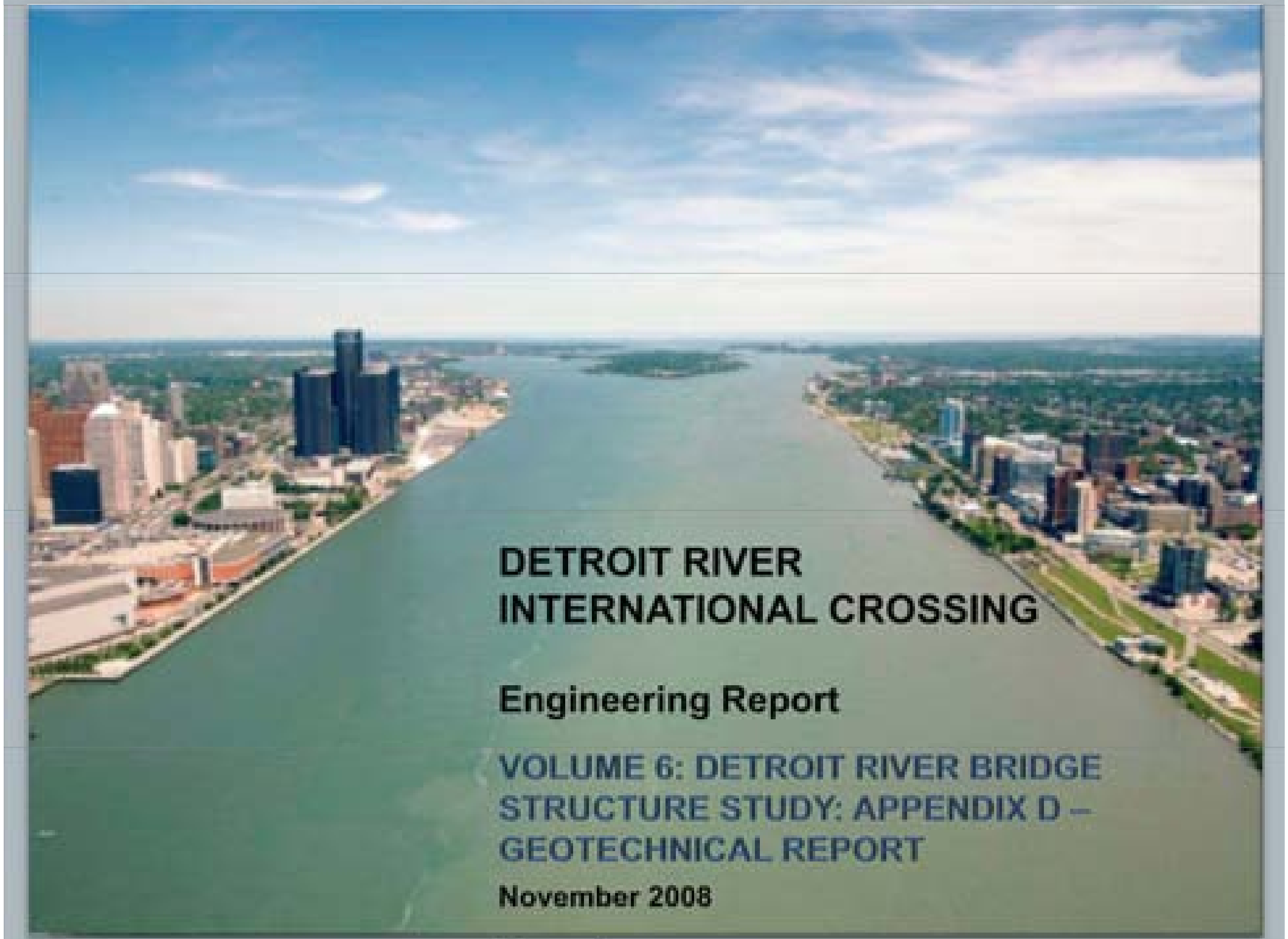
Detroit River
INTERNATIONAL CROSSING
PROJECT

A BORDER TRANSPORTATION PARTNERSHIP

Federal Highway Administration
Canada
MDOT
Ontario



DETROIT RIVER INTERNATIONAL CROSSING

Engineering Report

VOLUME 6: DETROIT RIVER BRIDGE STRUCTURE STUDY: APPENDIX D – GEOTECHNICAL REPORT

November 2008

Prepared by:

PARSONS

In association with:

benesch
NCI NORTHWEST CONSULTANTS, INC.
NTH Consultants, Ltd
Somat Engineering, INCORPORATED

Under agreement with:

CORRADINO
THE CORRADINO GROUP

Geotechnical Feasibility Evaluation Report
for Proposed Foundation Elements

Proposed Detroit River International Crossing
United States Shoreline

November 21, 2008

Prepared by:

NTH Consultants, Ltd.

Prepared for:

The Corradino Group
In Partnership with
Parsons Transportation Group

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EXECUTIVE SUMMARY

The Detroit River International Crossing (DRIC) Study is a bi-national effort to complete the environmental study processes for the border crossing between Detroit, Michigan and Windsor, Ontario for the United States, Michigan, Canada and Ontario governments. The study will identify solutions that support the region, state, provincial and national economies while addressing civil defense, national defense, and homeland security needs of the busiest trade corridor between the United States and Canada (Figure S-1).

Figure S-1: Detroit River International Crossing Study, Existing Detroit River International Crossings

Source: The Corradino Group of Michigan, Inc

The DRIC Study Draft Environmental Impact Statement (DEIS), previously released under separate cover, analyzes issues/impacts on the U.S. side of the crossing system over the Detroit River between Detroit, Michigan, and Windsor, Ontario, Canada. The alternatives are comprised of three components: the crossing, toll and customs plaza, and interchange connecting the plaza to I-75 (Figure S-2).

Figure S-2: Detroit River International Crossing Study, U.S. Area of Analysis for Crossing System

Source: The Corradino Group of Michigan, Inc.

Geotechnical Investigation Background

The initial geotechnical task performed by NTH for this project (Task 2330) was to study the Illustrative Alternative crossing locations from Belle Isle to the tip of Grosse Ile, including collecting the relevant available geotechnical data along the proposed project area and evaluating the data with respect to conceptual designs. The results of this task were presented in a report entitled Geotechnical Evaluation, Proposed Detroit River International Crossing, Task 2330, dated December 28, 2005.

After consideration of the available data, it became apparent that historical brine wells and associated cavities exist in the vicinity of the proposed bridge, but the exact locations of the historical solution mining were not known and the potential impacts to the proposed construction were not understood. As a result, the project team developed a Brine Well Cavity Investigation program (Brine Well Program) to delineate the size, locations, and shape of potential brine well cavities and to evaluate the possible impacts of such cavities. The Brine Well Program included the two proposed crossing corridors on the U.S. side of the river. An investigation plan was proposed and completed, utilizing a combined geophysical and geotechnical program including the drilling of multiple deep rock borings, performed in combination with cross-well seismic imaging. Forward modeling of geophysical methods in conjunction with preliminary historical rock mechanics analysis, borehole gravity, vertical seismic profiling (VSP), and downhole wireline logging were also included in the program. This report was entitled “Brine Well Cavity investigation Program Technical Report”, and was presented as a portion of the DRIC Draft Environmental Impact Statement (DEIS) in February 2008.

In a report entitled “Preliminary Draft Geotechnical Investigation Report” revised September 21, 2006, NTH summarized the historical data specifically relevant to proposed crossings corridors X-10 and X-11, and provided a summary of expected engineering and construction issues related to the crossing corridors. This current report is an update of the September 21, 2006 report, and includes site specific information and analysis based on our field investigation. The field portion of this investigation was conducted between April 24 and June 1, 2008 and consisted of drilling a total of eight test borings, followed by laboratory testing of samples, and analysis of conditions with respect to the proposed construction.

Purpose of the Report

The purpose of the report is to present the details of our investigation, and to provide study-level geotechnical analysis and recommendations with respect to the proposed construction concepts under consideration for the Detroit side of the proposed Detroit River International Crossing Bridge between Detroit, Michigan and Windsor, Ontario. This current report is not intended as a stand-alone document, and is intended for the use by the Corradino Group and Parsons Transportation to develop preliminary foundation concepts for the bridge design, as well as to evaluate probable construction costs and other impacts related to the project.

This report was developed to be placed in the Appendix of the complete Bridge Engineering Report, itself an attachment of the final DRIC Engineering report, which is part of the overall DRIC Environmental Impact Statement Report.

When this geotechnical investigation was undertaken (April 2008), two potential crossing corridors were under consideration, defined as Crossings X-10 and X-11 as shown on the attached Figure Nos. 1A and 1B, respectively, in Attachment A. The two subject crossings are in the same general vicinity, between the Ambassador Bridge, and Zug Island in southwest Detroit. The Crossing X-10 corridor generally consists of the area immediately north of Zug Island to historic Fort Wayne along the banks of the Detroit River. The Crossing X-11 corridor generally consists of the area along the banks of the Detroit River immediately north of historic Fort Wayne to the existing Mistersky Power Plant.

Crossing X-10 has now been selected as the preferred crossing alternative. While this report provides documentation of the investigation and data collected for both the crossing corridors, all evaluations and recommendations herein are made regarding Crossing X-10.

Executive Summary Conclusions

Based on the information gathered during this investigation, subsurface conditions appear variable in composition and thickness in the upper levels of the X-10 borings and become more consistent with depth as bedrock is approached. The subsoils generally consist of variable fill soils underlain by a thin fill layer of gravelly sand. Underlying the gravelly sand or fill is a relatively thick silty clay layer. The silty clay layer is underlain by clay or granular hardpan that extends to limestone and dolomitic limestone bedrock. The bedrock interface is generally characterized by a thin zone of low Rock Quality Designation (RQD) rock (<75%) that has intermittent layers of fragmented bedrock with gravel and silty clay. Underlying the low RQD bedrock at the interface, is competent (>75% RQD) limestone and dolomite bedrock extending to the explored depths.

Based on the results of the investigation, the existing fill deposits at both crossing locations are highly variable and are not considered suitable for support of any foundation elements.

The underlying silty clay or granular soils are not considered suitable for support of the heavy loading expected from primary or secondary bridge foundation elements, but may be sufficient for support of ancillary structures with light-to-moderate foundation loads. For the purposes of this document, primary foundation elements are defined as the main structural foundation for cable stay and suspension bridges and the anchorages for the suspension bridge. Secondary foundation elements are defined as foundation elements for the approach roadway piers. Ancillary structures include bridge approach elements such as retaining walls, signage foundations, etc.

Based on the overall evaluation of the subsurface data obtained during this investigation and consideration of the project background information, it is anticipated that deep foundation systems will be required to support primary and secondary bridge elements. Such systems may consist of large concrete elements cast within a deep shaft and bearing on sound bedrock, drilled straight-shaft concrete-filled caissons, or piles.

The hardpan soils underlying both corridors are considered well suited for the heavy foundation loading anticipated from proposed secondary structural elements of the bridge using deep foundation elements. The list below summarizes the nominal pile driving resistance values (R_{NDR}) for pipe piles and H-piles recommended in the Michigan Department of Transportation (MDOT) Bridge Design Manual (BDM). The dynamic resistance factor (ϕ_{DYN}) presented by the MDOT BDM is equal to 0.4, and assumes that pile driving criteria will be developed by using the Federal Highway Administration (FHWA) modified Gates Dynamic formula. We recommend considering the use of dynamic testing in developing driving criteria. If American Association of State Highway and Transportation Officials (AASHTO) guidelines for dynamic testing are followed, a dynamic resistance factor of 0.65 may be used instead of 0.4.

Pile	R_{NDR} (tons)
12” O.D. (0.25” wall)	175
14” O.D. (0.312” wall)	200
14” O.D. (0.438”wall)	250
HP12x53	200
HP14x102	400

If a drilled pier bearing on the hardpan is used, a nominal resistance value of 40 tons per square foot (3.8 MPa) can be used if a settlement of approximately 5% of the shaft end diameter is acceptable. A resistance factor of 0.55 should be used with the drilled shaft geotechnical design.

The upper, highly weathered bedrock (<75% RQD) underlying the hardpan soils is generally considered suitable for the heavy foundation loading anticipated from primary and secondary foundation elements of the bridge, although bearing capacities any higher than for the hardpan (as discussed above) are not recommended.

The competent bedrock (>75% RQD) underlying the hardpan soils and the weathered bedrock is well suited for the heavy foundation loading anticipated from primary and secondary foundation elements of the bridge.

The anticipated heavy foundation loading for proposed primary foundation elements may involve massive elements cast within circular concrete shafts or drilled concrete piers (also known as drilled caissons). Such foundation elements would be designed to extend through the upper fill, silty clay, granular soil layers, hardpan soils, and be founded on competent bedrock at least 5 feet (1.5 meters) into the competent limestone/dolomite bedrock, resulting in depths of approximately 100 to 120 feet (30 to 34 meters). Estimated load-settlement behavior is provided for drilled pier diameters of 2.5 meters (8.2 feet) and 3.3 meters (10.8 feet) at rock socket lengths of 5 feet (1.52 meters), 10 feet (3.05 meters), and 15 feet (4.57 meters). The ultimate nominal end resistance is approximately 300 tsf (28.7 MPa) while the ultimate nominal shaft side resistance in the bedrock is approximately 10.6 tsf (1.0 MPa). However, because the skin friction mobilizes at small strain, while the end resistance mobilizes at large strain, the ultimate values should not be summed to estimate the total resistance. Rather, a load resistance factor design (LRFD) procedure is outlined to estimate the total resistance that accounts for strain incompatibility. For the evaluation presented herein, an end resistance factor of 0.5 and a shaft side resistance factor of 0.65 are recommended, based on AASHTO and FHWA guidelines. If during final design, shaft side and end resistance values are obtained through the use of field load tests, the resistance factor for both end and shaft side resistance can be increased to 0.8.

Pipe piles to support the suspension bridge anchorage and/or main towers were also evaluated and could consist of 30-inch diameter reinforced concrete filled steel pipes. The pipe piles would be pre-drilled and driven to bear on or immediately above the bedrock, a reinforcing steel cage would then be placed within each pile, and then filled with concrete. For the concept design, it can be assumed that the bedrock end bearing resistance will be mobilized within a settlement of up to 5% of the pipe diameter, which will occur primarily as elastic settlement.

The nominal pile driving resistance values for vertical and battered (3V:1H) 30-inch pipe piles is summarized below. The values assume plugged conditions at the pile tip. The MDOT BDM presents a dynamic resistance factor (ϕ_{DYN}) equal to 0.4, which assumes that pile driving criteria will be developed by using the FHWA-modified Gates Dynamic formula. We recommend considering the use of dynamic testing in developing driving criteria. If AASHTO guidelines for dynamic testing are followed, a dynamic resistance factor of 0.65 may be used instead of 0.4.

	R_{NDR} (tons)		
Pile	Axial	Vertical Comp.	Horizontal Comp.
30” O.D. (0.625”)	990	939	312

This summary is general in nature and should not be considered apart from the entire text of the report with all the qualifications and considerations mentioned herein. All interpretations are for United States (US) side only and for Crossing X-10. It is also noted that the analysis and interpretations herein are with respect to the general feasibility and concept design for the bridge foundations. It is understood that the once the final design is undertaken, a more detailed geotechnical investigation and analysis will be conducted that will include additional test borings and laboratory testing. The additional investigation will consist primarily of additional soil and rock test borings (vertical and angled rock coreholes), in situ rock testing, in situ permeability testing for rock grouting design (if determined to be necessary), and additional rock core laboratory testing.

1.0 INTRODUCTION

The Detroit River International Crossing (DRIC) Study is a bi-national effort to complete the environmental study processes for the border crossing between Detroit, Michigan and Windsor, Ontario for the United States, Michigan, Canada and Ontario governments. The study proposes solutions that support the region, state, provincial and national economies while addressing civil national defense and homeland security needs of the busiest trade corridor between the United States and Canada (Figure 1-1).

Figure 1-1: Detroit River International Crossing Study, Existing Detroit River International Crossings

Source: The Corradino Group of Michigan, Inc.

The DRIC Draft Environmental Impact Statement (DEIS), previously released under separate cover, analyzes issues/impacts on the U.S. side of the border of the end-to-end crossing system over the Detroit River between Detroit, Michigan, and Windsor, Ontario, Canada. The alternatives are comprised of three components: The crossing, plaza (where tolls are collected and Customs inspections take place), and interchange connecting the plaza to I-75 (Figure 1-3). Figure 1-3 illustrates the approximate location of the Canadian sinkhole relative to the Detroit River International Crossing study.

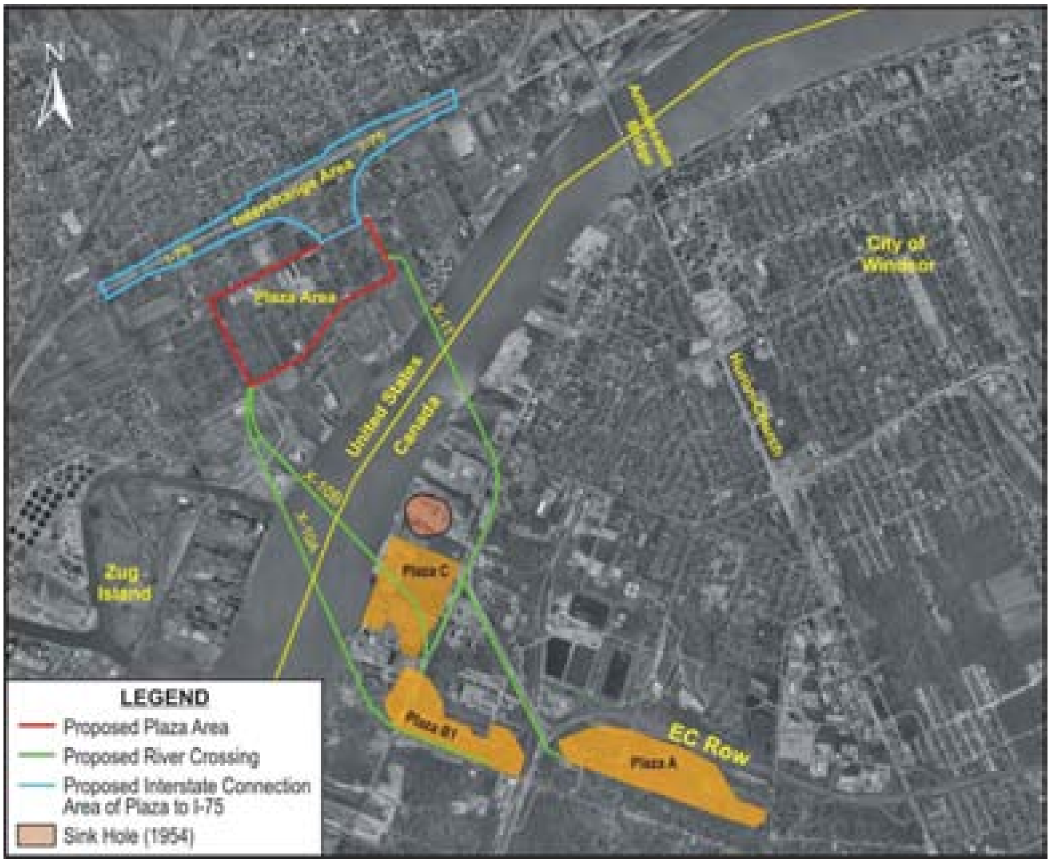


Figure 1-3: Detroit River International Crossing Study, Canadian Sinkhole

Source: URS Canada

1.1 PURPOSE OF THE REPORT

The purpose of the report is to present the details of our site specific investigation, and to provide study-level geotechnical analysis and recommendations with respect to the proposed construction concepts under consideration for the Detroit side of the proposed Detroit River International Crossing Bridge between Detroit, Michigan and Windsor, Ontario. This current report is not intended as a stand-alone document, and is intended for the use by the Corradino Group and Parsons Transportation to develop preliminary foundation concepts for the bridge design, as well as to evaluate probable construction costs and other impacts related to the project.

This report was developed to be placed in the Appendix of the complete Bridge Engineering Report, itself an attachment of the final DRIC Engineering report, which is part of the overall DRIC Environmental Impact Statement Report.

