Geotechnical Investigation Report

Whitfield Avenue Watermain
Manatee County, Florida

Prepared for: **Cardno TBE**
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Prepared By:
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Project No. T081317.194
April 2014

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GEOTECHNICAL • ENVIRONMENTAL
MATERIALS TESTING

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April 1, 2014

Mr. Dorian Modjeski, PE, BCEE
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Geotechnical Engineering Services Report
Whitfield Avenue Watermain
Manatee County, Florida
MC^{2} Inc. Project No. T081317.194

MC Squared, Inc. (**MC^{2}**) has completed the geotechnical engineering services for the referenced project. This study was performed in general accordance with **MC^{2}** proposal No. T081317.194 dated October 7, 2013. The services were authorized through a subcontract agreement between **MC^{2}** and **Cardno TBE.** The results of this exploration, together with our recommendations, are included in the accompanying report.

Often, because of design and construction details that occur on a project, questions arise concerning subsurface conditions. **MC^{2}** will be pleased to continue our role as geotechnical consultants during the construction phase of this project to provide assistance with construction materials testing and inspection services and to verify that our recommendations are implemented.

We trust that this report will assist you in the design and construction of the proposed project. We appreciate the opportunity to be of service to you on this project. Should you have any questions, please do not hesitate to contact us.

Respectfully submitted,
MC^{2}

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1.0 PROJECT INFORMATION

1.1 PROJECT AUTHORIZATION

Authorization to proceed with this project was issued by **Cardno TBE** thru an agreement for the services and is dated January 30, 2014. A formal contract has been executed between **Cardno TBE** and **MC^2** for these services.

1.2 PROJECT DESCRIPTION

Project information has been provided by Mr. Dorian Modjeski, PE of **Cardno TBE** through verbal and email communications including an aerial photo showing anticipated limits of the project. Based on our understanding, the proposed work includes the design of approximately 2400 LF of a new 16 inch waterline along Whitfield Ave. from 43rd Court East to Tuttle Ave.

Based on our discussions with **Cardno TBE**, we understand that a portion of the pipe along the eastern limits (approximately 1000 ft.) may be installed with horizontal directional drilling (HDD) methods or may be attached to the existing concrete wall adjacent to the roadway in this area. There is an existing box culvert (7) (8' x 3') located a distance ranging from 140 to 193 feet west of the centerline of 43rd Court East.

If any of this project description information is incorrect or has changed, please inform **MC^2** so that we may amend, if appropriate, the recommendations represented in this report.

1.3 SCOPE OF WORK AND SERVICES

Our geotechnical study began with a review of available subsurface test data including the USDA Manatee County Soil Survey and USGS Maps. The testing program consisted of the following services:

1. Conducted a visual reconnaissance of the project site. Reviewed the USDA Soil Survey for Manatee County and the USGS topographic maps.

2. Cleared utilities in the vicinity of the proposed boring locations.

3. Performed Standard Penetration Test (SPT) borings in areas of proposed pipeline as follows:

   - Along the eastern 1000 ft. (conservation area) in areas of proposed HDD, we performed three (3) SPT borings. Two (2) of the borings are proposed to a depth of twenty-five (25) feet and one (1) is proposed at 50 feet. The exact locations of the borings were determined by **Cardno TBE** after approval of the pipeline alignment. We performed borings on either end (25 feet deep) of the conservation area as well as one in the roadway (50 feet deep) along the midpoint of the conservation area. The borings were performed in areas accessible with our track mounted drill rig.

- Along the remainder of the project, we performed two (2) SPT borings to a depth of twenty (20) feet at an approximate spacing of about 600 feet.

4. As requested by **Cardno TBE**, performed two (2) pavement cores (instead of the originally proposed 4) in the existing roadway to evaluate existing pavement structure to assist designers with pavement replacement design. The pavement cores were performed as follows:

   - One (1) along Whitfield Avenue = PC-1
   - One (1) near 43rd Ct. E. = PC-2

5. Visually examined all recovered soil samples in the laboratory and performed laboratory tests on selected representative samples to develop the soil legend for the project using the Unified Soil Classification Systems, as appropriate. The laboratory testing included percent passing the -200 sieve, atterberg limit testing, natural moisture content determination and corrosion parameters tests.

The data was used in performing engineering evaluations, analyses, and for developing geotechnical recommendations in the following areas:

1. General assessment of area geology based on our past experience, study of geological literature and boring information.

2. General location and description of potentially deleterious materials encountered in the borings, which may interfere with the proposed construction or performance, including existing fills or surficial organics.

3. Address groundwater levels in the borings and estimate seasonal high groundwater.

4. Recommendations for construction including a summary of findings and analysis. We will also provide a summary of the pavement cores.

5. Discussed critical design and/or construction considerations based on the soil and groundwater conditions developed from the borings including earthwork recommendations, dewatering, hard soil conditions, need for sheet piles or bracing in open cut areas, potential settlement from sheeting or compaction to above ground structures, etc. We also provided soil design parameters including

estimated soil strength and density parameters, internal friction angles, dry and wet densities, cohesion and earth pressure coefficients (active and passive).

All information was provided in a Geotechnical Investigation Report which generally included the following:

a. Description of the proposed project.
b. Plot showing location of borings performed.
c. Boring logs including water table where encountered.
d. Description of surface and subsurface conditions encountered.
e. Internal friction angles, cohesion.
f. Active, passive and at rest soil pressures.
g. Recommendations for site preparation and engineered fill.
h. Recommendations for temporary sheet pile shoring design (not required).

The remainder of our services did not include an environmental assessment for determining the presence or absence of wetlands or hazardous or toxic materials in the soil, bedrock, groundwater, or air, on or below or around this site. Any statements in this report or on the boring logs regarding odors, colors, unusual or suspicious items or conditions are strictly for the information of our client.

2.0 LABORATORY TESTING

2.1 SOIL CLASSIFICATION TESTING

Representative soil samples collected from the SPT borings were visually reviewed in the laboratory by a geotechnical engineer to confirm the field classifications. The samples were classified and stratified in general accordance with the Unified Soil Classification System. Classification was based on visual observations with the results of the laboratory testing used to confirm the visual classification. Laboratory classification tests consisting of Atterberg limits, percent passing the No. 200 sieve and moisture content determinations were performed on selected soil samples believed to be representative of the materials encountered. A summary of the test results are provided in **Table 2** of our **Appendix.** In addition, limited corrosion tests were performed with the results provided in **Table 3.**

2.2 ATTERBERG LIMITS

The liquid limit and the plastic limit tests ("Atterberg Limits") were conducted in general accordance with the FDOT test designation FM 1-T089 and FM 1-T090, respectively (ASTM test designation D-4318). Atterberg plastic limit and liquid limit tests measure the moisture content at which a fine-grained soil changes from a semi-solid to plastic state and from a plastic to a liquid state, respectively. The plasticity index is the difference between the liquid and plastic limits. The plasticity index is an indication of the tendency of a soil to absorb water on the particle surfaces. Some clays have a strong affinity for water, and tend to swell when wetted and shrink when dried. The larger the plastic index, the greater the shrink-swell tendency.

2.3 PERCENT PASSING THE NO. 200 SIEVE

The wash gradation test measures the percentage of a dry soil sample passing the No. 200 sieve. By definition in the Unified Soil Classification System, the percentage by weight passing the No. 200 sieve is the silt and clay content. The amount of silt and clay in a soil influences its properties, including permeability, workability and suitability as fill. This test was performed in general accordance with ASTM D-1140 (Standard Test Methods for Amount of Material Finer than the No. 200 (75 μm) Sieve).

2.4 MOISTURE CONTENT

The laboratory moisture content test consists of the determination of the percentage of moisture contents in selected samples in general accordance with FDOT test designation FM 1-T265 (ASTM test designation D-2216). Briefly, natural moisture content is determined by weighing a sample of the selected material and then drying it in a warm oven. Care is taken to use a gentle heat so as not to destroy any organics. The sample is then removed from the oven and reweighed. The difference of the two weights is the amount of moisture removed from the sample. The weight of the moisture divided by the weight of the dry soil sample is the percentage by weight of the moisture in the sample.

