Test □
- Point Load Test U/A S/A
- Unconfined Compression U/L S/L

- Rock Quality Designation (RQD) ■
- Natural Moisture ×
- Plastic and Liquid Limit —○
- Undrained Triaxial at % Strain at Failure ⊕
- Penetrometer ▲

Drill Type: CME55 (Barge Mounted)

Datum: MLWS (ft)



LAGWGL02 (DEVANAND) GE00302101B.GPJ NEW.GDT 10/14/08

Continued Next Page



LOG OF BOREHOLE A2

Project No. INTL00302101C

Drawing No. 3

Project: Arawak Cay - Nassau

Sheet No. 2 of 2

G W L	S Y M B O L	Soil Description	ELEV. ft	D E P T H	N Value		Rock Quality Designation (RQD)			S A M P L E S	Rate of Coring (ft/min)	
					Unconfined Compressive Strength tsf		Natural Moisture Content % Atterberg Limits (% Dry Weight)					
					20	40	60	80	50	100		
					200	400	10 20 30					
	zones		-50.00	50								
	- calcarenite to calcilutite, tan, shell inclusions, moderately close to extremely close joint spacing, weak with extremely weak layers			51								
	- calcilutite, tan, vuggy and shell inclusions, moderately close to extremely close joint spacing, very weak to weak with occasional extremely weak zones			52								
	- calcilutite, tan, vuggy and shell inclusions, wide to close joint spacing, weak			53								
				54					■ RQD = 53% ■ Rec = 72%		0.943	
				55								
				56								
				57								
				58					■ RQD = 65% ■ Rec = 90%		0.850	
				59								
				60								
				61								
				62								
				63					■ RQD = 98% ■ Rec = 98%		0.474	
				64								
				65			548					
		END OF BOREHOLE	-65.2									

LAGWGL02 (DEVANAND) GE00302101B.GPJ NEW.GDT 10/14/08



LOG OF BOREHOLE A3

Project No. INTL00302101C

Drawing No. 4

Project: Arawak Cay - Nassau

Sheet No. 1 of 1

Location: Nassau Harbour UTM Zone 18R 0261327m east 2776126m north

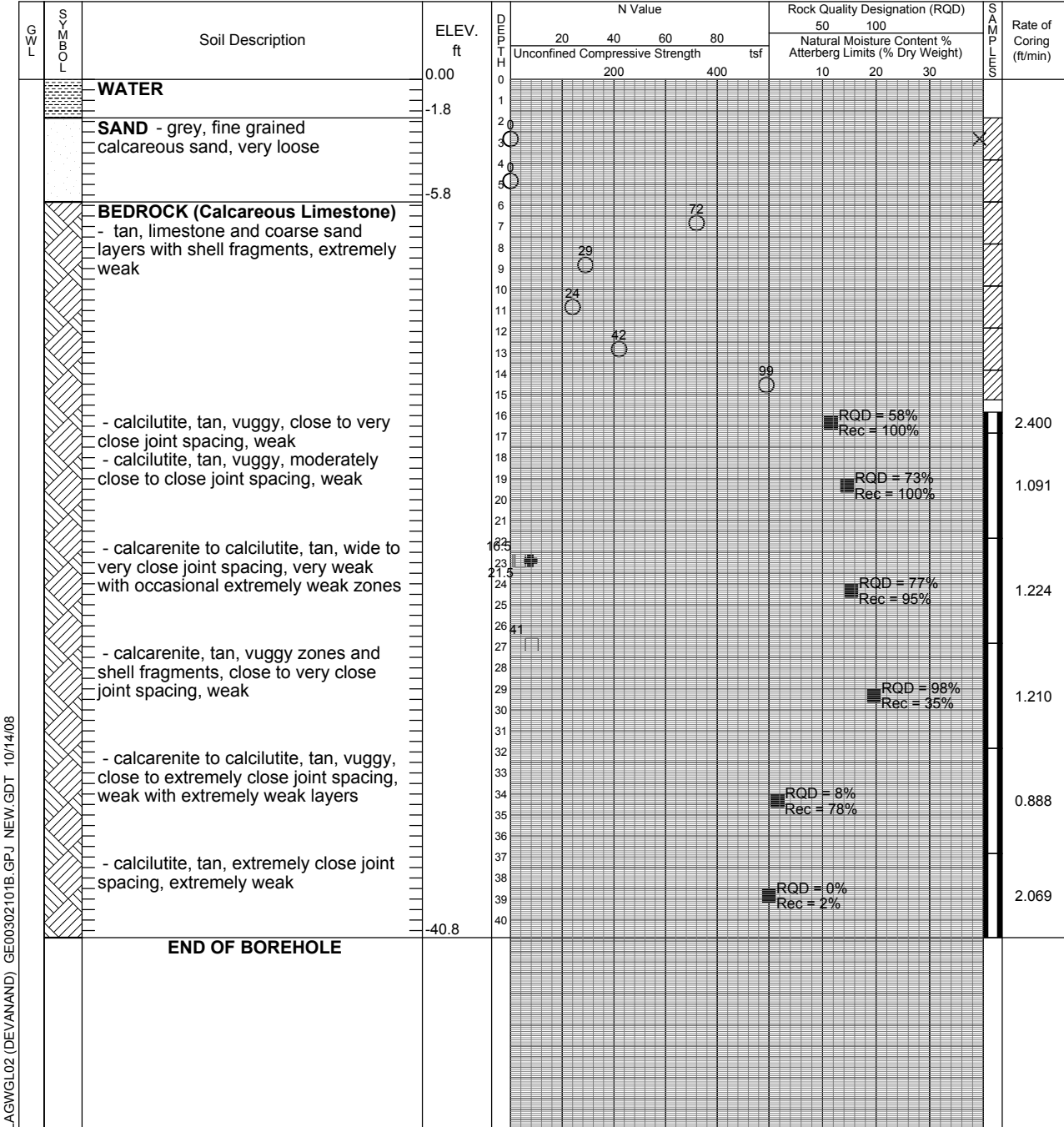
Date Drilled: July 30, 2008

Auger Sample
 SPT (N) Value
 Dynamic Cone Test
 Point Load Test
 Unconfined Compression

Rock Quality Designation (RQD)
 Natural Moisture
 Plastic and Liquid Limit
 Undrained Triaxial at % Strain at Failure
 Penetrometer

Drill Type: CME55 (Barge Mounted)

Datum: MLWS (ft)



LAGWGL02 (DEVANAND) GE00302101B.GPJ NEW.GDT 10/14/08



LOG OF BOREHOLE A4

Project No. INTL00302101C






Drawing No. 5






Project: Arawak Cay - Nassau

Sheet No. 1 of 2

Location: Nassau Harbour UTM Zone 18R 0261078m east 2776128m north

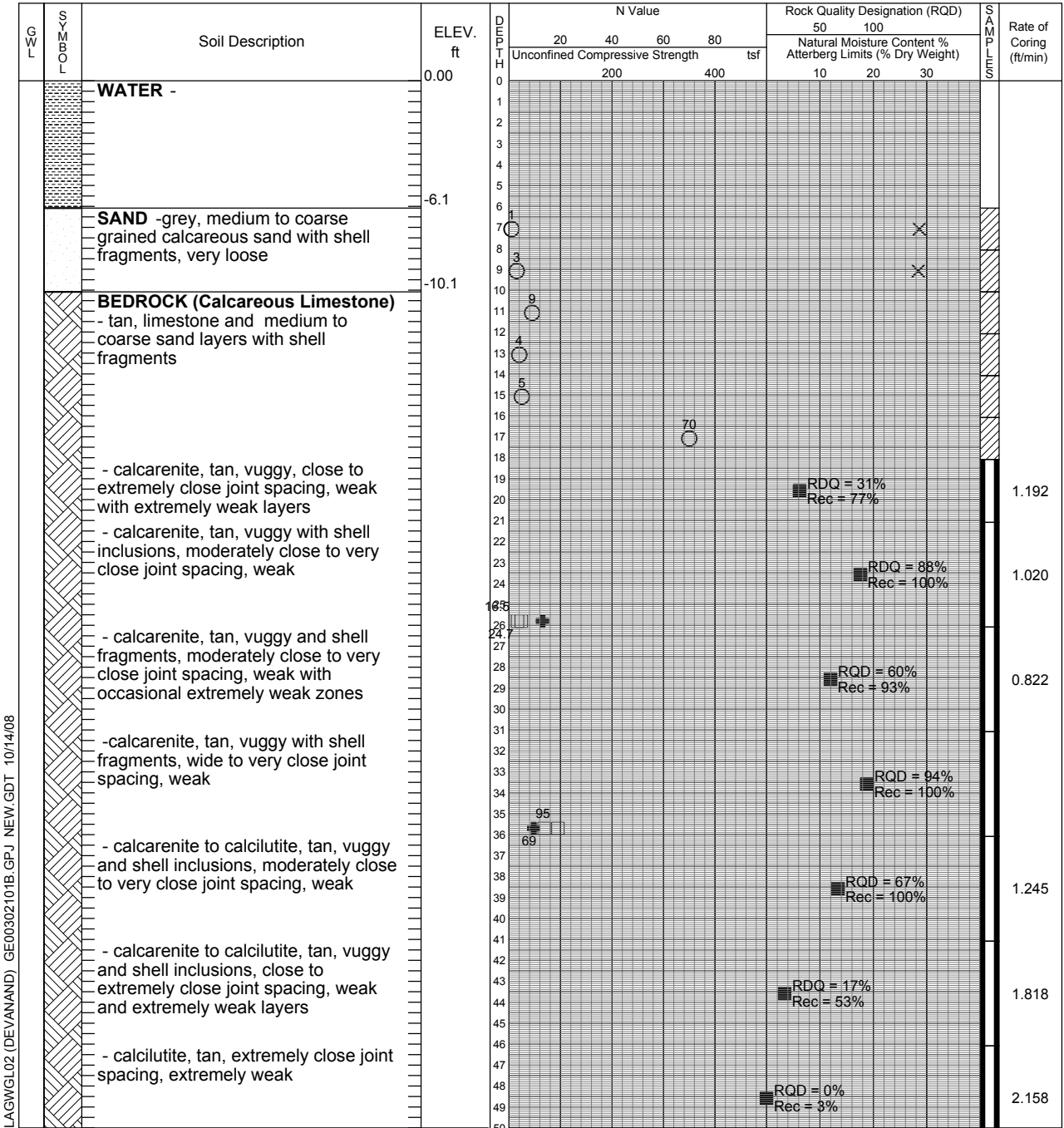
Date Drilled: July 29, 2008

Auger Sample 
 SPT (N) Value 
 Dynamic Cone Test 
 Point Load Test 
 Unconfined Compression 

Rock Quality Designation (RQD) 
 Natural Moisture 
 Plastic and Liquid Limit 
 Undrained Triaxial at % Strain at Failure 
 Penetrometer 

Drill Type: CME55 (Barge Mounted)

Datum: MLWS (ft)



LAGWGL02 (DEVANAND) GE00302101B.GPJ NEW.GDT 10/14/08

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Appendix A

Photographs of Bedrock Cores



BH A1 Box 1



BH A1 Box 2



BH A1 Box 3



BH A2 Box 1



BH A2 Box 2



BH A2 Box 3



BH A3 Box 1



BH A3 Box 2



BH A4 Box 1



BH A4 Box 2



BH A4 Box 3