When this geotechnical investigation was undertaken (April 2008), two potential crossing corridors were under consideration, defined as Crossings X-10 and X-11 as shown on the attached Figure Nos. 1A and 1B, respectively, in Attachment A. The two subject crossings are in the same general vicinity, between the Ambassador Bridge, and Zug Island in southwest Detroit. The Crossing X-10 corridor generally consists of the area immediately north of Zug Island to historic Fort Wayne along the banks of the Detroit River. The Crossing X-11 corridor generally consists of the area along the banks of the Detroit River immediately north of historic Fort Wayne to the existing Mistersky Power Plant.

Crossing X-10 has now been selected as the preferred crossing alternative. While this report provides documentation of the investigation and data collected for both the crossing corridors, all evaluations and recommendations herein are made regarding Crossing X-10.

2.0 SITE CONDITIONS

The two subject crossings are in the same general vicinity, between the Ambassador Bridge and Zug Island in southwest Detroit, and are described as follows:

2.1 X-10 CROSSING CORRIDOR

The Crossing X-10 corridor generally consists of the area immediately north of Zug Island to historic Fort Wayne along the banks of the Detroit River. The area is generally flat with a slight drop in elevation at the river, with large vacated areas, parking lots, and paved/unpaved roads. Current land use includes light-to-moderate industrial areas, including a cement terminal, a major trucking terminal, truck ferry operation, and aggregate storage areas. Residential areas exist north of Jefferson Avenue, but are generally intermingled with light commercial and industrial uses. Historic land use includes light-to-heavy industrial areas, including a major chemical processing plant and power plant operations, along with two suspected solution well operations identified during the literature search portion of the Brine Well Program. Known solution mining wells exist outside the influence zone of the crossing X-10 primary and secondary bridge elements; adjacent to the Rouge River along the south portion of the corridor, as well as possible undocumented solution mining wells adjacent to the current Fort Wayne property. Historic maps also indicate the original shoreline of the Detroit River to be set back approximately 16 to 80 feet (5 to 24 meters) from its current position, with potentially abandoned and buried docks and former boat slips prevalent throughout.

The X-10 Crossing bridge foundation borings (TB-101 through TB-107) were located on the former Detroit Coke site, which was used for coke oven and coke oven gas by-products operations from early 1900 until 1991. The project site has been the subject of environmental investigations performed by others not related to this project. The property is now subdivided between several owners including Lafarge, Inc., McCoig Concrete (Koenig/Michigan Foundation, Inc.), the Detroit Economic Growth Corporation (DEGC), and Yellow Trucking, Inc.

Boreholes TB-102 through TB-104 are located on the Lafarge North America Detroit Cement Terminal facility located at 1301 Springwells Court along the Detroit River northeast of the McCoig aggregate facility. The Lafarge property consists primarily of open land with a storage silo and related structures adjacent to the Detroit River.

The remaining X-10 Crossing boreholes, TB-101 and TB-105 through TB-107, were drilled on property that is currently vacant land owned by the DEGC. This property is sparsely vegetated and mostly open, with several overhead and underground utilities.

2.2 X-11 CROSSING CORRIDOR

Although the X-11 corridor is no longer under consideration for this project, one of the test borings was drilled in this area, and as such, some site information is provided.

The Crossing X-11 corridor generally consists of the area along the banks of the Detroit River immediately northeast of historic Fort Wayne and southwest of the existing Mistersky Power Plant; between Jefferson Avenue and the river. The area is generally flat and vacant, with a slight drop in elevation at the river. Current land use in the immediate area includes light to moderate industrial regions, the Mistersky power generation facilities, and some residential use to the north. The residential areas exist north of Jefferson Avenue and are generally intermingled with light commercial and industrial areas. Historic land use includes light to heavy industrial areas, including a major copper and brass fabrication operation, along with two suspected solution well operations identified during the literature search portion of the Brine Well Program. Historic maps indicate that potential solution mining operations exist directly to the north of the historic copper and brass fabrication facility and the northern portion of the corridor, in what is now intermingled residential and commercial areas. Based on the results of the brine well investigation, it has been confirmed that no brine wells exist within the influence zone of the primary elements of the proposed bridge alignment. Historic maps also indicate the original shoreline of the Detroit River in the X-11 area to be set back approximately 10 to 50 feet (3 to 15 meters) from its current position, with possible docks and former boat slips prevalent throughout.

Test boring TB-108 was drilled on the former Revere Copper and Brass (Revere) site located at 5851 West Jefferson Avenue. The Revere site exists as vacant, unoccupied land with some lightly wooded areas south of West Jefferson, to the southeast of the Mistersky Power Plant. Previous environmental investigations conducted by others not related to this project have identified the Revere site as impacted with the byproducts of former industrial use.

3.0 PROJECT BACKGROUND

3.1 REGIONAL SALT AND SOLUTION MINING ACTIVITIES

Salt (halite) has historically been solution mined in the area of the X-10 Crossing Corridor. As part of this solution mining process, fresh water was injected into the ground, natural salt beds were dissolved, and the resulting brine was brought to the surface and evaporated to make salt. The solution mining of salt layers ranging from approximately 900 to 1,600 feet (270 to 500 meters) below the ground surface was typically conducted in an uncontrolled method before standardized record keeping was common practice. This created underground cavities of unknown location, size, and shape. A solution mining cavity collapsed to the surface and formed a sinkhole on the Windsor side of the study area in 1954. At least two additional sinkholes occurred at Point Hennepin (on Grosse Ile) south of the DRIC crossing site on the U.S. side of the river. Also, settlement of several feet was observed near the Wyandotte, MI brinefield location to the south of the X-10 Corridor.

3.1.1 Brine Well Program

The Brine Well Program was developed to delineate the size, locations, and shape of potential brine well cavities in the X-10 and X-11 Crossing corridors on the U.S. side of the river. Approval was obtained from MDOT for a combined geophysical and geotechnical program, which included the drilling of 13 deep-rock borings in combination with cross-well seismic imaging. Forward modeling of geophysical methods in conjunction with preliminary historical rock mechanics analysis, borehole gravity, vertical seismic profiling (VSP), and downhole wireline logging were also included in the program.

Based on criteria established by MDOT in January 2006 and further defined at a June 2006 Geotechnical Advisory Group meeting, the proposed bridge in Corridors X-10 and X-11 requires: 1) foundations be located outside of the influence of any rock cavities that could have impact on the foundations, including those produced by solution mining activities; and, 2) foundations be built on competent bedrock. The brine well investigation program was developed and implemented to define conditions in the corridors to determine if these criteria could be satisfied.

A total of 12 cross-well seismic imaging profiles were performed for the X-10 corridor. The processed cross-well images and other geophysical data show two anomalies of interest, neither of which are of significant concern. A report detailing the findings was presented under separate cover in the DRIC DEIS, as mentioned previously.

3.1.2 Rock Mechanics Investigation Results

A preliminary model of geotechnical rock mass characteristics was also completed as part of the Brine Well Program to evaluate the potential instability of possible solution cavities of similar shape and size of anomalies discovered during the geophysical investigation program. The evaluation was also based on review of the historical instability of existing solution cavities in the Detroit-Windsor vicinity and on the results of a three-dimensional, distinct-element (3DEC) analysis of suspected or potential solution cavity geometry.

3.1.3 Combined Geophysical Investigation and Rock Mechanics Results

Based on the observations made during the deep drilling and subsequent geophysical investigations, there is no evidence of cavities of concern in the X-10 Crossing Corridor, nor evidence of potential instability of the rock mass. In fact, the analysis shows that the observed anomalies have probably been filled by one or a combination of several mechanisms. In addition, even for the largest of the anomalies located, and assuming an unfilled cavity, the analysis shows the anomaly is stable and will not progress upward any significant distance to affect a nearby bridge foundation.

3.2 SUMMARY OF ENVIRONMENTAL CONDITIONS IN X-10

Based on experience along the Detroit River shoreline and within the Detroit River sediments, some environmental issues will be present for any excavations along the United States shorelines and within the upper 5 to 10 feet (1.5 to 3 meters) of river sediment. Along the shoreline, fill soils to depths of 5 to 30 feet (1.5 to 9 meters) from previous activity are typically contaminated requiring disposal in Type II landfills. Within the river, sediments along the river bottom are also typically contaminated increasing in risk and contamination levels especially south of the downtown Detroit area.

The former Detroit Coke Site, originally owned by the Solvay Processing Company (Solvay), occupies most of the Crossing X-10 landing area between Jefferson Avenue and the Detroit River. The Detroit Coke Site was used for coke oven and coke oven gas by-products operations from early 1900 until 1991. Due to the presence of regulated deep underground injection wells in the western part of the property, it was also identified as a Resource Conservation and Recovery Act (RCRA) facility. Associated environmental impacts with the coke oven and coke oven gas by-products operations included tar, free phase hydrocarbons (free product), and soil and groundwater contamination. Almost the entire site has been impacted by the former industrial operations.

Previous investigations by others have indicated site soils are contaminated with volatile organic compounds (VOCs), semi volatile organic compounds (SVOCs), ammonia, cyanide, and metals at concentrations exceeding the Michigan Department of Environmental Quality (MDEQ) industrial criteria for indoor and ambient air, direct contact, particulate inhalation, and surface water protection. Site groundwater is contaminated with VOCs, SVOCs, ammonia, cyanide, and metals at concentrations exceeding the MDEQ industrial criteria for indoor air, direct contact, and surface water protection. Previous investigations have also indicated significant soil and groundwater contamination, including possible free phase coal tar. The underlying clay layer vertically confines the contamination (aquitard). The Michigan Department of Environmental

Quality (MDEQ) and the former site owner (who is under a consent decree to implement remedial actions) have both expressed concern of the potential for future construction to allow existing contamination to vertically migrate through the aquitard.

Honeywell, the current owner of the Detroit Coke Site and the RCRA primary responsible party, has installed a demarcation membrane in certain areas, and approximately 6 to 12 inches (15 to 30 centimeters) of clean fill material has been placed over the membrane to prevent contact with the impacted soil. However, this membrane and clean fill layer may not be present throughout the entire site as observed during field work activities. Honeywell has also installed groundwater collection trenches to limit impacted groundwater from discharging to the Rouge River and/or Detroit River.

The site may also have been used as a brine well processing facility, without any documented environmental impacts attributed to that operation.

3.2.1 X-10 Disposal Wells

Research for Brine Well Investigation Program uncovered the existence of three previously operated deep-injection disposal wells on the former Solvay/Honeywell (Crossing X-10) parcel. The wells were drilled from 1969 to 1978 to depths of greater than 4,000 feet (1,219 meters). The wells were used to inject hazardous waste into permeable formations (Munising Group) deep within the ground. Wells #1 and #3 were plugged and abandoned according to the Michigan Department of Environmental Quality (MDEQ) and court records, resolving how the operators of the hazardous waste injection operation were prosecuted for illegal activities. Well #2 was reportedly plugged during the winter of 2008, according to the Michigan Department of Environmental Quality (MDEQ). These former deep-injection disposal locations are presented on Figure No 1A in Attachment A.

3.3 REGIONAL GEOLOGIC SUMMARY

The generalized subsurface geology for the area is summarized in this section of the report.

3.3.1 Overburden

The bedrock in the project corridors is overlain by glacial drift soils, which have been deposited either directly by glacial ice (till), glacial meltwater streams (glaciofluvial deposits), or impounded glacial lakes (lacustrine deposits). The upper soil formations along the alignment generally consist of a relatively thick mantle of Wisconsin-aged lacustrine clays (10,000 to 50,000 years ago) that, with the exception of the near-surface deposits are typically very soft to soft in consistency. The lacustrine soils were deposited as sediments from a series of glacial lakes impounded between the ice front and the Inner Defiance Moraine located near the northwest corner of Wayne County. The upper 10 to 20 feet (3 to 6 meters) of these deposits (where still present) have been desiccated during historical low-water periods, resulting in soils of very stiff to hard consistency near the surface. The clay soils frequently contain intermittent sand and gravel layers that were produced from glacial rivers carrying coarser sediments as lake levels fluctuated. Localized alluvial soils are present along existing rivers and streams that drain the inland areas. In some locations, lake shorelines are identified by relatively thick layers of sand and gravel.

The lacustrine deposits are typically underlain by a thin layer of highly over-consolidated glacial till, generally consisting of sand, silt, and gravel within a matrix of clay. This formation is locally termed “hardpan” and usually overlies the bedrock formation. Depending on the amount of clay binder contained in the hardpan, the material may range in nature from cohesive to granular. The hardpan is generally believed to be from the Illinoian Ice Age (200,000 years ago) and can also contain calcium carbonate producing a cemented condition. Given the glacial origins of the hardpan layer, occasional cobbles and large boulders are typically present in this layer. Methane and hydrogen sulfide gas may also be encountered in this layer.

The total glacial drift thickness along the X-10 Crossing Corridor varies from approximately 94.5 to 99 feet (Elevations 483 to 494 feet; or 28.8 to 30.2 meters). The surface topography was formed during the Wisconsin stage (youngest) of Pleistocene Series glaciations of the Cenozoic Era, and has been somewhat modified by surface erosion since that time.

3.3.2 Hydrogen Sulfide Occurrence and Project History

Hydrogen sulfide is a colorless gas that smells like rotten eggs and can paralyze the sense of smell (olfactory paralysis). Olfactory paralysis then prevents the recognition of its presence. Hydrogen sulfide in the atmosphere can blacken exposed materials and irritate the eyes, causing them to become swollen or very sensitive to light.

Hydrogen sulfide gas has a history of occurrence in the DRIC Crossing areas. On many recent and historical projects, the gas has caused toxic conditions during deep excavation and tunneling operations, even causing death to construction workers in some cases.

The Southwest Intake Rock Tunnel was constructed in 1957 as part of a water intake system for the southern portion of Wayne County. The tunnel was constructed within the bedrock from an intake structure located in the Detroit River at the middle portion of Fighting Island approximately 4.5 miles (7.2 kilometers) downriver from the X-10 bridge alignment. From the intake structure, the tunnel was mined to a shore shaft on the west bank of the Detroit River approximately 1 mile (1.6 km) away. The tunnel consists of a 12-foot (3.7 m) finished inside diameter rock tunnel which varies in depth from approximately 127 feet at the intake structure to 176 feet at the shore shaft, corresponding to Elevation 445 to 400 feet (38 m to 65 m, corresponding to Elevation 135 m to 122 m). The tunnel is located approximately 65 to 115 feet (20 to 35 m) below the rock/soil interface at the intake and shore shafts, respectively. The tunnel was constructed using drill-and-shoot methods to excavate the rough opening.

Based on discussions with an individual who worked on the project, the major problems encountered during construction were the presence of hydrogen sulfide gas together with large inflows of groundwater. During construction, the contractor’s personnel were reportedly required to wear gas masks. Further, due to the highly fractured nature of the bedrock, inflows of groundwater and hydrogen sulfide required significant grouting activities during the construction. These efforts, although not entirely effective, provided sufficient control to complete the construction of the rock tunnel. It should be noted that during construction of the related shore tunnel to the west of the rock tunnel, a worker died after being overcome by hydrogen sulfide gas.

Several other projects that involved excavation into the bedrock have been constructed within 1 to 2 miles (1.6 to 3.2 km) of the project site, mostly related to the nearby Detroit Water and Sewerage Department's wastewater treatment plant. These projects include the Pump Station 2A project (1991), Pump Station 1 project (1960s), and the DRO2 project (construction terminated in 2004 due to catastrophic inflow of hydrogen sulfide laden groundwater into the unfinished tunnel). For each of these projects, dissolved sulfide levels within groundwater were reported to be 80 ppm or higher. It is valuable to note that contaminated water inflow was controlled effectively by pre-excavation grouting of the bedrock. In the case of the DRO2 project, the uncontrolled inflows occurred during tunneling in which grouting was performed during the excavation (i.e., pre-excavation grouting was not performed).

3.3.3 Bedrock

The proposed crossing corridor is located at the geologically-termed southeast margin of the Michigan Basin geomorphic province and within the Erie-Huron glacial lowland. The Michigan Basin is termed as such due to the structural basin shape of the bedrock, in which layers of Paleozoic era sedimentary rock that overlay the Precambrian Basement Complex, dip inwards to the center of the Lower Peninsula of Michigan from each direction as a series of bowls. The youngest layers of bedrock are first encountered in the center of the state, with older rock layers progressing outwards to the outer margins.

The Michigan Basin was initially formed during the early Cambrian Period, when the remnants of the mountains formed during the Cambrian-Penokean Orogeny remained in a belt extending from Ontario, Canada, across the central part of the Upper Peninsula to present-day Wisconsin. The erosion of these "northern highlands" began the series of depositions and erosions that constitute the modern basin. The later effects of the Appalachian Orogeny likely caused the structural deformation and localized downward movement in what had been a relatively stable interior continental region.

As a result, several intracratonic structural basins were formed throughout the central lowland areas of North America creating arches and domes. The Michigan Basin is bounded on the west by the Wisconsin Arch and Wisconsin Dome; on the north and northeast by the Canadian Shield; on the east and southeast by the Algonquin Arch in Ontario and the Findlay Arch in Ohio; and by the Kankakee Arch in northern Indiana and Illinois.

Based on the position of Detroit, Michigan, along the southeast rim of the Michigan Basin, the Paleozoic rocks that comprise the basin in this area typically dip to the northwest, with each formation being buried by successive younger formations in the direction of the dip. The regional dip is slight, and is estimated at approximately 30 to 50 feet per mile (6 to 10 meters per kilometer).

The topography of the bedrock surface within the area is somewhat variable and characterized by numerous irregular features in the bedrock surface. These features include many synclinal and anticlinal structures believed to have developed before the Pleistocene Epoch and subsequently modified by repetitive glacial action. The bedrock features also include the existence of ancient stream valleys and numerous healed faults that cut the bedrock surface. Based on historical information, the bedrock features are understood to be fairly broad, and become narrow as they reach the terminus of the Erie/Huron Lowlands.

Due to the movement of the earth’s crust, these strata are seamed and fissured with vertical and horizontal joints that permit movement of ground water. Where carbon dioxide dissolved within the groundwater-filled cracks, solution cavities typically developed within the limestone, and to some degree within the dolomite. Both the limestone and dolomite formations are known to contain dissolved sulfides, which can produce hydrogen sulfide gas upon exposure to atmospheric conditions. The natural decay of organic compounds that also existed within the ancient seas became trapped within cavities formed in the limestone and dolomites and is evident today as petroleum, carbon monoxide, and methane. Small amounts of petroleum found within the limestone and dolomite tend to cause discoloring, staining, and associative odors.

3.4 GROUNDWATER

The near surface granular deposits and fill layers in the Detroit area typically contain groundwater, which is perched above the underlying clay strata. This groundwater forms an intermittent unconfined aquifer, which varies seasonably in depth and extent. In addition, confined groundwater is often contained within relatively thin granular layers that are occasionally present within the thick cohesive deposits and / or hardpan present throughout the corridor areas. Such confined aquifers are usually limited in extent, and therefore, have limited recharge capabilities. However, surficial granular layers near the Detroit River shoreline can obtain hydraulic communication with the river, sometimes requiring extensive dewatering programs as discussed later during the restoration of the TB-108 location.

Groundwater in the X-10 Crossing Corridor can typically be distinguished according to its chemical constituency and can be sub-divided into fresh and mineral in the explored depths.

3.4.1 Fresh Groundwater

Fresh water is potable and is free of any deleterious, naturally-occurring chemicals or dissolved salts or solids. Fresh water aquifers generally exist in the upper glacial drift. In the project area, the fresh water aquifer is discontinuous, and often contaminated as a result of human activities. Where the fresh water aquifer is present in the glacial drift, groundwater generally flows toward the Detroit River, which generally behaves as a regional discharge feature. The deepest freshwater aquifer in the explored area is considered to be the base of the glacial drift.

3.4.2 Mineral Groundwater

Mineral water contains dissolved minerals or constituents that may alter to gas upon being exposed to the atmosphere. The dissolved compounds of interest expected for this investigation consisted of hydrogen sulfide, methane, and carbon monoxide which exist naturally in some mineral ground waters. Mineral waters are common in the lower glacial drift (hardpan) and upper Middle Devonian bedrock (Dundee Limestone and Detroit River Group - Lucas Formation). In the project area, mineral waters are found in the hardpan and bedrock, and typically exhibit flowing artesian conditions when encountered.

3.5 REGIONAL SEISMOLOGY

According to historical seismic risk maps published by the United States Geodetic Survey, Michigan is located within Seismic Risk Zone No. 1 and, as such, posses a relatively low risk for earthquake occurrence. While tremors from earthquakes with epicenters in other regions have been recorded in Michigan, only 34 earthquakes with epicenters in Michigan have been recorded since 1872. With the exception of two seismic events that occurred in the Keweenaw Peninsula at the turn of the 20th century, all recorded events had recorded intensities of less than IV on the modified Mercalli scale. This corresponds to approximately magnitude 4.7 on the Richter scale.

According the Geologic Survey Division of the Michigan Department of Environmental Quality, the majority of the above referenced seismic events resulted in slippage along deep-seated Pre-Cambrian Faults and is not believed to involve slippage along the faulting of the overlying Paleozoic units.

4.0 PROPOSED CONSTRUCTION

The geotechnical investigation was planned and carried out to provide a general understanding of the feasibility and concept-level design requirements for bridge concepts as presented in the Detroit River International Crossing, Bridge Conceptual Engineering Report, Revised February 2008 by Parsons Transportation.

4.1 BRIDGE CONCEPTS

The project team has developed the crossing concept as a three-lane each way (for a total of 6 lanes with shoulders and a central median) clear-span bridge with an anticipated clearance of approximately 133 feet (40.5 m) at the river's edge. The bridge is anticipated to be engineered for restriping from six to eight lanes with a center median. The overall structure is designed to achieve a 120-year structure life, with replaceable components being designed for those not able to achieve the intended lifespan.

Given the navigational requirements, the bridge is anticipated to be a suspension bridge or cable-stayed bridge with primary piers on or near the shoreline. For the purposes of this document, primary foundation elements are defined as the main structural foundation elements for cable stay and suspension bridge towers. Primary piers would most likely be supported on drilled concrete piers (also termed "drilled caissons").

Suspension bridge anchorages would be located approximately 1,000 to 1,500 feet (300 to 450 meters) behind the primary piers, which would be located at or near the river's edge. Foundations for suspension bridge anchorages would be on competent bedrock. These foundations could be constructed as large-diameter sunken caissons.

Secondary foundation elements would consist of concrete piers supported on deep foundations. Secondary foundations are defined as elements for the approach roadway piers to the bridge. The final spacing of the secondary foundation elements would be developed during the final design, but for the purposes of the concept evaluation, it is anticipated that these elements would be approximately 100 to 200 feet (30 to 60 meters) apart.

4.1.1 Suspension Bridge

For a suspension bridge, the main cables are constructed between two large towers or piers and are anchored to massive anchorage structures often founded on bedrock. These cables form the primary load-bearing structure for the bridge deck. The cables are under tension from only their own weight before the deck is placed. Suspender ropes support the deck from the main cables. The tension on the cables is then transferred to the earth via the anchorages.

A diagram of a typical suspension bridge can be found below in Figure No. 4-1 and also in the attached Figure No. 38 in Attachment A.

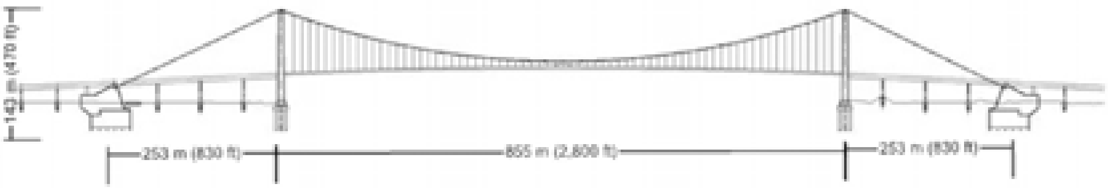


Figure No. 4-1: Typical Suspension Bridge Structure
(Modified from Bridge Conceptual Engineering Report, Rev Feb 2008 by Parsons)

4.1.1.1 Currently Proposed Alignment X-10 Suspension Bridge

The proposed Detroit side pylon is approximately 463 feet (141 m) tall with respect to the top of footing and is located on land adjacent to the existing rail spur currently servicing the Lafarge Terminal. The main span consists of an approximate 2,800-foot (855 m) suspended deck, with a US-side approach backstay span of nearly 830 feet (253 m). The planned main anchorage is located to the north of Springwells Court, on the Lafarge property, and is proposed to resist the suspension cable pull through a combination of dead weight, passive soil resistance, and direct load transfer to underlying bedrock.

4.1.2 Cable-Stayed Bridge

In the cable-stayed structure, the main piers or pylons are considered the primary foundation elements. The bridge deck is supported by steel cables running directly from the deck structure to the towers. The pylons are currently envisioned in two forms, namely an A-Frame and Inverted Y shape to generally limit second order effects and to increase resistance to wind forces. A diagram of a typical cable-stayed bridge is presented below in Figure No. 4-2 and also in the attached Figure No. 37 in Attachment A.

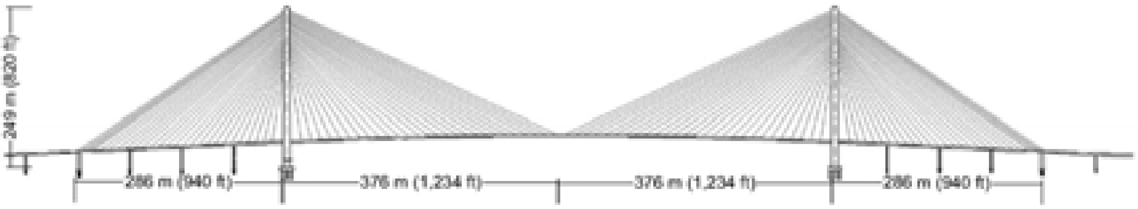


Figure No. 4-2: Typical Cable-Stayed Bridge Structure
(Modified from Bridge Conceptual Engineering Report, Rev Feb 2008 by Parsons)

4.1.2.1 Currently Proposed Alignment X-10 Cable-Stayed Bridge - This option consists of a 2,760-foot (840 m) main span with symmetric 1,050-foot (320 m) side spans. The heavier concrete box girder allows the side spans to be shorter than one half the main span length and act as counterweights when loaded with traffic, effectively eliminating uplift on the anchor piers. Two pylon configurations have been proposed, with both extending approximately 820 feet (250 m) above their footings. The proposed pylon is located on land adjacent to the existing rail spur currently servicing the Lafarge Terminal.

4.2 FOUNDATION CONCEPTS

Similar to the bridge concepts, the Bridge Conceptual Engineering Report developed general concepts for foundations for both the general bridge types being proposed.

4.2.1 Suspension Bridge Foundation Concepts

As discussed previously, the suspension bridge concept includes two major types of primary foundation elements; pier foundations supporting the main towers; and anchorages resisting the tensile forces from the main cables.

4.2.1.1 Suspension Bridge Towers

The proposed tower foundations generally consist of drilled piers and a concrete footing (or pier cap) at the base of each tower leg. A tie-beam then connects the two adjacent pier caps. The footings typically consist of regularly reinforced mass concrete and are generally poured in a single monolithic pour at each tower location. A diagram of a typical tower foundation is presented below in Figure No. 4-3.

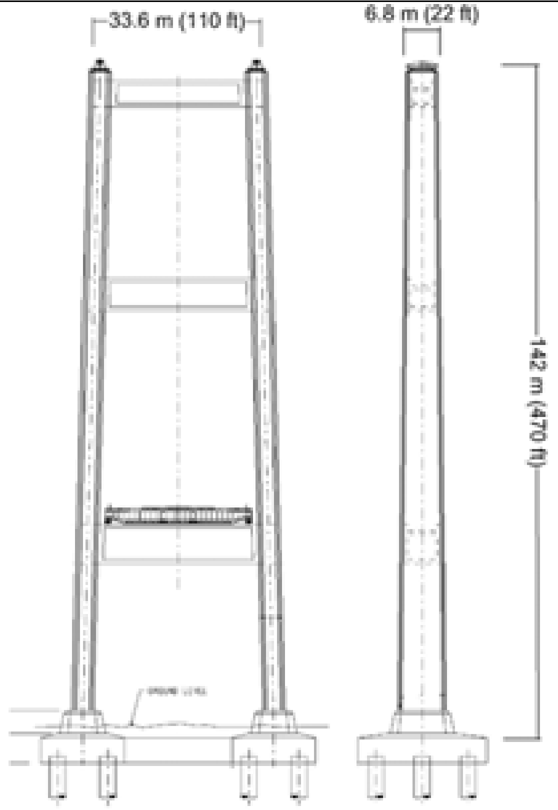


Figure No. 4-3: Typical Suspension Bridge Tower Configuration
(Modified from Bridge Conceptual Engineering Report, Rev Feb 2008 by Parsons)

4.2.1.2 Suspension Bridge Anchorages

The proposed suspension bridge anchorages for this project consist generally of a mass concrete anchorage block with a splay chamber for each cable where it is secured to anchor rods. These anchorages are gravity-type anchorages, which use the dead weight of the concrete to resist the pull of the main cables. The anchorages are generally assumed to be founded on bedrock with longitudinal resistance to the main cable provided by direct transfer to the bedrock and conservative estimates of passive soil and rock resistance. An example of a typical suspension bridge anchorage is presented below in Figure No. 4-4.

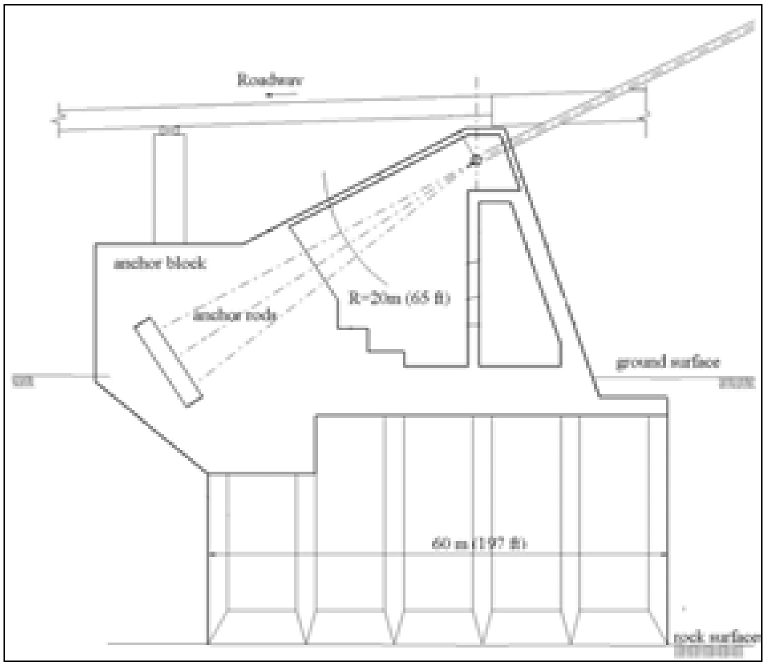


Figure No. 4-4: Typical Suspension Bridge Anchorage Structure

(Modified from Bridge Conceptual Engineering Report, Rev Feb 2008 by Parsons)

4.2.2 Cable-Stayed Bridge Foundation Concepts

The Bridge Conceptual Engineering Report considered the following general foundation types for the cable stayed bridge option.

4.2.2.1 Pylon Foundations

The proposed pylon foundations generally consist of drilled piers and a footing (pier cap) at the base of each tower leg. A tie beam then connects the two adjacent pier caps. The footings typically consist of regularly reinforced mass concrete. A diagram of a typical pylon foundation is presented below in Figure No. 4-5.

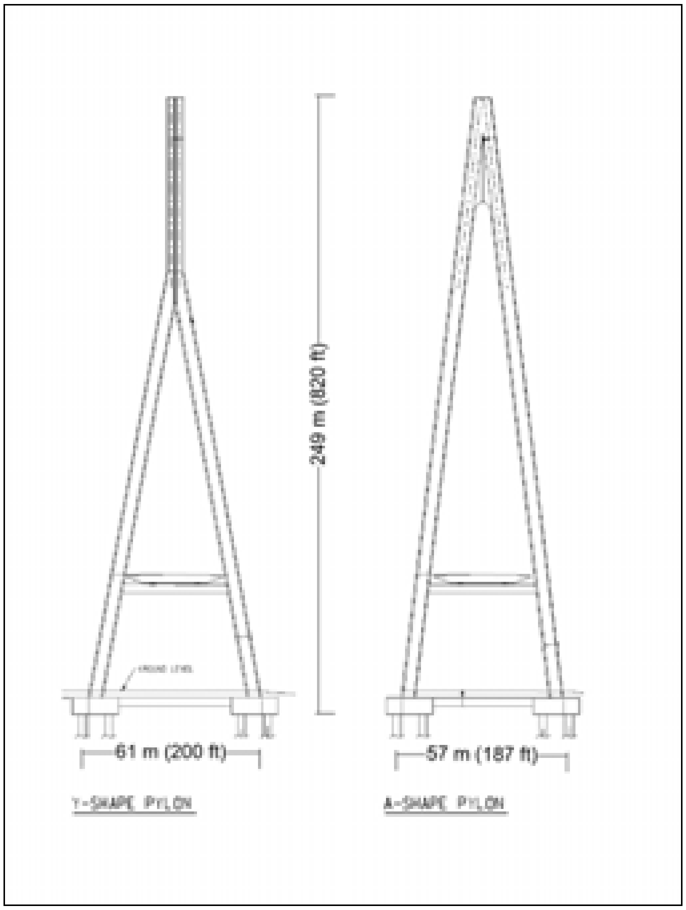


Figure No. 4-5: Typical Cable-Stayed Bridge Pylon

(Modified from Bridge Conceptual Engineering Report, Rev Feb 2008 by Parsons)

4.2.2.2 Anchor Piers

Conceptually, anchor piers for a cable-stayed bridge system consist of drilled shafts with a respective cast in place footing or pier cap. At this time, the footing is envisioned to be reinforced concrete and located entirely below grade. The subsequent pier is constructed using solid cast in place concrete columns with multiple lifts and splicing of column reinforcing steel. This type of anchorage is much smaller than for the suspension bridge concept.

4.2.3 Approach Piers

Approach piers for the roadway are independent of chosen span type and are currently envisioned to be constructed similar to the procedure described in Section 4.2.1.1.

Generally, the piers are founded on deep foundations consisting of drilled caissons (piers) resting on or socketed into bedrock (possibly hardpan soils depending on anticipated loading). Drilled shafts are conceptually 5 to 10 feet (1.5 to 3.0 m) in diameter depending on the anticipated loading.

5.0 FIELD INVESTIGATION

The field portion of this investigation was conducted between April 24 and June 1, 2008 and consisted of drilling a total of eight test borings. Prior to drilling, clearances for underground utilities were obtained through the Miss Dig System. The test borings were designated as TB-101 through TB-108. The test borings were drilled to depths varying between 112.5 and 152.5 feet (34.3 and 46.5 meters), under the full time supervision of NTH field staff in liaison with the project engineer. The test boring locations were selected by Parsons Transportation Group and located in the field based on NTH's knowledge of the proposed and existing structures. The approximate as-drilled locations of the borings are shown in Figures 5-1 and 5-2, and again on the attached Test Boring Location Plans, Figure Nos.1A and 1B in Attachment A. Elevations of test borings were surveyed in the field after restoration activities, and as such, should be considered approximate.

Figure No. 5-1: Test Boring Location Plan for Crossing X-10.

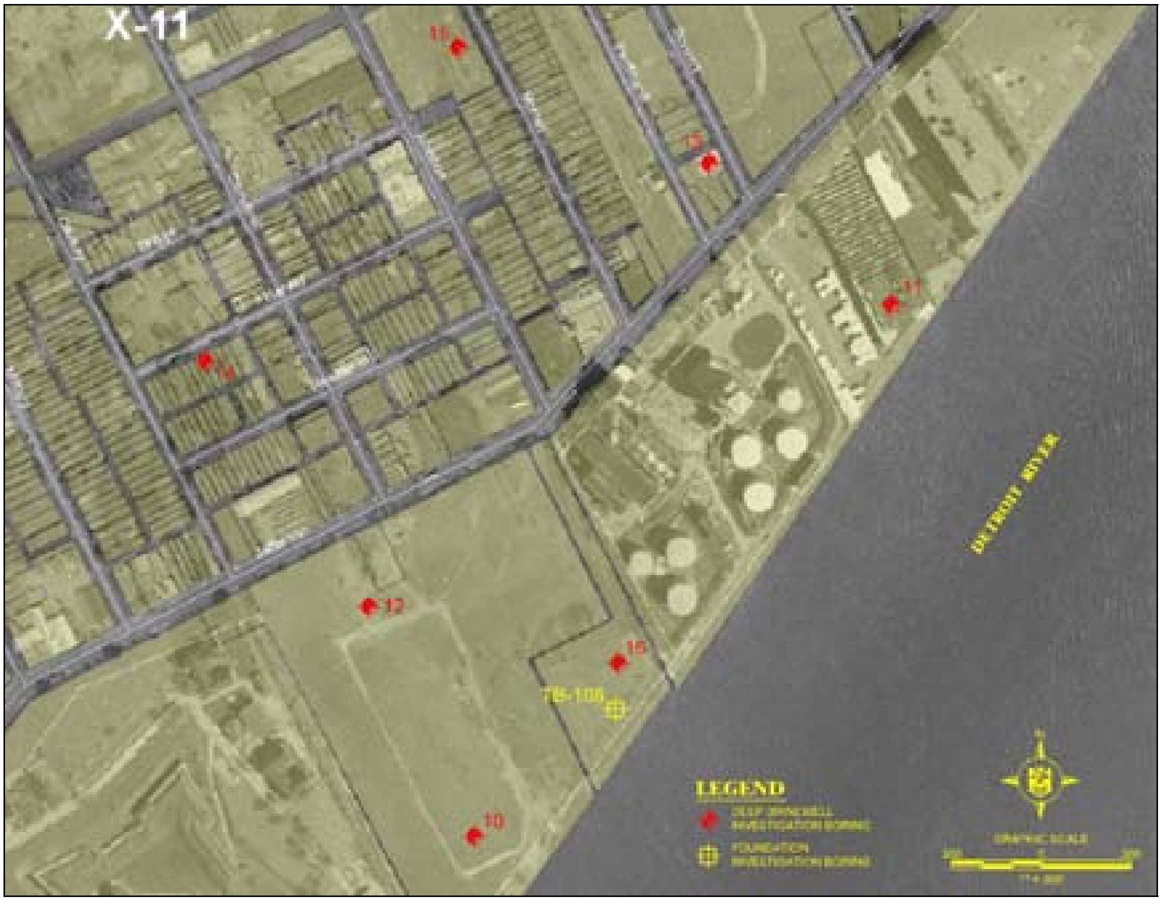


Figure No. 5-2: Test Boring Location Plan for Crossing X-11.

5.1 DRILLING PROCEDURES

The test borings were drilled with an ATV-mounted drilling rig using a combination of 3 ¼-inch (8.3 centimeters) inside diameter (ID) hollow stem augers, 12 ¼-inch (31.1 centimeters) wash rotary tri-cone rotary bits, NWJ rods, and NQ/NX wire-line diamond rock coring techniques. TB-101 through 107 and TB-108 were drilled in X-10 and X-11 Crossing Corridors, respectively, and were drilled for a total depth of 112.5 to 152.5 feet (34.3 to 46.5 meters). TB-102 was terminated at 112.5 feet (Elevation 468.5 feet) (34.3 meters, Elevation 142.8 meters) due to broken coring equipment blocking the borehole. TB-104 and TB-105 were offset several times from the originally drilled locations due to demolition debris encountered below the surface. At test boring location TB-104E, the surficial concrete obstruction was cored to advance the boring. The soil samples were obtained by either the Standard Penetration Test Method or by advancing a thin walled Shelby tube into undisturbed soil using a standard Shelby tube sampler.

Once the underlying bedrock formation was encountered, wire-line rock coring techniques were used to extend the boring to its termination depth.

Soil and rock conditions encountered in each of the test borings have been evaluated and are presented in the Logs of Test Boring, Figure Nos. 3 through 10, attached in Attachment A. The boring logs present information relating to sample data, standard penetration test results, groundwater conditions observed in the borings, personnel involved, and other pertinent data. The logs included in this report have been prepared on the basis of laboratory testing, as well as visual classification of the soil and rock samples. Terminology used to classify subsurface conditions is presented as Figure No. 2A, General Notes, presented in Attachment A.

The stratification shown on the test boring logs represents the soil and rock conditions at the actual explored locations. Variations in subsoil conditions may occur between the borings. Additionally, the stratigraphic lines represent the approximate boundary between the soil or rock types; however, the transition may be more gradual than what is shown.

5.1.1 Standard Penetration Testing

The Standard Penetration Test (SPT) (ASTM D1586) consists of driving a 2.0-inch (5.1 centimeters) outside diameter split-barrel sampler into the soil with a 140-pound (63.5 kilograms) weight falling freely a distance of 30 inches (76.2 centimeters). The sampler is generally driven three successive 6-inch (15.2 centimeters) increments, with the number of blows for each increment being recorded. The number of blows required to advance the sampler the last 12 inches (30.5 centimeters) is termed the Standard Penetration Resistance (N) and is presented on the individual Logs of Test Boring. As added information, the blow counts for each 6-inch (15.2 centimeters) increment are also presented on the Logs of Test Boring.

The split barrel sampler generally contains a 1-3/8-inch (3.5 centimeters) inside diameter and 3-inch (7.6 centimeters) long liner insert. Soil samples recovered in these liners are designated as "LS" on the respective test boring logs, whereas samples recovered directly from the split barrel are designated as “S.” All soil and rock samples obtained with the split-barrel sampler were sealed in jars and transported to NTH’s laboratory for further classification and testing.

5.1.2 Thin-Walled Shelby Tube Sampling

Within two of the test borings, Shelby tube samples were obtained within the soft cohesive soil zones. The samples were obtained using a standard Shelby tube sampler and are considered relatively undisturbed. The sampling method consists of advancing a thin-walled steel Shelby tube into the soil using hydraulic pressure. The sampling was performed in accordance with the Standard Practice for Thin-Walled Tube Sampling of Soils (ASTM D1587). Standard Shelby tube samples are designated as “ST” on the Logs of Test Boring. Samples were sealed within the steel tubes and transported to NTH’s laboratory for further classification and testing.

5.2 ROCK CORING AND TESTING PROCEDURES

The sampling procedures used within the rock portion of the test borings included continuous NQ wireline core sampling. Brief descriptions of these methods are presented in the following paragraphs.

Continuous rock core samples were obtained in general accordance with ASTM D2113. Diamond core drilling was accomplished at each test boring location with NQ/NX wire line core tooling. Double-tube, solid, swivel type core barrels with diamond tipped bottom discharge bits were used. Each core barrel was capable of obtaining a core run length of at least five feet with actual cores obtained in 0.7 to 10-foot (0.2 to 3 meters) coring runs.

Upon removal of each core from the borehole, the cores were placed in boxes. All core boxes were provided with longitudinal separators and recovered cores were laid out from left to right and top to bottom. Spacer blocks or plugs were inserted into the core column to mark the beginning of each successive core run.

After placing the core into its respective box, the field engineer prepared selected samples for laboratory testing, then recorded the percent recovery, fractures per foot of run, prepared selected portions of the core for laboratory testing, lithology, and determined the Rock Quality Designation (RQD) value for each core run. The recovery is defined as the total length of core retrieved from the barrel divided by the total distance the barrel was advanced during coring. The RQD is defined as the total length of all intact rock core pieces greater than four inches in length divided by the total advanced distance. The fractures per foot were determined by the NTH field engineer immediately after sampling by evaluating fractures that appeared natural and were not obviously mechanical (caused by coring, handling of the core, or by intentional breakage).

A detailed Log of Core Boring for each core run was prepared based on visual classification and the logs are presented as Figure Nos. 11 through 18 in Attachment A. The Logs of Core Boring include information for each core run, including a detailed rock description, a description of fractures and mechanical breaks noted by the NTH field engineer at the time of sampling, results of sampling for hydrogen sulfide gas and methane during sampling.

The Logs of Core Boring present information relating to sample data, rock recovery, rock quality designation (RQD), personnel involved, and other pertinent data. Definitions related to the information presented on the Log of Core Boring are presented on the Summary of Rock Log Nomenclature, Figure No. 2B, attached in Attachment A.

The stratification shown on the Log of Core Boring represents the rock conditions at the actual explored locations. Variations in bedrock conditions may occur between the borings. Additionally, the stratigraphic lines represent the approximate boundary between the rock types; however, the transition may be more gradual than what is shown.

5.3 PIEZOMETER INSTALLATION

Following the completion of the rock coring in TB-101 and 102, pneumatic piezometers, designated as PZ-101 and PZ-102 were installed within the respective boreholes to provide continued groundwater information within the bedrock. The piezometers consist of two flexible tubes, separated by a porous stone and flexible rubber diaphragm exposed to the surrounding groundwater. The piezometers were installed in a sand pack within the bedrock at predetermined depths, sealed with a small section of bentonite plug, and then tremie grouted to the surface. Groundwater head was measured by applying a pressure across the flexible diaphragm through one of the tubes until air is observed escaping from the remaining tube. The pressure at which air returns to the surface was recorded as the equivalent of the groundwater pressure at that depth. The corresponding groundwater head was then calculated based on the tip elevation of the piezometer. Schematics containing the details of the piezometer installation and associated water level readings are attached as Figure Nos. 19 and 20 in Attachment A.

6.0 LABORATORY TESTING

The details of laboratory testing for soil and rock are discussed as follows.

6.1 SOIL TESTING

Representative soil samples obtained from the test borings were subjected to laboratory testing to determine pertinent engineering characteristics. The testing program included the determination of the dry density and natural moisture content, unconfined compressive strength, Atterberg Limits, and particle size distributions of selected soil samples. All testing was performed in accordance with current ASTM standards. The dry density, moisture content, and unconfined compressive strength values are presented on the Logs of Test Boring, as well as on the Tabulation of Laboratory Test Data, attached as Figure No. 1 in Attachment A. The Atterberg Limits and particle-size distribution values are presented in the Tabulation of Laboratory Test Data. In addition, Grain Size Analyses are presented graphically as Grain Size Distribution Curves, Figure Nos. 2 through 15 in Attachment A.

6.2 ROCK TESTING

A total of 14 representative rock samples obtained during the field investigation were subjected to Uni-Axial Compressive Strength (UCS) testing. During these tests, on eight of the 14 samples, attempts were made to measure and record both the axial and lateral deformation of the sample. However, when the data was processed, it became apparent that the strain gauges had failed, apparently producing errant lateral deformation values. The purpose of this test was to measure the compressive strength and determine the Poisson’s ratio of the rock (estimations of modulus of elasticity can be obtained from the testing), although this value was also determined as part of tri-axial testing and acoustic testing discussed below.

Two representative rock samples obtained during the field investigation were subjected to Tri-Axial Compressive Strength testing. This type of test simulates the behavior of rock underground, as significant confining pressure is applied to the sample during loading. Five tests are typically conducted for each test sample, each at a different confining pressure. The resulting sample strengths are then plotted on a stress difference versus axial strain curve, and on a Mohr circle, to compute the design strength parameters.

A total of 4 representative rock samples obtained during the field investigation were subjected to Indirect (Brazilian) Tensile Strength testing, which provide a measure of rock toughness, as well as tensile strength.

Two representative rock samples obtained during the field investigation, which were prepared for UCS testing, were subjected to testing where the velocities of compressive and shear ultrasonic waves passing through the core sample were measured. These values were used to calculate the elastic modulus and Poisson's ratio as an indication of the competency of the rock. From the wave velocities and the sample bulk density, the dynamic elastic modulus, and dynamic Poisson’s ratio are then calculated.

The tests were performed at the Earth Mechanics Institute (EMI) of the Colorado School of Mines, Golden, Colorado, in general accordance with ASTM D7012, D 3967, and D 2845. All samples were tested at their "as-received" moisture content. The tests were performed on a number of selected rock core samples and were intended to determine the general nature of intact properties for the upper bedrock formations. The results of the rock tests are presented on EMI’s rock testing summary and report, presented as Figure No. 36 in Attachment A. The results of testing are also presented on the individual Logs of Core Borings. Included within the EMI report are detailed test results, sample measurements, procedures, and photographs of the specimens after testing.

7.0 SUBSURFACE CONDITIONS

Based on the information gathered during this investigation, it is determined that the subsurface conditions vary in composition and thickness in the upper soil horizons and become more consistent with depth. The subsoils generally consist of variable fill soils underlain by a thin fill layer of gravelly sand. Underlying the gravelly sand or fill is a relatively thick soft silty clay layer. The clay layer is underlain by clay hardpan that extends to bedrock. The bedrock interface is generally characterized by a thin zone of low RQD (RQD <75%) bedrock that has intermittent layers of fragmented bedrock with gravel and silty clay. Underlying the low RQD bedrock, is competent limestone/dolomite bedrock (RQD >75%) extending to the explored depths.

A generalized soil and rock profile is presented for illustration in Figure 7-1. This summary profile is intended for illustration only. For conceptual design or evaluation purposes, the reader should refer to the individual test borings.

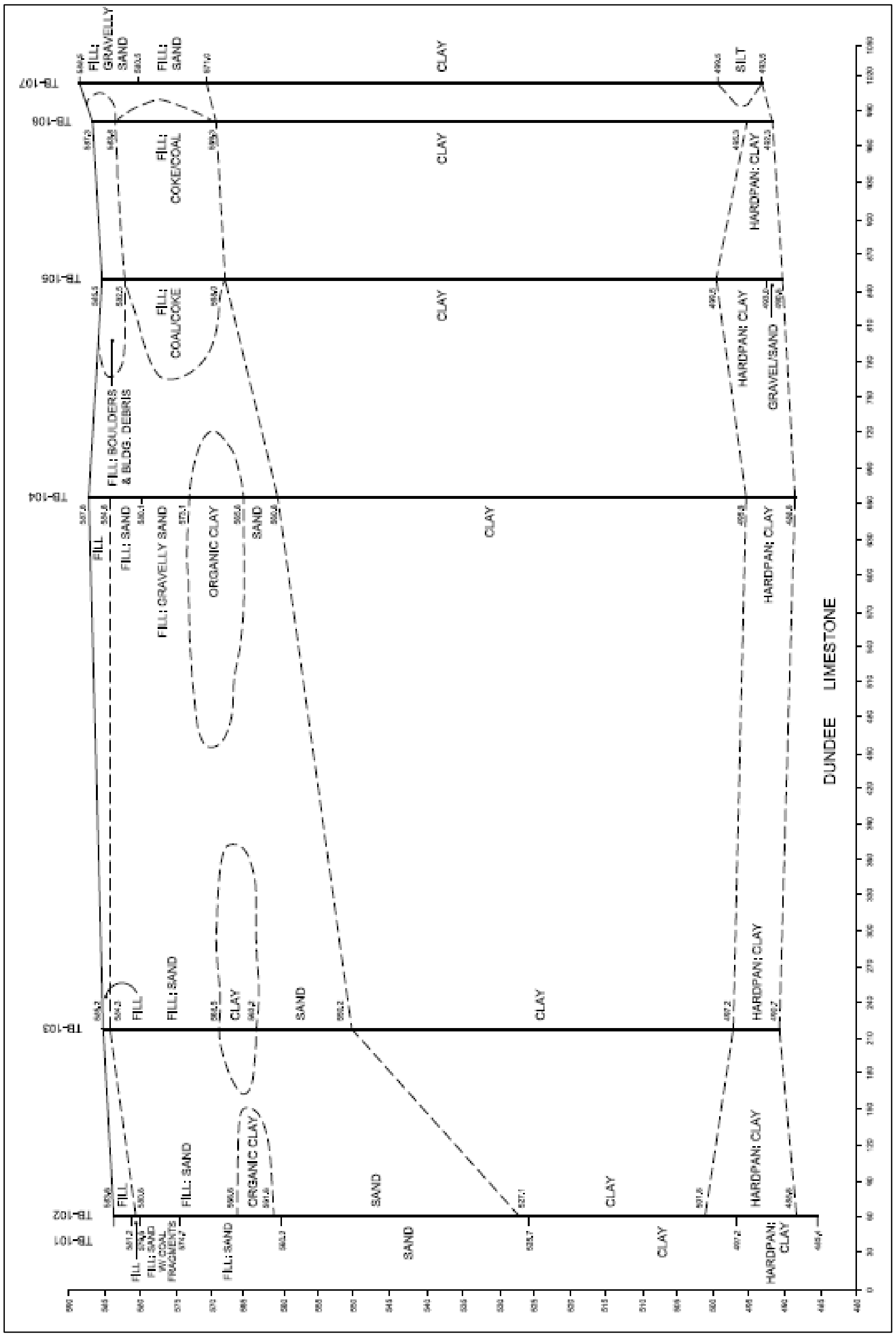


Figure No. 7-1: Generalized Soil and Rock Profile.

7.1 SOIL CONDITIONS

As discussed in the Subsurface Investigation section of this report, soil conditions were investigated for both of the proposed crossing corridors.

7.1.1 X-10 Crossing Corridor

Test Boring TB-101 through TB-107 were drilled within the X-10 crossing corridor. Granular fill was encountered at each test boring location to depths of 14.5 to 21 feet (4.4 to 6.4 meters) below the existing ground surface, which corresponds to Elevations 560.2 to 573.1 feet (Elevations 170.7 to 174.7 meters). The fill typically consists of loose to very compact silty sand, gravelly sand, clayey sand, and sand. Coal fragments, coke, coke tar, ash, and construction debris (variable size brick and concrete fragments) were encountered in the fill along with cobbles and boulders. Soft organic clay was encountered in test boring TB-102 at depths between 17 and 22 feet (5.2 and 6.7 meters), and very soft organic clay was encountered in TB-104E at depths between 14.5 and 22 feet (4.4 and 6.7 meters)

Native granular soils were found to underlie the fill soils at test borings TB-101 through TB-104 and extended 27 to 56.5 feet (8.2 to 17.2 meters) below the existing ground surface, which corresponds to Elevations 560.6 to 525.7 feet (Elevations 170.9 to 160.2 meters). The native granular soils generally consist of loose to compact sand and silty sand. However, a layer of very compact silty sand was encountered in test boring TB-102 at a depth of 46 to 56.5 feet (14 to 17.2 meters), corresponding to Elevation 537.6 to 527.1 feet (Elevation 163.9 to 160.7 meters) and a layer of very loose silty sand was encountered in test boring TB-103 at a depth of 22 to 28 feet (6.7 to 8.5 meters), corresponding to Elevation 563.2 to 557.2 feet (Elevation 171.7 to 169.8 meters).

Native cohesive soils were encountered below the granular soils in test borings TB-101 through TB-104 as well as directly below the fill at test boring locations TB-105 through TB-107. Very soft to medium silty clay, sandy clay, and clay was encountered in each test boring, extending to depths of 82 to 92 feet (25 to 28 meters) below the ground surface, corresponding to Elevations 499 to 490 feet (Elevations 152.1 to 149.4 meters). The very soft to medium clays are underlain by hardpan at test boring locations TB-101 through TB-106. The hardpan layer consists of very stiff to very hard silty, sandy, and gravelly clay and extends to bedrock, although a layer of very compact gravel and sand was encountered above the bedrock within TB-105A. Within TB-107, a layer of native granular soil consisting of medium compact gray clayey silt was encountered beneath the very soft to medium native cohesive soils.

7.1.2 X-11 Crossing Corridor

Test Boring TB-108 was drilled within the X-11 crossing corridor. Approximately 0.2 feet of topsoil was encountered at the surface in test boring TB-108. Fill was encountered below the topsoil and extended to a depth of 12 feet (3.7 meters) below the existing ground surface, corresponding to Elevation 565 feet (Elevation 171.6 meters). The fill consists of loose to medium compact silty sand and gravel, as well as very stiff sandy clay.

Below the fill in TB-108, native cohesive soils were encountered, consisting of soft to medium silty clay. This stratum extend to a depth of 87 feet (Elevation 490 feet), or (26.5 meters (Elevation 149.4 meters). The soft to medium clay is then underlain by a layer of stiff silty clay to 92 feet (Elevation 485 feet), or 28 meters (Elevation 147.8 meters). The cohesive soils are further underlain by very compact silty sand hardpan which extends to bedrock.

7.2 BEDROCK CONDITIONS

The bedrock consists of amorphous to fine-grained, fossiliferous limestone and dolomite of the Dundee Limestone and Detroit River Group (Lucas Formation). The bedrock extends to the explored depths of 113.5 to 152.5 feet (Elevations 470.2 to 429.2 feet) (34.6 to 46.5 meters, Elevations 143.3 to 130.8 meters). Within the test borings, bedrock consists of light gray amorphous limestone with some fossiliferous zones.

RQD values generally ranged from 50% to 100% with several notable exceptions, which are summarized in Table 7.1. Recoveries in the remaining core runs ranged between 80 and 100%, with the exception of Run 1, performed at TB-101 from 95.8 to 98 feet, which had a recovery of 56.8%.

Table No. 7-1: Core Runs With An RQD Value Less Than 50%.

Core Boring	Run	Depth	RQD (%)	Recovery (%)
TB-101	2	99-102 ft (30.2-31.1 m)	18	33
TB-102	1	97-101 ft (29.6-30.8 m)	48	61
	4	111-113.5 ft (33.8-34.6 m)	49	49
TB-103	1	94.5-100 (28.8-30.5 m)	8	81
	2	100-105 (30.5-32 m)	36	36
TB-104E	1	99-102.5 ft (30.2-31.2 m)	33	28
TB-105A	1	95-100 ft (29-30.5 m)	12	75
TB-106	4	115-125 ft (35.1-38.1)	31	97
TB-108	2	108-109 ft (32.9 – 33.2 m)	0	28

Fractures per foot ranged from 0 to 2.6 for all eight borings except Run 1, performed at TB-103 from 94.5 to 100 feet (28.8 feet and 30.5 feet) and Run 2 performed at TB-107 from 100.7 to 105.7 feet (30.7 to 32.2 feet), which had 10 and 22 fractures per foot, respectively (intensely fractured).

7.3 CANADIAN SUBSURFACE CONDITIONS

For the purposes of comparison, the team has reviewed the generalized subsurface conditions that were determined from the Canadian Brine Well Program. It should be noted that the Canadian DRIC team has not yet conducted an investigation specifically for the purpose of evaluating bridge foundation support issues, and so this summary is based on information gathered during the Canadian Brine Well Program. Because the brine well investigation was focused on the bedrock, there is little information on the soil overburden.

7.3.1 Canadian Overburden Conditions

The subsurface conditions on the Canadian side of the proposed crossing corridors appear to be quite similar to those on the United States side. Conditions generally consist of bedrock overlain by glacial drift, which has been deposited either directly as till, glaciofluvial, or lacustrine deposits.

The area of Canadian investigation, as well as the area of the United States investigation, is generally located in what is commonly referred to as the St. Clair Clay Plain. Late Pleistocene era unconsolidated deposits, typically from the last major glaciations (Wisconsinan Stage, 10,000 to 14,000 years ago), overly the Dundee Limestone bedrock. The sediments typically consist of basal till and a sequence of lacustrine deposits. In some areas, the lacusterine deposits contain sand and gravel sequences overlying the bedrock, but generally consist of 65 to 100 feet (20 to 30 m) of fine-grained silt and clay materials.

7.3.2 Canadian Bedrock Conditions

The proposed crossing corridor on the Canadian side, as well as the area of the United States investigation, is located at the geologically-termed southeast margin of the Michigan Basin geomorphic province. Dundee Limestone (Dundee), a light gray, moderately hard, petroliferous, fossiliferous, laminated to thinly bedded, stylotic, pitted and vuggy limestone to dolomitic limestone is commonly encountered underlying the overburden. The Dundee in this area (from historical reports) typically exhibits unconfined compressive strengths on the order of 7 to 14.5 ksi (50 to 100 MPa), similar to values achieved on the Unites States side. RQD values for core obtained for the Canadian X-10 and X-11 Crossing locations ranged from 19 to 82%, with an average of approximately 44%, though the upper 6 feet (2 m) often exhibit values as low as 5 to 10%, which is consistent with observations made on the United States side.

7.3.3 Planned Additional Investigation

The Canadian DRIC team has indicated that to better understand surficial soil and bedrock conditions on the Canadian side of the proposed crossing corridors, an additional investigation consisting of approximately 35 test borings and 35 Cone Penetration Test borings (CPT) are planned in the near future. This investigation will supplement the deep rock/brine well investigation that has already been performed by the Canadian team.

7.4 EVALUATIONS

Based on the information gathered during this United States investigation, subsurface conditions appear variable in composition and thickness in the upper levels of the borings and become more consistent with depth as bedrock is approached. The subsoils generally consist of variable fill soils underlain by a thin fill layer of gravelly sand. Underlying the gravelly sand or fill is a relatively thick silty clay layer. The clay layer is underlain by clay or granular hardpan that extends to limestone and dolomitic limestone bedrock. The bedrock interface is generally characterized by a thin zone of low RQD bedrock (RQD <75%) that has intermittent layers of fragmented bedrock with gravel and silty clay. Underlying the low RQD bedrock is more competent (RQD >75%) limestone and dolomite bedrock extending to the explored depths.

Based on the results of this investigation, the existing fill deposits at both crossing locations are highly variable and are not considered suitable for support of any foundation elements.

The underlying silty clay or granular soils are not considered suitable for support of the heavy loading expected from primary or secondary bridge foundation elements, but may be sufficient for support of ancillary structures with light to moderate foundation loads.

The hardpan soils underlying both corridors are considered well suited for the heavy foundation loading anticipated from proposed secondary structural elements of the bridge.

The upper highly weathered bedrock (RQD >75%) underlying the hardpan soils is considered suitable for the heavy foundation loading anticipated from primary and secondary foundation elements of the bridge, although it is recommended that preliminary design bearing capacities are not any higher than for the hardpan.

The competent bedrock (RQD >75%) underlying the hardpan soils and the weathered bedrock is well suited for the heavy foundation loading anticipated from primary and secondary foundation elements of the bridge.

8.0 GROUND WATER CONDITIONS AND CONTROL

Groundwater was encountered during drilling at depths of 12.5 to 19.5 feet (Elevations 567.5 to 574 feet) (3.8 to 5.9 meters, Elevations 173 to 175 meters) in the test borings performed at the X-10 Crossing Corridor and at a depth of 6.5 feet (Elevation 570.5 feet) (2 meters, Elevation 173.9 meters) in TB-108 for the X-11 Crossing Corridor. Further groundwater level measurements during drilling and at completion were precluded due to the use of drilling fluids.

Pneumatic piezometers were installed in TB-101 (PZ-101) and TB-102 (PZ-102). PZ-101 was installed at Elevation 431 (131.4 meters) within the lower bedrock zone. PZ-102 was installed within the upper bedrock zone at approximately EL 470.5 (143.4 meters). Refer to Figure Nos. 19 and 20 in Attachment A for diagrams of the well installation. Based on the readings taken subsequent to June 17, 2008, hydrostatic artesian pressure head in the bedrock was recorded at 8.0 to 11.5 feet (2.4 to 3.5 meters) above ground surface at PZ-102 and PZ-101, respectively, corresponding to Elevation 592.7 feet (180.7 meters).

8.1 GROUNDWATER CONTROL

Due to the depth of the proposed excavations, groundwater control will be required to address groundwater conditions (and related gas conditions) within the granular soil portions of the soft ground profile above the hardpan, as well as artesian conditions within the hardpan and bedrock.

8.1.1 Groundwater in the Soil Horizon

Within the X-10 corridor (in particular TB-101, TB-102, and TB-103), relatively deep water bearing granular soils are present in close proximity to the Detroit River shoreline, which will act as a significant source of recharge to the groundwater within this granular aquifer. Based on analysis of the grain size testing for these soils, it appears that the permeability will be as high as approximately 0.01 centimeter per second (cm/sec). This permeability is considered quite high and on the margin of what is considered feasible to dewater. In addition, it is likely that any dewatering effort within this aquifer would cause migration of existing known contamination on the former Detroit Coke site toward the river, and would require extensive investigation and evaluation in order to accommodate permitting by regulating agencies. Further, any dewatering discharge would require significant treatment to remove dissolved hydrogen sulfide prior to disposal, which has proven to be extremely expensive for nearby projects where this method was used.

For these reasons, it is anticipated that the only feasible method for groundwater control for large open excavations (such as for anchorages) within the granular soils present in the upper 50 to 60 feet (15 to 18 meters), will be to install a groundwater cutoff wall consisting of steel sheeting, a slurry wall, or possibly sinking caisson wall. Such a wall may be incorporated as part of the temporary earth retention system for installing the anchorage elements. In any case, a system will need to be designed that prevents cross contamination of upper aquifers (where present), and which is acceptable to regulating agencies.

For drilled or pre-drilled foundation elements (drilled piers or pre-drilled piles, respectively), such as for the main piers or for approach piers, it is expected that excavation can most likely be accomplished through the use of slurry during drilling (as discussed in later sections). It is unlikely that any other method (such as driving cutoff casing) will be successful without the use of slurry, due to the depth of the water bearing granular layers, and the depth to which the casing would need to be driven to provide cutoff. As discussed above for large excavations, any design for drilled piers or piles will need to consider the potential for cross contamination of upper aquifers. It is likely that for any design incorporating piles or drilled piers, the most practical approach would be to install a single perimeter cutoff wall around the pile group supporting the subject pier, rather than installing cutoff casing around each foundation element.

8.1.2 Groundwater and Gas in the Hardpan and Bedrock

The groundwater within the hardpan and bedrock typically contains dissolved sulfides, which can create hydrogen sulfide gas upon exposure to the atmosphere and groundwater discharge concerns if not addressed. Likewise, toxic or explosive gases may be potentially present in localized areas throughout the site.

Based on the highly fractured nature of the upper rock, as well as NTH's experience with this formation on a number of sites in the immediate vicinity of the proposed construction, it is estimated that the upper rock mass has permeability in the range of 0.01 to 0.001 cm/sec, which is considered relatively high. Given the anticipated relatively high rock mass permeability, it appears groundwater in the bedrock may require control during foundation construction. For applications that require exposure of large areas of the bedrock to atmospheric conditions (such as for suspension bridge anchorages), this is typically accomplished by either rock grouting, or sometimes by dewatering.

For foundation installation methods other than drilled piers (discussed in subsequent sections), groundwater control in the hardpan and bedrock units by dewatering is not considered to be feasible for this project. This is because the foundations will be relatively massive and would be expected to involve at least 12 months of underground construction and related dewatering. For such a prolonged dewatering effort, consolidation settlement would be induced in the thick cohesive strata overlying the hardpan, creating the potential for damage to surrounding infrastructure. In addition, the volume of water and the associated groundwater treatment to remove sulfides and other contaminants prior to disposal (as discussed above for the soil overburden) would be significant and very costly.

Rock grouting has proven to be a very cost-effective means of controlling groundwater and gas at the soil rock interface on several projects in close proximity to the DRIC, where construction involved excavation and exposure of the bedrock surface. Rock grouting of the Dundee Limestone would allow for exposing and cleaning the rock surface to confirm the primary and secondary foundation elements are founded on an adequate bearing surface. However, rock grouting of the upper weathered rock will require consideration for the presence of granular soils where encountered immediately over the bedrock. This was the encountered condition at TB-105A and TB-107 in X-10, and TB-108 in X-11. The granular soils have the potential to fill the upper rock fractures, which may inhibit grout penetration during a rock grouting program. This can be addressed by the use of low viscosity chemical grouting such as acrylimide, or hot bitumen in the upper rock.

For the purposes of the conceptual engineering, and if exposure of a large area of the rock surface is necessary to accommodate the construction, it is recommended that a rock grouting program be developed, with a concept design including a three stage, cementious, top down rock grouting program, with a secondary chemical grouting program of the upper rock to address granular infilling of fractures in the rock.

Even if such a rock grouting program is effectively implemented, some amount of groundwater will almost certainly be produced from the surface of the grouted rock. Any groundwater produced is expected to require treatment for dissolved sulfides and hydrogen sulfide prior to disposal. Additionally, odor control may be required for airborne hydrogen sulfide gas, although substantial efforts for odor control are not typically necessary or required for industrial areas such as the X-10 corridor. When necessary, such odor control typically involves collecting and treating the water before it has the opportunity to let gas escape, and/or masking the odor chemically.

9.0 SITE PREPARATION

It is anticipated that final design grades will be close to existing site grades. However, due to the presence of fill on the site, some earthwork may be required to achieve final design grades. It is anticipated that some of the on-site fill can be reused for earthwork operations, but only in green belt areas.

The concept engineering should consider that at the start of earthwork operations, and after any demolition is complete, all existing pavement, vegetation, layers of topsoil, and any other exposed organic soils should be stripped and removed from within the proposed foundation footprint. In addition, any utilities present within the foundation area should be re-routed and then properly abandoned and removed following outside of the footprint.

A special consideration for the X-10 site will be the presence of the demarcation barrier and associated groundwater collection system, which will probably need to be maintained outside the immediate construction area. In addition, special handling and excavation methods will be required for any excavations through the upper granular soil layers to prevent exacerbation of contamination on the site. It is likely that installation of a contamination cutoff wall will be necessary, which could be coincident with the groundwater cutoff wall mentioned in Section 7.1.1.

9.1 TEMPORARY EXCAVATIONS

The concept engineering should required that all excavations deeper than 5 feet (1.5 meters) be properly sloped or otherwise structurally retained to provide stable and safe working conditions. In all cases, applicable regulations prescribed by the Michigan Department of Consumer and Industry Services (MDCIS), formerly known as MIOSHA, will need to be followed and adequate protection for workers, structures and utilities provided.

10.0 FOUNDATION RECOMMENDATIONS

Recommendations for support of bridge foundations have been developed on the basis of the currently proposed cable-stayed and suspension bridge concepts as developed in the Detroit River International Crossing, Bridge Conceptual Engineering Report, by Parsons Transportation.

Based on the concepts developed in the Bridge Conceptual Engineering Report, the main pier elements for both suspension and cable-stayed bridge options, along with secondary and approach pier foundation elements, may be supported on long slender deep foundation elements. For the purposes of this report, such foundation elements will consist of drilled piers, or pre-drilled or driven piles. In all cases, multiple pier or pile elements would be required to support a main pile cap, in turn supporting the pier or other structural element.

For conceptual design purposes, the Michigan Department of Transportation's (MDOT) Bridge Design Manual (BDM) is used to develop nominal pile driving resistance values (R_{NDR}) for the recommendations presented herein. The driving resistance values presented in the MDOT BDM assume that the FHWA-modified Gates dynamic formula is used to develop driving criteria. As such, both AASHTO (2007) LRFD Bridge Design Specifications and the MDOT BDM recommend a dynamic resistance factor (ϕ_{DYN}) equal to 0.4. It should be noted that the FHWA manual Design and Construction of Driven Pile Foundations, FHWA-HI-97-013 and -014, states the following:

> *",,,dynamic formulas do not provide information on pile driving stresses and, in many circumstances, have proven unreliable in determining pile capacity. Therefore, their continued use is not recommended on significant projects. Dynamic test methods using signal matching techniques can be used to: calculate pile installation stresses, determine pile integrity, estimate static pile capacity, and determine relative soil*

> *resistance distribution on the pile. This is also an appropriate means of establishing driving criteria."*

If AASHTO guidelines are followed to establish driving criteria from dynamic testing with signal matching, a dynamic resistance factor of 0.65 may be used instead of 0.4. We recommend that consideration be given to establishing pile driving criteria using dynamic testing.

10.1 SUSPENSION BRIDGE ANCHORAGE OPTIONS

If the main structure is chosen to be a suspension bridge, multiple options have been proposed and evaluated for the anchorage foundation elements. Those options are discussed as follows.

10.1.1 Two Rectangular Caissons (Option I)

In this case, two large rectangular sinking caissons are designed and constructed to serve as both the temporary earth retention system and permanent anchorage structures for the proposed suspension bridge. The proposed caissons measure approximately 33 feet x 197 feet (10 meters x 60 meters) in plan dimension. A diagram has been prepared to illustrate Option I and is presented as Figure 10-1 below and Figure No. 39 in Attachment A.

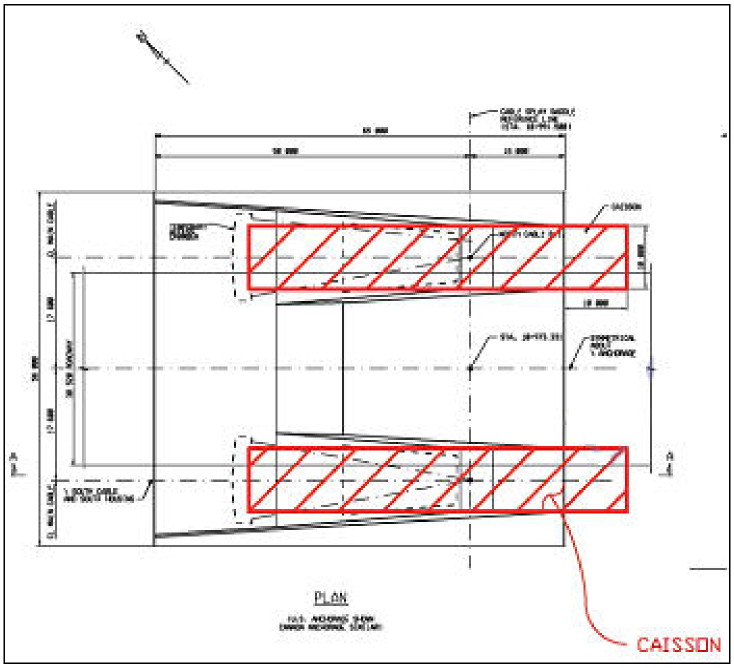


Figure No. 10-1: Schematic of Option I – Two Rectangular Caissons.

10.1.2 Single Circular Caisson (Option II)

In this case, one large diameter sinking caisson would be designed and constructed to serve as both the temporary earth retention system and permanent anchorage structure for the proposed suspension bridge. The proposed caisson has a diameter of approximately 165 feet (50 meters). A diagram has been prepared to illustrate Option II and is presented as Figure 10-2 below and Figure No. 40 in Attachment A.

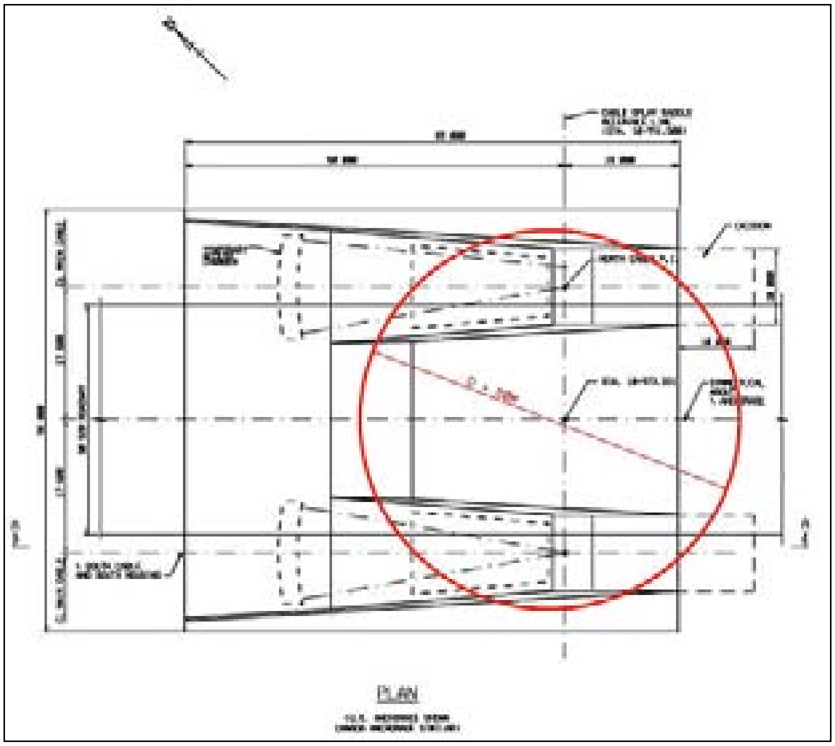


Figure No. 10-2: Schematic of Option II – Single Circular Caisson.

10.1.3 Drilled Shafts and Two Circular Caissons (Option III)

In this case, two circular sinking caissons, in combination with 12 drilled piers are designed and constructed to serve as the permanent anchorage structure for the proposed suspension bridge. The proposed sinking caissons measure approximately 65 feet (20 meters) in plan diameter and the drilled piers would be 10-feet (3 meters) in diameter. A diagram has been prepared to illustrate Option III and is presented as Figure 10-3 below and Figure No. 41 in Attachment A.

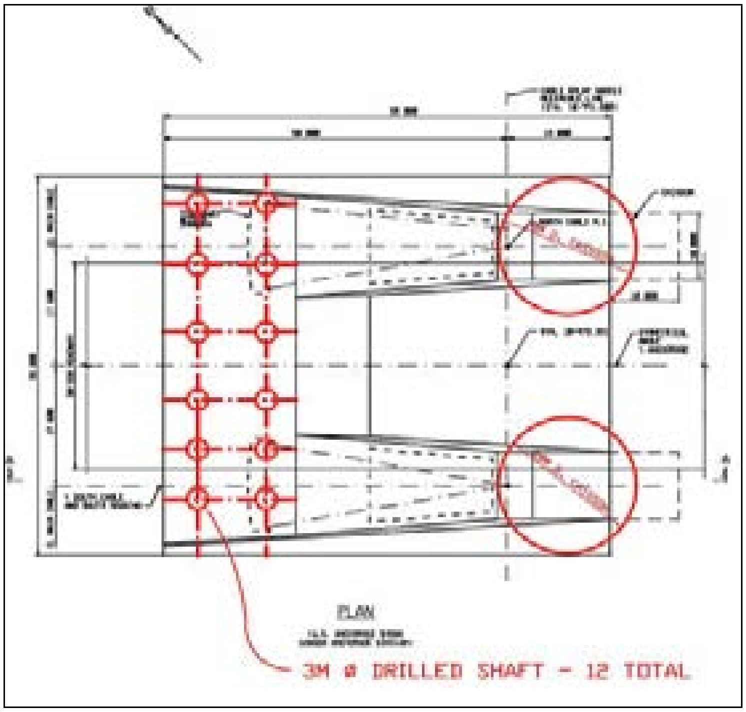


Figure No. 10-3: Schematic of Option III – Drilled Shafts and Two Circular Caissons.

10.1.4 Drilled Shafts and Two Rectangular Caissons (Option IV)

In this case, two rectangular sinking caissons in combination with 12 drilled piers are designed and constructed to serve as the permanent anchorage structure for the proposed suspension bridge. The proposed caissons measure approximately 30 feet x 82 feet (9 meters x 25 meters) with nominal 10-feet (3 meters) diameter drilled shafts. A diagram has been prepared to illustrate Option IV and is presented as Figure 10-4 below and Figure No. 42 in Attachment A.

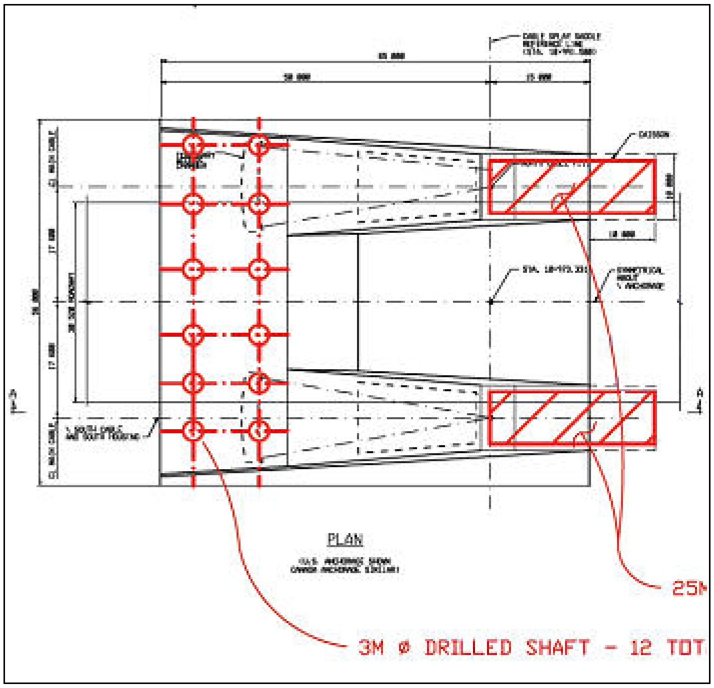


Figure No. 10-4: Schematic of Option IV – Drilled Shafts and Two Rectangular Caissons.

10.1.5 Pipe Piles (Option V)

Pipe piles to support the suspension bridge anchorage could consist of reinforced concrete filled steel pipes, which are typically equipped with a driving boot. The pipe piles would be pre-drilled to the top or very near the top of bedrock, and mandrel driven to bear on top of bedrock. A reinforcing steel cage would then be placed within each pile and the pile filled with concrete. For the concept design, it can be assumed that the bedrock geotechnical end bearing resistance will be mobilized within a settlement of up to 5% of the pipe diameter, and that any settlement would occur primarily as elastic settlement.

Table 10-1 summarizes the recommended nominal and factored pile driving resistance values for 30-inch diameter pipe piles. The values presented in Table 10-1 assume that the tip of the pipe pile is plugged. The vertical and horizontal components are also provided for battered piles assuming a 3V:1H batter. The dynamic resistance factor

(ϕ_{DYN}) presented by the MDOT BDM is equal to 0.4, and assumes that pile driving criteria will be developed by using the FHWA-modified Gates Dynamic formula.

Table No. 10-1: Conceptual Driving Resistance Values for Cast-in-place (C.I.P. Pipe Piles.

Pile	Axial (tons)		Vert. Component (tons) 3V:1H batter		Horiz. Component (tons) 3V:1H batter	
	R_{NDR}	$\phi_{DYN}R_{NDR}$	R_{NDR}	$\phi_{DYN}R_{NDR}$	R_{NDR}	$\phi_{DYN}R_{NDR}$
30” O.D. 0.625” Wall	990	396	939	376	314	126
ϕ_{DYN} = 0.4 based on using the FHWA-modified Gates dynamic formula to establish driving criteria.						

We recommend considering the use of dynamic testing in developing driving criteria. If AASHTO guidelines for dynamic testing are followed, a dynamic resistance factor of 0.65 may be used instead of 0.4.

The pipes should be spaced approximately 2 diameters apart or greater, to prevent significant reduction in capacity and increases in settlement due to grouping effects. The actual spacing could potentially be closer (or potentially further), based on final design analysis and in consideration of the final actual loading group configuration, pipe boot details, etc.

10.2 DEEP (SLENDER) FOUNDATION ELEMENTS FOR PRIMARY FOUNDATIONS

Drilled piers (also termed drilled caissons or drilled shafts) used for support of the primary foundations would extended through the upper fill, silty clay, granular soil layers, hardpan soils, and be founded at least 5 feet (1.5 meters) into the underlying competent limestone bedrock formation, resulting in depths of approximately 100 to 110 feet (30 to 34 meters). Extending these foundation elements will minimize uncertainties in the concept design by providing a uniform and reliable bottom pier elevation bearing on competent bedrock.

The drilled shaft evaluation summarized herein was performed using LRFD methodology. AASHTO and FHWA (report number FHWA-IF-99-025) guidelines were followed. The following statement taken from FHWA-IF-99-025 summarizes the assumptions used for evaluation.

> *"Assume that the settlement of the drilled shaft (pier) above the rock socket is due only to elastic compression of the drilled shaft (pier) material and is negligible. It is also assumed that the load transferred in the overburden above the rock is minimal. That is, all of the load is transferred in the socket. This assumption will ordinarily result in overpredicted settlements, since some load is invariably transferred in the overburden."*

Tables 10-2 and 10-3 summarize the nominal ultimate end and nominal ultimate side resistance values for the evaluated drilled piers. The nominal ultimate base resistance, R_{BN}, and nominal ultimate side resistance values, R_{SN}, are computed by the following equations.

$$R_{BN} = A_E(q_{max}) \qquad \text{[Eqn. 10.1]}$$

$$R_{SN} = A_S(f_{max}) \qquad \text{[Eqn. 10.2]}$$

Where: A_E Drilled pier end area, and
A_S Drilled pier shaft area within bedrock.

Table No. 10-2: Nominal Ultimate End Resistance Values.

	B = 2.5 m (8.2 ft)	**B = 3.3 m (10.8 ft)**
Maximum End Resistance, q_{max} (tsf)	300	
Maximum Nominal End Resistance, R_{BN} (tons)	15,843	27,605
Resistance Factor, ϕ	0.50	

Table No. 10-3: Nominal Ultimate Side Resistance Values.

	Socket Length (ft)	B = 2.5 m (8.2 ft)	B = 3.3 m (10.8 ft)
Maximum Side Resistance, f_{max} (tsf)	---	10.6	
Maximum Nominal Side Resistance, R_{SN} (tons)	5	1,370	1,800
	10	2,730	3,600
	15	4,100	5,400
Resistance Factor, ϕ	---	0.65	
B = Pier diameter			

For final drilled pier design, if the end and side resistance values are determined using a field load test, both resistance factors can be increased to 0.8. The nominal total base resistance, R_T, is computed by summing the nominal end and side resistance values. However, the ultimate nominal values of end and side resistance should not be added as these ultimate resistance values will not be mobilized at the same strain, as illustrated in Figure 10-5. The ultimate side resistance (point A) is mobilized at small strain, while the ultimate end resistance (point B) is mobilized at large strain. Once the end bearing resistance is fully mobilized, the shaft resistance has usually reduced to residual strength conditions (near point C). The load-settlement evaluation presented herein accounts for this strain incompatibility.

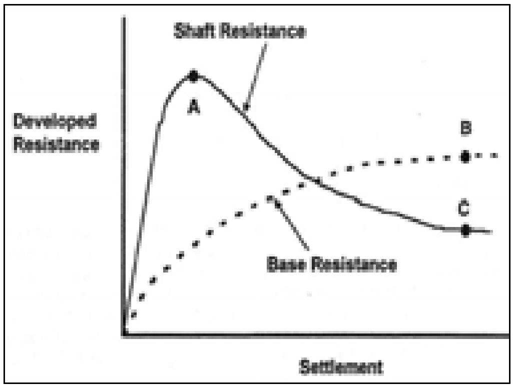


Figure No. 10-5: Load-Settlement Behavior under Compression Loading (FHWA-IF-99-025).

Figures 10-6 and 10-7 present the load-settlement behavior for pier diameters of 2.5 m (8.2 ft) and 3.3 m (10.8 ft), respectively. The load-settlement relation is modeled using three linear segments; although, the actual behavior is likely non-linear. The first segment presumes the behavior is elastic until side socket shear failure occurs. During the second segment, after side shear failure and before complete failure of base (plunging), it is assumed that the base behavior is elastic and the side resistance reduces to its residual strength value. The behavior in the third segment is plastic (plunging) where uncontrolled deformation occurs with very little to no additional loading.

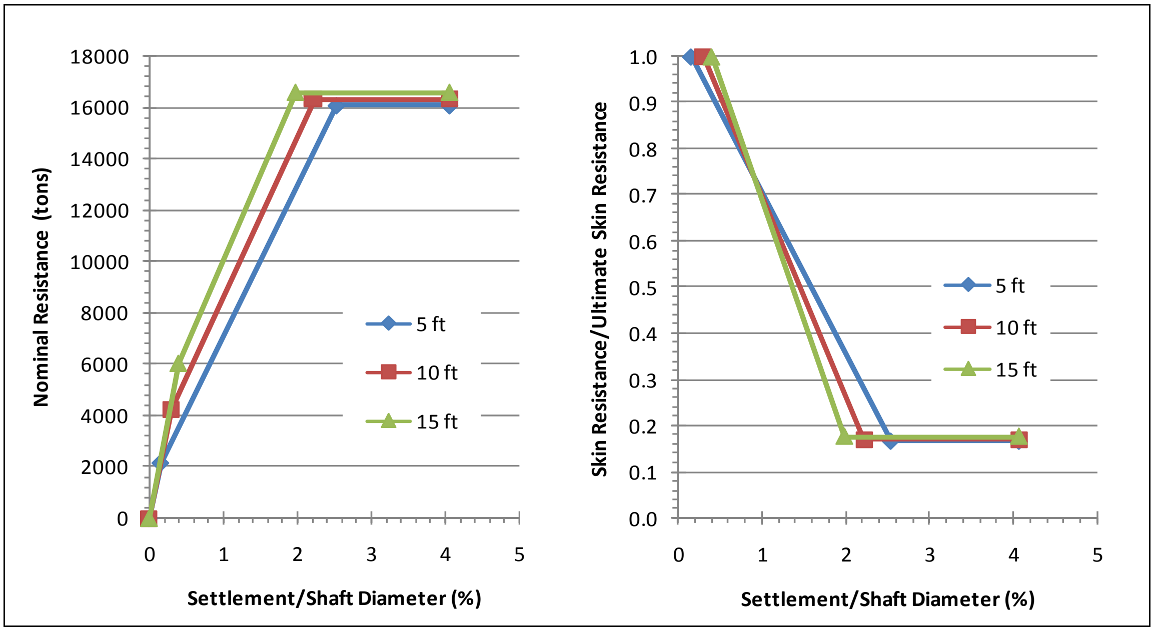


Figure No. 10-6: Drilled Pier Results Summary for a Diameter of 2.5 m (8.2 ft).

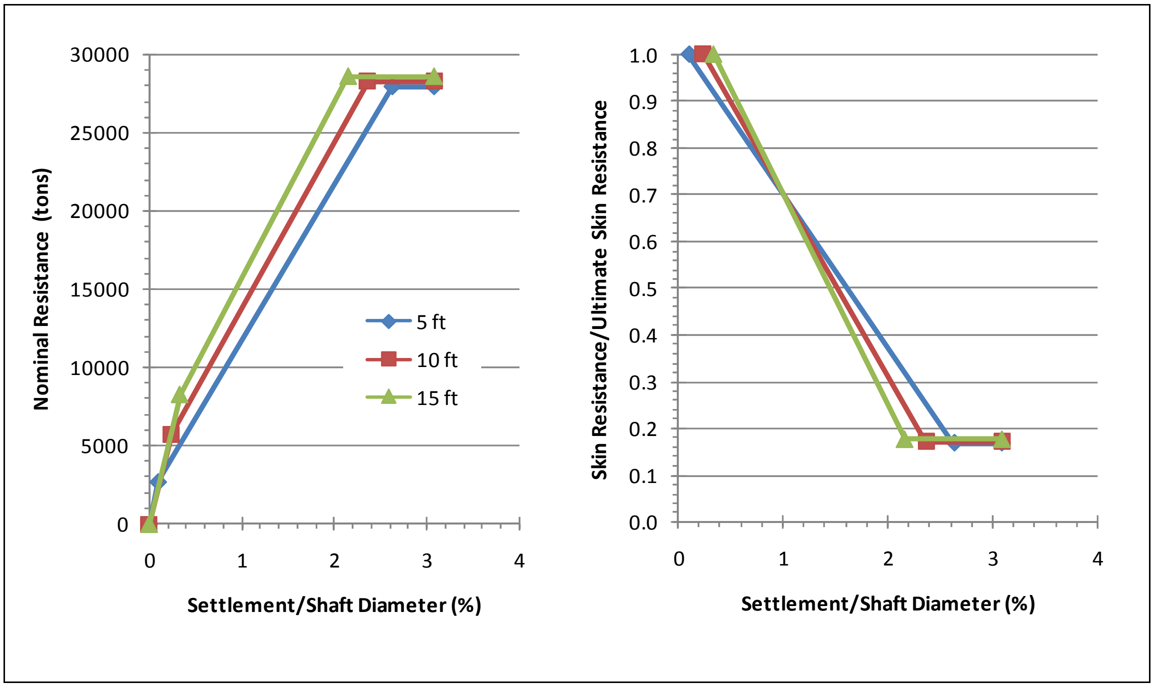


Figure No. 10-7: Drilled Pier Results Summary for a Diameter of 3.3 m (10.8 ft).

For Figures 10-6 and 10-7, the computed normalized skin friction transfer relationship is also provided. At small strain, the mobilized skin resistance is near 100% of its ultimate value. Allowing continued strain reduces the mobilized skin resistance to its residual strength value, which is approximately 17% of its ultimate value. Figures 10-6 and 10-7 are used to estimate the nominal and factored resistance values at a specified strain, considering strain incompatibility between skin resistance and end bearing resistance.

Tables 10-4 and 10-5 summarize in a matrix format the graphical results that are presented in Figures 10-6 and 10-7, for various rock socket lengths (L). The column δ/D represents the pier settlement normalized by the pier diameter found in the x-axis of Figures 10-6 and 10-7. In addition, the values presented in Tables 10-2 and 10-3 are used to develop factored resistance values that are listed in Tables 10-3 and 10-4. In Tables 10-4 and 10-5, R_N is the nominal total load, R_S is the nominal shaft skin friction load, $R_{S(U)}$ is the nominal ultimate shaft skin friction load, and R_B is the nominal end bearing load.

Table No. 10-4: Summary of Nominal and Factored Resistance Values for a 2.5 m (8.2 ft) Drilled Pier.

δ/D (%)	R_N (tons)	$R_S/R_{S(U)}$	R_S (tons)	R_B (tons)	ϕR_N (tons)
L = 5 feet					
0.6	4,700	0.84	1,147	3,553	2,522
1.0	7,200	0.70	956	6,244	3,743
1.4	9,600	0.56	765	8,835	4,915
2.5	16,072	0.17	229	15,843	8,071
L = 10 feet					
0.6	6,100	0.87	2,362	3,738	3,404
1.0	8,800	0.69	1,884	6,916	4,683
1.4	11,300	0.53	1,447	9,853	5,867
2.2	16,313	0.17	470	15,843	8,227
L = 15 feet					
0.6	7,400	0.89	3,645	3,755	4,247
1.0	10,200	0.68	2,785	7,415	5,518
1.4	13,000	0.48	1,966	11,034	6,795
2.0	16,565	0.18	721	15,844	8,391

Table No. 10-5: Summary of Nominal and Factored Resistance Values for a 3.3 m (10.8 ft) Drilled Pier.

δ/D (%)	R_N (tons)	$R_S/R_{S(U)}$	R_S (tons)	R_B (tons)	ϕR_N (tons)
L = 5 feet					
0.6	7,500	0.83	1,133	6,367	3,920
1.0	11,500	0.70	956	10,544	5,893
1.4	15,700	0.57	778	14,922	7,967
2.63	27,907	0.17	229	27,678	13,988
L = 10 feet					
0.6	9,400	0.86	2,335	7,065	5,050
1.0	13,700	0.70	1,912	11,789	7,137
1.4	18,000	0.54	1,475	16,525	9,221
2.36	28,225	0.17	470	27,755	14,183
L = 15 feet					
0.6	11,100	0.87	3,564	7,537	6,085
1.0	15,700	0.69	2,826	12,874	8,274
1.4	20,300	0.51	2,089	18,211	10,463
2.15	28,558	0.18	721	27,837	14,387

The evaluation presented herein is preliminary and is subject to verification based on performing additional soil borings and rock coring at the final locations of the bridge foundation units. Once the final foundation locations are selected, and loading information, settlement tolerance, and construction methods are known, this evaluation can be finalized to design the required drilled pier diameter and rock socket length for each foundation unit.

It is understood that a preliminary shaft diameter of approximately 98 to 168 inches (2.5 to 3.3 meters) is planned from a foundation load standpoint. For planning purposes, drilled piers should be spaced a minimum of 2 diameters apart (edge-to-edge, including any bells), although the final spacing should be confirmed on the basis of actual pier layout and geometry, loading, design depth, etc.

During the conceptual engineering of foundation systems expected to be subjected to lateral loading, preliminary values for the modulus of lateral subgrade reaction can be applied as follows:

Soil Stratum	Lateral Subgrade Reaction Modulus (pounds per cubic inch)
Granular Layers	60
Cohesive Layers	100
Hardpan	2,000
Bedrock (weathered)	2,500

If used in the conceptual engineering structural analyses, the above moduli (Welch and Reese, 1972, 1975) should be used in conjunction with caisson diameters, modeling method used, etc., to determine appropriate lateral caisson capacities.

Within test borings TB-105A, TB-107, and TB-108, layers of granular soils were encountered below the hardpan or below the cohesive soil layers extending to the bedrock. It is expected that the predominately sandy layer(s) will generally possess little to no standup time. In locations with this condition, it is expected that drilling slurry (as recommended above to control groundwater), will be necessary to preserve sidewall stability. Likewise, where this condition exists, the use of belled caissons in the soil will be difficult or impossible, since the bells cannot be relied on to be self supportive within predominantly granular layers.

In order to evaluate the feasibility of the drilled excavations in the soft to medium clay soil zones, probable overload factors (also termed stability numbers and defined as the ratio of overburden stress to soil shear strength), were calculated. Overload factors on the order of six to eight (6 to 8) typically indicate marginal sidewall stability, and values greater than eight (8) typically indicate squeezing conditions. Based on the soil data, it is estimated that the overload factors will approach twenty (20) as the excavation depth below the ground surface reaches the hardpan layers. This indicates that squeezing conditions will be present within the shaft excavations unless the shafts are drilled under slurry or fully cased. As such, the concept engineering should include provisions for the use of specialized drilling slurry for the full depth of the caissons through clay, as well as through granular layers overlying the hardpan and bedrock. In addition, the use of full-length steel casing may be preferred for isolated foundation elements in areas of known soil and groundwater contamination (particularly in the upper fill layers), as discussed earlier in this report. For larger groups of drilled pier elements, a perimeter cutoff wall will likely be more cost effective.

Based on local experience with subsurface conditions in the Detroit Area, as well as observations from test boring programs on nearby parcels, the possibility of random occurrence of toxic, noxious, and explosive gases in caisson excavations cannot be precluded, although proper gas monitoring will minimize the risk associated with such events.

10.3 DEEP (SLENDER) FOUNDATION ELEMENTS FOR SECONDARY FOUNDATIONS

Based on the concepts developed in the Bridge Conceptual Engineering Report, the secondary and approach pier foundation elements may be supported on long slender deep foundation elements. For the purposes of this report, such foundation elements will consist of drilled piers, or pre-drilled or driven piles. In all cases, multiple pier or pile elements would be required to support a main pile cap, in turn supporting the pier or other structural element.

10.3.1 Drilled Piers

Drilled piers would extended through the upper fill, silty clay, granular soil layers, and be founded at least 2 feet (0.6 meters) into the underlying hardpan soils, resulting in depths of approximately 90 to 100 feet (27 to 30 meters). The drilled pier should be designed for end bearing in the hardpan. For a drilled shaft constructed in this fashion, the nominal end resistance should be approximately 40 tsf (3.8 MPa) for conceptual design purposes, which corresponds to a settlement of approximately 5 percent of the shaft end diameter. A resistance factor of 0.55 should be used. Invariably, some load will be distributed into the overburden soils along the drilled pier. This load is difficult to quantify with certainty, and will be small relative to the end resistance mobilized in the hardpan. As a result, overpredicted settlements are likely due to some load transfer into the overburden soils.

10.3.2 Driven Piles

Based on the current bridge concepts, some bridge foundation elements (most likely approach piers) could potentially be supported on piles, which could involve various sizes of mandrel or top-driven concrete filled pipe piles, or top driven H-piles.

10.3.2.1 Pipe Piles

Pipe piles for this project could consist of concrete filled steel pipes, which are typically equipped with a driving boot. The pipe piles are then pre-drilled to within approximately 5 feet (1.5 m) of hardpan, mandrel driven to bear within the hardpan, and then filled with concrete. The concept engineering should include provisions for the use of drilling slurry for the full depth of the pre-drilled hole through clay, as well as through granular layers overlying the hardpan and bedrock.

Table 10-6 summarizes the nominal and factored pile driving resistance values for pipe piles recommended in MDOT BDM. The dynamic resistance factor (ϕ_{DYN}) presented by the MDOT BDM is equal to 0.4, and assumes that pile driving criteria will be developed by using the FHWA-modified Gates Dynamic formula.

Table No. 10-6: Conceptual Driving Resistance Values for C.I.P. Pipe Piles.

Pile	Axial (tons)	
	R_{NDR} (t)	$\phi_{DYN}R_{NDR}$
12" O.D. 0.25" wall	175	70
14" O.D. 0.312" wall	200	80
14" O.D. 0.438" wall	250	100
ϕ_{DYN} = 0.4 based on using the FHWA-modified Gates dynamic formula to establish driving criteria.		

We recommend considering the use of dynamic testing in developing driving criteria. If AASHTO guidelines for dynamic testing are followed, a dynamic resistance factor of 0.65 may be used instead of 0.4.

The pipe piles would need to be spaced at least 3 diameters apart to prevent significant reduction in capacity and increases in settlement due to grouping effects.

10.3.2.2 H-Piles

H-piles for this project would be top driven into the hardpan. Table 10-7 summarizes the nominal and factored pile driving resistance values for driven H-piles recommended in the MDOT BDM. As with pipe piles, the dynamic resistance factor (ϕ_{DYN}) presented by the MDOT BDM is equal to 0.4, and assumes that pile driving criteria will be developed by using the FHWA-modified Gates Dynamic formula.

Table No. 10-7: Conceptual Driving Resistance Values for C.I.P. Pipe Piles.

Pile	Axial (tons)	
	R_{NDR} (t)	$\phi_{DYN}R_{NDR}$
HP 12x53	200	80
HP14x102	400	160
ϕ_{DYN} = 0.4 based on using the FHWA-modified Gates dynamic formula to establish driving criteria.		

We recommend considering the use of dynamic testing in developing driving criteria. If AASHTO guidelines for dynamic testing are followed, a dynamic resistance factor of 0.65 may be used instead of 0.4.

Due to the relatively deep depth to the hardpan and because pre-drilling is not typically practical for H-piles, sweep of the piles during driving operations could be problematic. The piles would need to be spaced at least 5 dimensions apart in consideration of the potential for sweep, and to prevent significant reduction in capacity and increases in settlement due to grouping effects.

10.3.2.3 Environmental Considerations for Piles

Research indicates that there is a potential for pile foundations to enhance vertical migration of contamination through aquitards. The research indicates that a variety of factors may influence the potential for vertical migration, including:

- Pile shape (round, square, or H)
- Pile diameter
- Shape of pile bottom (flat or pointed)
- Installation method (driven or cast in place)
- Pile material (steel, concrete, or wood)

- Soil type of aquitard (stiff clay or soft clay)
- Thickness of aquitard
- Amount of aquitard penetration (partial or full)

Research indicates that installation of certain types of piles under certain conditions may lead to a ten times increase in vertical contaminant flow through the aquitard. In general, bored piles may lead to more vertical flow than driven piles. Also, H piles that are driven may lead to more vertical flow than round piles that are driven.

During the brine well and geotechnical investigations, oversized steel environmental casings and specialized cementing methods were used to prevent vertical migration. Since these investigations required open boreholes, this environmental casing was deemed an appropriate precaution. However, for permanent piles, such preventative measures may or may not be required. Due to the numerous factors that may affect the potential for increased vertical migration, it would not be appropriate to make a recommendation at this time. However, based on the concerns of the MDEQ and former liable owner, and the high visibility of this project; the potential for piles to increase vertical migration of the existing contamination should be considered during the final foundation design.

10.4 DEEP EXCAVATIONS FOR FOUNDATION INSTALLATION

For several of the bridge foundation concepts that have been developed (particularly for installation of a suspension bridge anchorage), a relatively large excavation will need to be extended into the bedrock. Based on the concepts developed as part of the Bridge Conceptual Engineering Report (as discussed above), such excavations could potentially consist of a circular shaft 165 feet (50 m) in diameter or two rectangular shafts 35 by 200 feet (10 by 60 m) in plan dimension. Two hybrid designs are also proposed that consist of smaller diameter circular and rectangular caissons, in combination with drilled piers. In any case, the excavation support systems would extend to the top of (or into) rock, which can be expected to be approximately 100 feet (30 m) below ground surface. Such excavations would require a significant earth retention system, which could potentially consist of an internally braced tangent pile shaft, structural slurry wall, or sinking caisson. In addition, internally braced steel sheeting has been considered, although this option is not considered to be practical. These options are discussed below.

10.4.1 Tangent Pile Shaft

A tangent pile shaft involves augering a series of holes, tangent to one-another (usually staggered in plan view), through the overburden soils to bedrock in pattern around the planned shaft location. Heavy steel members or reinforcement rods are then placed vertically into the holes, with the annulus tremie-grouted with concrete. The steel effectively provides resistance to bending in the vertical direction. As excavation proceeds, steel reinforcing beams (rectangular shaft) or reinforced concrete “ring beams” (circular shaft) are cast-in-place around the inside perimeter of the tangent pile walls. The steel bracing or concrete ring beams act as horizontal compression members, resisting the resulting soil pressures acting on the outside of the shaft.

Due to the large soil loads, the steel bracing or concrete beams would need to be placed at close vertical spacing (probably in the range of 5 to 10 feet or 1.5 to 3 meters), resulting in a large number of bracing levels over the entire depth of the shaft. Also, the shaft must be designed such that the tangent pile walls can resist lateral earth loads below the base of the excavation during construction. Such forces for the expected necessary size shaft would be large, with associated steel members also large. A major risk of this method involves inward creep of the tangent piles and squeezing of soil through gaps between tangent piles; both issues are due to high soil pressure and associated high overload factors as the shaft depth approaches the hardpan. A generalized plan view of a typical tangent pile shaft is shown in Figure 10-8.

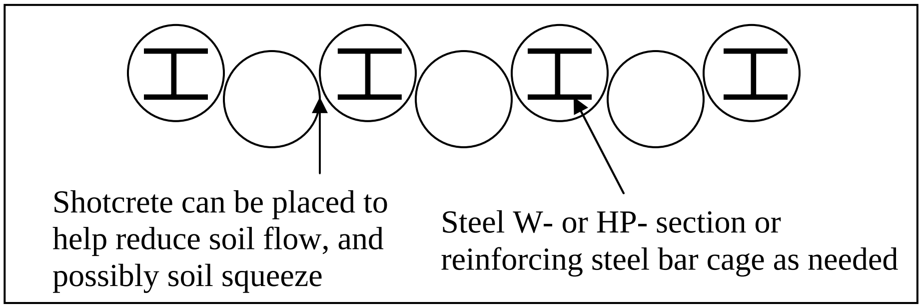


Figure No. 10-8: Schematic Plan View of a Typical Tangent Pile Wall.

A hybrid of this alternative, termed a secant pile shaft, consists of a similar installation of augered bore holes that are spaced having a clear distance less than one bore diameter away from each other. These initial spaced holes are backfilled with lean concrete and have no steel reinforcing. After these initial lean concrete piles have cured, the space between the lean concrete piles is augered, along with a portion of the adjacent lean concrete piles. Reinforcing steel and structural concrete is placed in these intersecting (i.e., secant) piles. The main support derives from the reinforced piles, while the lean concrete piles act as lagging. Similar risks are associated with a secant pile shaft as were noted for the tangent pile shaft. The benefit of using a secant pile shaft is that the amount of soil squeeze between structural support elements should be reduced. A generalized plan view of a typical secant pile shaft is shown in Figure 10-9.

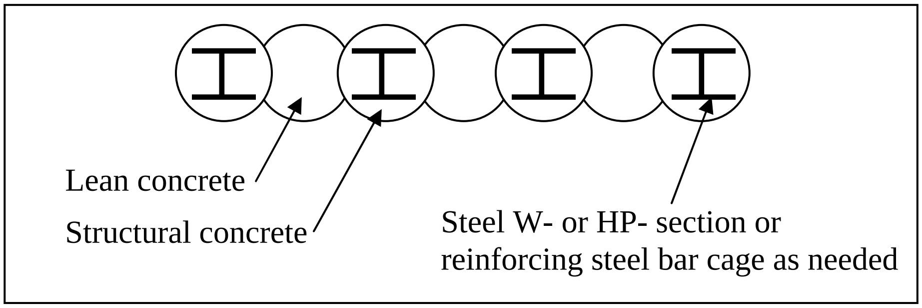


Figure No. 10-9: Schematic Plan View of a Typical Secant Pile Wall.

10.4.2 Steel Sheeted Shaft

A steel sheeted shaft would be similar in concept to a tangent pile shaft, with the steel sheeting most likely being driven within a starter pit to reduce sheeting lengths. Similar to the tangent pile shaft concept, as the excavation proceeds, steel bracing or reinforced “ring beams” are then placed around the inside perimeter of the sheeting. Also similarly, the supports would need to be placed at close vertical spacing and the sheeting would need to be designed to resist lateral earth loads below the base of the excavation during construction. Major risks involved with this method involve jumping of interlocks in the sheets during driving, resulting in squeezing of soil into the shaft through gaps, as well as inward creep due to high ground pressure. In addition, pre-drilling may be required to remove or dislodge cobbles and thus allow adequate sheeting penetration. The noted risks probably make the use of this method impractical for the DRIC project.

10.4.3 Slurry Wall Shaft

A slurry wall shaft, also known as a diaphragm wall, is constructed by excavating to or into bedrock a series of deep trenches, each of a short finite length, to the designed bottom depth of the shaft. Each trench is excavated and kept open using slurry material, usually a mixture of bentonite, water, and additional additives that modify the slurry’s dynamic properties. Following excavation, steel reinforcement is placed in the trench, and the trench is then tremie grouted with structural concrete. Following the concrete placement, the next trench is excavated, and the process is repeated. Such walls can be constructed in a circular plan configuration, which resists soil and water loads primarily through compression, with some secondary bending affects. However due to large compression loads and buckling effects, larger shafts constructed in this manner often utilize internal support, such as internal ring beams. For a rectangular shaft, a concrete slurry wall would probably require installation of internal support as the shaft excavation proceeds.

When considering the size of the proposed shaft and the prevailing soil conditions, slurry wall construction methods offer a few distinct disadvantages. Although these types of walls can carry significant load in compression, they are sensitive to unsymmetrical loading. This is critical when considering the trench excavations would take place in soft soils. Slurry trenches in soft soils have historically resulted in poor control of squeezing within deep excavations, although this can usually be controlled through careful construction practices. Another potential disadvantage is that slurry wall construction is a technique somewhat unfamiliar to the local construction community.

10.4.4 Sinking Caisson Construction

A sinking caisson could either be constructed in as a circular shaft or rectangular shaft. A circular shape is much more common and is very efficient structurally, because the walls act as a circular compression beam. A rectangular or square caisson typically requires very thick and highly reinforced walls, although this can sometimes be reduced through the use of internal diaphragm walls cast between the outside long-span walls to provide support.

For this project, the caisson would be constructed through variable fill soils, soft to medium cohesive soils, loose to very compact silty/sandy soils, very stiff to very hard clayey hardpan, then very stiff to stiff silty clay, and finally bedrock. During excavation, bentonite injection is typically be used to advance the shaft downward. The general sequence of the installation would then involve completing the shaft by underpinning the shaft to bedrock. Excavating and/or blasting the underlying rock will be necessary to install the lower portion of the shaft into competent bedrock. The general concept for the sinking caisson construction method by stage is shown on Figure 10-10 and Figure 43 in Attachment A, and is discussed in greater detail in the following paragraphs.

Figure No. 10-10: Sinking Caisson Construction Schematic.

The sinking caisson construction method generally begins with the removal of any existing structures, sewer pipes, piles, and other obstructions within the proposed shaft area, followed by a bedrock and soil/rock interface grouting program (discussed previously). Following the grouting program, a cutoff wall may be installed for groundwater control purposes, and a steel sheet pile wall retaining system may be used to execute the launching pit excavation.

Given the existence of variable fill material, a concept design involving a sinking caisson should include the caisson being launched from a shaft starter pit excavated larger than the shaft footprint. The launching pit for this project would almost certainly be constructed within a braced excavation, and due to groundwater, environmental and face instability concerns of the fill soils, the excavation bracing would probably consist of steel sheet piling. Given the large diameter of the caissons, it likely that internal bracing for a launching pit will consist of cast-in-place concrete ring beams. All fill soils within the shaft starter pit would be removed prior to constructing the launching platform, and a pad of engineered fill would be constructed on native or otherwise suitable fill soil. The launching pit excavation would then be backfilled to a specified elevation such that flooding during construction can be avoided. Steel sheeting installed to cut-off potential connection to the river and near surface groundwater (perched) may also be installed.

The concept design should consider that any sheets that are driven through the fill soils will encounter multiple obstructions and result in very difficult driving, misalignment, and jumping sheet pile interlocks. As such, the concept design should include provisions that, prior to pile driving, the line of sheeting be pre-excavated and filled in limited sections to remove major obstructions.

Once the launching pit is completed, construction is started on the first couple of lifts of the caisson, on the launching platform within the starter pit. The first lift contains a “cutting shoe” that consists of a tapered edge to allow the tip to cut into the soil as soil is removed from the interior of the shaft. After the first and second lifts have gained appropriate strength, the launching platform is segmentally removed and the caisson begins to sink into the soil profile generally via a controlled soil failure at the cutting shoe tip. The shaft is subsequently sunk to a pre-determined depth by excavating within the shaft to reduce bearing of the cutting shoe and frictional resistance on the shaft wall, adding concrete lifts, and injecting bentonite to the shaft exterior to decrease skin resistance. The actual sinking of the caisson occurs whenever the total weight of the constructed portion exceeds the soil bearing support of the cutting shoe, hydrostatic uplift pressure, and the overall skin resistance of the soil on the shaft walls. During construction, a proper balance between these forces must be maintained to ensure shaft sinking is controlled.

Sinking the caisson in the wet has been considered to reduce the groundwater cutoff requirements, although this method is often problematic when sinking through harder soil layers at great depth, and is not recommended for this project.

Caissons sink incrementally and the large shaft diameters required for the primary foundation elements will require substantial effort to maintain plumbness. Maintaining roundness is also an issue for circular sinking caissons, since the wall elements are typically very thin in comparison to the diameter, and may be subject to buckling if unbalanced loading occurs. Rectangular caissons typically have thicker walls with respect to the wall length, and are less susceptible to bucking. These issues are generally controlled by excavating in particular areas of the caisson interior as required. At times the excavation may be asymmetrical to achieve the desired result in the caisson structure.

Sinking a caisson through the anticipated loose to very compact granular layers encountered above the hardpan soils in some locations will require the use of a cut-off wall to form a barrier between the caisson sinking operation and the existing groundwater and contaminated soils (also expected to be variably impacted). This could consist of a slurry wall as discussed above, or a jet grout cut-off wall extending down into hardpan expected near Elevation 500 feet (Elevation 152.4 meters). If used, such a wall should be designed to also intercept the intermittent granular layers above/below the hardpan where encountered (specifically on Crossing X-10). The jet grout cut-off wall would have such dimensions as to allow sufficient room for caisson sinking activities and subsequent ground settlement impacts. In any case, provisions for the concept design should include monitoring the effectiveness of the cut-off wall, such as monitoring wells on each side of the cut-off wall screen within the granular soils zone.

Potential risks associated with the sinking caisson method include the potential that untimely injection of the bentonite lubricating fluid may result in undesirable movement of the caisson; there is some potential for ground settlement; and that the success of the system is strongly dependent on development of, and adherence to a well thought-out sinking plan.

Although there are a number of local contractors that are very familiar with the sinking caisson method, there are others less familiar. Since the method requires diligent control of the ground and careful excavation to maintain plumbness (and roundness in the case of circular caissons), it would be prudent for the final design, that very stringent qualification requirements be included in the project specifications.

10.4.5 Shaft Wall Construction at Soil/Rock Interface

Specialized construction methods will be required to construct the shaft walls at the bedrock interface and to the desired elevation within the bedrock. The shaft will probably be advanced by interior soil excavation until rock is encountered beneath the shaft wall at approximately Elevation 485 to 494 feet (Elevation 147.8 to 150.6 meters)

For the Sinking Caisson option discussed in the previous section, the shaft would then be stabilized by replacing the lubricating bentonite on the exterior of the caisson with cement grout injected through the bentonite injection pipes. As the bedrock surface is expected to vary beneath the shaft, ground stabilization (jet grouting or ground freezing) would then be employed to stabilize the zone between the caisson shoe and the bedrock interface prior to underpinning the caisson. The same method would also hold true for a structural slurry wall designed shaft. Depending on the continuity and success of the bedrock grouting program, the ground stabilization must be designed for groundwater cutoff, partial support of the shaft weight during underpinning (sinking caisson option only), and soil/hydrostatic pressures.

Depending on the final anchorage design, the sinking caisson shaft may require advancement into the bedrock, either for bearing, or to provide added resistance to lateral load. This is typically accomplished by excavating into the rock after the caisson itself is underpinned and locked into place. The entire shaft bottom, or only the perimeter, can then be excavated into the rock, depending on the bearing requirements and lateral passive resistance requirements. Passive resistance may also be developed by constructing key-ways in the rock.

Where passive resistance is needed to provide a reaction to lateral anchorage loads, the following lateral passive equivalent fluid pressure values are suggested for conceptual design purposes.

Soil Stratum	Equivalent Fluid Pressure (pounds per cubic foot)
Overlying soil material	135
Hardpan	257
Bedrock	330

This may be combined with a coefficient of frictional resistance component between rock and the concrete anchorage foundation of 0.65. It is expected that these values may be refined based on further testing and evaluation that would be conducted during the final design.

10.5 FOUNDATION COST CONSIDERATIONS

For development of concept level cost estimates for piles and drilled piers, the use of RSMeans (annual construction cost estimating publication) or local bid tabulation data is recommended. Costs for tangent pile walls and braced sheeting are also probably best developed from local bid tabulations for similar work.

Cost estimates for slurry wall foundations can be developed from RSMeans. However, this type of construction is not well known to the local construction community, and as a result, costs in the Detroit area may be greater than those in areas where this method is commonly used. For a recent project in the Detroit area, the bid tabulation indicated a unit cost for a slurry wall of similar depth in similar soils at $100 per square foot.

Based on our experience, caisson construction is generally more cost-effective than other methods for projects of this type in similar soil and groundwater conditions. For the purposes of developing conceptual cost estimates, examples of cost comparisons for caisson construction are available in “Sinking Caissons as an Effective Means of Construction Shafts”, 1997 Rapid Excavation and Tunneling Conference in Las Vegas, Nevada, June 22-25, 1997, presented as Figure No. 23 in Attachment A. The paper presents the results of a study of case histories on projects where sinking caissons were selected to deal with the impact of soil and groundwater conditions (commonly called bad ground conditions) on the performance and constructability of different shaft systems. The paper also looks at factors that have led to the success or failure of the systems. Cost data for five projects have been reviewed and compared to cost of shaft installation by other means.

For cost estimating drilled piers, Means may also be used, although local bid tabulation information may be more useful to provide up-to-date regional unit cost data. Based on recent projects we are familiar with in the Detroit area involving large diameter drilled piers extending to rock and drilled under slurry, the following unit costs for developing a concept-level cost estimate for the drilled piers are suggested:

- Assuming full length steel cages and slurry installation (necessary for the known site conditions), assume about $1,400/cy of concrete, in-place. If the steel cage can be reduced to only extend about 50 feet deep, the unit rate can be reduced by 10 percent.

- For piers to be socketed into rock (necessary for the main piers), assume an additional $2,700/cy for the rock excavation and removal (under slurry). For the main piers, it is suggested herein that the piers be socketed at least 5 feet. For estimating purposes, a slightly deeper socket may be considered to provide for variable conditions.

For all foundation installation, the presence of contamination in the upper water bearing granular layers should be considered in developing cost estimates. As indicated elsewhere in this report, the most practical method for preventing cross contamination of upper aquifers is, most likely, the use of a perimeter cutoff wall such as steel sheeting or slurry wall, surrounding each group of piles or drilled piers. The cost of such a wall can be developed from RSMeans or similar cost data, or from local bid tabulations for similar construction.

In any case, concept level cost estimates for foundations should consider what may be fluctuating prices for steel, concrete and fuel, which have resulted in several recent projects being bid significantly higher than the design estimate.

10.6 FOUNDATION CONCLUSIONS AND RECOMMENDATIONS

Various methods for constructing the several different foundation elements are discussed above. Although a number of different construction methods are considered feasible for the various foundation elements, certain methods of construction appear to offer advantages in terms of least risk and probable lowest cost. The following sections provide recommended foundation types and construction methods for the individual foundation elements. It should be noted that these recommended methods are for the purpose of this feasibility report and for feasibility-level cost estimation. It is expected that upon final design, these suggested methods will be refined and modified to reflect the final design requirements.

10.6.1 Cable Stay and/or Suspension Towers

Drilled piers with pier cap and tie-beams are considered the most likely foundation type for these elements. The piers should be drilled under slurry, with the upper fill (20 to 40 feet, depending on location) cased to reduce the possibility collapse of unstable material. The pier area should be isolated using a cutoff wall, to prevent cross contamination of upper aquifers. The piers should be socketed at least 5 feet into rock. Final pier diameter and rock socket length should be determined based on the applied loading and settlement tolerance.

10.6.2 Suspension Anchorages

All of the options presented in the DRIC Engineering Report (and in Section 10.1, above) are considered feasible for installation of the suspension anchorages. However, the specific anchorage loads and sizes are not known at this time and thus detailed cost estimating is not possible at this time.

10.6.3 Bridge Approaches

Drilled piers with a pier cap are considered the most likely foundation type for these elements. Similar to the main pier foundations, the piers should be drilled under slurry, with the upper fill (20 to 30 feet, depending on location), cased to reduce the possibility for collapse of unstable material. Where these foundation elements are isolated or not closely spaced, it may be practical to design the upper casing method to satisfy the requirement for prevention of cross-contamination of aquifers (i.e., a perimeter cutoff wall may not be the most cost effective solution for such cases). The piers may be supported in hardpan or on the top of rock, depending on the required settlement limitations. Final pier diameter should be determined by applied loading and settlement tolerance.

11.0 SEISMIC DESIGN CRITERIA

The 2003 Michigan Building Code (MBC) states that the site shall be classified as one of the site classes defined in Table 1615.1.1. Where the soil shear velocity is not known, site class shall be determined, as permitted in Table 1615.1.1, from standard penetration resistance or from soil undrained shear strength, calculated in accordance with Section 1615.1.5. The site soil profile does not contain any soils having one or more of the characteristics that would require the site to be classified as Site Class F. Therefore, according to Section 1615.1.5.1 of the MBC, and for the purposes of the concept level design, the site should be classified as Site Class E. A site-specific investigation including determination of shear wave velocities for the various soil layers should be performed prior to actual design of foundation elements to confirm this assessment.

12.0 LIMITATIONS

The evaluations and preliminary recommendations presented in this report have been formulated on the basis of generalized data in the vicinity of the proposed bridge crossings, together with current preliminary concepts for the bridge and foundations. As such, all of the preliminary conclusions presented herein are considered appropriate for concept-level evaluations of the design and for concept-level cost estimating. Experience indicates that the actual sub-soil conditions at the actual final locations of all the primary and secondary foundation elements may vary from those explored and presented in this report. Therefore, a comprehensive final design-specific geotechnical investigation should be performed to provide geotechnical exploration and analysis at the locations of each primary and secondary foundation element.

The scope of the present investigation was limited to the preliminary site specific evaluation of subsurface conditions for the support of the proposed bridge foundations. Considerations relating to environmental concerns beyond those specifically mentioned in this text, or other possible regulatory restrictions on development, were not included in the scope of this investigation.

Respectfully Submitted,

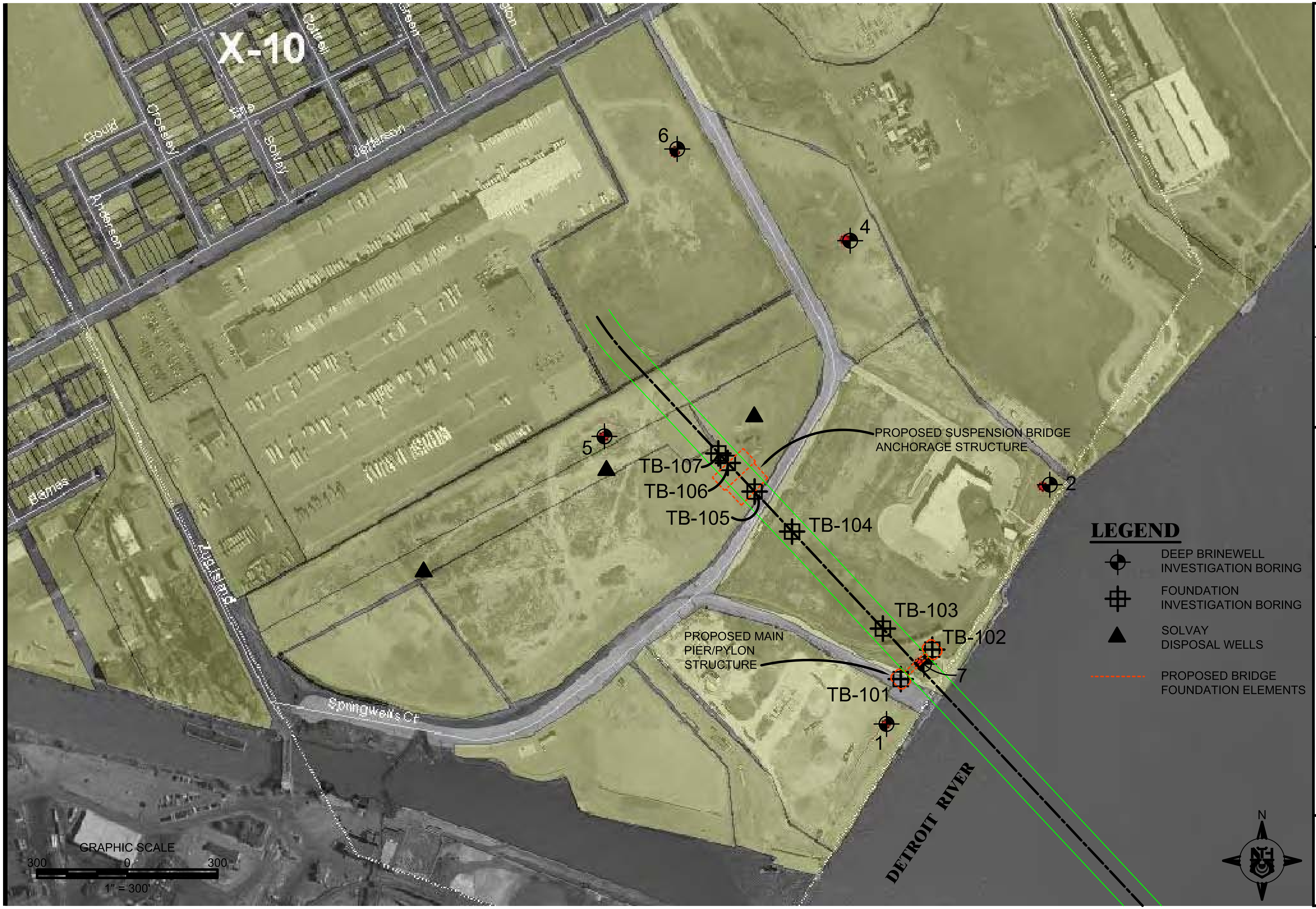
NTH Consultants, Ltd.

Craig R. Johnson Project Engineer Geotechnical Investigation	Donald C. Wotring, Ph.D., P.E. Project Engineer Geotechnical Analysis

Fritz J. Klingler, P.E.
Project Manager

JNS/CRJ/FJK

Attachments



NTH Consultants, Ltd.
Infrastructure Engineering
and Environmental Services

NTH PROJECT No.: 15-050014-12
CAD FILE NAME: D05001457
DESIGNED BY: CRJ
INCEP DATE: 10 MAR 2008
DRAWN BY: SHB
DRAWING SCALE: 1" = 300'
CHECKED BY: FJK
PLOT DATE: 15 SEP 2008

PRELIMINARY TEST BORING LOCATION PLAN SITE X-10 FOUNDATION BORINGS

DETROIT RIVER INTERNATIONAL CROSSING
DETROIT, MICHIGAN

FIGURE No.
1A

LEGEND

⌖	DEEP BRINEWELL INVESTIGATION BORING
⊞	FOUNDATION INVESTIGATION BORING

NTH Consultants, Ltd.
Infrastructure Engineering
and Environmental Services

NTH PROJECT No.: 15-050014-12
CAD FILE NAME: D05001460
DESIGNED BY: CRJ
INCEP DATE: 17 MAR 2008
DRAWN BY: SHB
DRAWING SCALE: 1" = 300'
CHECKED BY: FJK
PLOT DATE: 20 AUG 2008

PRELIMINARY TEST BORING LOCATION PLAN
SITE X-11 FOUNDATION BORINGS

DETROIT RIVER INTERNATIONAL CROSSING
DETROIT, MICHIGAN

FIGURE No.
1B

GENERAL NOTES

TERMINOLOGY

Unless otherwise noted, all terms utilized herein refer to the Standard Definitions presented in ASTM D 653.

PARTICLE SIZES

Boulders	-	Greater than 12 inches (305mm)
Cobble	-	3 inches (76.2mm) to 12 inches (305mm)
Gravel - Coarse	-	3/4 inches (19.05mm) to 3 inches (76.2mm)
Fine	-	No. 4 - 3/16 inches (4.75) to 3/4 inches (19.05mm)
Sand - Coarse	-	No. 10 (2.00mm) to No. 4 (4.75mm)
Medium	-	No. 40 (0.425mm) to No. 10 (2.00mm)
Fine	-	No. 200 (0.074mm) to No. 40 (0.425mm)
Silt	-	0.005mm to 0.074mm
Clay	-	Less than 0.005mm

CLASSIFICATION

The major soil constituent is the principal noun, i.e., clay, silt, sand, gravel. The second major soil constituent and other minor constituents are reported as follows:

Second Major Constituent (percent by weight)	Minor Constituents (percent by weight)
Trace - 1 to 12%	Trace - 1 to 12%
Adjective - 12 to 35% (clayey, silty, etc.)	Little - 12 to 23%
	Some - 23 to 33%
And - Over 35%	

COHESIVE SOILS

If clay content is sufficient so that clay dominates soil properties, clay becomes the principal noun with the other major soil constituent as modified; i.e., silty clay. Other minor soil constituents may be included in accordance with the classification breakdown for cohesionless soils; i.e., silty clay, trace of sand, little gravel.

Consistency	Unconfined Compressive Strength (psf)	Approximate Range of (N)
Very Soft	Below 500	0 - 2
Soft	500 - 1000	3 - 4
Medium	1000 - 2000	5 - 8
Stiff	2000 - 4000	9 - 15
Very Stiff	4000 - 8000	16 - 30
Hard	8000 - 16000	31 - 50
Very Hard	Over 16000	Over - 50

Consistency of cohesive soils is based upon an evaluation of the observed resistance to deformation under load and not upon the Standard Penetration Resistance (N).

COHESIONLESS SOILS

Density Classification	Relative Density %	Approximate Range of (N)
Very Loose	0 - 15	0 - 4
Loose	16 - 35	5 - 10
Medium Compact	36 - 65	11 - 30
Compact	66 - 85	31 - 50
Very Compact	86 - 100	Over 50

Relative density of cohesionless soils is based upon the evaluation of the Standard Penetration Resistance (N), modified as required for depth effects, sampling effects, etc.

SAMPLE DESIGNATIONS

AS	-	Auger Sample - directly from auger flight
BS	-	Miscellaneous Sample - bottle or bag
S	-	Split Spoon Sample - ASTM D 1586
LS	-	Split Spoon Sample S with Liner Insert 3 inches in length
ST	-	Shelby Tube Sample - 3 inch diameter unless otherwise noted
PS	-	Piston Sample - 3 inch diameter unless otherwise noted
RC	-	Rock Core - NX core unless otherwise noted
CS	-	Continuous Sample - from rock core barrel or continuous sampling device

STANDARD PENETRATION TEST (ASTM D 1586) - A 2.0" outside-diameter, 1-3/8" inside-diameter, split barrel sampler is driven into undisturbed soil by means of a 140-pound weight falling freely through a vertical distance of 30 inches. The sampler is normally driven three successive 6-inch increments. The total number of blows required for the final 12 inches of penetration is the Standard Penetration Resistance (N).

Figure No. 2A

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2000 Brush Street
Detroit, Michigan 48226
Phone: 313.237.3900
Fax: 313.237.3909

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SUMMARY OF ROCK LOG NOMENCLATURE

RUN NUMBER
The number of the individual coring interval starting at the rock interface.

ROCK TYPE/DESCRIPTION
Description of the color, grain size or texture, bedding, foliation, lithology and mineralogy.

Color - When describing the color, use only common colors such as gray, brown, green, etc., or simple combinations of these (e.g., yellow-brown). The degree of color (light vs. dark) should also be employed.

Grain Size/Texture - Terminology used to identify size, shape, and arrangement of the constituent elements: e.g., porphyritic, glassy, amygdaloidal, etc.

Where applicable, the following size classification is utilized:

• Amorphous	Particles too small to be seen with the naked eye.
• Fine grained	Particles barely seen with naked eye.
• Medium grained	Particles barely seen with naked eye to 1/8 in.
• Coarse grained	Particles between 1/8 in. and 1/4 in.
• Very coarse	Particles greater than 1/4 in.

Bedding or Foliation - A bed (or foliation) is the smallest diversion of a stratified series, and marked by a well defined divisional plane from strata or layers above and below. Bedding is the collective term signifying the existence of beds or laminae.

The relative thickness of the bedding planes shall be described as follows:

Bedding Planes	Spacing
• Laminated	Less than 0.4 in. (1 cm)
• Very thin	0.4 inch (1 cm)
• Thin	2 to 12 inches
• Medium	1 to 3 feet
• Thick	3 to 10 feet

Lithology – Rock name or classification and modifiers such as Limestone, Shaly Limestone, Shale, Calcareous Shale, etc.

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Detroit, Michigan 48226
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WEATHERING/ALTERATION

Weathering (alteration) of the rock (mineral fabric) is caused by mechanical and chemical action (temperature variations, water, bacteria, physical and chemical attack) and produces deterioration of the rock fabric leading eventually to a disaggregated mass resembling soil. The terms used to describe the relative degree of weathering are as follows:

• F - Fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
• SW - Slight	Discoloration indicates weathering of rock material and discontinuity surfaces may be somewhat weaker externally than in its fresh condition.
• MW - Moderate	Less than half the rock material is decomposed and or disintegrated to a "soil”. Fresh or slight weathered rock present either as a continuous framework or as corestones. Large pieces cannot be broken by hand.
• HW - High	More than half the rock material is decomposed and/or disintegrated to a soil. Rock so weakened by weathering that fairly large pieces can be crumbled by hand. Fresh or discolored rock (slight) may be present as a discontinuous framework or as corestones.
• CW - Complete	Rock reduced to "soil". Rock "fabric" not discernible or discernible only in small scattered locations.
• RS - Residual Soil	The original minerals of the rock have been entirely altered to secondary minerals and the original rock fabric is not apparent.

FIELD HARDNESS

A measure of resistance to scratching or abrasion. The descriptions of the relative degrees of hardness are as follows:

• S - Soft	Reserved for plastic material only.
• F - Friable	Easily crumbled by hand, pulverized or reduced to powder and is too soft to be cut with a pocket knife.
• LH - Low Hardness	Can be gouged deeply or carved with a pocketknife.
• MH - Moderately Hard	Can be readily scratched by a knife blade. Scratch leaves heavy trace of dust and scratch is readily visible after the powder is blown away.
• H - Hard	Can be scratched with difficulty; scratch produces little powder and is often faintly visible; traces of the knife steel may be visible.
• VH - Very Hard	Cannot be scratched with pocketknife; leaves knife steel marks on surface.

GRAPHIC LOG OF FRACTURES

A scaled representation of fractures and discontinuities observed along the length of the core run. Fracture angles with respect to the longitudinal axis of the core run shall be noted were applicable.

Figure No. 2B

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Detroit, Michigan 48226
Phone: 313.237.3900
Fax: 313.237.3909

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DESCRIPTION OF ROCK DEFECTS

Description of rock defects shall include information regarding **discontinuities** as well as **solution cavities** or **voids**.

Discontinuities - Surface representing breaks or fractures separating the rock mass into discrete units.

The types of discontinuities are as follows:

• Crack	A partial or incomplete fracture.
• Joint	A simple fracture along which no visible shear displacement has occurred. May occur with parallel joints to form a joint set.
• Shear	A fracture along which differential movement has taken place parallel to the surface sufficient to produce slickensides, striations or polishing. May be accompanied by a zone of fractured rock (shear zone).
• Fault	A major fracture along which there has been measurable/observable displacement; often accompanied by clayey gouge and/or a severely fractured adjacent zone of rock.
• Shear or Fault Zone	A band or zone of parallel or sub-parallel shears and/or faults.

Discontinuity Spacing – The spacing should be measured in feet to the nearest tenth perpendicular to the plane in the set.

- IF – Intensely Fractured - <0.3ft
- CF – Closely Fractured – 0.3 to 1.0ft
- MF – Moderately Fractured – 1.0 to 3.0ft
- WF – Widely Fractured – 3.0 to 6.0ft
- VWF – Very Widely Fractured - >6ft

Surface Roughness - The terms used to describe the relative degree of surface roughness of the discontinuity are as follows:

• VR - Very Rough	Near "vertical" steps and ridges occur on the discontinuity surface.
• R - Rough	Some ridges and side-angle steps are evident; asperities are clearly visible; discontinuity surface feels very abrasive.
• SR - Slightly Rough	Asperities on the discontinuity surface are distinguishable and can be felt.
• S - Smooth	Surface appears smooth
• SLK - Slickensides	Visual evidence of striations or a smooth glassy-appearing finish.

Other terms used for surface roughness can include stepped, planar, and undulating.

Figure No. 2B

Dip/Attitude - The terms used to describe the angle of inclination of the discontinuities with respect to the plane normal to the longitudinal axis of the core run are as follows:

• Horizontal	0 to 5 degrees
• Low Angle	5 to 35 degrees
• Moderately dipping	35 to 55 degrees
• Steep or high angle	55 to 85 degrees
• Vertical	85 to 90 degrees

Discontinuity Infilling - A description of the mineralogy, thickness and hardness of observed discontinuity infilling should be noted.

The terms used to define the relative degree of infilling are as follows:

- ST - Surface stain
- Sp - Spotty
- P - Partially filled; half of surface or opening is filled
- F - Filled (partially)
- H - Healed

Solution Cavities and Voids - Open spaces in the subsurface are generally due to removal of rock material by chemical dissolution or the action of running water. Since most of these voids result from the action of groundwater, the openings are not usually equi-dimensional, but rather are elongated in the horizontal plane.

The relative size of voids and cavities are as follows:

- Pit or pitted - Voids barely seen with the naked eye to 1/4 in.
- Vug - Voids 1/4 in. to 2 in. in diameter
- Cavity - Holes 2 in. to 2 ft. in diameter
- Cave - Holes 2 ft. and larger in diameter

PERCENT CORE RECOVERY
The amount of core actually recovered divided by the length of the run (expressed as a percentage). Both intact and weak rock including gravel sized pieces are included in the percent recovery.

RQD (ROCK QUALITY DESIGNATION)
Total length of all "intact" pieces of core greater than 4-inches in length measured along the centerline of the core, divided by the total length of the run. Mechanical discontinuities such as those resulting from the core operation or handling of the core sample should not be included in the length measurements for RQD.

FRACTURES/FOOT
The number of naturally occurring fractures observed over the length of the recovered core divided by the length of the total core run.

CORE BOX NUMBER
The box number in which the core is stored.

COMMENTS
Comments include information on drilling water losses, reasons for core loss or fracture, gas readings, average pull-down pressure used to advance the run, total time required to complete the run and any other data pertinent to the core operation and/or condition of the core.

Miscellaneous Features - Any additional characteristics to further identify and evaluate the rock from the standpoint of engineering properties: secondary mineralization, fossils, swelling and slaking properties, etc.

LOG OF TEST BORING NO: TB-101

NTH Consultants, Ltd.

Project Name: *DRIC Preliminary Foundation Investigation*

NTH Proj. No.: *15-050014-00*

Project Location: *Detroit, MI*

Checked By

SUBSURFACE PROFILE

SURFACE ELEVATION: 581.2

ELEV (FT)	ELEV	DESCRIPTION	DEPTH
580	579.6	FILL - Brown SANDY SILT, Trace Gravel	1.5
575	574.7	FILL - Medium Compact to Compact Black SAND, GRAVEL, and COAL Fragments	6.5
570			
565			
560	560.2	FILL - Loose to Medium Compact Brown to Grey SAND with Some Gravel	21.0
555			
550			
545		Loose to Medium Compact Grey SAND with Trace Silt	

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/6 INCHES	[illegible] (IN)	[illegible] (IN)	FIELD TEST	MOIST. CONTENT (%)	DRY DENSITY (PCF)	UNCONF. COMPR. (PSF)
	LS-1	21 21 17	38	10				>9000*
5	LS-2	6 6 6	12	12				
	LS-3	7 10 7	17	15				
10	LS-4	2 4 5	9	10				
15	LS-5	3 4 5	9	17				
20	LS-6	4 4 6	12	17				
	LS-7	4 4 6	10	9				
25								
	LS-8	1 3 7	10	15				
30								
	LS-9	6 6 8	14	15.5				
35								

Total Depth: *152.0 FT*
Drilling Date: *5-19-08 to 5-30-08*
Inspector: *M. Flanagan/S. Emery*
Contractor: *DLZ*
Driller: *D. Hoskins/J. Pollnest*

Drilling Method
CME 750 ATV Mounted Drill Rig with a combination of 3 1/4 inch Inside Diameter Hollow Stem Auger, 12 1/4 inch Tri Cone Rotary Bit, NQ/NX Wireline Tooling, and NWJ Rods

Plugging Procedure
Pneumatic Piezometer, