Geotechnical Investigation,  
Part 2, Mooring Dolphins  
Nassau Harbour Port Improvement Project

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 dredging of Nassau Harbour in the Bahamas and for the installation of three mooring dolphins at the ends of piers at Prince George Wharf by means of receipt of a mobilization payment on April 17, 2008. Formal Authorization was subsequently received in the form of a signed authorization sheet. This report documents information and suggestions regarding geotechnical aspects for the design of the mooring dolphins.

The fieldwork for this investigation consisted of drilling and sampling 15 boreholes to 5 feet below the proposed dredge grades of El. -38 feet MLWS inside the harbour and El. -40 feet in the entrance channel plus three boreholes to El. -100 feet at or near the location of the mooring dolphins. The overburden and the upper levels of the bedrock were sampled by carrying out Standard Penetration Resistance 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 and grain size distribution 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, it was determined that the correlation multiplication factors are 9 and 12 for the shallow and deep bedrock respectively. Point load tests on as received cores and the same core run soaked for at least 24 hours in artificial seawater indicated that differences in strength due to sample preparation are significantly less than differences in the natural strength of the bedrock on the same core run.

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.

The surficial stratum at the mooring dolphins is very loose medium to coarse grained calcareous sand. It was encountered between El. -21.1 feet at the East Dolphin and at Els. -41.0 to -41.8 feet at the West Dolphins. It is 0.4 to 4.0 feet thick. Calcareous limestone bedrock underlies the sand at Els. -25 feet (East Dolphin) to -45 feet (West Dolphins). It is extremely weak above El. -57.5 feet at the north west location dipping to El. -73.7 feet in the east location. Below these levels it is generally interlayered weak and extremely weak material. Below El. -86 feet, there is a 10 feet to at least 14 feet thick layer of fair to excellent quality, strong to weak bedrock with very close to wide joint spacing. Extremely
weak bedrock was encountered below this stronger layer at Els. -96 to -99 feet at the west mooring dolphins.

Cox and SHAL Consultants (C&S) are considering three design options for the mooring dolphins. Geotechnical related information about the support mechanism for each of these is summarized below:

- Sixteen 36 inch diameter by 0.75 inch wall thickness steel open ended pipe piles installed vertically and filled with reinforced concrete to 10 feet above their bottoms – The vertical working load on each pile is 125 to 130 kips. This requires that the piles be driven to between Els. -86 and -90 feet to gain adequate support with reasonable certainty. The lateral stability analyses indicated that the piles should be driven to Els. -80 and -70 feet at the West and East Dolphins respectively. At the design maximum lateral load of 43 kips per pile, it is estimated that the head deflection for 0.625 inch wall thickness piles with their bottoms at these levels will be 7.6 and 4.3 inches at the West and East Dolphins respectively with resultant maximum moments of 25,800 and 21,200 kips-in. The deflections and moments will be less for 0.75 inch wall thickness piles. Piles driven to below El. -86 feet, as required to achieve adequate bearing capacity with reasonable certainty, should therefore adequately resist the lateral design loads.

- Twenty-five 18 inch diameter by 0.625 inch wall thickness steel open ended pipe piles with some driven vertically and others on a batter to obtain 202 kips compression and tension support from the bedrock – The steel area for the bottom three feet of these piles should be doubled by welding a section of the piles around the outside. The piles should then be driven to between Els. -86 and -87 feet to obtain compressive support from a more competent 10 feet thick layer of bedrock at this level. Tension support should be provided by 10 feet long rock anchors installed below the base of the piles. The allowable bond between the grout for the rock anchors and the bedrock may be assumed to be 100 psi.

- Forty-six feet diameter steel sheet pile cells driven to 5 feet below the design harbour bottom levels and filled with dredged material – Design details for bearing capacity, friction, sliding resistance and overturning are detailed in the text of the report. It should be expected that there will be significant settlement of the cells as they are filled with dredgings.

The mooring dolphins will have pedestrian bridges to the existing piers. These will each have a single central pier supported by four vertical 20 inch diameter by 0.625 inch wall thickness open ended steel pipe piles with a design compression load of 35 kips per pile and a design lateral load of 11.5 kips per pile. The highest bottom of pile elevations, at which adequate lateral support is available, is -70 and -65 feet at the west and east locations respectively. Depending on whether the piles gain support from the more competent zones in
the bedrock, the required bearing capacity may be available at this level for the west pier piles. However, the piles for the east pier would have to be driven to at least El. -79 feet to obtain adequate compressive support. If the extremely weak bedrock zones dominate the bearing capacity properties at these levels, both the east and west piles may have to be driven to approximate El. -86 feet to achieve adequate support. In view of the uncertainty about achieving adequate bearing above this level, it is suggested that it should be assumed that the piles will be driven to between Els. -86 and -90 feet.

Comments about pile installation procedures and monitoring and testing requirements for the various options are presented in the text of the report.
2. Introduction

On April 17, 2008, Trow International Ltd. (Trow) 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 purpose of the investigation was to provide geotechnical data about the stratigraphy above the proposed dredge grades in the harbour and entrance channel to Nassau and for three proposed dolphins, which are planned at both ends of the northerly pier within Prince George Wharf and the west end of the middle pier. Additionally, geotechnical parameters were to be provided for the design of the mooring dolphins.

The proposed mooring dolphins will be located 300 feet off the ends of the piers to extend their effective length. Support for the dolphins, which are being considered, include:

- Large diameter vertical piles;
- Smaller diameter vertical and batter piles; and
- Steel sheet piles cells.

As well as the data obtained during this investigation, information from a previous investigation for the northerly pier at Prince George Wharf and associated dredging has been used to assess the geotechnical conditions in the area. This report is entitled “Nassau Harbour Expansion and Family Islands Harbour Improvements”, a geotechnical study written by Woods Engineering Consulting, Inc., dated August 17, 1989 (Woods Report). Because it is considered that discrepancies between the methods of description used in this report and the Woods Report might result in interpretive confusion, the borehole logs from the Woods Report are not included with this report.

During the production of the report, it was agreed that the comments about the dredging and about the design of the mooring dolphins would documented separately. This report comments on the design of the mooring dolphins. The comments about the dredging are presented in a report dated August 8, 2008 (the Dredging Report).

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, as presented in the proposal for the project, dated November 2007, were to:

- Drill and sample 18 boreholes to 45 feet depth in four areas that had been identified to be dredged in the Request for Proposals by the Government of Bahamas, Ministry of Works & Transport and 3 boreholes to 100 feet depth for 3 mooring dolphins. Location of the boreholes was to be by GPS methods. The elevations were to be established from tidal gauges.

- 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.

- The site was to be visited by the project engineer in the early stages of the fieldwork to: review site conditions and drilling procedures and modify them, if required; meet with personnel of other members of the consortium and have discussions with Government geologists about the expected stratification at the site.

- Geotechnical laboratory testing would consist of: moisture contents on all soil samples; grain size distribution tests on one soil sample per borehole; and unconfined compression tests on two sections of bedrock per borehole.

- 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: graphical presentation of the grain size distribution results, tabulation of the unconfined compression tests results; analysis of the dredgability of the materials above 40 feet depth; and provision of geotechnical engineering parameters for the mooring dolphins.

- 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.
During production of the report, it was agreed that it would be presented in two parts: all the data obtained, interpretation of that data, and comments about the dredging aspects of the construction; and a subsequent report providing geotechnical parameters for the design of the mooring dolphins. This report covers the second part of the complete report.

For reasons outside the control of Trow, the project manager, Mr. C. D. Thompson, was unable to visit the site during the drilling. Instead, he fulfilled all the functions of the site visit, with the exception of meetings with Government geologists, by very close communication with the Trow engineer on site and with Mr. T. Hluchan of SHAL Consulting Engineers Ltd. Dr. S. Micic, Ph.D., P.Eng., a senior geological engineer from Trow, visited the site on two occasions to fulfill these requirements.

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 10 boreholes (BHs 1, 3, 6 to 10 and 12 to 15) to approximately El. -43 feet MLWS, 5 boreholes (BHs 11, 14 and 16 to 18) to approximate El. -45 feet MLWS and 3 boreholes (BHs 2, 4 and 5) to El. -100 feet MLWS. The drilling and sampling was carried out by Toney Drilling Supplies, Inc. under the direction of Trow International Ltd. between May 22 and June 9, 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. This sampling was carried out continuously in some boreholes and at up to 5 feet intervals in others. 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 and the depth of the water surface below the top of the cope wall for the northerly pier at Prince George Wharf at as close to the same time as practical. The elevation of the top of the cope wall was assumed to be +7.5 feet MLWS based on information provided by C&S. 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 geological 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 19). 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 and grain size distribution tests on selected soil samples and unconfined compression and point load tests on the bedrock cores. As there were few soil samples and most of them were given to Blue Engineering for environmental
testing, only 5 moisture content and 5 grain size distribution tests (Drawings 20 to 24) were carried out. Furthermore, the samples were small and it is possible that the samples had dried out somewhat, even though they were stored in airtight bags.

A total of 31 unconfined compression tests were performed on bedrock core, as received from the field. Point load tests were undertaken on the cores both axially (in the vertical direction) and laterally (in the horizontal direction) both as received in the laboratory and after they had been soaked in artificial seawater for at least 24 hours. The point load test results were calibrated for unconfined compression strength by comparing all those, on which unconfined compression tests had been undertaken, with the results of that testing. A total of 61 point load tests were carried out. The as received unit weight of the bedrock cores was measured for 38 samples. The moisture content of 19 samples, which had been soaked in artificial seawater for at least 24 hours, was measured. The results of the testing are presented in Table 5.
5. Interpretation of Factual Information

The factual information obtained during the investigation 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

The point load tests (PLT) were correlated with the unconfined compression (U/C) test results on similar samples, which were as received in the laboratory. Initially, all the PLT results were used and they were separated into axial and lateral test results. While this indicated that the axial correlation multiplier was likely slightly greater than the lateral multiplier, it revealed that the correlation multiplier for the shallow bedrock cores above the design dredge grades was likely significantly less than in the deeper bedrock at the mooring dolphins. This difference was greater than between the axial and lateral multiplier correlation difference. Consequently, it was considered that it was more appropriate to determine PLT correlation multipliers for all results at above and below El. -58 feet, which is the highest level, at which corable deeper bedrock was encountered in the drilling. The resultant correlation multipliers are 9 for shallow bedrock and 12 for deep bedrock.

5.2 Impact on Strength of Soaking Bedrock Cores

The strengths of the bedrock were compared to assess whether they were impacted by air drying. PLTs on as received samples were compared with those that had been soaked for at least 24 hours in artificial seawater and the moisture content after soaking was measured for selected samples. Overall, there is a slight tendency for the soaked PLTs to be greater than the unsoaked results. However, the scatter of results is so great that it is considered that strength tests are directly comparable for both preparation procedures. This is in agreement with similar comparisons by Trow on other projects in the Bahamas Archipelago.

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 5. 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 proposed dredging and mooring dolphins consisted of 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 is difficult. Consequently, the boreholes have been undertaken to assess ranges of engineering properties that can be expected, both in the dredging and the installation of the mooring dolphins rather than the precise stratification at the site. The data from them should be interpreted and analyzed on this basis.

This section of the report describes the geology and subsurface conditions, as follows:

1. Geology of the Bahamas.
2. Subsurface Conditions near the Proposed Mooring Dolphins.

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. Recrystalization 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.

Ooliths form around discrete particles such as silica. The ooliths 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.

### 6.2 Subsurface Conditions near the Proposed Mooring Dolphins

Due to difficulties keeping out of the way of cruise ship traffic, only BH 2 was drilled at the exact location of a mooring dolphin. However, based on the data from Boreholes 2, 4 and 5 and from Woods Report BHs 1 to 11 NAS, the geotechnical conditions are sufficiently consistent to extrapolate conclusions about them to the locations of the mooring dolphins, as long as appropriate consideration is given to the natural variability of the bedrock.

At BH 2, there is 4 feet of very loose sand below the harbour bottom, which is at El. -21.1 feet. It is grey, medium to coarse grained and calcareous. The harbour has been previously dredged at BHs 4 and 5 to between Els. -41.0 and -41.8 feet. Indications are that material removal has occurred deeper in some locations, as there is 4 feet of very loose sand overlying the bedrock at BH 4. At BH 5, there is a thin veneer of 0.4 feet of sand over the bedrock. The Standard Penetration Resistances (SPRs) of the sand range from 1 to 4 blows per foot. The moisture content of one sample was measured to be 33%.

Calcareous limestone bedrock underlies the sand at Els. -25.1 feet (BH 2) to -45 feet (BH 4) and extends to the bottom of the boreholes at Els. -100.0 to -100.8 feet. Its texture ranges from calcarenite to calcilutite. It is tan in colour. It is predominantly vuggy where it was cored.