2.5 ENVIRONMENTAL CORROSION TESTS

Environmental corrosion tests were conducted in accordance with the FDOT test designations FM 5-550, FM 5-551, FM 5-552 and FM 5-553. These tests were performed on one recovered soil sample obtained from the SPT borings performed and one water sample obtained from standing water at approximately station 21+00 45 LT. Environmental corrosion tests measure parameters such as pH, resistivity, sulfate content and chloride content. Test results obtained are presented in **Table 3** in the **Appendix.** Based on the laboratory test results and the FDOT Structures Design Guidelines, the environment of the proposed pipeline alignment soils and water samples have classification ranging from moderately to extremely aggressive for steel and from slightly to moderately aggressive for concrete. We recommend using the FDOT Structures Design Guidelines and FDOT Standard Specifications for corrosion protection measures.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 MANATEE COUNTY SOIL SURVEY

The U.S. Department of Agriculture - Soil Conservation Service now known as Natural Resources Conservation Service (NRCS), has mapped the shallow soils in this area of Manatee County. This information was outlined in a report titled *The Soil Survey of Manatee County, Florida* using Version 9, dated December 19, 2013. The aerial images were photographed between February 10, 2010 and March 18, 2011. The Soil Survey describes the soils at the different intersections as described in **Table 1 in the Appendix,** which also includes the normal high groundwater table. Small areas of other soil types may be present within the mapping unit.

The USDA Soil Survey is not necessarily an exact representation of the soils on the site. The mapping is based on interpretation of aerial maps with scattered shallow borings for confirmation. Accordingly, borders between mapping units are approximate and the change may be transitional. Differences may also occur from the typical stratigraphy, and small areas of other similar and dissimilar soils may occur within the soil mapping unit. As such, there may be differences in the mapped description and the boring descriptions obtained for this report. The survey is, however, a good basis for evaluating the shallow soil conditions of the area.

3.2 SUBSURFACE CONDITIONS

The subsurface conditions were explored using five (5) SPT borings drilled to depths ranging from 20 to 50 feet below ground surface (BGS). The approximate boring locations and the depths were selected by **Cardno TBE** and provided to us. The borings were located in the field by **MC^2** personnel measuring distances from existing site features. The approximate boring locations are presented on **Sheets 1 through 5** in the **Appendix** and the approximate boring elevations obtained from a drawing provided by the client showing spot elevations.

The SPT borings were conducted in general accordance with ASTM D-1586 (Standard Test Method for Penetration Test and Split Barrel Sampling of Soils) using the rotary wash method, where a clay slurry (“drill mud” or “drill fluid”) was used to flush and stabilize the borehole. The initial 4 feet of the borings were advanced with a hand auger to further explore for underground utilities. Then, Standard Penetration sampling was performed at closely spaced intervals in the upper 10 feet and at 5-foot intervals thereafter. After seating the sampler 6 inches into the bottom of the borehole, the number of blows required to drive the sampler one foot further with a standard 140 pound hammer is known as the “N” value or blowcount. The blowcount has been empirically correlated to soil properties. The recovered samples were placed into containers and returned to our office for visual review.
Select soil samples were tested in the laboratory to determine material properties for our

evaluation. Laboratory testing was accomplished in general accordance with ASTM standards. Laboratory test results are shown on the soil profiles and summarized in **Table 2** presented in the **Appendix**.

The surface description discussed below is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The soil profiles included in the **Appendix** should be reviewed for specific information at individual boring locations. These profiles include soil description, stratification, penetration resistances, and laboratory test results. The stratification shown on the boring profiles represents the conditions only at the actual boring location. Variations may occur and should be expected between boring locations.

- **Borings B-1 and B-2 (Western Portion of the Proposed Watermain Alignment to be installed by open-trench methodology)**

  In general, all the borings indicated very loose to dense fine sands and/or slightly silty fine sands and/or slightly clayey fine sand (SP/SP-SM)/SP-SC) to depths ranging from 17 feet to the boring termination depth of 20 feet. These sands occasionally contained traces of shell. Boring B-1 terminated in medium dense fine sands and/or slightly silty fine sands and/or slightly clayey fine sand (SP/SP-SM)/SP-SC). Boring B-2 entered medium dense clayey fine sand (SC) extending to depths ranging from 17 to 20 feet. An isolated thin layer of loose clayey fine sand (SC) was encountered in boring B-1 from depths ranging from 6 to 7 feet.

- **Borings B-3 trough B-5 (Eastern Portion of Proposed Watermain Alignment to be installed by HDD methodology)**

  In general, all the borings indicated very loose to medium dense fine sands and/or slightly silty fine sands and/or slightly clayey fine sand (SP/SP-SM)/SP-SC) to depths ranging from 12 to 17 feet. Next, the borings entered firm to very stiff clay to sandy clay (CL/CH) and/or medium dense clayey fine sand (SC) extending to depths ranging from 17 feet to the boring termination depth of 50 feet. Boring B-5 was terminated in medium dense fine sands and/or slightly silty fine sands and/or slightly clayey fine sand (SP/SP-SM)/SP-SC) extending to the boring termination depth of 25 feet. In addition, boring B-4 encountered an isolated layer of moderately hard highly weathered limestone extending from depths ranging from 37 to 42 feet.

3.3 GROUNDWATER INFORMATION

Due to the use of drilling fluids, groundwater is typically unable to be measured accurately in SPT borings unless encountered at very shallow depths. Generally speaking, groundwater levels tend to fluctuate during periods of prolonged drought and extended rainfall and are affected by man-made influences such as drainage conveyance systems. In addition, a seasonal effect will also occur in which higher groundwater levels are normally recorded in rainy seasons. If the groundwater level is critical to design or construction, temporary observation wells should be installed along the alignment in order to monitor groundwater fluctuations over a period of time and permit more accurate determinations of wet and dry seasonal levels.

Our services for this project included performing five (5) SPT borings along the project alignment. Based on these borings and our review of the soils in the area, we have estimated SHWT levels along the pipeline route with the results summarized and presented in **Table 1** in the **Appendix.** These estimates are based on the soil stratigraphy, measured groundwater levels in the SPT borings, USDA information and past experience. In areas where subsurface soil conditions were disturbed, normal indications such as “stain lines” were not evident. Ground surface elevations, for the borings, have been estimated from the topographic contours provided to us.

Fluctuation of the groundwater levels should be anticipated. We recommend that the Contractor determine the actual groundwater levels at the time of the construction to determine groundwater impact on the construction procedure.

3.4 PAVEMENT CORES INFORMATION

Two (2) pavement cores were performed (labeled PC-1 and PC-2) and the approximate locations shown in **Sheets 2 and 5**. Pictures of the cores are also included in the **Appendix** of this report.

Summary of Pavement Core Results				
Core No.	Station and offset (ft)	Type of Material and Averaged Thickness		
		Friction Course Thickness FC-6[1] (in.)	Structural Course Thickness Type S[1] (in.)	Field Measurement of Shell Base (in.)
PC-1	19+00 / 40 RT	0.48	3.51	8.00
PC-2	28+15 / 10 LT	0.90	1.30	13.25
Avg. Layer Thickness (in.)		**0.69**	**2.41**	**10.63**
Layer Thickness Range (in.)		**0.48 – 0.90**	**1.30 – 3.51**	**8.00 – 13.25**
Notes:				
1.	**Assumed asphalt type, friction and structural courses type based on visual observation.**			
2.	**The thicknesses were measured at four locations around the core using a calibrated caliper and averaged.**			

4.0 EVALUATION AND RECOMMENDATIONS

4.1 GENERAL

The following design recommendations have been developed on the basis of the previously described project characteristics and subsurface conditions encountered. If there are any changes in these project criteria, including project location on the site, a review must be made by **MC²** to determine if any modifications in the recommendations will be required. The findings of such a review should be presented in a supplemental report.

Once final design plans and specifications are available, a general review by **MC²** is strongly recommended as a means to check that the evaluations made in preparation of this report are correct and that earthwork and foundation recommendations are properly interpreted and implemented.

4.2 PIPELINE CONSIDERATIONS

Borings B-1 and B-2 indicated very loose to dense fine sands and/or slightly silty fine sands and/or slightly clayey fine sand (SP/SP-SM)/SP-SC) to depths ranging from 17 feet to the boring termination depth of 20 feet. These sands occasionally contained traces of shell. Boring B-2 terminated in medium dense fine sands and/or slightly silty fine sands and/or slightly clayey fine sand (SP/SP-SM)/SP-SC). Next, the borings entered medium dense clayey fine sand (SC) extending to depths ranging from 17 to 20 feet. An isolated thin layer of loose clayey fine sand (SC) was encountered in boring B-1 from depths ranging from 6 to 7 feet.

Borings B-3 through B-5 indicated very loose to medium dense fine sands and/or slightly silty fine sands and/or slightly clayey fine sand (SP/SP-SM)/SP-SC) to depths ranging from 12 to 17 feet. Next, the borings entered firm to very stiff clay to sandy clay (CL/CH) and/or medium dense clayey fine sand (SC) extending to depths ranging from 17 feet to the boring termination depth of 50 feet. Boring B-5 was terminated in medium dense fine sands and/or slightly silty fine sands and/or slightly clayey fine sand (SP/SP-SM/SP-SC), extending to the boring termination depth of 25 feet. In addition, boring B-4 encountered an isolated layer of moderately hard highly weathered limestone extending from depths ranging from 37 to 42 feet.

Based on the borings performed, settlement due to the presence of the pipeline should be minimal unless the subsoil is excessively disturbed during the installation, or the phreatic surface is lowered for a substantial period of time, or if new loads are placed above or near the pipeline. **Uplift pressure from the groundwater should be considered when the bottom of the pipeline is significantly below the existing groundwater level.**

Surface water and groundwater control will be necessary during construction of the pipeline to establish a stable sand bottom in which to bed the pipeline. Dewatering consisting of sump pumps and/or well pointing has been successful in the past. Dewatering must be conducted with care to avoid settlement of nearby structures, roads or utilities, and in such a manner that the areas possibly affected are as small as possible.

Depending upon shallow groundwater levels and the effectiveness of dewatering at the time of construction, seepage may enter the excavated trenches from the bottom and sides. Such seepage will act to loosen soils and create difficult working conditions. Groundwater levels should be determined immediately prior to construction. Shallow groundwater should be kept at least 12 inches below the working area to facilitate proper material placing and compaction. Organic soils and clayey soils should be removed (if encountered) within 24 inches from the bottom of the pipeline and replaced with properly compacted clean sands (SP/SP-SM/SP-SC). Additional borings may be required during construction to better determine (delineate) the horizontal and vertical extent of any organic soils and their impact on the project.

A density of at least 98% of the modified Proctor maximum dry density (ASTM D-1557) is recommended for all fill materials and natural subgrade under the pipeline. The subgrade soils should be firm and stable prior to placement of the pipe. Once the pipeline is placed, it is recommended that backfill around the sides be placed and compacted in equal lifts with a vibratory tamper. Lifts should not exceed 6-inches (loose density) to avoid laterally displacing the pipeline. Failure to compact the backfill will result in future settlement of the ground surface.

Pipeline backfill should be clean, fine sand (free of clay, rubble, organics and debris) with less than 12% passing the No. 200 sieve and placed in compacted lifts. Some contractors like to place a gravel working bed in wet areas. Fine gravel, such as No. 57, and No. 67 stone may be used in limited areas. A continuous gravel bed should not be placed for the full pipe length to prevent a flow conduit under the pipeline. The gravel, where used, should be compacted and the compaction confirmed by visual observation.

The non-organic clean fine sands, slightly silty fine sands or slightly clayey fine sand (SP/SP-SM/SP-SC), encountered in the project, with less than 12 percent passing the No. 200 sieve will be suitable for backfill soils.

It should be mentioned that water seepage through construction joints in the completed pipeline may have a tendency to erode soil from around the pipeline. The designer should consider the use of a geotextile around joints if this is a concern.

4.3 STRUCTURE EXCAVATIONS

All structure excavations, if applicable, should be observed by the Geotechnical Engineer or his representative to explore the extent of any fill and excessively loose, soft, or otherwise undesirable materials. If the excavation appears suitable as load bearing materials, the soils should be prepared for construction by compaction to a dry density of at least 98% of the modified Proctor maximum dry density (ASTM D-1557) for a depth of at least 1 foot below the foundation base.

If soft pockets are encountered in the bottom of the structure excavations, the unsuitable materials should be removed and the proposed foundation elevation re-established by backfilling after the undesirable material has been removed. This backfilling may be done with a very lean concrete or with a well-compacted, suitable fill such as clean sand, gravel, or crushed #57 or #67 stone. Sand backfill should be compacted to a dry density of at least 98% of the modified Proctor maximum dry density (ASTM D-1557), as previously described. Gravel, or crushed #57 or #67 stone, if used, should be compacted and the compaction confirmed by visual observation.

4.4 LATERAL EARTH PRESSURES

The pipeline and walls will be subject to lateral earth pressures. For walls (if applicable) which are restrained and adjacent to moderately compacted backfill, design is usually based on “at-rest” earth pressures. Active pressures are usually employed for unrestrained retaining wall design. Several earth pressure theories could be utilized. One of the most straightforward is the equivalent fluid pressure or Rankine Theory.

Pipes and walls (if applicable) constructed below existing grades or which have adjacent compacted fill will be subjected to lateral at-rest or active earth pressures. Walls which are restrained at the top and bottom will be subjected to at-rest soil pressures equivalent to a fluid density of 55 pounds per cubic foot (pcf). Walls which are not restrained at the top and where sufficient movement may mobilize active earth pressures, an equivalent fluid density of 36 pcf can be used. At locations where the base of the walls extends below the groundwater table, soil pressures can be calculated using half (½) the equivalent fluid densities given above (see table below for actual values). However, hydrostatic and seepage forces must then also be included. The above pressures do not include any surcharge effects for sloped backfill, point or area loads behind the walls and assume that adequate drainage provisions have been incorporated. The lateral earth pressures acting on below grade walls will be resisted by the sliding resistance forces along the base of the wall footing and the passive resistance resulting from footing embedment at the wall toe. Passive resistance could be neglected for a safer design (due to possible excavation or erosion in front of the wall at a future time).

Earth Pressure Condition	**Coefficient of Earth Pressure (K)**	**Unsubmerged Fluid Density (1) (pcf)**	**Submerged Fluid Density (pcf) (2)**
At-Rest (K_o)	0.50	55	24
Active (K_a)	0.33	36	16
Passive (K_p)	3.00	330	143

(1) These fluid densities are based on a clean sand backfill with an average internal friction angle of 30 degrees and a moist unit weight of 110 pcf.
(2) Hydrostatic and seepage forces should be added to the submerged fluid densities when calculating total forces acting on retaining walls.

4.5 STRUCTURAL FILL

All materials to be used for backfill or compacted fill construction should be evaluated and, if necessary, tested by **MC^2** prior to placement to determine if they are suitable for the intended use. Based on the borings performed, the majority of the on-site sandy materials encountered in the ***upper depths (top 12 to 17 feet))*** of the borings are suitable for use as structural fill and as general subgrade fill and backfill. Suitable structural fill materials should consist of fine to medium sand with less than 12% passing through the No. 200 sieve and be free of rubble, organics, clay, debris and other unsuitable material. **Table 2** in our **Appendix** includes a summary of the laboratory test results performed indicating depths and locations of soils with % passing the No. 200 sieve indicated.

All structural fill should be compacted to a dry density of at least 98 percent of the modified Proctor maximum dry density (ASTM D-1557). In general, the compaction should be accomplished by placing the fill in maximum 6-inch loose lifts and mechanically compacting each lift to at least the specified minimum dry density. A representative of **MC^2** should perform field density tests on each lift as necessary to assure that adequate compaction is achieved.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 General

It is recommended that **MC^2** be retained to provide observation and testing of construction activities involved in the foundation, earthwork, and related activities of this project to ensure that the recommendations contained herein are properly interpreted and implemented. If **MC^2** is not retained to perform these functions, we cannot be responsible for the impact of those conditions on the performance of the project.

5.2 Fill Placement and Subgrade Preparation

The following are our general recommendations for overall site preparation and mechanical densification work for the proposed project based on the anticipated construction and our boring results. These recommendations should be used as a guideline for the project general specifications by the Design Engineer.

1. The excavated subgrade (dewatered trench bottom) for the pipes and associated structures (if applicable) should be leveled, cut to grade if necessary, and then compacted with a vibratory compactor. Careful observations should be made during compaction to help identify any areas of soft yielding soils that may require overexcavation and replacement. If unsuitable material, such as organic or clayey soils, is encountered at the bottom of the pipe or structure embedment depth, overexcavation of an additional 2 and 3 feet of the material is recommended for the pipe and structure, respectively. The excavation should then be backfilled to foundation grade with clean sands in controlled lifts not exceeding 6-inches and compacted to a density of at least 98 percent of the maximum density, as determined by ASTM D-1557. Care should be used when operating the compactor to avoid transmission of vibrations to existing structures or other construction operations that could cause settlement damage or disturb occupants. Dewatering may also have an effect on adjacent structures. A preconstruction survey with video and/or photographs of adjacent residences/structures is recommended to check for existing cracking prior to construction and during construction. Vibration and groundwater levels monitoring are also recommended.

2. Prior to beginning compaction, soil moisture contents may need to be

controlled in order to facilitate proper compaction. A moisture content within 2 percentage points of the optimum indicated by the modified Proctor test (ASTM D-1557) is recommended.

3. Following satisfactory completion of the initial compaction on the excavation bottom, the construction areas may be brought up to finished subgrade levels. Fill should consist of fine sand with less than 12% passing the No. 200 sieve, free of rubble, organics, clay, debris and other unsuitable material. Fill should be tested and approved prior to acquisition and/or placement. Approved sand fill should be placed in loose lifts not exceeding 6-inches in thickness and should be compacted to a minimum of 98% of the maximum modified Proctor dry density (ASTM D-1557). Density tests to confirm compaction should be performed in each fill lift before the next lift is placed.

4. It is recommended that a representative from our firm be retained to provide on-site observation of earthwork activities. The field technician would monitor the placement of approved fills and compaction and provide compaction testing. Density tests should be performed in subgrade sands after rolling and in each fill lift. It is important that **MC^2** be retained to observe that the subsurface conditions are as we have discussed herein, and that construction and fill placement is in accordance with our recommendations.

5.3 GROUNDWATER CONTROL

Dewatering will be necessary to achieve the required depth of excavation and compaction of backfill. Groundwater can normally be controlled in excavations with a sump pump and/or well pointing as previously discussed. For deep excavations, dewatering using temporary well points or temporary sheet pile walls may be necessary.

Surface water and groundwater control will be necessary during construction to permit establishment of a stable sand bottom. If a pump is used, a standby pump is recommended.

Soils exposed in the bases of all satisfactory excavations should be protected against any detrimental change in conditions such as from physical disturbance or rain. Surface run-off water should be drained away from the excavations and not be allowed to pond. If possible, all drainage structures should be placed the same day the excavation is made. If this is not possible, the excavations should be adequately protected.

Groundwater levels should be determined by the contractor immediately prior to construction. Shallow groundwater should be kept at least 12 inches below the lowest working area to facilitate proper material placement and compaction.

5. 4 TEMPORARY SLOPES

Side slopes for temporary excavations may stand near one (1) horizontal to one (1) vertical (1H:1V) for short dry periods of time and a maximum excavation depth of four (4) feet. Where restrictions do not permit slopes to be constructed as recommended above, the excavation should be shored in accordance with current OSHA requirements. In addition, any open cut excavations adjacent to existing structures should be evaluated by a geotechnical engineer on a case by case basis. During construction, excavated materials should not be stockpiled at the top of the slope within a horizontal distance equal to the excavation depth.

Excavation slopes should conform to OSHA, State of Florida and any other local regulations. The dewatering system chosen for use on this project should consider the nature of the permeable upper sands encountered at the project site. The contractor should also assess equipment loads and vibrations when considering slopes or excavation bracing.

6. 0 REPORT LIMITATIONS

The recommendations detailed herein are based on the available soil information obtained by **MC^2** and information provided by **Cardno TBE** for the proposed project. If there are any revisions to the plans for this project, or if deviations from the subsurface conditions noted in this report are encountered during construction, **MC^2** should be notified immediately to determine if changes in the foundation, or other, recommendations are required. In the event that **MC^2** is not retained to perform these functions, **MC^2** cannot be held responsible for the impact of those conditions on the performance of the project.

The geotechnical engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are more complete, the geotechnical engineer should be provided the opportunity to review the final design plans and specifications to assess that our engineering recommendations have been properly incorporated into the design documents. At that time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of **Cardno TBE.**

APPENDIX

- **Table 1 – Borings and Groundwater Levels**
- **Table 2 – Summary of Laboratory Test Results**
- **Table 3 – Summary of Corrosion Test Results**
- **Boring Location Plan – Sheets 1 through 5**
- **Report of Core Borings – Sheet 6**
- **Pavement Core Photos**
- **Test Procedures**

Table 1
Summary of Borings and Groundwater Levels
Whitfield Avenue Watermain
Manatee County, Florida
Mc Squared No. T081317.194

Boring No.	Station and offset (ft)	Boring Elev.[1] (ft)	USDA Soil Type	USDA Seasonal High Groundwater Table Depth (ft)	Boring Depth (ft)	Measured GW Depth (ft) Mar 2014	Measured GW elev. (ft)	Estimated Seasonal High Groundwater Levels Depth/elev. (ft)
B-1	6+15/44 RT	18.0	(No. 20) EauGallie fine sand	0.5 – 1.5	20.0	5.0	13.0	1.0 (perched above the Sandy Clay Loam)/12.0
B-2	12+88/45 RT	16.0			20.0	GNM	GNM	
B-3	19+00/17 LT	16.0	(No. 7) Canova, Anclote, and Okeelanta soils	0.0	25.0	6.0	10.0	0.5 (perched above the Sandy Loam and Muck)/10.0
B-4	23+70/8 RT	15.0		0.0	50.0	7.0	8.0	
B-5	28+46/25 LT	11.5		0.0	25.0	3.0	8.5	
Notes:								
1.0	Boring elevation was estimated from plan provided by **Cardno TBE** with spot elevations and is approximate.							
2.0	GNM = Groundwater not measured due to the introduction of drilling mud.							

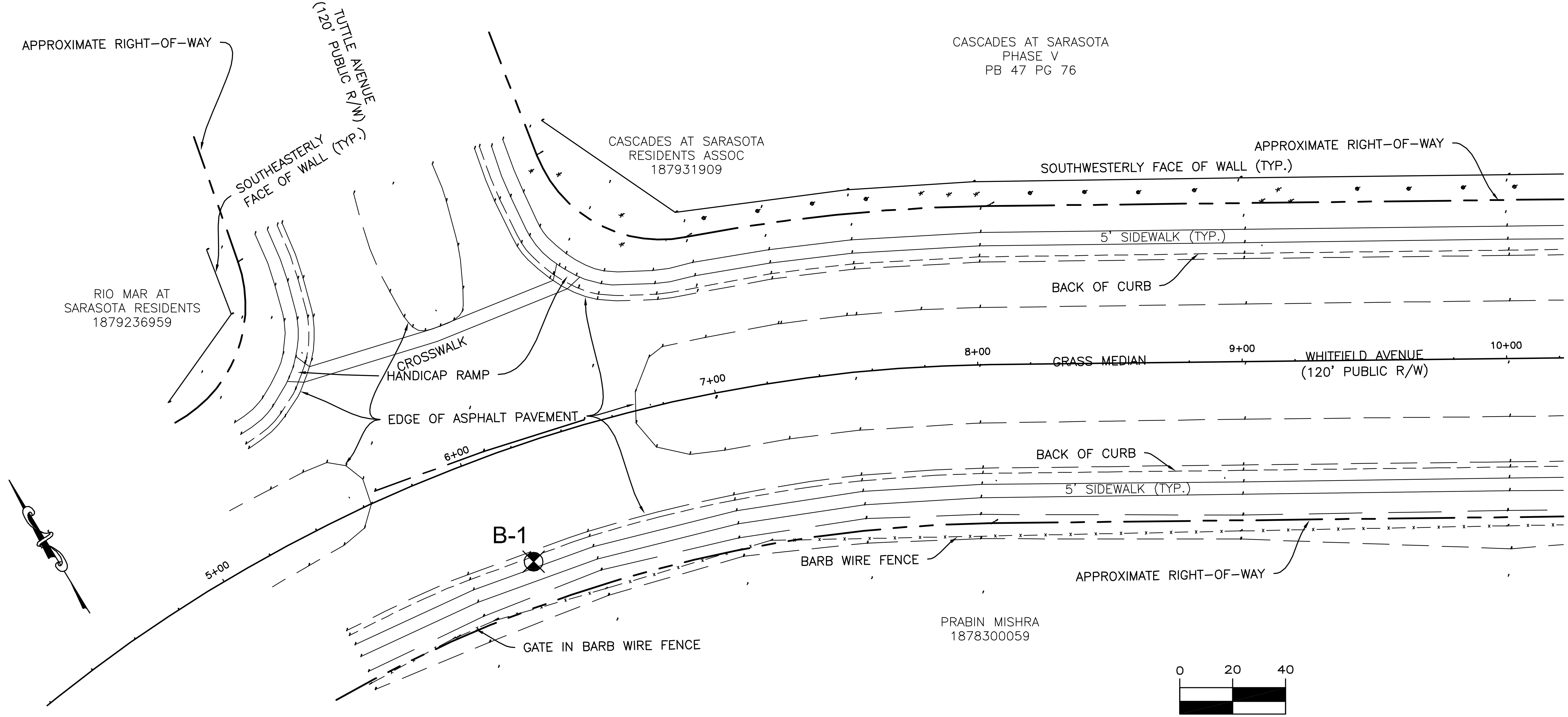
Table 2
Summary of Laboratory Test Results
Whitfield Avenue Watermain
Manatee County, Florida
Mc Squared No. T081317.194

Boring No.	Depth (ft)	USCS Classi.	Sieve Analysis (% Passing)							Liquid Limit (%)	Plastic Index (%)	Organic Content (%)	Natural Moisture Content (%)
			#10	#20	#40	#60	#100	#140	#200				
B-1	6.0 – 7.0	SC							16				23
B-2	2.0 – 4.0	SP/SP-SM/SP-SC							8				14
B-2	18.5 – 20.0	SC							20				21
B-3	23.5 – 25.0	CL/CH							67	64	44		60
B-4	2.0 – 4.0	SP/SP-SM/SP-SC							5				11
B-4	28.5 – 30.0	CL/CH							61	73	39		77
B-5	2.0 – 4.0	SP/SP-SM/SP-SC							10				25
B-5	13.5 – 15.0	SC							33	NP	NP		41

Table 3
Summary of Corrosion Parameters Test Results
Whitfield Avenue Watermain
Manatee County, Florida
Mc Squared No. T081317.194

Boring No.	Sample Depth (Feet)	Unified Soil Class.	pH (FM 5-550)	Resistivity (ohm-cm) (FM 5-551)	Chloride (ppm) (FM 5-552)	Sulfates (ppm) (FM 5-553)	Environmental Classification* (Soil Sample)	
							Steel	Concrete
B-2 (soil)	2.0 – 6.0	SP/SP-SM/SP-SC	8.3	2600	211.5	150.0	Moderately Aggressive	Moderately Aggressive
B-4 (soil)	2.0 – 6.0	SP/SP-SM/SP-SC	8.7	4300	157.5	114.0	Moderately Aggressive	Slightly Aggressive
							(Water sample)	
Water Sample	Standing water at approximately station 21+00/ 45 LT	Water	8.0	865	265.0	77.0	Extremely Aggressive	Moderately Aggressive
Notes:								

***= As per FDOT Structures Design Guidelines, Table 1.3.2.-1 (Criteria for Substructure Environmental Classifications), January 2014**



LEGEND

APPROXIMATE SPT BORING LOCATION

APPROXIMATE PAVEMENT CORE LOCATION

DATE	NAME	REVISION	APPROVED BY:

MC SQUARED, INC.
Geotechnical Consultants
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GEOTECHNICAL • ENVIRONMENTAL MATERIALS TESTING

FLORIDA ENGINEERING CERTIFICATE OF AUTHORIZATION No. 9191
Kermit Schmidt, P.E.
FLORIDA LICENSE No. 45603

	NAME	DATE
DESIGNED BY:	IR	03/14
DRAWN BY:	IR	03/14
CHECKED BY:	KS	03/14
SUPERVISED BY:		

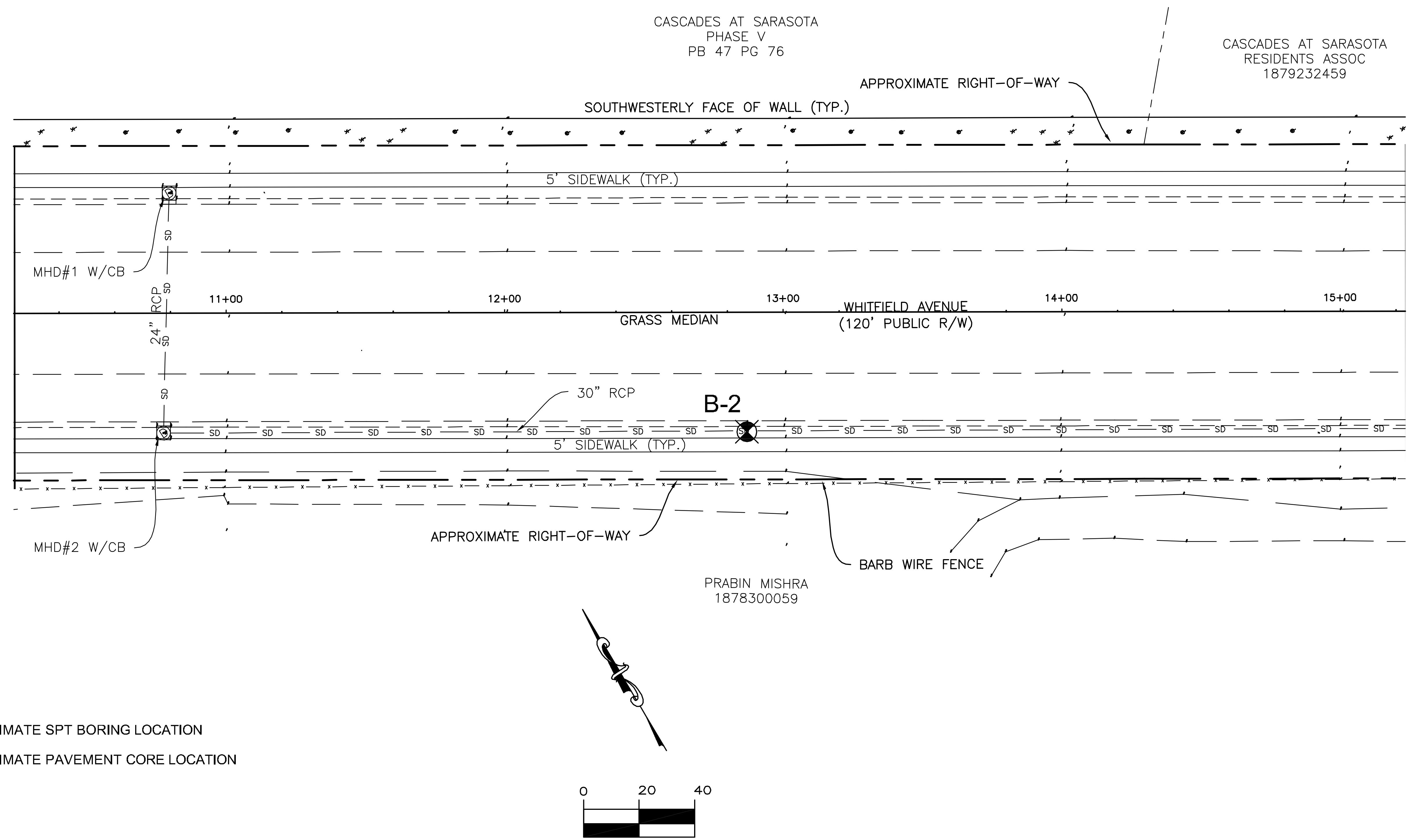
BORING LOCATION PLAN

WHITFIELD AVENUE WATERMAIN
MANATEE COUNTY, FLORIDA

PROJECT NO. T081317.194

SHEET NO. 1

STORM DRAIN STRUCTURES			
ID	RIM	INV(DIR)	INV(DIR)
MHD#1	15.93'	8.61(S)	
MHD#2	15.93'	8.40'(N)	8.28'(E)



LEGEND

APPROXIMATE SPT BORING LOCATION

APPROXIMATE PAVEMENT CORE LOCATION

DATE	NAME	REVISION	APPROVED BY:

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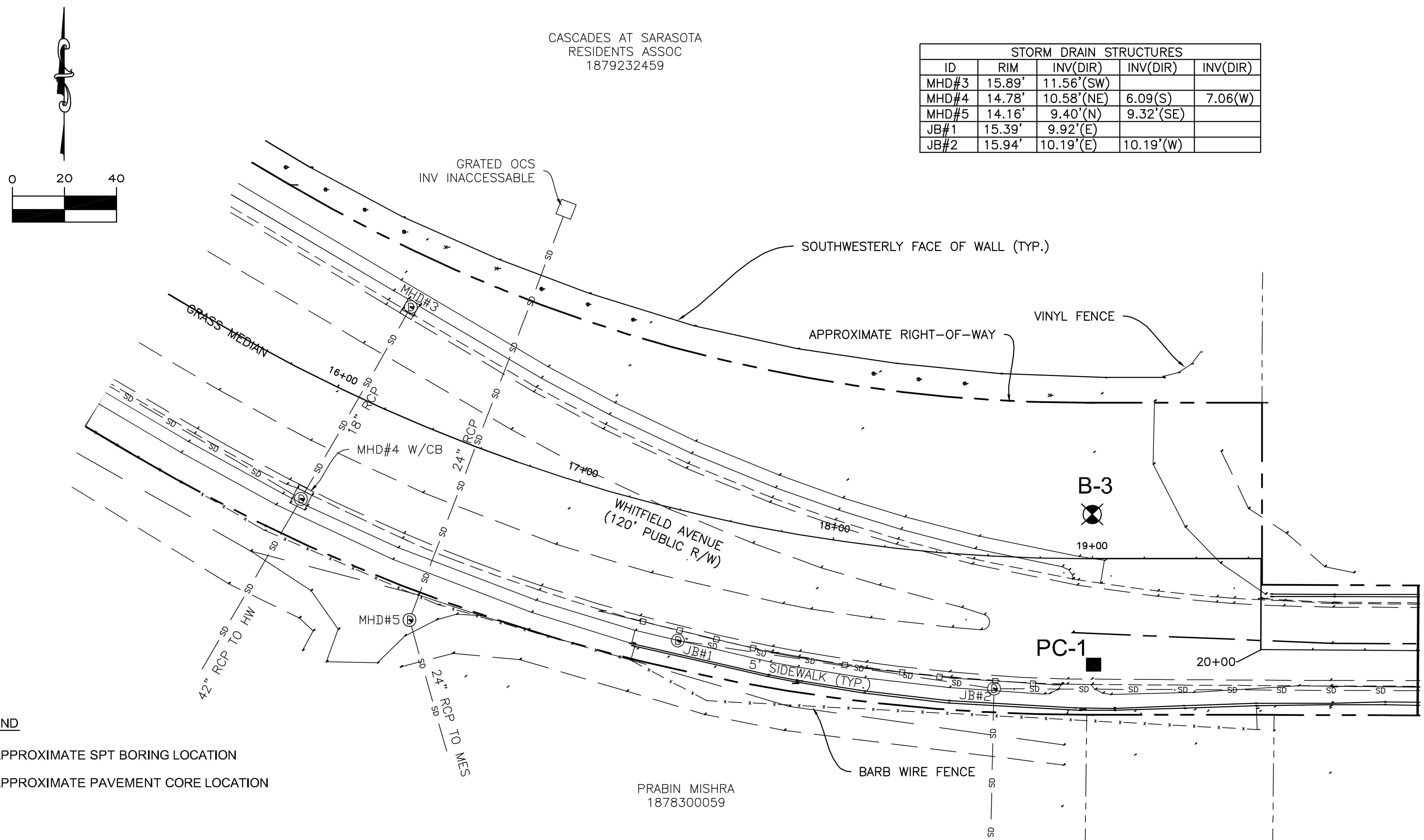
BORING LOCATION PLAN

WHITFIELD AVENUE WATERMAIN
MANATEE COUNTY, FLORIDA

PROJECT NO.	SHEET NO.
T081317.194	2



STORM DRAIN STRUCTURES				
ID	RIM	INV(DIR)	INV(DIR)	INV(DIR)
MHD#3	15.89'	11.56'(SW)		
MHD#4	14.78'	10.58'(NE)	6.09(S)	7.06(W)
MHD#5	14.16'	9.40'(N)	9.32'(SE)	
JB#1	15.39'	9.92'(E)		
JB#2	15.94'	10.19'(E)	10.19'(W)	

LEGEND

APPROXIMATE SPT BORING LOCATION

APPROXIMATE PAVEMENT CORE LOCATION

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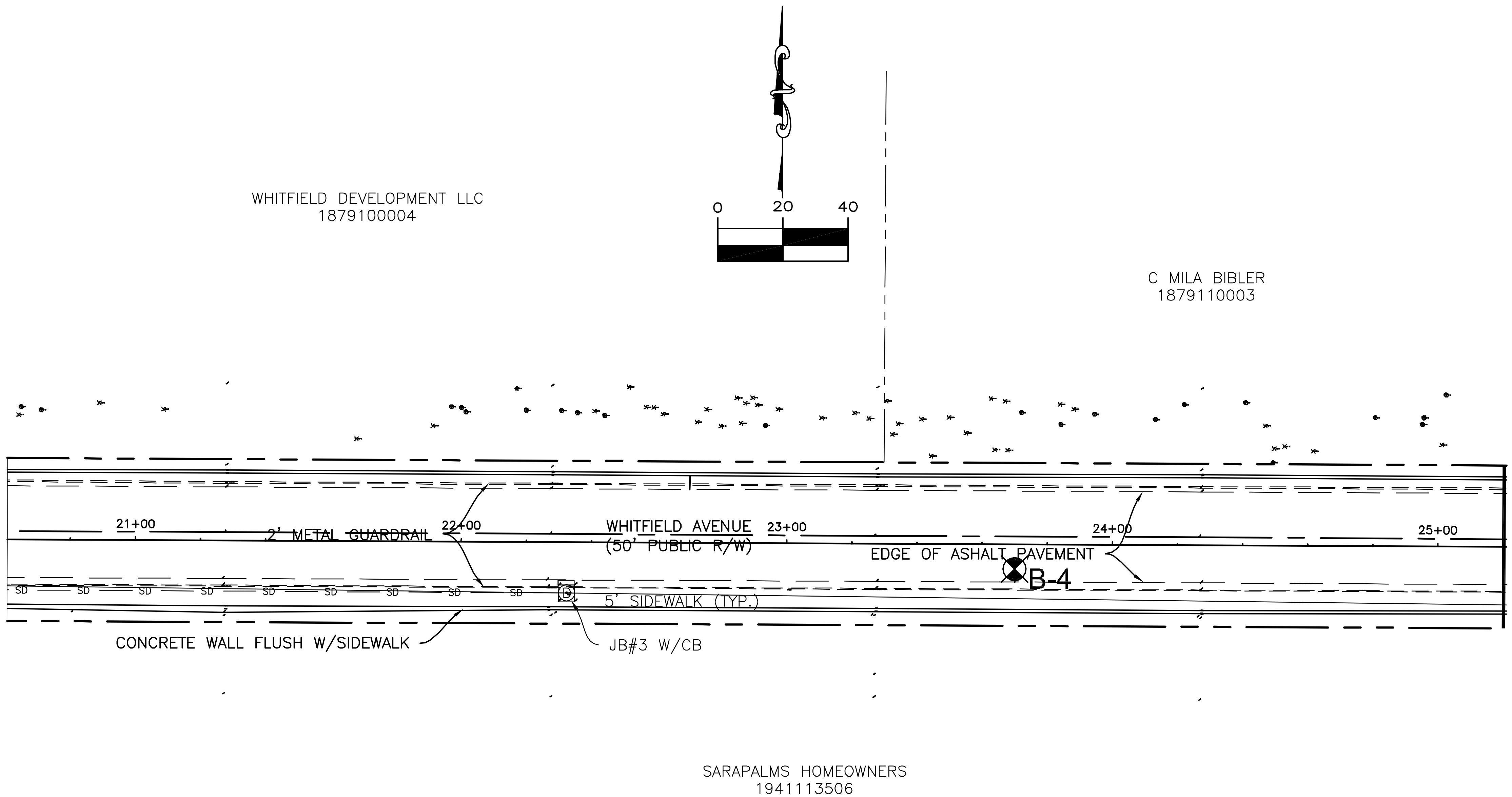
SUPERVISED BY:		

BORING LOCATION PLAN

WHITFIELD AVENUE WATERMAIN
MANATEE COUNTY, FLORIDA

PROJECT NO. T081317.194

SHEET NO. 3



STORM DRAIN STRUCTURES		
ID	RIM	INV(DIR)
MHD#3	15.89'	11.56'(SW)

LEGEND

APPROXIMATE SPT BORING LOCATION

APPROXIMATE PAVEMENT CORE LOCATION

DATE	NAME	REVISION	APPROVED BY:

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MC²
GEOTECHNICAL • ENVIRONMENTAL
MATERIALS TESTING

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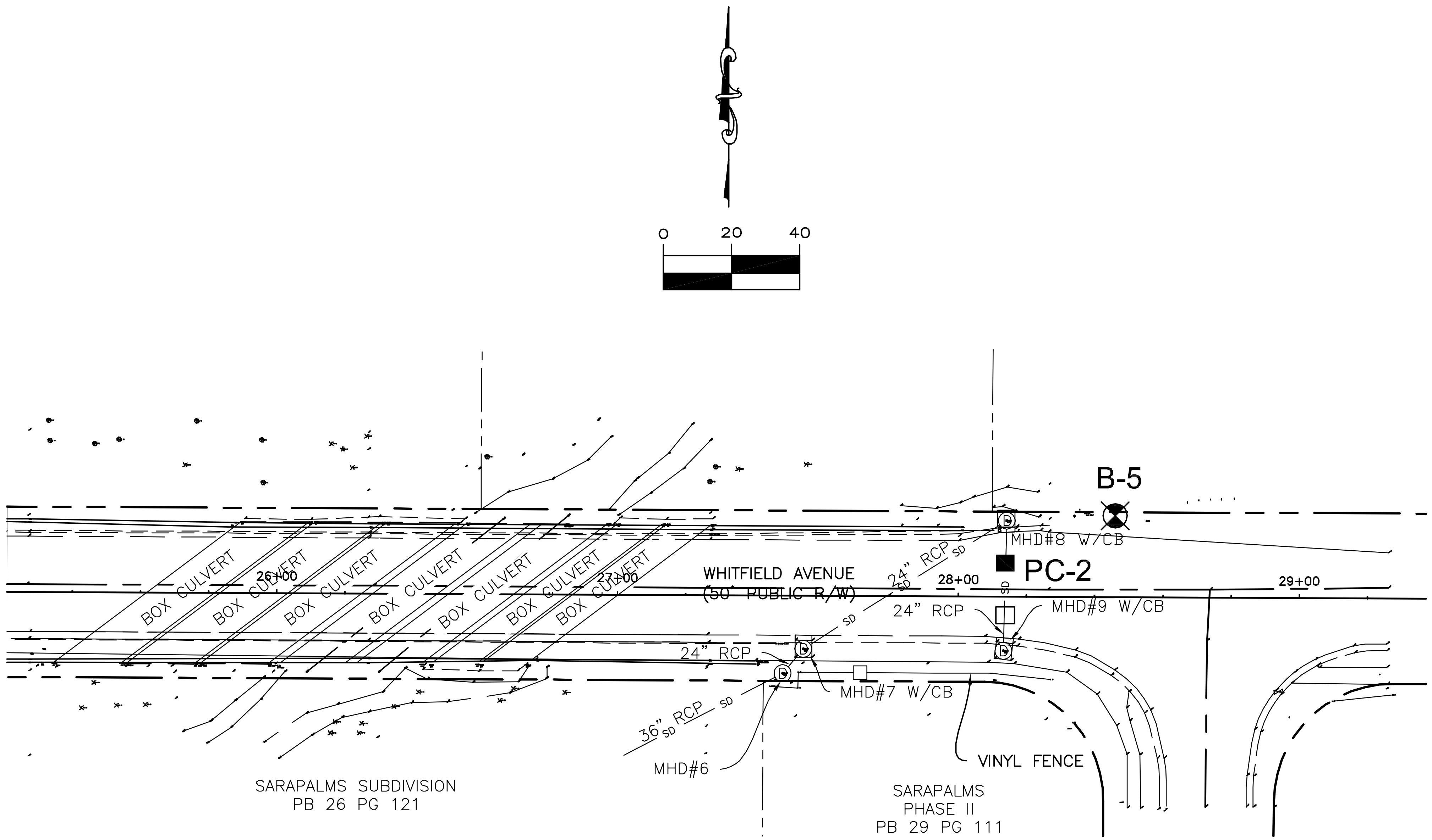
CHECKED BY:	KS	03/14
SUPERVISED BY:		

BORING LOCATION PLAN

WHITFIELD AVENUE WATERMAIN
MANATEE COUNTY, FLORIDA

PROJECT NO. T081317.194

SHEET NO. 4



STORM DRAIN STRUCTURES			
ID	RIM	INV(DIR)	INV(DIR)
MHD#6	13.01'	7.28'(NE)	5.77'(SW)
MHD#7	12.76'	6.88'(SW)	7.42'(NE)
MHD#8	12.31'	7.55'(SW)	7.55'(S)
MHD#9	11.99'	7.75N)	

LEGEND

APPROXIMATE SPT BORING LOCATION

APPROXIMATE PAVEMENT CORE LOCATION

DATE	NAME	REVISION	APPROVED BY:

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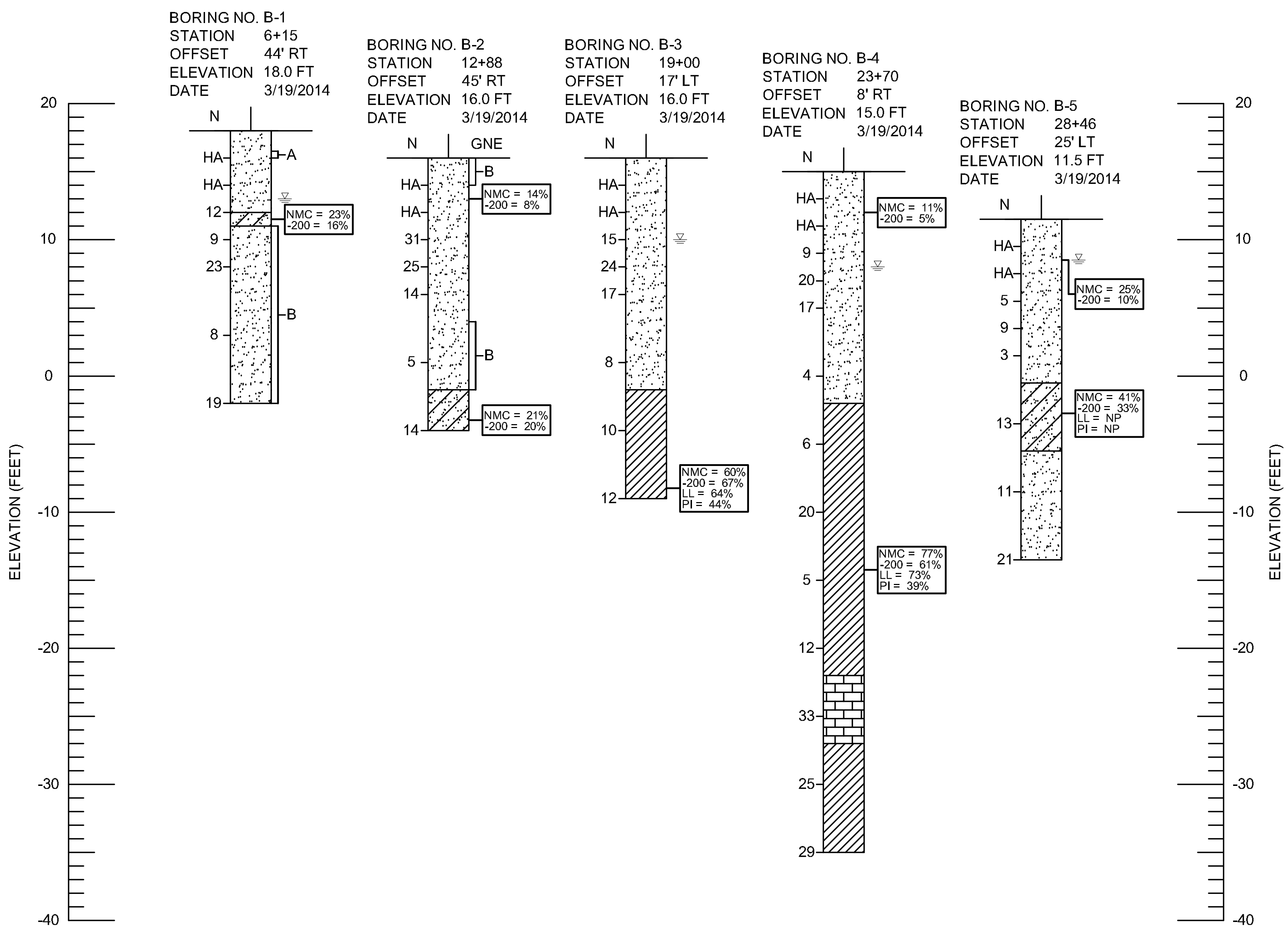
CHECKED BY:	KS	03/14
SUPERVISED BY:		

BORING LOCATION PLAN

WHITFIELD AVENUE WATERMAIN
MANATEE COUNTY, FLORIDA

PROJECT NO. T081317.194

SHEET NO. 5



0 10

SCALE: 1" = 10'

LEGEND

- (SP/SP-SM/SP-SC) GRAY OR BROWN FINE SAND OR SLIGHTLY SILTY FINE SAND OR SLIGHTLY CLAYEY FINE SAND.
- (SC) GREENISH GRAY CLAYEY FINE SAND..
- (CL/CH) GREENISH GRAY SANDY CLAY TO CLAY.
- PALE BROWN HIGHLY WEATHERED LIMESTONE.

NOTES:

A	WITH TRACES OF WOOD DEBRIS
B	WITH TRACES OF SHELL
▽	WATER TABLE
GNE	GROUNDWATER NOT ENCOUNTERED
NMC	NATURAL MOISTURE CONTENT (%)
-200	PERCENT PASSING A NO. 200 SIEVE (%)
LL	LIQUID LIMIT (%)
PI	PLASTICITY INDEX (%)
NP	NON-PLASTIC SOILS

GRANULAR MATERIALS- RELATIVE DENSITY	SPT (BLOWS/FT)
VERY LOOSE	LESS THAN 4
LOOSE	5-10
MEDIUM	11-30
DENSE	31-50
VERY DENSE	GREATER THAN 50

SILTS AND CLAYS CONSISTENCY	SPT (BLOWS/FT)
VERY SOFT	LESS THAN 2
SOFT	3-4
FIRM	5-8
STIFF	9-15
VERY STIFF	16-30
HARD	30-50
VERY HARD	GREATER THAN 50

DATE	NAME	REVISION	APPROVED BY:

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	NAME	DATE
DESIGNED BY:	IR	03/14
DRAWN BY:	IR	03/14
CHECKED BY:	KS	03/14
SUPERVISED BY:		

REPORT OF CORE BORINGS

WHITFIELD AVENUE WATERMAIN
MANATEE COUNTY, FLORIDA

PROJECT NO. T081317.194

SHEET NO. 6


MC2
Geotechnical • Environmental
Materials Testing
PC-1
Whitfield Avenue Watermain
T081317.194
Manatee County, FL


MC2
Geotechnical • Environmental
Materials Testing
PC-2
Whitfield Avenue Watermain
T081317.194
Manatee County, FL

TEST PROCEDURES

The general field procedures employed by MC Squared, Inc. (**MC^2**) are summarized in the American Society for Testing and Materials (ASTM) Standard D420 which is entitled "Investigating and Sampling Soil and Rock". This recommended practice lists recognized methods for determining soil and rock distribution and groundwater conditions. These methods include geophysical and in-situ methods as well as borings.

Standard Drilling Techniques
To obtain subsurface samples, borings are drilled using one of several alternate techniques depending upon the subsurface conditions. Some of these techniques are:

In Soils:

a) Continuous hollow stem augers.
b) Rotary borings using roller cone bits or drag bits, and water or drilling mud to flush the hole.
c) "Hand" augers.

In Rock:

a) Core drilling with diamond-faced, double or triple tube core barrels.
b) Core boring with roller cone bits.

The drilling method used during this exploration is presented in the following paragraph.

Hollow Stem Augering: A hollow stem augers consists of a hollow steel tube with a continuous exterior spiral flange termed a flight. The auger is turned into the ground, returning the cuttings along the flights. The hollow center permits a variety of sampling and testing tools to be used without removing the auger.

Core Drilling: Soil drilling methods are not normally capable of penetrating through hard cemented soil, weathered rock, coarse gravel or boulders, thin rock seams, or the upper surface of sound, continuous rock. Material which cannot be penetrated by auger or rotary soil-drilling methods at a reasonable rate is designated as “refusal material”. Core drilling procedures are required to penetrate and sample refusal materials.

Prior to coring, casing may be set in the drilled hole through the overburden soils, to keep the hole from caving and to prevent excessive water loss. The refusal materials are then cored according to ASTM D-2113 using a diamond-studded bit fastened to the end of a hollow, double or triple tube core barrel. This device is rotated at high speeds, and the cuttings are brought to the surface by circulating water. Core samples of the material penetrated are protected and retained in the swivel-mounted inner tube. Upon completion of each drill run, the core barrel is brought to the surface, the core recovery is measured, and the core is placed, in sequence, in boxes for storage and transported to our laboratory.

Sampling and Testing in Boreholes
Several techniques are used to obtain samples and data in soils in the field; however the most common methods in this area are:

a) Standard Penetration Testing
b) Undisturbed Sampling
c) Dynamic Cone Penetrometer Testing
d) Water Level Readings

The procedures utilized for this project are presented below.

Standard Penetration Testing: At regular intervals, the drilling tools are removed and soil samples obtained with a standard 2 inch diameter split tube sampler connected to an A or N-size rod. The sampler is first seated 6 inches to penetrate any loose cuttings, and then driven an additional 12 inches with blows of a 140 pound safety hammer falling 30 inches. Generally, the number of hammer blows required to drive the sampler the final 12 inches is designated the "penetration resistance" or "N" value, in blows per foot (bpf). The split barrel sampler is designed to retain the soil penetrated, so that it may be returned to the surface for observation. Representative portions of the soil samples obtained from each split barrel sample are placed in jars, sealed and transported to our laboratory.

The standard penetration test, when properly evaluated, provides an indication of the soil strength and compressibility. The tests are conducted according to ASTM Standard D1586. The depths and N-values of standard penetration tests are shown on the Boring Logs. Split barrel samples are suitable for visual observation and classification tests but are not sufficiently intact for quantitative laboratory testing.

Water Level Readings: Water level readings are normally taken in the borings and are recorded on the Boring Records. In sandy soils, these readings indicate the approximate location of the hydrostatic water level at the time of our field exploration. In clayey soils, the rate of water seepage into the borings is low and it is generally not possible to establish the location of the hydrostatic water level through short-term water level readings. Also, fluctuation in the water level should be expected with variations in precipitation, surface run-off, evaporation, and other factors. For long-term monitoring of water levels, it is necessary to install piezometers.

The water levels reported on the Boring Logs are determined by field crews immediately after the drilling tools are removed, and several hours after the borings are completed, if possible. The time lag is intended to permit stabilization of the groundwater level that may have been disrupted by the drilling operation.

Occasionally the borings will cave-in, preventing water level readings from being obtained or trapping drilling water above the cave-in zone.

BORING LOGS

The subsurface conditions encountered during drilling are reported on a field boring log prepared by the Driller. The log contains information concerning the boring method, samples attempted and recovered, indications of the presence of coarse gravel, cobbles, etc., and observations of groundwater. It also contains the driller's interpretation of the soil conditions between samples. Therefore, these boring records contain both factual and interpretive information. The field boring records are kept on file in our office.

After the drilling is completed a geotechnical professional classifies the soil samples and prepares the final Boring Logs, which are the basis for our evaluations and recommendations.

SOIL CLASSIFICATION

Soil classifications provide a general guide to the engineering properties of various soil types and enable the engineer to apply his past experience to current problems. In our investigations, samples obtained during drilling operations are examined in our laboratory and visually classified by an engineer. The soils are classified according to consistency (based on number of blows from standard penetration tests), color and texture. These classification descriptions are included on our Boring Logs.

The classification system discussed above is primarily qualitative and for detailed soil classification two laboratory tests are necessary; grain size tests and plasticity tests. Using these test results the soil can be classified according to the AASHTO or Unified Classification Systems (ASTM D-2487). Each of these classification systems and the in-place physical soil properties provides an index for estimating the soil's behavior. The soil classification and physical properties are presented in this report.

The following table presents criteria that are typically utilized in the classification and description of soil and rock samples for preparation of the Boring Logs.

Relative Density of Cohesionless Soils From Standard Penetration Test		Consistency of Cohesive Soils	
Very Loose	≤ 4 bpf	Very Soft	≤ 2 bpf
Loose	5 - 10 bpf	Soft	3 - 4 bpf
Medium Dense	11 - 30 bpf	Firm	5 - 8 bpf
Dense	31 - 50 bpf	Stiff	9 - 15 bpf
Very Dense	> 50 bpf	Very Stiff	16 - 30 bpf
		Hard	30 – 50 bpf
(bpf = blows per foot, ASTM D 1586)		Very Hard	> 50 bpf

Relative Hardness of Rock		Particle Size Identification	
Very Soft	Hard Rock disintegrates or easily compresses to touch; can be hard to very hard soil.	Boulders	Larger than 12"
		Cobbles	3" - 12"
Soft	May be broken with fingers.	Gravel	
		Coarse	3/4" - 3"
Moderately Soft	May be scratched with a nail, corners and edges may be broken with fingers.	Fine	4.76mm - 3/4"
		Sand	
		Coarse	2.0 - 4.76 mm
Moderately Hard	Light blow of hammer required to break samples.	Medium	0.42 - 2.00 mm
		Fine	0.42 - 0.074 mm
Hard	Hard blow of hammer required to break sample.	Fines (Silt or Clay)	Smaller than 0.074 mm

Rock Continuity		Relative Quality of Rocks	
$\textbf{RECOVERY} = \dfrac{\text{Total Length of Core}}{\text{Length of Core Run}} \times 100\ \%$		$\textbf{RQD} = \dfrac{\text{Total core, counting only pieces > 4" long}}{\text{Length of Core Run}} \times 100\ \%$	
Description	Core Recovery %	Description	RQD %
Incompetent	Less than 40	Very Poor	0 - 25 %
Competent	40 - 70	Poor	25 - 50 %
Fairly Continuous	71 - 90	Fair	50 - 75 %
Continuous	91 - 100	Good	75 - 90 %
		Excellent	90 - 100 %