The bedrock was sampled with split barrel samplers to between El. -57.5 feet (BH 4), El. -58.3 feet (BH 5) and El. -73.7 feet (BH 2), indicating that the extremely weak upper bedrock likely deepens to the east, as was also assessed from the boreholes for the dredging. The SPRs of the upper extremely weak bedrock ranged from 6 to 52 blows/foot and are predominantly about 10 blows per foot. This, in equivalent soil consistency terms, would be firm to hard with estimated unconfined compression (U/C) values of 0.5 to 5 tsf or, in
equivalent soil compactness terms, would be generally loose increasing to very dense at some locations and levels.

The core recovery above approximate El. -86 feet ranged from 50 to 100% with assumed extremely weak bedrock filling the unrecovered sections. The RQD Classification quality of this upper corable bedrock ranges from very poor (0%) to good (90%) and the joint spacing is extremely close (less than 0.8 inches) to moderately close (8 to 24 inches). The upper corable bedrock is generally interlayered weak (U/C of 98 to 273 tsf) and extremely weak material (U/C of less than 10 tsf). At some levels, the weak bedrock predominates and at others the extremely weak bedrock.

For 10 feet (BHs 4 and 5) to more than 14 feet (BH 2) below approximate El. -86 feet, the core recovery ranged from 90 to 100%. The RQD Classification quality is fair (57%) to excellent (96%). The joint spacing is very close (0.8 to 2.4 inches) to wide (more than 24 inches). This bedrock stratum is strong to weak (U/Cs of 116 to 740 tsf) calcilutite that does not contain extremely weak zones. It extends to the bottom of BH 2.

The more competent bedrock encountered at El. -86 feet is underlain at approximate Els. -96 (BH 5) to -99 feet (BH 4) by extremely weak bedrock (U/C of less than 10 tsf). The core recovery of this bedrock was zero.
7. Design of Mooring Dolphins

Information from C&S indicates that three designs are being considered for the mooring dolphins. For each dolphin, these are:

- Sixteen 36 inch diameter by 0.75 inch wall thickness steel pipe piles installed vertically open ended and filled with reinforced concrete to 10 feet above the bottom, supporting a 6 feet thick concrete pile cap with a top elevation of +9.5 feet (Large Diameter Vertical Piles);

- Twenty-five open ended eighteen inch diameter by 0.625 inch wall thickness steel pipe piles installed vertically and on a batter resisting the mooring loads in compression and tension plus possible rock anchors drilled below the bottom of the piles. The piles would support a 6 feet thick concrete cap with a top elevation of +9.5 feet (Smaller Diameter Vertical and Batter Piles); and

- Forty-six feet diameter steel sheet pile cells using web arch piles driven 5 feet into the bedrock and filled with dredged material and covered with a 4 feet thick concrete cap with a top elevation of +9.5 feet (Steel Sheet Pile Cells).

There will be pedestrian bridges from the existing piers to the mooring dolphins. Central piers will be constructed for these bridges and they will be supported by four vertical or near vertical open ended 20 inch diameter by 0.625 inch wall thickness pipe piles.

All the pipe piles are open ended to ensure that they can be installed to sufficient depths to obtain adequate lateral support from the bedrock and/or to be able to obtain the required tension resistance.

The design harbour bottom is El. -42 feet at the West Dolphins and El. -30 feet at the East Dolphin. The geotechnical design has assumed that the very loose sand above El. -45 feet in the vicinity of the West Dolphins will not always be present.

Prior to carrying out a geotechnical analysis for the alternative designs, the subsurface profile was examined to establish geotechnical parameters for the strata at the dolphins. These are presented in Table 6.

The design alternatives and the support for the bridge piers are discussed in Sections 7.1 to 7.4 below. The discussion of the preferred design alternative of large diameter vertical piles (Section 7.1) and of the bridge piers (Section 7.4) is more detailed than of the other design alternatives. If one of the other design alternatives is subsequently employed, Trow should be contacted to assess whether further geotechnical related comments should be made.
7.1 Large Diameter Vertical Pile Support

The procedure for the design of the large diameter vertical pile support of the mooring dolphins consisted of an iterative process between C&S and Trow. Initially C&S provided conceptual design sketches for the dolphins. Trow then analyzed the lateral support characteristics of the proposed piles installed to different penetrations below harbour bottom to assess the required depth of penetration and the resulting deflections and moments in a single pile for a range of lateral loads at the top of the pile. The top of the pile was assumed to be rigidly held by the mooring dolphin cap. This analysis was undertaken for both 36 inch diameter by 0.625 inch wall thickness pipe piles then proposed for the mooring dolphins and 18 inch diameter by 0.625 inch wall thickness pipe piles then proposed for the bridge piers. All the lateral loading analyses were undertaken using the FLPIER computer program. The results of the analyses are presented in Appendix B. In the output shown in Appendix B, the Z Nodal Coordinates in the pile displacement graphs are the depths, in inches, below the bottom of the pile cap at El. +3.5 feet. In the analyses, EI was used for the steel and 0.35 EI for the concrete, where E is the modulus of elasticity and I is the moment of inertia.

A sensitivity analysis for the lateral loading analysis was undertaken by increasing the geotechnical parameters and determining the resultant impact on the lateral movements of the 36 inch diameter piles. For this purpose, the angles of internal friction for the top two strata described in Table 6 were increased by five degrees and the cohesion of the third stratum was increased to 40 tsf. The sensitivity analysis indicated that a substantial increase in the assumed soil strengths reduced the lateral movements at the seabed by approximately 20%. This was not considered to be significant, thereby confirming that the geotechnical parameters (Table 6), which were fairly conservative due to allowances for the crushable calcareous nature of the materials providing the lateral support, were providing reasonable values.

Based on data from research notes from Caltrans’ GeoResearch Group, GRG Vol. 1, No. 6, May 2003, the impact of group action was assessed. Applying these data to a four by four pile group at pile spacings of three pile diameters, it was analyzed that the impact of the group action would be to increase the lateral deflections and moments by a multiplier of 1.5 at a specified load. Conversely, the lateral deflections and moments in the single pile analysis in Appendix B would be generated at two thirds of the analyzed loads, when applied in a four by four pile group.

The results of the preliminary analyses were discussed with C&S and, subject to final analytical verification, the required depth of penetration to provide adequate lateral support was established to be to Els. -80 feet and -70 feet at the West and East Dolphins respectively. Further it was decided that the piles would be filled with reinforced concrete to 10 feet above the bottom of the piles, which was assessed to be the approximate level, above which the bedrock will provide lateral resistance to the direction of load application (below this level the bedrock is stopping the piles from rotating).
Analysis of the “final” structural design of the 36 inch diameter by 0.625 inch wall thickness piles was then undertaken using the geotechnical parameters in Table 6 and the following data supplied by C&S:

- Zero tension and compression and a maximum lateral load of 43 kips, which was analyzed as the equivalent to 65 kips on a single pile;
- Fixity at the top of the piles at El. +3.5 feet;
- Design harbour bottom grades of El. -45 feet at the West Dolphins and El. -30 feet at the East Dolphin;
- Bottom of the piles at El. -80 feet at the West Dolphins and El. -70 feet at the East Dolphin;
- Loss of 0.125 inches of steel due to corrosion over the lifespan of the dolphins; and
- Reinforcement in the concrete in the piles of 12 #11 bars in a 29 inch diameter circle with #4 ties at 12 inch centres.

The analyses indicate that, at the lateral design load, the maximum lateral deflections, including those in the bedrock, and the maximum moments are at the top of the piles and are:

- 7.6 inches and 25,800 kips-in at the West Dolphins; and
- 4.3 inches and 21,200 kips-in at the East Dolphin.

The lateral deflections at the design dredge grades are 1.4 and 1.5 inches at the West and East Dolphins respectively.

The design has subsequently been changed to the piles having 0.75 inch wall thickness. This and the compression loading will decrease the deflections and moments presented above.

The working compression loads on the piles are 130 kips at the West Dolphins and 125 kips at the East Dolphin. There will be zero uplift loading.

The friction on these piles at a specific depth can be determined from the equation below using the parameters, which are presented in Table 6.

\[ q_s = \beta \sigma_v' \]

Where:
- \( q_s \) = the ultimate unit frictional resistance in psf
- \( \beta \) = the combined shaft resistance factor
- \( \sigma_v' \) = the vertical effective stress in psf
The ultimate end bearing stress of the extremely weak bedrock is estimated to be 300 psi or less above El. -58 feet and -74 feet at the West and East Dolphins respectively. Between these levels and approximate El. -86 feet, the bedrock is interlayered weak to very weak bedrock with an estimated ultimate bearing strength of 3 ksi and extremely weak bedrock with an estimated ultimate bearing strength of 300 psi. The impact of the interlayering cannot readily be assessed. However, previous experience would indicate that the strength properties of the extremely weak (300 psi bearing) material may govern the end bearing on the piles. Consequently, it is unlikely that the piles will achieve allowable capacities of 125 to 130 kips above approximate El. -86 feet.

Consequently, it is suggested that the piles be driven into the more competent bedrock below El. -86 feet to achieve the required capacity. They should not be driven deeper than El. -90 feet, as below this level there is a danger that they might drive through the bearing stratum. High strain dynamic testing with a Pile Driving Analyzer or similar equipment and associated wave equation analysis based on its data should be carried out to verify the bearing capacity. Any wave equation analysis using the data from the high strain dynamic testing should take into account only end bearing on the steel area of the piles, i.e., it should not assume any bearing resistance from soil or bedrock plugs, if they occur during the pile driving, as these may relax with time. The high strain dynamic testing should establish the required penetration resistance to achieve the design working load with a factor of safety of 2.5.

It should be possible to drive the 36 inch diameter by 0.75 inch wall thickness open ended pipe piles to the planned levels by using a pile driving hammer capable of developing 30 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 Table 6. WEAP analysis of a number of Delmag hammers has indicated that a Delmag D30-32 with a rated energy of 75.4 foot kips per blow is just suitable for this purpose. A Delmag D36-32 with a rated energy of 90.4 foot kips per blow would provide greater flexibility and is unlikely to overstress the piles, as long as it is properly operated.

The actual hammer used for pile driving should have a similar or greater rated energy than the D 30-32, 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 be at least 90 foot kips, even though a lower rated energy hammer could theoretically be used.

The penetration resistance required to achieve a design working load of 130 kips with the Delmag D36-32 hammer has been assessed by the WEAP analysis and is estimated to range
from 18 blows per foot with a piston stroke of 11.3 feet to 24 blows per foot with a piston stroke of 7.5 feet, as long as a soil or bedrock plug does not form. If one does form, the piles should be driven to a penetration resistance of at least 20 blows per inch. The actual required penetration resistance should be determined from high strain dynamic testing and associated wave equation analysis. It should be recorded for three consecutive inches of penetration showing increasing resistances. These resistances should be achieved in the bearing stratum between Els. -86 and -90 feet. If the piles drive to El. -90 feet without showing the required penetration resistance, Trow should be consulted about the most suitable procedures to ensure that they have adequate capacity.

An auger of effectively the same diameter as the inside of the piles will be required to remove the material above 10 feet above their bottoms. If there is difficulty driving the piles to El. -86 feet, this auger can be used to auger out any soil and bedrock plug and loosen the bedrock ahead of the pile. However, it should not be used below El. 86 feet.

Once the piles have been augered out to 10 feet above their bottoms, the loose material remaining in them above this level should be cleaned out until water flows clear, when flushing the piles under high pressure. The concrete should have at least a seven inch slump and should be placed by tremie methods.

### 7.2 Smaller Diameter Vertical and Batter Pile Support

C&S have indicated a number of design parameters that they require for the 18 inch diameter open ended steel pipe piles. The piles must provide both compressive capacity and uplift resistance depending on the loading configuration. C&S has indicated that the required working load on the 18 inch diameter piles is 202 kips in both tension and compression. It is proposed that these piles be driven open ended and, if necessary, it is planned to install rock anchors below their bottoms to provide the tension resistance.

From past experience with driven piles in the type of stratigraphy encountered in the borings, both end bearing and friction can be fairly low even though the borings indicate reasonably competent founding materials. The reasons for this are that piles tend to slice through and crush the weak calcareous limestone, rather than displacing it. For instance, it is possible that the ultimate end bearing capacity of 18 inch diameter open ended pipe piles driven to the top of the more competent rock at El. -86 feet could be less than 6 ksi times the steel area. This results in a calculated ultimate end resistance of approximately 210 kips for the open ended 18 inch diameter by 0.625 inch wall thickness pipe piles. From the data in Table 6, the ultimate friction resistance of the piles, when founded at El. -86 feet, has been calculated to be approximately 100 and 130 kips at the West and East Dolphins respectively. Applying a factor of safety of three, which is considered appropriate if no form of load testing is undertaken, the piles do not have adequate capacity to resist the required loads. The factor of safety can be reduced, depending on the extent of testing of the piles. However, it is unlikely that acceptable factors of safety in compression could be achieved by a testing programme.
Driving the piles deeper into the more competent bedrock would increase the end bearing such that the required compressive capacity may be achieved. While the piles should not be driven significantly deeper than El. -86 feet, it should be recognized that there is the potential for some inaccuracy in determination of this level. Consequently, if it appears from the pile driving that the penetration resistance is such that piles have not reached the more competent bedrock stratum, they may be driven a foot or so deeper. However, driving the piles significantly deeper would result in a loss of depth of the more competent bedrock, in which to achieve bond resistance for rock anchors, and the increase in frictional resistance on the piles would be minor.

The compressive capacity of the piles can be increased by increasing the end bearing area of the piles. If a section of the 18 inch diameter by 0.625 inch wall thickness pile is cut longitudinally and welded in place around the bottom of the pile, the end bearing resistance will be doubled. A three feet long section of pile should be used for this purpose. While this will reduce the frictional resistance to possibly half of the values present above, the frictional capacity is anyway so low that rock anchors are needed to resist the tension loads. If sufficient testing is undertaken, the factor of safety can be reduced to approximately two. Consequently, as we recommend tension testing of the rock anchors and these would be accomplished by applying the same compression load to the piles, adequate compression testing can readily be accomplished. It is therefore considered that the required working load of 202 kips in compression can be obtained by doubling the end area of the pipe by the means suggested above.

It is recommended that 18 inch diameter pipe piles be driven with a pile driving hammer capable of developing at least 30 ksi stress per blow at practical refusal. The pile driving hammer may be selected by undertaking wave equation analysis for the proposed piles assuming the geotechnical parameters presented in Table 6. The piles should be driven to El. -86 to -87 feet. It is expected that the open ended piles should be drivable to El. -86 feet with a pile driver of the suggested capacity as long as a soil plugs do not form in them. If these do form, there is a reasonable chance that the piles will meet practical refusal at higher levels. In this case, the piles may be extended deeper by augering ahead of the piles and/or preaugering prior to pile driving. Augering should terminate at El. -83 feet, so that the bedrock at the pile tip is not disturbed.

The tension capacity of the piles should be achieved by rock anchors without any assumption of frictional resistance from the piles. If the anchor is drilled with equipment that creates a rough sided hole, such as an airtrack drill, the allowable bond strength between the concrete and the bedrock may be taken as 100 psi. The rock anchors should be extended to El. -96 feet. If 6 inch diameter holes are drilled, the calculated allowable bond capacity of a 10 feet long anchor is 226 kips.

Before filling the piles with concrete, at least two anchors at each mooring dolphin should be load tested to twice the required capacity and all the rock anchors should be proof tested to
1.33 times the required capacity. If these tests result in failure of the bond between the concrete and the bedrock, the diameter of the rock anchor hole should be increased, using a factor of safety of two on the bond strength proved in the test. On completion of the tests, 10% of the required tension load should be locked into the rock anchor system for seating purposes.

If the installation of the rock anchors has disturbed the bedrock, it is possible that the testing will result in movement of the piles. This is not deemed to be critical, as long as the piles stop moving within six inches penetration. The penetration can be decreased by pouring a three feet long concrete plug in the base before the rock anchor is installed. In this case, care should be taken that the installation procedure creates a larger diameter hole through the concrete plug than the maximum diameter of the rock anchor hole and that there is no contact between the grout in the rock anchor hole and the concrete in the plug.

The suggested installation and testing programme will effectively proof test most of the piles in both compression and tension. However, there will be some piles, which will be loaded only in compression and therefore do not require rock anchors and the consequent proof testing. These piles may be driven up to 5 feet into the more competent bedrock below El. - 86 feet to gain support, if required. It is suggested that high strain dynamic testing and analysis with a Pile Driving Analyzer or similar equipment be used for verifying the capacity of the piles.

The concrete in the piles should have at least a seven inch slump and should be placed by tremie methods.

7.3 Steel Sheet Pile Cells

Steel sheet pile cells would be constructed by threading them together in a 46 feet diameter ring and then driving them to approximately 5 feet below the design harbour grades. They would then be filled with dredgings.

The sheet pile cells will act as large footings on the extremely weak upper calcareous limestone bedrock, which is considered to behave as a loose sand. While it is likely that the cells will be safe against rotational shear failure, the safe bearing pressure should be calculated according to the equation below:

\[ F_s Q_s = 0.5 \gamma' B N_{\gamma} + q' N_{q} \]

Where:
- \( F_s \) = Factor of Safety = 3
- \( Q_s \) = Safe Bearing Capacity, psf
- \( \gamma' \) = Effective Unit Weight of Bedrock = 55 pcf
- \( B \) = Diameter of Sheet Pile Cell = 46 feet
- \( N_{\gamma} \) = Bearing Capacity Factor = 11
- \( N_{q} \) = Bearing Capacity Factor = 11
\( q' = \text{Effective Overburden Pressure at the bottom of the sheeting} = 5 \times \gamma' \)

\( N_q = \text{Bearing Capacity Factor} = 14 \)

The settlements of the sheet pile cells may be larger than is normally tolerated for building structures. They will occur quickly after the loads are applied. For estimating purposes, it is suggested that the settlement be assumed to be one inch per two ksf maximum effective vertical pressure at the base of the sheeting and that the top of the sheeting be left the resultant amount high until the cell has been filled with dredgings.

Dredgings backfill may be assumed to have a bulk density of 100 pcf in the as placed condition with a coefficient of active earth pressure of 0.40. If the backfill is compacted by vibroflotation techniques or similar, the bulk density should increase to approximately 120 pcf with a coefficient of active earth pressure of 0.27.

The very loose sand between Els. -41 and -45 feet at the West Dolphins and between Els. -21 and -25 feet at the East Dolphin may be assumed to have a bulk density of 90 pcf and a coefficient of active earth pressure of 0.50. The bedrock below the very loose sand is estimated to have a bulk density of 120 pcf and coefficients of active and passive earth pressure of 0.4 and 2.5 respectively.

The frictional resistance on the sheeting may be calculated by multiplying the effective lateral pressures from the soil and bedrock on the piles by: 0.25 for the very loose sand; 0.3 for the uncompacted dredgings and the bedrock; and 0.45 for the compacted dredgings.

A factor of safety of at least 2 should be used with the resisting parameters presented above.

Resistance to sliding of the sheet pile cells will be provided by the passive resistance on the upload side of the cells less the active loads on the download side plus friction across the base of the cells. The coefficient of net (passive – active) earth pressure of the bedrock may be taken as 1.9. The friction on the base can be estimated to be the effective dead load pressures less any uplift pressures from the live loading on the cell times 0.45. In the sliding analysis, a factor of safety of 1.5 should be used with the resisting parameters.

The overturning impact from the mooring loads should be such that the resultant eccentricity of the load on the base is not more than one sixth of the diameter of the cell from its centre at any time.

### 7.4 Bridge Piers

The pedestrian bridges from the existing piers to the mooring dolphins will be supported in the middle by piers set on four vertical or near vertical 20 inch diameter by 0.625 inch wall thickness open ended pipe piles. The required bearing capacity of each pile is 35 kips. The total required lateral resistance is 46 kips or 11.5 kips per pile. The piles will derive their
lateral support from the adjacent bedrock. Consequently an analysis of their lateral loading properties was carried out by the same methods as for the 36 inch diameter piles and with the same general assumptions. These are shown in Table 6 and Appendix B. Appendix B shows the impact of installing the 20 inch diameter piles to varying penetrations below the harbour bottom. This type of analysis was also undertaken for the preliminary design of the 36 inch diameter piles before “final” structural design information was available.

The elevations, to which the analyses in Appendix B indicate that the piles should be driven to obtain adequate lateral support, are -70 feet at the West Bridge Piers and -65 feet at the East Bridge Pier. If the required single pile lateral resistance is multiplied by 1.5 to account for group action, the applicable resistance in the single pile analysis is 17.25 kips. As this multiplication factor was determined for a four by four pile group, it is conservative. The analyses indicate that, at the lateral design load, the maximum lateral deflections, including those in the bedrock, and the maximum moments are at the top of the piles and are respectively:

- 9.3 inches and 6,400 kips-in at the bridge piers to the West Dolphins; and
- 4.6 inches and 5,000 kips-in at the bridge pier to the East Dolphin.

For piles driven to the minimum penetration, at which both the required bearing capacity and lateral stability are achieved, the lateral deflection at the design dredge grades is 1.3 inches at both the West and East Dolphins.

Based on the data in Table 6 and a factor of safety of three, the calculated allowable frictional capacity of the piles is 11.5 and 17.5 kips at the highest elevations required for lateral support at the west and east bridge piers respectively. Using an ultimate end bearing stress of 3 ksi and a factor of safety of three, the calculated allowable end bearing load of the cores from the upper portions of the corable bedrock above El. -86 feet bedrock is approximately 40 kips on 20 inch diameter by 0.625 inch wall thickness open ended pipe piles. However, this bedrock contains extremely weak zones, in which the achievable bearing stress is only approximately one tenth of the more competent bedrock at an estimated 300 psi. Consequently, the allowable end bearing load for these piles is in the order of 4 kips. The bedrock, which could not be cored above these levels, has a similar or lower end bearing capacity.

Based on the theoretical bearing capacity calculations, the piles for the bridge piers to the West Dolphins may achieve an allowable bearing capacity of more than 35 kips at El. -70 feet, the elevation needed for lateral support, as long as the piles are supported by the more competent bedrock at this level. However, those at the east bridge pier would have to be driven at least 5 feet into the corable bedrock below El. -74 feet to achieve an adequate capacity. If the extremely weak zones result in reduced end bearing for the piles, they may have to be driven to below approximate El. -86 feet to achieve an allowable capacity of 35 kips.
Consequently, it is suggested that the piles be driven into the more competent bedrock below El. -86 feet to achieve the required capacity. They should not be driven deeper than El. -90 feet, as below this level there is a danger that they might drive through the bearing stratum.

High strain dynamic testing with a Pile Driving Analyzer or similar equipment and associated wave equation analysis based on its data should be carried out to verify the bearing capacity. Any wave equation analysis using the data from the high strain dynamic testing should take into account only end bearing on the steel area of the piles, i.e., it should not assume any bearing resistance from soil or bedrock plugs, if they occur during the pile driving, as these may relax with time. The high strain dynamic testing should also establish the required penetration resistance to achieve the design working load with a factor of safety of 2.5.

It should be possible to drive the 20 inch diameter by 0.625 inch wall thickness open ended pipe piles to the planned levels by using a pile driving hammer capable of developing 30 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 Table 6. WEAP analysis of a number of Delmag hammers has indicated that a Delmag D 19-42 with a rated energy of 43.2 foot kips per blow is just suitable for this purpose. A Delmag D22-23 with a rated energy of 51.3 foot kips per blow would provide greater flexibility and is unlikely to overstress the piles, as long as it is properly operated.

The actual hammer used for pile driving should have a similar or greater rated energy than the D 19-42, depending on its efficiency, which is defined as the delivered energy as a percentage of the rated energy, of the hammer, 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 this analysis, it may be appropriate to specify that the rated energy of the pile driving hammer be at least 50 foot kips, even though a smaller hammer could theoretically be used.

The penetration resistance required to achieve a design working load of 35 kips with the Delmag D 22-23 hammer has been assessed by the WEAP analysis and is estimated to range from 6 blows per foot with a piston stroke of 11.5 feet to 8 blows per foot with a piston stroke of 7.5 feet, as long as a soil or bedrock plug does not form. If one does form, the piles should be driven to a penetration resistance of at least 20 blows per inch. The actual required penetration resistance should be determined from high strain dynamic testing and associated wave equation analysis. It should be recorded for three consecutive inches of penetration showing increasing resistances. These resistances should be achieved in the bearing stratum between Els. -86 and -90 feet. If the piles drive to El. -90 feet without showing the required penetration resistance, Trow should be consulted about the most suitable procedures to ensure that they have adequate capacity.
An auger of effectively the same diameter as the inside of the piles will be required to remove the material above 10 feet above their bottoms. If there is difficulty driving the piles to El. -86 feet, this auger can be used to auger out any soil and bedrock plug and loosen the bedrock ahead of the pile. However, it should not be used below El. 86 feet.

Based on the depths required for lateral support, the piles should be augered out to Els. -60 and -55 feet at the West and East Dolphin bridge piers respectively. Once the piles have been augered out to these levels, the loose material remaining in them should be cleaned out until water flushing the piles under high pressure flows clear. The concrete should have at least a seven inch slump and should be placed by tremie methods.

7.5 Other Comments

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.

Technical Director President
Geotechnical Engineering 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, siditic etc. should be used.

**ADDITIONAL DESCRIPTIVE TERMS BASED ON ORIGIN OF CONSTITUENT PARTICLES**

<table>
<thead>
<tr>
<th>Degree of Induration</th>
<th>Approximate Unconfined Compressive Strength</th>
<th>NOT DISCERNIBLE</th>
<th>BIOCLASTIC (Organic)</th>
<th>OOLITE (Inorganic)</th>
<th>SHELL (Organic)</th>
<th>CORAL (Organic)</th>
<th>ALGAL (Organic)</th>
<th>PISOLITES (Inorganic)</th>
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<tr>
<td></td>
<td></td>
<td>CARBONATE MUD</td>
<td>CARBONATE SILT</td>
<td>CARBONATE SAND</td>
<td>CARBONATE GRAVEL</td>
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<tr>
<td>Non-indurated</td>
<td>Very soft to hard (1 to &lt;36 kN/m²)</td>
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<td>Mixed carbonate and non-carbonate GRAVEL</td>
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<td>Slightly indurated</td>
<td>Hard to moderately weak (12.5 to 125 MN/m²)</td>
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<td></td>
<td>Silica SAND (calcareous silica SAND)</td>
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<td></td>
<td>Calcarenite (calcareous sandstone)</td>
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<td></td>
<td>Calcirudite (calcareous conglomerate or breccia)</td>
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<tr>
<td>Moderately indurated</td>
<td>Moderately strong to strong (100 to 700 MN/m²)</td>
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<td>Fine-grained LIMESTONE or detrital LIMESTONE</td>
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<td>Conglomerate LIMESTONE</td>
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<td>Highly indurated</td>
<td>Very strong to extremely strong (700 to &gt;2000 MN/m²)</td>
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<td></td>
<td>Crystalline LIMESTONE or MARBLE (tends towards uniformity of grain size and loss of original texture)</td>
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Conventional metamorphic nomenclature applies in this section.
Table 2: Classification of Rock with Regard to Strength

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

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

<table>
<thead>
<tr>
<th>SPACING CLASSIFICATION</th>
<th>SPACING WIDTH (m)</th>
<th>SPACING WIDTH (inch)</th>
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</thead>
<tbody>
<tr>
<td>Extremely close</td>
<td>&lt; 0.02</td>
<td>&lt; 0.8</td>
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<tr>
<td>Very close</td>
<td>0.02 - 0.06</td>
<td>0.8 - 2.4</td>
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<td>Close</td>
<td>0.06 - 0.20</td>
<td>2.4 - 8</td>
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<tr>
<td>Moderately close</td>
<td>0.2 - 0.6</td>
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<tr>
<td>Wide</td>
<td>0.6 - 2.0</td>
<td>24 - 80</td>
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<tr>
<td>Very wide</td>
<td>2.0 - 6.0</td>
<td>80 - 240</td>
</tr>
<tr>
<td>Extremely wide</td>
<td>&gt; 6</td>
<td>&gt; 240</td>
</tr>
</tbody>
</table>
### Table 4: Classification of Rock with Regard to RQD-Value

<table>
<thead>
<tr>
<th>RQD CLASSIFICATION</th>
<th>RQD-VALUE (%)</th>
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<tbody>
<tr>
<td>Very poor quality</td>
<td>&lt; 25</td>
</tr>
<tr>
<td>Poor quality</td>
<td>25 - 50</td>
</tr>
<tr>
<td>Fair quality</td>
<td>50 - 75</td>
</tr>
<tr>
<td>Good quality</td>
<td>75 - 90</td>
</tr>
<tr>
<td>Excellent Quality</td>
<td>90 - 100</td>
</tr>
</tbody>
</table>
## Table 5: Bedrock Core Test Results

<table>
<thead>
<tr>
<th>Sample Number</th>
<th>Elevation (ft)</th>
<th>Unit Weight (pcf)</th>
<th>Moisture Content (%)</th>
<th>Point Load Test Results (kPa)</th>
<th>Unconfined Compression Strength (U/C) (tsf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(ft)</td>
<td></td>
<td></td>
<td>U/A</td>
<td>U/L</td>
</tr>
<tr>
<td>BH 2-1</td>
<td>-74.9</td>
<td>127.5</td>
<td>8</td>
<td>430</td>
<td>985</td>
</tr>
<tr>
<td>BH 2-2</td>
<td>-88.9</td>
<td>136.2</td>
<td></td>
<td>430</td>
<td>985</td>
</tr>
<tr>
<td>BH 2-3</td>
<td>-95.8</td>
<td>157.7</td>
<td>1.5</td>
<td>430</td>
<td>985</td>
</tr>
<tr>
<td>BH 2-4</td>
<td>-98.6</td>
<td>142.8</td>
<td></td>
<td>430</td>
<td>985</td>
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<tr>
<td>BH 3-1</td>
<td>-16.4</td>
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<td>14.1</td>
<td>219</td>
<td>239</td>
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<td>-24.8</td>
<td>113.7</td>
<td></td>
<td>430</td>
<td>985</td>
</tr>
<tr>
<td>BH 4-1</td>
<td>-72.1</td>
<td>133.9</td>
<td>7.2</td>
<td>1108</td>
<td>0</td>
</tr>
<tr>
<td>BH 4-2</td>
<td>-87.6</td>
<td>135.7</td>
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<td>430</td>
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<td>BH 4-3</td>
<td>-92.1</td>
<td>155.6</td>
<td>1.4</td>
<td>4262</td>
<td>2186</td>
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<td>-93.8</td>
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<td>4368</td>
<td>2306</td>
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<tr>
<td>BH 5-1</td>
<td>-68.1</td>
<td>121.4</td>
<td>8.2</td>
<td>75</td>
<td></td>
</tr>
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<td>BH 5-2</td>
<td>-130.2</td>
<td></td>
<td></td>
<td>194</td>
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</tr>
<tr>
<td>BH 5-3</td>
<td>-90.8</td>
<td>134.3</td>
<td>2.1</td>
<td>1119</td>
<td>2548</td>
</tr>
<tr>
<td>BH 6-1</td>
<td>-20.9</td>
<td>111.4</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>BH 6-2</td>
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<td>111.9</td>
<td>2.9</td>
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<td>BH 6-3</td>
<td>-22.9</td>
<td>114.3</td>
<td>209</td>
<td>21</td>
<td>23</td>
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<td>BH 9-1</td>
<td>-23.1</td>
<td>105</td>
<td></td>
<td>575</td>
<td>176</td>
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<td>BH 9-2</td>
<td>-30.1</td>
<td>108.5</td>
<td>13.1</td>
<td>37</td>
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</tr>
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<td>BH 10-1</td>
<td>-21.2</td>
<td>132.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH 10-2</td>
<td>-22.3</td>
<td>105.5</td>
<td>13.8</td>
<td></td>
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</tr>
<tr>
<td>BH 11-1</td>
<td>-20.8</td>
<td>133.3</td>
<td></td>
<td>408</td>
<td>343</td>
</tr>
<tr>
<td>BH 11-2</td>
<td>-35.3</td>
<td>120.7</td>
<td>7.2</td>
<td>2401</td>
<td>1163</td>
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<tr>
<td>BH 11-3</td>
<td>-38.1</td>
<td>122.7</td>
<td></td>
<td>834</td>
<td>1102</td>
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<tr>
<td>BH 12-1</td>
<td>-29.6</td>
<td>122.4</td>
<td>11.1</td>
<td></td>
<td></td>
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<tr>
<td>BH 12-2</td>
<td>-95</td>
<td>74.1</td>
<td></td>
<td>1478</td>
<td>155</td>
</tr>
<tr>
<td>BH 13-1</td>
<td>-27.1</td>
<td>131.4</td>
<td>12.1</td>
<td>833</td>
<td></td>
</tr>
<tr>
<td>BH 13-2</td>
<td>-35.4</td>
<td>118.9</td>
<td></td>
<td>41</td>
<td></td>
</tr>
<tr>
<td>BH 13-3</td>
<td>-42.6</td>
<td>121.5</td>
<td>1.4</td>
<td>3845</td>
<td>2818</td>
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<tr>
<td>BH 14-1</td>
<td>-18.7</td>
<td>124.4</td>
<td></td>
<td>394</td>
<td></td>
</tr>
<tr>
<td>BH 14-2</td>
<td>-30</td>
<td>133.1</td>
<td></td>
<td>72</td>
<td></td>
</tr>
<tr>
<td>BH 14-3</td>
<td>-39.2</td>
<td>118</td>
<td></td>
<td>117</td>
<td></td>
</tr>
<tr>
<td>BH 14-4</td>
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<td>107.9</td>
<td>3.6</td>
<td>1119</td>
<td>1773</td>
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<tr>
<td>BH 15-1</td>
<td>-19.6</td>
<td>129.4</td>
<td></td>
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<tr>
<td>BH 15-2</td>
<td>-28.5</td>
<td>121.9</td>
<td>8.2</td>
<td>1153</td>
<td>2153</td>
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<tr>
<td>BH 16-1</td>
<td>-25.5</td>
<td>152.6</td>
<td></td>
<td>1713</td>
<td>420</td>
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<tr>
<td>BH 16-2</td>
<td>-35.2</td>
<td>101.9</td>
<td>7.8</td>
<td>286</td>
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</tr>
<tr>
<td>BH 16-3</td>
<td>-40.3</td>
<td>101.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BH 18-1</td>
<td>-30.8</td>
<td>122.3</td>
<td>11.7</td>
<td>1215</td>
<td>1156</td>
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<tr>
<td>BH 18-2</td>
<td>-34.7</td>
<td>122.3</td>
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<td>375</td>
<td>1156</td>
</tr>
<tr>
<td>BH 18-3</td>
<td>-42.9</td>
<td>1.9</td>
<td></td>
<td>796</td>
<td>1363</td>
</tr>
</tbody>
</table>

**Notes:**
1. Moisture Content is for samples that were soaked for at least 24 hours in artificial saltwater.
2. 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)
   - S/A Soaked in Artificial Seawater for at least 24 hours/Axial (Vertical)
   - S/L Soaked in Artificial Seawater for at least 24 hours/Lateral (Horizontal)
3. Point load test results multiplied by 9 above El. -58 feet and by 12 below El. -58 feet to estimate Unconfined Compression Strength.
4. Unconfined Compression Tests, Unsoaked Point Load Tests and Unit Weight determinations were carried out on the samples as received in the laboratory.
## Table 6. Estimated Geotechnical Parameters
### Mooring Dolphins

<table>
<thead>
<tr>
<th>Description</th>
<th>Elev. From (ft)</th>
<th>Elev. To (ft)</th>
<th>Elev. From (ft)</th>
<th>Elev. To (ft)</th>
<th>φ (°)</th>
<th>c^2 (tsf)</th>
<th>γ (pcf)</th>
<th>k (kci)</th>
<th>β^5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calcareous Limestone, extremely weak with SPTs ranging from 6 to 52 but mainly around 10 blows/ft</td>
<td>-45</td>
<td>-58</td>
<td>-30</td>
<td>-74</td>
<td>25</td>
<td>0</td>
<td>120</td>
<td>0.06</td>
<td>0.3</td>
</tr>
<tr>
<td>Calcareous Limestone, weak (c = approx. 50 tsf) layered with extremely weak layers (SPTs may be 6 to 10 blows/ft)</td>
<td>-58</td>
<td>-86</td>
<td>-74</td>
<td>-86</td>
<td>0</td>
<td>25</td>
<td>125</td>
<td>0.80</td>
<td>0.4 to 0.8</td>
</tr>
<tr>
<td>Calcareous Limestone, weak to strong (c = approx. 200 tsf) with occasional extremely weak layers (recovery over 80% and mostly close to 100%)</td>
<td>-86</td>
<td>-96</td>
<td>-86</td>
<td>-100</td>
<td>0</td>
<td>200</td>
<td>140</td>
<td>0.80</td>
<td>0.4 to 1.2</td>
</tr>
<tr>
<td>Calcareous Limestone, extremely weak</td>
<td>-96</td>
<td>-100</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
1. φ is the angle of internal friction.
2. c is the cohesion.
3. γ is the bulk density.
4. k is the modulus of lateral soil reaction.
5. β is the combined shaft resistance factor.
Drawings
<table>
<thead>
<tr>
<th>Soil Description</th>
<th>ELEV. ft</th>
<th>SPT (N) Value</th>
<th>Undrained Triaxial % Strain at Failure</th>
<th>Natural Moisture Content %</th>
<th>Plastic and Liquid Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>WATER</td>
<td>0.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SAND - grey, fine to coarse grained calcareous sand with shell fragments, loosely packed</td>
<td>15.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BEDROCK (Calcareaus Limestone) - tan, limestone and coarse sand layers with shell fragments, extremely weak</td>
<td>15.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- tan, limestone and coarse to medium sand layers, extremely weak</td>
<td>15.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>END OF BOREHOLE</td>
<td>43.9</td>
<td></td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>
**LOG OF BOREHOLE 2**

**Project No.:** INTL00302101A  
**Project:** Nassau Harbour Port Improvement Project  
**Location:** Nassau Harbour UTM Zone 18R  0264218m east   2776060m north

**Date Drilled:** May 31, 2008  
**Drill Type:** CME55 (Barge Mounted)  
**Datum:** MLWS (ft)

<table>
<thead>
<tr>
<th>ELEV. ft</th>
<th>Soil Description</th>
<th>% Value</th>
<th>Unconfined Compressive Strength (tsf)</th>
<th>Rock Quality Designation (RQD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>WATER</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>21.1</td>
<td>SAND - grey, medium to coarse grained calcareous sand with shell fragments, very loose</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25.1</td>
<td>BEDROCK (Calcereous Limestone) - tan, limestone and coarse sand layers, extremely weak</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>- tan, limestone and coarse to medium sand layers with shell fragments, extremely weak</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Project No. 2 1 of 2 Nassau Harbour Port Improvement Project Sheet No. 8/13/08**
<table>
<thead>
<tr>
<th>Soil Description</th>
<th>ELEV. ft</th>
<th>N Value</th>
<th>Unconfined Compressive Strength (psi)</th>
<th>Rock Quality Designation (RQD)</th>
<th>Rate of Coring (ft/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>- tan, limestone and coarse sand layers, extremely weak</td>
<td>-50.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- tan, limestone and coarse sand layers, extremely weak</td>
<td>-50.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- calcarenite, tan, vuggy, extremely close to close joint spacing, weak with occasional extremely weak zones</td>
<td>13.48</td>
<td>72.69</td>
<td>100 200 400 600 800</td>
<td>RQD = 75%</td>
<td>3.048</td>
</tr>
<tr>
<td>- calcarenite to calcilutite, tan, vuggy, extremely close to close joint spacing, weak with extremely weak zones</td>
<td>13.48</td>
<td>72.69</td>
<td>100 200 400 600 800</td>
<td>RQD = 75%</td>
<td>3.048</td>
</tr>
<tr>
<td>- calcarenite to calcilutite, tan, vuggy, extremely close to moderately close joint spacing, weak with occasional extremely weak zones</td>
<td>13.48</td>
<td>72.69</td>
<td>100 200 400 600 800</td>
<td>RQD = 75%</td>
<td>3.048</td>
</tr>
<tr>
<td>- calcilutite, tan, vuggy zones and shell inclusions, very close to close joint spacing, weak to medium strong</td>
<td>13.48</td>
<td>72.69</td>
<td>100 200 400 600 800</td>
<td>RQD = 75%</td>
<td>3.048</td>
</tr>
<tr>
<td>- calcilutite, tan, vuggy zones and shell inclusions, very close to wide joint spacing, strong with occasional extremely weak zones</td>
<td>13.48</td>
<td>72.69</td>
<td>100 200 400 600 800</td>
<td>RQD = 75%</td>
<td>3.048</td>
</tr>
<tr>
<td>- calcilutite, tan, vuggy zones and shell inclusions, close to wide joint spacing, weak</td>
<td>13.48</td>
<td>72.69</td>
<td>100 200 400 600 800</td>
<td>RQD = 75%</td>
<td>3.048</td>
</tr>
</tbody>
</table>

END OF BOREHOLE
LOG OF BOREHOLE 4

Date Drilled: June 9, 2008
Drill Type: CME55 (Barge Mounted)
Datum: MLWS (ft)

WATER -
-41.0

SAND - grey, medium to coarse grained calcareous sand with shell fragments, very loose
-45.0

BEDROCK (Calcareous Limestone) - tan, limestone and coarse to fine sand layers with shell fragments

PROJECT:
INTL00302101A

Drawing No. 5
Sheet No. 1 of 2

Trow
<table>
<thead>
<tr>
<th>ELEV. ft</th>
<th>Soil Description</th>
<th>N Value</th>
<th>Rock Quality Designation (RQD)</th>
<th>Rate of Coring (ft/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-50.00</td>
<td>calcarenite, tan, vuggy and shell inclusions, very close to extremely close joint spacing, weak with occasional extremely weak zones</td>
<td></td>
<td>RQD = 76%</td>
<td>1.266</td>
</tr>
<tr>
<td></td>
<td>calcarenite, tan, vuggy and shell inclusions, extremely close to moderately close joint spacing, weak and extremely weak layers</td>
<td></td>
<td>RQD = 88%</td>
<td>2.292</td>
</tr>
<tr>
<td></td>
<td>calcarenite, tan, vuggy and shell inclusions, extremely close to moderately close joint spacing, weak with occasional extremely weak zones</td>
<td></td>
<td>RQD = 57%</td>
<td>1.002</td>
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<tr>
<td></td>
<td>calcilutite, tan, vuggy and shell inclusions, generally moderately close joint spacing, weak to medium strong</td>
<td></td>
<td>RQD = 52%</td>
<td>0.852</td>
</tr>
<tr>
<td></td>
<td>calcilutite, tan, vuggy and shell inclusions, extremely close to moderately close joint spacing, weak with occasional extremely weak zones</td>
<td></td>
<td>RQD = 63%</td>
<td>1.134</td>
</tr>
<tr>
<td></td>
<td>calcarenite, tan, vuggy, extremely close to very close joint spacing, weak and extremely weak layers</td>
<td></td>
<td>RQD = 59%</td>
<td>1.296</td>
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<td>calcarenite to calcilutite, tan, vuggy and shell inclusions, very close to moderately close joint spacing, weak</td>
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<td>RQD = 67%</td>
<td>0.834</td>
</tr>
<tr>
<td></td>
<td>calcilutite to calcilutite, tan, vuggy zones and shell inclusions, moderately close joint spacing, medium strong to strong</td>
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<td>RQD = 71%</td>
<td>0.702</td>
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<tr>
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<td>calcilutite to calcilutite, tan, vuggy, very close to close joint spacing, medium strong to strong with occasional extremely weak zones</td>
<td></td>
<td>RQD = 83%</td>
<td>1.008</td>
</tr>
</tbody>
</table>

**END OF BOREHOLE**

**NOTES:**

**Soil Description**

- **Atterberg Limits (% Dry Weight)**
  - **G**
  - **W**
  - **L**
  - **S**
  - **Y**
  - **M**
  - **B**
  - **O**
  - **D**
  - **E**
  - **P**
  - **T**

**Unconfined Compressive Strength (tsf)**

- **ELEV. ft**
- **ft**

**Natural Moisture Content %**

- **Rate of Coring (ft/min)**
- **tsf**

**Rec**

- **50**
- **100**
- **200**

**Project No.**

- **22**

**Project:**

- Nassau Harbour Port Improvement Project

**Drawing No.:**

- 5

**Sheet No.:**

- 2 of 2
**LOG OF BOREHOLE 5**

**Project No.:** INTL00302101A  
**Project:** Nassau Harbour Port Improvement Project  
**Location:** Nassau Harbour UTM Zone 18R 0263643m east  2776027m north

**Date Drilled:** June 5, 2008  
**Drill Type:** CME55 (Barge Mounted)  
**Datum:** MLWS (ft)

<table>
<thead>
<tr>
<th>Soil Description</th>
<th>ELEV. ft</th>
<th>% Value</th>
<th>Unconfined Compressive Strength</th>
<th>Rate of Coring (ft/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WATER</td>
<td>0.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SAND - grey, medium to coarse grained calcareous sand with shell fragments, very loose</td>
<td>41.8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BEDROCK (Calcareous Limestone) - tan, limestone and coarse sand layers, extremely weak</td>
<td>42.2</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Auger Sample**
- Natural Moisture
- Plastic and Liquid Limit
- Undrained Triaxial at % Strain at Failure
- Penetrometer

**Rock Quality Designation (RQD)**
- U/L
- S/L
- S/A
- M
- P
- L

**Unconfined Compression**
- N Value

**Point Load Test**
- U/A

**Dynamic Cone Test**

**Undrained Triaxial**

**Atterberg Limits (% Dry Weight)**
- G
- W
- L
- S
- Y
- M
- B
- O
- D
- P
- T
- H

**Unconfined Compressive Strength (tsf)**

**ELEV. ft**

**Rate of Coring (ft/min)**

---

Continued Next Page

---

Trow
### Soil Description

- Calcarenite, tan, vuggy, extremely close to wide joint spacing, weak to medium strong with occasional extremely weak zones.

- Calcarenite, tan, vuggy, very close to wide joint spacing, weak to medium strong with occasional extremely weak zones.

- Calcarenite to calcilutite, tan, vuggy and shell inclusions, close to moderately close joint spacing, weak to medium strong.

- Calcarenite, tan, vuggy and shell inclusions, very close to wide joint spacing, weak to medium strong with occasional extremely weak zones.

- No recovery, extremely weak.

### Rock Quality Designation (RQD)

- RQD = 40%
- RQD = 25%
- RQD = 64%
- RQD = 90%
- RQD = 56%
- RQD = 52%
- RQD = 96%
- RQD = 76%
- RQD = 0%

### Rate of Coring (ft/min)

- 1.788
- 2.292
- 1.620
- 1.356
- 1.344
- 1.200
- 1.764
- 3.528

### Unconfined Compressive Strength

- 50 tsf

### Atterberg Limits (% Dry Weight)

- GWC
- LW
- SW
- ML
- OL
- LL
- PL

### Natural Moisture Content %

- 10
- 20
- 30
- 40
- 60
- 80
- 100

### Project:

- Nassau Harbour Port Improvement Project

### Drawing No.

- 6

### Sheet No.

- 2

### Project No.

- 22 of Nassau Harbour Port Improvement Project Sheet No.

---

**LOG OF BOREHOLE 5**
**LOG OF BOREHOLE 6**

**Project No.:** INTL00302101A  
**Project:** Nassau Harbour Port Improvement Project  
**Location:** Nassau Harbour UTM Zone 18R 0263587m east 2775859m north

**Date Drilled:** May 28, 2008  
**Drill Type:** CME55 (Barge Mounted)  
**Datum:** MLWS (ft)

---

### Soil Description

<table>
<thead>
<tr>
<th>Soil Description</th>
<th>ELEV. ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>WATER</td>
<td>0.00</td>
</tr>
<tr>
<td>BEDROCK (Calcareous Limestone)</td>
<td>- tan, limestone and fine to coarse sand layers with shell fragments, extremely weak</td>
</tr>
<tr>
<td>END OF BOREHOLE</td>
<td>-43.6</td>
</tr>
</tbody>
</table>

### Water Levels

<table>
<thead>
<tr>
<th>G.W.L.</th>
<th>S.M.B.L.</th>
</tr>
</thead>
<tbody>
<tr>
<td>WATER</td>
<td>0.00</td>
</tr>
</tbody>
</table>

### Test Results

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Auger Sample</td>
<td></td>
</tr>
<tr>
<td>SPT (N) Value</td>
<td></td>
</tr>
<tr>
<td>Dynamic Cone Test</td>
<td></td>
</tr>
<tr>
<td>Point Load Test</td>
<td></td>
</tr>
<tr>
<td>Unconfined Compression</td>
<td></td>
</tr>
<tr>
<td>Natural Moisture</td>
<td></td>
</tr>
<tr>
<td>Plastic and Liquid Limit</td>
<td></td>
</tr>
<tr>
<td>Undrained Triaxial at % Strain at Failure Penetrometer</td>
<td></td>
</tr>
</tbody>
</table>

### Rock Quality Designation (RQD)

- Unconfined Compressive Strength (tsf)
- Natural Moisture Content (% Dry Weight)
- Atterberg Limits (% Dry Weight)
- Other Test Results

### Project Details

- **Project:** Nassau Harbour Port Improvement Project
- **Drawing No.:** 7
- **Sheet No.:** 1
- **Location:** Nassau Harbour UTM Zone 18R 0263587m east 2775859m north
- **Date Drilled:** May 28, 2008
- **Drill Type:** CME55 (Barge Mounted)
- **Datum:** MLWS (ft)
<table>
<thead>
<tr>
<th>Soil Description</th>
<th>ELEV. ft</th>
<th>% Value</th>
<th>Unconfined Compressive Strength (psi)</th>
<th>Rock Quality Designation (RQD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WATER</td>
<td>0.00</td>
<td>D</td>
<td>20 40 60 80</td>
<td></td>
</tr>
<tr>
<td>SAND</td>
<td>-10.9</td>
<td>D</td>
<td>200 400</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-24.9</td>
<td>D</td>
<td>10 20 30</td>
<td></td>
</tr>
<tr>
<td>BEDROCK (Calcereous Limestone)</td>
<td>-44.9</td>
<td>D</td>
<td>50 100</td>
<td></td>
</tr>
</tbody>
</table>

- WATER: WATER appears to be the first soil layer at 0.00 ft, indicating the presence of water or a water-bearing layer.
- SAND: SAND is described as grey becoming tan, fine to coarse grained calcareous sand, very loose to loose. The layer starts at -10.9 ft and continues downwards.
- BEDROCK (Calcereous Limestone): BEDROCK is characterized as tan, limestone and coarse sand layers with shell fragments, extremely weak. This bedrock layer begins at -24.9 ft and is the last soil layer noted.

END OF BOREHOLE is marked at -44.9 ft, indicating the deepest point reached during the drilling process. The location is designated as Nassau Harbour UTM Zone 18R 0263183m east 2775953m north.
## LOG OF BOREHOLE 8

**Project No.** INTL00302101A  
**Project:** Nassau Harbour Port Improvement Project  
**Location:** Nassau Harbour UTM Zone 18R 0263000m east 2775995m north

**Date Drilled:** May 26, 2008  
**Drill Type:** CME55 (Barge Mounted)  
**Datum:** MLWS (ft)

<table>
<thead>
<tr>
<th>Soil Description</th>
<th>ELEV.</th>
<th>% Value</th>
<th>Rock Quality Designation (RQD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WATER</td>
<td>0.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SAND - tan, fine to medium grained</td>
<td>-8.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>calcareous sand, very loose to loose</td>
<td>-15.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BEDROCK (Calcareous Limestone)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- tan, limestone and coarse sand</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>layers, extremely weak</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- calciudite, tan, very close</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>to close joint spacing, very weak</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>and extremely weak layers</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- calciudite, tan, extremely close</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>to close joint spacing, very weak</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- calciudite, tan, very close</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>to close joint spacing, very weak</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- calciudite, tan, extremely close</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>to close joint spacing, very weak</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- calciudite, tan, with shell</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>inclusions, extremely close</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>to very close joint spacing, very weak</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>and extremely weak zones</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- calciudite, tan, extremely close</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>to close joint spacing, extremely weak</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>with occasional very weak zones</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>END OF BOREHOLE</td>
<td>40.9</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Rate of Coring (ft/min):**  
- 3.750  
- 1.362  
- 1.764  
- 1.500  
- 1.638

**Penetrometer:**  
- 21  
- 2  
- 2  
- 5  
- 28

**Drill Sample:**  
- 8 inch

**Other Tests:**  
- Auger Sample
- SPT (N) Value
- Dynamic Cone Test
- Point Load Test
- Unconfined Compression
- % Strain at Failure
- Penetrometer

**Project No.:** 1 of Nassau Harbour Port Improvement Project  
**Sheet No.:** 9 of 1

**Location:**  
- Nassau Harbour UTM Zone 18R 0263000m east 2775995m north

**Datum:** MLWS (ft)

**Rate of Coring (ft/min):**  
- 3.750  
- 1.362  
- 1.764  
- 1.500  
- 1.638

**Penetrometer:**  
- 21  
- 2  
- 2  
- 5  
- 28

**Drill Sample:**  
- 8 inch

**Other Tests:**  
- Auger Sample
- SPT (N) Value
- Dynamic Cone Test
- Point Load Test
- Unconfined Compression
- % Strain at Failure
- Penetrometer

**Project No.:** 1 of Nassau Harbour Port Improvement Project  
**Sheet No.:** 9 of 1
<table>
<thead>
<tr>
<th>ELEV. ft</th>
<th>Soil Description</th>
<th>Unconfined Compressive Strength (tsf)</th>
<th>Natural Moisture Content %</th>
<th>Plastic and Liquid Limit</th>
<th>Undrained Triaxial at % Strain at Failure</th>
<th>Penetrometer</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>WATER</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18.6</td>
<td>SAND - tan, fine to medium grained calcareous sand,</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>loose</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20.6</td>
<td>BEDROCK (Calcereous Limestone) - tan, limestone</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>medium to coarse sand, extremely weak</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>- calcarenite, tan, vuggy and shell inclusions,</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>extremely close to close joint spacing, very weak</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>with occasional extremely weak zones</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>- no recovery, extremely weak</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>49.6</td>
<td>- tan, limestone and sand layers, extremely</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>weak</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>- no recovery, extremely weak</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>- tan, limestone and sand layers, extremely</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>weak</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## LOG OF BOREHOLE 9

**Project No.** INTL00302101A  |  **Drawing No.** 10  
**Project:** Nassau Harbour Port Improvement Project  |  **Sheet No.** 2 of 2

<table>
<thead>
<tr>
<th>Soil Description</th>
<th>ELEV. ft</th>
<th>DIA.</th>
<th>N Value</th>
<th>Rock Quality Designation (RQD)</th>
<th>Natural Moisture Content %</th>
<th>Atterberg Limits (% Dry Weight)</th>
<th>Rate of Coring (ft/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>END OF BOREHOLE</td>
<td>-50.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## LOG OF BOREHOLE 10

**Project No.:** INTL00302101A  
**Project:** Nassau Harbour Port Improvement Project  
**Location:** Nassau Harbour UTM Zone 18R 0262763m east 2776135m north

### Date Drilled:
May 25, 2008

### Drill Type:
CME55 (Barge Mounted)

### Datum:
MLWS (ft)

### Soil Description

<table>
<thead>
<tr>
<th>ELEV. ft</th>
<th>WATER</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SAND - grey fine to medium grained calcareous sand, very loose</td>
</tr>
<tr>
<td></td>
<td>- tan, fine to medium grained calcareous sand, loose</td>
</tr>
<tr>
<td></td>
<td>BEDROCK (Calcareous Limestone)</td>
</tr>
<tr>
<td></td>
<td>- calcarenite, tan, very close to close joint spacing, weak with occasional extremely weak zones</td>
</tr>
<tr>
<td></td>
<td>- calcirudite to calcarenite, tan, with shell inclusions, very close moderately close joint spacing, weak to very weak with occasional extremely weak zones</td>
</tr>
<tr>
<td></td>
<td>- calcarenite, tan, very close to close joint spacing, extremely weak with very weak zones</td>
</tr>
<tr>
<td></td>
<td>- no recovery, extremely weak</td>
</tr>
<tr>
<td></td>
<td>- no recovery, extremely weak</td>
</tr>
<tr>
<td></td>
<td>- limestone and fine sand layers, extremely weak</td>
</tr>
<tr>
<td>0.00</td>
<td>END OF BOREHOLE</td>
</tr>
</tbody>
</table>

### Notes:

- 1.200 ft
- 1.362 ft
- 1.674 ft
- 1.302 ft
- 3.948 ft

### Rock Quality Designation (RQD)

- Natural Moisture
- Plastic and Liquid Limit
- Undrained Triaxial at % Strain at Failure
- Penetrometer

### Project No.
1 of

### Sheet No.
1

### Drawing No.
11

### Unconfined Compressive Strength

<table>
<thead>
<tr>
<th>% Value</th>
<th>20</th>
<th>40</th>
<th>60</th>
<th>80</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined Compressive Strength</td>
<td>200</td>
<td>400</td>
<td>600</td>
<td>800</td>
<td></td>
</tr>
</tbody>
</table>

### Rock Quality Designation (RQD)

<table>
<thead>
<tr>
<th>% Value</th>
<th>50</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural Moisture Content %</td>
<td>10</td>
<td>20</td>
</tr>
</tbody>
</table>

### Atterberg Limits (% Dry Weight)

<table>
<thead>
<tr>
<th>% Value</th>
<th>50</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plastic and Liquid Limit</td>
<td>20</td>
<td>40</td>
</tr>
</tbody>
</table>

### Unconfined Compression

<table>
<thead>
<tr>
<th>% Value</th>
<th>50</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undrained Triaxial at Strain at Failure</td>
<td>20</td>
<td>40</td>
</tr>
</tbody>
</table>

### Rate of Coring (R/min)

<table>
<thead>
<tr>
<th>% Value</th>
<th>50</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetrometer</td>
<td>10</td>
<td>20</td>
</tr>
</tbody>
</table>
**LOG OF BOREHOLE 11**

**Project No.** INTL00302101A  
**Location:** Nassau Harbour UTM Zone 18R 0262668m east 2776261m north

**Date Drilled:** May 25, 2008  
**Drill Type:** CME55 (Barge Mounted)  
**Datum:** MLWS (ft)

<table>
<thead>
<tr>
<th>Value</th>
<th>SPT (N) Value</th>
<th>Dynamic Cone Test</th>
<th>Point Load Test</th>
<th>Penetrometer</th>
</tr>
</thead>
<tbody>
<tr>
<td>RQD</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| Value |  
|-------|-------|
| 1.092 | 1.224 |
| 3.258 | 4.164 |
| 64%   | 58%   |
| 23%   | 80%   |
| 7%    | 71%   |
| 0%    | 58%   |
| 18%   | 78%   |

**Soil Description:**
- **WATER**
- **SAND** - tan, fine to medium grained calcareous sand, compact
- **BEDROCK (Calcareous Limestone)** - tan, limestone and medium sand layers, extremely weak
  - calcitite, tan, very close to moderately close joint spacing, weak with extremely weak zones
  - calcitite, tan, with shell inclusions, extremely close to close joint spacing, weak and extremely weak layers
- calcitite to calcarenite, tan, with shell inclusions, extremely close to close joint spacing, weak with occasional extremely weak zones
- calcarenite, tan, with shell inclusions, extremely close joint spacing, extremely weak with occasional weak zones
- calcarenite, tan, vuggy and shell inclusions, extremely close to close joint spacing, weak with extremely weak zones
- calcarenite, tan, vuggy and shell inclusions, extremely close to close joint spacing, weak and extremely weak layers
- calcarenite, tan, vuggy and shell inclusions, extremely close to very close joint spacing, extremely weak

**Natural Moisture Content %**

- **Atterberg Limits (% Dry Weight)**
- **SPT**
- **Dynamic Cone Test**
- **Point Load Test**

**Unconfined Compressive Strength**

- **50, 100**
- **10, 20, 30**
- **% Strain at Failure**

**Rate of Coring (ft/min)**

- **1.092**
- **3.258**
- **4.164**
- **1.224**
- **0.912**
- **1.332**
- **1.578**

Continued Next Page
<table>
<thead>
<tr>
<th>Soil Description</th>
<th>ELEV. ft</th>
<th>Rate of Coring (ft/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>with weak zones</td>
<td>-50.00</td>
<td></td>
</tr>
</tbody>
</table>

**END OF BOREHOLE**

<table>
<thead>
<tr>
<th>N Value</th>
<th>Rock Quality Designation (RQD)</th>
<th>Natural Moisture Content %</th>
<th>Unconfined Compressive Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
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<td>20</td>
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<td>30</td>
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<td>70</td>
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</tr>
<tr>
<td>80</td>
<td>80</td>
<td>80</td>
<td>80</td>
</tr>
</tbody>
</table>

**Project:** Nassau Harbour Port Improvement Project

**Drawing No.:** INTL00302101A

**Sheet No.:** 2 of 2

**Date:** 8/13/08
### LOG OF BOREHOLE 12

**Project No.:** INTL00302101A  
**Drawing No.:** 13  
**Sheet No.:** 1 of 1  

**Project:** Nassau Harbour Port Improvement Project  
**Location:** Nassau Harbour UTM Zone 18R 0263281m east 2776430m north

**Date Drilled:** May 22, 2008  
**Drill Type:** CME55 (Barge Mounted)  
**Datum:** MLWS (ft)

| Depth (ft) | Soil Description | ELEV. ft | % Value | Unconfined Compressive Strength (tsf) | Rock Quality Designation (RQD) | Natural Moisture | Plastic and Liquid Limit | &lt;/br&gt;| Rate of Coring (ft/min) |
|-----------|------------------|---------|---------|--------------------------------------|-------------------------------|-----------------|--------------------------|--------------------------|
| 0.00      | WATER            |         |         |                                      |                               |                 |                          |                          |                        |
| 4.0       | SAND             |         |         |                                      |                               |                 |                          |                          |                        |
| 0.630     | Calcarenite, tan, medium to coarse-grained calcareous sand, loose |       |         |                                      |                               |                 |                          |                          |                        |
| 0.942     | BEDROCK (Calcareous Limestone) |   |         |                                      |                               |                 |                          |                          |                        |
| 0.732     | - calcarenite, tan, very close to moderately close joint spacing, extremely weak |       |         |                                      |                               |                 |                          |                          |                        |
| 0.630     | - calcitrudite to calcarenite, tan, very close to moderately close joint spacing, weak with extremely weak zones |       |         |                                      |                               |                 |                          |                          |                        |
| 0.504     | - calcarenite, tan, vuggy, extremely close to close joint spacing, weak with extremely weak zones |       |         |                                      |                               |                 |                          |                          |                        |
| 0.588     | - calcitrudite, tan, vuggy, extremely close to close joint spacing, extremely weak with occasional weak zones |       |         |                                      |                               |                 |                          |                          |                        |
| 0.504     | - calcarenite to calcitrudite, tan, vuggy, extremely close to close joint spacing, extremely weak and very weak to weak layers |       |         |                                      |                               |                 |                          |                          |                        |

**END OF BOREHOLE**

---

**Trow**
LOG OF BOREHOLE 13

Date Drilled: May 23, 2008
Drill Type: CME55 (Barge Mounted)
Datum: MLWS (ft)

<table>
<thead>
<tr>
<th>Soil Description</th>
<th>ELEV. ft</th>
<th>% Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>WATER</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SAND - tan, fine to medium grained calcareous sand, very loose</td>
<td>6.4</td>
<td></td>
</tr>
<tr>
<td>BEDROCK (Calcareous Limestone)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- tan, limestone and medium to coarse sand, extremely weak</td>
<td>7.9</td>
<td></td>
</tr>
<tr>
<td>- calcirudite to calcarenite, tan, extremely close to close joint spacing, extremely weak with weak zones</td>
<td>14.1</td>
<td></td>
</tr>
<tr>
<td>- calcirudite to calcarenite, tan, close to moderately close joint spacing, weak</td>
<td>23.4</td>
<td></td>
</tr>
<tr>
<td>- calcarenite to calcilutite, tan vuggy, extremely close to close joint spacing, extremely weak with very weak zones</td>
<td>29.7</td>
<td></td>
</tr>
<tr>
<td>- calcarenite, tan, vuggy and shell inclusions, extremely close to close joint spacing, weak and extremely weak layers</td>
<td>35.4</td>
<td></td>
</tr>
<tr>
<td>- calcarenite, tan, extremely close to close joint spacing, medium strong to weak and extremely weak layers</td>
<td>46.4</td>
<td></td>
</tr>
</tbody>
</table>

END OF BOREHOLE

Trow
### Water Sand
- Grey, medium to coarse grained calcareous sand, very loose to loose

### Bedrock (Calcareous Limestone)
- Calcirudite, tan, weak with weak zones
- Calcirudite, tan, very close to moderately close joint spacing, weak
- Calcirudite, tan, extremely close to moderately close joint spacing, weak
- Calcarenite, tan, vuggy, extremely close to close joint spacing, weak with extremely weak zones
- Calcarenite, tan, vuggy and shell inclusions, extremely close to very close joint spacing, extremely weak with weak zones
- Calcarenite, tan, vuggy and shell inclusions, extremely close to close joint spacing, weak with extremely weak zones

### End of Borehole
### LOG OF BOREHOLE 15

**Project No.:** INTL00302101A  
**Project:** Nassau Harbour Port Improvement Project  
**Location:** Nassau Harbour UTM Zone 18R  0262556m east   2775961m north  
**Date Drilled:** May 27, 2008  
**Drill Type:** CME55 (Barge Mounted)  
**Datum:** MLWS (ft)

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<tr>
<th>ELEV.</th>
<th>Soil Description</th>
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<tbody>
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<td>0.00</td>
<td>WATER</td>
</tr>
<tr>
<td>-8.0</td>
<td>WATER</td>
</tr>
<tr>
<td>-39.0</td>
<td>WATER</td>
</tr>
<tr>
<td>-39.0</td>
<td>BEDROCK (Calcereous Limestone)</td>
</tr>
<tr>
<td>-39.0</td>
<td>- tan, limestone and coarse sand layers</td>
</tr>
<tr>
<td>-39.0</td>
<td>- calcirudite, tan, shell inclusions, very close to close joint spacing, weak with extremely weak zones</td>
</tr>
<tr>
<td>-39.0</td>
<td>- calcirudite to calcarenite, tan, with shell inclusions, extremely close to moderately close joint spacing, weak with extremely weak zones</td>
</tr>
<tr>
<td>-39.0</td>
<td>- calcirudite, tan, extremely close to close joint spacing, weak with extremely weak zones</td>
</tr>
<tr>
<td>-39.0</td>
<td>- calcirudite to calcarenite, tan, close to moderately close joint spacing, weak</td>
</tr>
<tr>
<td>-39.0</td>
<td>- calcarenite to calcilutite, tan, vuggy zones, close joint spacing, weak</td>
</tr>
<tr>
<td>-39.0</td>
<td>- calcarenite to calcilutite, tan, vuggy and shell inclusions, very close to close joint spacing, weak</td>
</tr>
<tr>
<td>-39.0</td>
<td>- calcirudite to calcarenite, tan, vuggy and shell inclusions, very close to close joint spacing, weak</td>
</tr>
</tbody>
</table>

**Rate of Coring (ft/min):**

- 2.730
- 3.156
- 3.798
- 1.536
- 2.220
- 1.536

**Auger Sample**

- Natural Moisture
- Plastic and Liquid Limit
- Undrained Triaxial at % Strain at Failure
- Penetrometer

**Rock Quality Designation (RQD):**

- RQD = 12%
- RQD = 14%
- RQD = 61%
- RQD = 51%
- RQD = 61%
- RQD = 12%
- RQD = 21%
- RQD = 12%
- RQD = 100%
- RQD = 100%
- RQD = 100%
- RQD = 100%

**Rock Quality Designation (RQD) (tsf):**

- RQD = 100%
- RQD = 100%
- RQD = 100%
- RQD = 100%
- RQD = 100%
- RQD = 100%
- RQD = 100%
- RQD = 100%
- RQD = 100%
- RQD = 100%
- RQD = 100%
- RQD = 100%
- RQD = 100%

**Rate of Coring (ft/min):**

- 2.730
- 3.156
- 3.798
- 1.536
- 2.220
- 1.536

**Rate of Coring (ft/min):**

- 2.730
- 3.156
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- 2.220
- 1.536

**Rate of Coring (ft/min):**

- 2.730
- 3.156
- 3.798
- 1.536
- 2.220
- 1.536

**Rate of Coring (ft/min):**

- 2.730
- 3.156
- 3.798
- 1.536
- 2.220
- 1.536

**Rate of Coring (ft/min):**

- 2.730
- 3.156
- 3.798
- 1.536
- 2.220
- 1.536
LOG OF BOREHOLE 16

Project No: INTL00302101A
Project: Nassau Harbour Port Improvement Project
Location: Nassau Harbour UTM Zone 18R 0262419m east 2776624m north

Date Drilled: June 2, 2008
Drill Type: CME55 (Barge Mounted)
Datum: MLWS (ft)

WATER
- tan, limestone and fine to coarse sand layers

BEDROCK (Calcareous Limestone)
- tan, limestone and fine to coarse sand layers
- calcarenite, tan, vuggy and shell inclusions, very close to moderately close joint spacing, weak to medium strong with extremely weak zones
- calcarenite, tan, vuggy and shell inclusions, extremely close to close joint spacing, weak to very weak and extremely weak layers
- calcarenite, tan, vuggy, extremely close to moderately close joint spacing, weak to very weak with extremely weak zones
- calcarenite, tan, vuggy and shell inclusions, extremely close to moderately close joint spacing, weak with extremely weak zones
- calcarenite, tan, vuggy and shell inclusions, extremely close to close joint spacing, weak with extremely weak zones

END OF BOREHOLE
WATER

BEDROCK (Calcereous Limestone) - tan, limestone and medium to coarse sand layers with shell fragments, extremely weak

END OF BOREHOLE

Date Drilled: June 2, 2008
Drill Type: CME55 (Barge Mounted)
Datum: MLWS (ft)
WATER

BEDROCK (Calcareous Limestone)
- calcarenite to calcilutite, tan, with shell inclusions, very close to close joint spacing, weak with occasional extremely weak zones
- calcarenite, tan, with shell inclusions, very close to moderately close joint spacing, weak with occasional extremely weak zones
- calcarenite to calcilutite, tan, vuggy and shell inclusions, very close to moderately close joint spacing, weak with occasional extremely weak zones
- calcarenite, tan, vuggy and shell inclusions, very close to moderately close joint spacing, weak
- calcarenite, tan, vuggy and shell inclusions, extremely close to moderately close joint spacing, weak

END OF BOREHOLE
**Grain Size Analysis Test Report**

Sample Test No.: 115892-2

Project No.: lagm00307285a
Project Name: Nassau Harbour Port Improvement Project

**Sample Information**
- Borehole No.: BH 3
- Sample Method: SS
- Sample No.: 2
- Depth: 2 - 4ft
- Sample Description: Dev. Pawaroo
- Sampled By: Dev. Pawaroo
- Sampling Date: 28-May-2008
- Date Received: 27-Jun-2008
- Client Sample ID: 
- Comments: 

**Date Reported:** 14-Jul-2008

<table>
<thead>
<tr>
<th>Sieve Size (mm)</th>
<th>% Passing Sample</th>
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</thead>
<tbody>
<tr>
<td>26.5</td>
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<tr>
<td>22.4</td>
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<tr>
<td>19.0</td>
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<td>16.0</td>
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<tr>
<td>13.2</td>
<td>100.0</td>
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<tr>
<td>4.75</td>
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<tr>
<td>0.150</td>
<td>39.1</td>
</tr>
<tr>
<td>0.075</td>
<td>24.5</td>
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<tr>
<td>0.053</td>
<td>17.7</td>
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Notes: *Out of Specification

---

**UNIFIED SOIL CLASSIFICATION SYSTEM**

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<thead>
<tr>
<th>Clay and Silt Fine</th>
<th>Sand Medium</th>
<th>Gravel Coarse</th>
</tr>
</thead>
</table>

- **GRAIN SIZE IN MICROMETERS**
  - 0.001
  - 0.01
  - 0.1
  - 1
  - 10
  - 100

- **SIEVE DESIGNATION (Imperial)**
  - 0.001
  - 0.01
  - 0.1
  - 1
  - 10
  - 100

**Project Manager:** Mukul (Mickey) Misra  **Approved By:** Original Signed By  **Date Approved:** 14-Jul-2008

Willie Rodych
Sample Test No.: 115893-2
Project No.: JAGM00307285a
Project Name: Nassau Harbour Port Improvement Project

Sample Information
Borehole No.: BH 6
Sample Method: SS
Sample No.: 2
Depth: 4 - 6ft.
Sample Description: Dev. Pawaroo
Sampled By: Dev. Pawaroo
Sampling Date: 28-May-2008
Date Received: 27-Jun-2008
Client Sample ID:
Comments:

Date Reported: 14-Jul-2008

Grain Size Analysis
Test Report

Sieve Size (mm) | % Passing Sample
--- | ---
26.5 | 100.0
22.4 | 100.0
19.0 | 100.0
16.0 | 100.0
13.2 | 100.0
12.7 | 100.0
9.5 | 100.0
6.7 | 100.0
4.75 | 99.5
2.00 | 95.1
0.850 | 77.2
0.425 | 52.6
0.250 | 31.7
0.180 | 24.3
0.150 | 22.2
0.075 | 19.3
0.053 | 18.5

Notes: *Out of Specification

Unified Soil Classification System

Grain Size in Micrometers

Percent Passing

Grain Size (mm)

Project Manager: Mukul (Mickey) Misra
Approved By: Original Signed By
Date Approved: 14-Jul-2008

Willie Rodych
Sample Test No.: 115894-2  
Project No.: lmg00302785a  
Project Name: Nassau Harbour Port Improvement Project

Sample Information
Borehole No.: BH 7  
Sample Method: SS  
Sample No.: 3  
Depth: 4 - 6ft.  
Sample Description:  
Sampled By: Dev. Pawaroo  
Sampling Date: 27-May-2008  
Date Received: 27-Jun-2008  
Client Sample ID:  
Comments:

Date Reported: 14-Jul-2008

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<th>Sieve Size (mm)</th>
<th>% Passing Sample</th>
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<tbody>
<tr>
<td>26.5</td>
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<tr>
<td>22.4</td>
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</tr>
<tr>
<td>19.0</td>
<td>100.0</td>
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<tr>
<td>16.0</td>
<td>100.0</td>
</tr>
<tr>
<td>13.2</td>
<td>100.0</td>
</tr>
<tr>
<td>12.7</td>
<td>100.0</td>
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<td>9.5</td>
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<tr>
<td>0.250</td>
<td>84.2</td>
</tr>
<tr>
<td>0.180</td>
<td>64.3</td>
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<td>0.150</td>
<td>51.3</td>
</tr>
<tr>
<td>0.075</td>
<td>25.7</td>
</tr>
<tr>
<td>0.053</td>
<td>19.8</td>
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Notes: "Out of Specification"

UNIFIED SOIL CLASSIFICATION SYSTEM

GRANITE IN MICROMETERS | SAND | GRAVEL

SIEVE DESIGNATION (Imperial)
1 3 5 10 30 #200 #100 #50 #16 #4 3/8" 1/2" 3/4" 1" 3"
Sample Test No.: 115895-2
Project No.: lagm00307285a
Project Name: Nassau Harbour Port Improvement Project

Sample Information
Borehole No.: BH 8
Sample Method: SS
Sample No.: 3
Depth: 4 - 6 ft.
Sample Description: Dev. Pawaroo
Sampled By: Dev. Pawaroo
Sampling Date: 26-May-2008
Date Received: 27-Jun-2008
Client Sample ID: 
Comments:

---

**Grain Size Analysis Test Report**

**Date Reported:** 14-Jul-2008

<table>
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<th>Sieve Size (mm)</th>
<th>% Passing Sample</th>
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</thead>
<tbody>
<tr>
<td>26.5</td>
<td>100.0</td>
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<td>22.4</td>
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<td>19.0</td>
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<td>16.0</td>
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Notes: *Out of Specification*

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**UNIFIED SOIL CLASSIFICATION SYSTEM**

<table>
<thead>
<tr>
<th>CLAY AND SILT</th>
<th>SAND</th>
<th>GRAVEL</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Fine</td>
<td>Medium</td>
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<td></td>
<td>Fine</td>
<td>Coarse</td>
</tr>
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</table>

**GRAIN SIZE IN MICROMETERS**

1 3 5 10 30 50 #200 #100 #50 #16

**SIEVE DESIGNATION (Imperial)**

3/8" 1/2" 3/4" 1" 3"
Sample Test No.: 115896-2
Project No.: lagm00307285a
Project Name: Nassau Harbour Port Improvement Project

Sample Information
Borehole No.: BH 10
Sample Method: SS
Sample No.: 3
Depth: 4 - 6ft.
Sample Description: Sampled By: Dev. Pawaroo
Sampling Date: 25-May-2008
Date Received: 27-Jun-2008
Client Sample ID:
Comments:

Date Reported: 14-Jul-2008

Sieve Size (mm) | % Passing Sample
--- | ---
26.5 | 100.0
22.4 | 100.0
19.0 | 100.0
16.0 | 100.0
13.2 | 100.0
12.7 | 100.0
9.5 | 100.0
6.7 | 100.0
4.75 | 100.0
2.00 | 98.4
0.850 | 92.7
0.425 | 80.7
0.250 | 60.2
0.180 | 39.5
0.150 | 30.3
0.075 | 15.2
0.053 | 12.0

Notes: *Out of Specification

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT

SAND

GRAVEL

GRAIN SIZE IN MICROMETERS

PERCENT PASSING

GRAIN SIZE (MM)

SIEVE DESIGNATION (Imperial)

Project Manager: Mukul (Mickey) Misra
Approved By: Original Signed By
Date Approved: 14-Jul-2008

Willie Rodych
Appendix A
Photographs of Bedrock Cores
BH4 Box 3

BH5 Box 1
Appendix B
Lateral Loading Analyses
NASSAU
EAST DOLPHIN
NASSAU
EAST DOLPHIN
36 inch PILES

Casing and Reinforced concrete

Only Casing

+3.5'

0

-30'

-60'

-70'
**NASSAU DOLPHIN**

36" Pipe Pile

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<tbody>
<tr>
<td>Modulus of elasticity of steel, $E_s$</td>
<td>30000000 psi</td>
</tr>
<tr>
<td>Modulus of elasticity of concrete, $E_c$</td>
<td>3333333 psi</td>
</tr>
<tr>
<td>External Diameter of pile, $D_e$</td>
<td>36 in</td>
</tr>
<tr>
<td>Thickness of casing of pile, $t$</td>
<td>0.53 in (after corrosion)</td>
</tr>
<tr>
<td>Inner Diameter of steel casing of pile, $D_i$</td>
<td>34.94 in</td>
</tr>
<tr>
<td>Cross sectional area of concrete, $A_c$</td>
<td>958.82 in$^2$</td>
</tr>
<tr>
<td>Cross sectional area of steel, $A_s$</td>
<td>59.06 in$^2$</td>
</tr>
<tr>
<td>Total Area, $A_c + A_s$</td>
<td>1017.88 in$^2$</td>
</tr>
<tr>
<td>Moment of Inertia of steel, $I_s$</td>
<td>9290 in$^4$</td>
</tr>
<tr>
<td>Moment of Inertia of concrete, $I_c$</td>
<td>73158 in$^4$</td>
</tr>
<tr>
<td>$E_s * I_s$</td>
<td>2.787E+11 lb-in$^2$</td>
</tr>
<tr>
<td>$E_c * I_c$</td>
<td>2.439E+11 lb-in$^2$</td>
</tr>
<tr>
<td>$0.35 * E_c * I_c$</td>
<td>8.535E+10 lb-in$^2$</td>
</tr>
<tr>
<td>$(E * I)_{equivalent}$</td>
<td>$E_s * I_s + 0.35 * E_c * I_c = 3.641E+11$ lb-in$^2$</td>
</tr>
</tbody>
</table>
Note: Graphs show pile displacement along embedded section. Deflection of free standing section is not shown. Z-coordinates are measured from the top of the pile.
36" Pile (40' embedment) EAST DOLPHIN

Z Nodal Coordinate, in.

Pile Displacement, in.

Load 1 = 8.0 Kips
Load 2 = 20.0 Kips
Load 3 = 35.0 Kips
Load 4 = 65.0 Kips
Load 5 = 85.0 Kips
Load 6 = 110.0 Kips
MAXIMUM MOMENT VALUES
36" PILES

<table>
<thead>
<tr>
<th>Embedment</th>
<th>#</th>
<th>Load</th>
<th>Max. Moment</th>
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<tr>
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<td></td>
<td>-5995</td>
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<td>3</td>
<td>35</td>
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<tr>
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<td>65</td>
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<td>-28210</td>
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<tr>
<td>6</td>
<td>110</td>
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<td>-37180</td>
</tr>
</tbody>
</table>
NASSAU
EAST DOLPHIN
20" PILES
NASSAU DOLPHIN
20" Pipe Pile

Modulus of elasticity of steel, \( E_s = \) 30000000 psi
Modulus of elasticity of concrete, \( E_c = \) 3333333 psi

External Diameter of pile, \( D_e = \) 20 in
Thickness of casing of pile, \( t = \) 0.625 in
Inner Diameter of steel casing of pile, \( D_i = \) 18.75 in
Cross sectional area of concrete, \( A_c = \) 276.12 \( \text{in}^2 \)
Cross sectional area of steel, \( A_s = \) 38.04 \( \text{in}^2 \)
Total Area, \( A_c + A_s = \) 314.16 \( \text{in}^2 \)

Moment of Inertia of steel, \( I_s = \) 1787 \( \text{in}^4 \)
Moment of Inertia of concrete, \( I_c = \) 6067 \( \text{in}^4 \)

\( E_s * I_s = \) 5.361E+10 lb-in\(^2\)
\( E_c * I_c = \) 2.022E+10 lb-in\(^2\)
\( 0.35 * E_c * I_c = \) 7.078E+09 lb-in\(^2\)

\( (E_s I_s)_{equivalent} = \) \( E_s * I_s + 0.35 * E_c * I_c = \) 6.069E+10 lb-in\(^2\)
Note: Graphs show pile displacement along embedded section. Deflection of free standing section is not shown. Z-coordinates are measured from the top of the pile.
20” Pile (15' Embedment) - EAST DOLPHIN

Z Nodal Coordinate, in.

Load 1 = 4.0 Kips
Load 2 = 10.0 Kips
Load 3 = 20.0 Kips
20" Pile (20' Embedment) - EAST DOLPHIN

Pile Displacement, in.

Z Nodal Coordinate, in.

Load 1 = 4.0 Kips
Load 2 = 10.0 Kips
Load 3 = 20.0 Kips
Load 4 = 35.0 Kips
20" Pile (25' Embedment) - EAST DOLPHIN

Load 1 = 4.0 Kips
Load 2 = 10.0 Kips
Load 3 = 20.0 Kips
Load 4 = 35.0 Kips
Load 5 = 50.0 Kips
Pile Displacement, in.

Z Nodal Coordinate, in.

Load 1 = 4.0 Kips
Load 2 = 10.0 Kips
Load 3 = 20.0 Kips
Load 4 = 35.0 Kips
Load 5 = 50.0 Kips
Load 6 = 65.0 Kips
20" Pile (35' Embedment) - EAST DOLPHIN

- Z Nodal Coordinate, in.
- Pile Displacement, in.

- Load 1 = 4.0 Kips
- Load 2 = 10.0 Kips
- Load 3 = 20.0 Kips
- Load 4 = 35.0 Kips
- Load 5 = 50.0 Kips
- Load 6 = 65.0 Kips
20" Pile (45' Embedment) - EAST DOLPHIN

Z Nodal Coordinate, in.

Pile Displacement, in.

- Load 1 = 4.0 Kips
- Load 2 = 10.0 Kips
- Load 3 = 20.0 Kips
- Load 4 = 35.0 Kips
- Load 5 = 50.0 Kips
- Load 6 = 65.0 Kips
MAXIMUM MOMENT VALUES
EAST DOLPHIN 20" PILES
# EAST DOLPHIN

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NASSAU
WEST DOLPHIN
36 inch PILES

Casing and Reinforced concrete

Only Casing
NASSAU DOLPHIN
36” Pipe Pile

Modulus of elasticity of steel,

\[ E_s = 30000000 \text{ psi} \]

Modulus of elasticity of concrete,

\[ E_c = 3333333 \text{ psi} \]

External Diameter of pile,

\[ D_e = 36 \text{ in} \]

Thickness of casing of pile,

\[ t = 0.53 \text{ in (after corrosion)} \]

Inner Diameter of steel casing of pile,

\[ D_i = 34.94 \text{ in} \]

Cross sectional area of concrete,

\[ A_c = 958.82 \text{ in}^2 \]

Cross sectional area of steel,

\[ A_s = 59.06 \text{ in}^2 \]

Total Area,

\[ A_c + A_s = 1017.88 \text{ in}^2 \]

Moment of Inertia of steel,

\[ I_s = 9290 \text{ in}^4 \]

Moment of Inertia of concrete,

\[ I_c = 73158 \text{ in}^4 \]

\[ E_s \times I_s = 2.787E+11 \text{ lb-in}^2 \]

\[ E_c \times I_c = 2.439E+11 \text{ lb-in}^2 \]

\[ 0.35 \times E_c \times I_c = 8.535E+10 \text{ lb-in}^2 \]

\[ \left( E \times I \right)_{\text{equivalent}} = E_s \times I_s + 0.35 \times E_c \times I_c = 3.641E+11 \text{ lb-in}^2 \]
36" Pile - WEST DOLPHIN

Head Displacement, in.

Loads, Kips.
Note: Graphs show pile displacement along embedded section. Deflection of free standing section is not shown. Z-coordinates are measured from the top of the pile.
36" Pile (35' embedment) - WEST DOLPHIN

Z Nodal Coordinate, in.

Pile Displacement, in.

Load 1 = 3.0 Kips
Load 2 = 8.0 Kips
Load 3 = 15.0 Kips
Load 4 = 25.0 Kips
Load 5 = 40.0 Kips
Load 6 = 65.0 Kips
MAXIMUM MOMENT VALUES
36” PILES

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NASSAU
WEST DOLPHIN
20” PILES

Casing and
Reinforced concrete

Only Casing

+3.5’

-45’

10’
NASSAU DOLPHIN
20" Pipe Pile

Modulus of elasticity of steel,  \( E_s = \) 30000000 psi
Modulus of elasticity of concrete,  \( E_c = \) 3333333 psi

External Diameter of pile,  \( D_e = \) 20 in
Thickness of casing of pile,  \( t = \) 0.625 in
Inner Diameter of steel casing of pile,  \( D_i = \) 18.75 in
Cross sectional area of concrete,  \( A_c = \) 276.12 in\(^2\)
Cross sectional area of steel,  \( A_s = \) 38.04 in\(^2\)
Total Area,  \( A_c + A_s = \) 314.16 in\(^2\)

Moment of Inertia of steel,  \( I_s = \) 1787 in\(^4\)
Moment of Inertia of concrete,  \( I_c = \) 6067 in\(^4\)

\( E_s \times I_s = \) 5.361E+10 lb-in\(^2\)
\( E_c \times I_c = \) 2.022E+10 lb-in\(^2\)
\( 0.35 \times E_c \times I_c = \) 7.078E+09 lb-in\(^2\)

\( (E \times I)_{equivalent} = \)  
\( E_s \times I_s + 0.35 \times E_c \times I_c = \) 6.069E+10 lb-in\(^2\)
Note: Graphs show pile displacement along embedded section. Deflection of free standing section is not shown. Z-coordinates are measured from the top of the pile.
20" Pile (10' Embedment) - WEST DOLPHIN

Z Nodal Coordinate, in.

Load 1 = 4.0 Kips
Load 2 = 10.0 Kips
Load 3 = 20 Kips
20" Pile (15' Embedment) - WEST DOLPHIN

Load 1 = 4.0 Kips
Load 2 = 10.0 Kips
Load 3 = 15.0 Kips
Load 4 = 25.0 Kips
20" Pile (20' Embedment) - WEST DOLPHIN

Z Nodal Coordinate, in.

Load 1 = 4.0 Kips
Load 2 = 10.0 Kips
Load 3 = 15.0 Kips
Load 4 = 25.0 Kips
Load 5 = 35.0 Kips
Load 6 = 45.0 Kips

Pile Displacement, in.
20" Pile (25' Embedment) - WEST DOLPHIN

Load 1 = 4.0 Kips
Load 2 = 10.0 Kips
Load 3 = 15.0 Kips
Load 4 = 25.0 Kips
Load 5 = 35.0 Kips
Load 6 = 45.0 Kips
20" Pile (35' Embedment) - WEST DOLPHIN

Pile Displacement, in.

Z Nodal Coordinate, in.

Load 1 = 4.0 Kips
Load 2 = 10.0 Kips
Load 3 = 15.0 Kips
Load 4 = 25.0 Kips
Load 5 = 35.0 Kips
Load 6 = 45.0 Kips
20" Pile (40' Embedment) - WEST DOLPHIN

Load 1 = 4.0 Kips
Load 2 = 10.0 Kips
Load 3 = 15.0 Kips
Load 4 = 25.0 Kips
Load 5 = 35.0 Kips
Load 6 = 45.0 Kips
20" Pile (45' Embedment) - WEST DOLPHIN

Pile Displacement, in.

Z Nodal Coordinate, in.

Load 1 = 4.0 Kips
Load 2 = 10.0 Kips
Load 3 = 15.0 Kips
Load 4 = 25.0 Kips
Load 5 = 35.0 Kips
Load 6 = 45.0 Kips
MAXIMUM MOMENT VALUES
WEST DOLPHIN 20” PILES
### MAXIMUM MOMENT VALUES

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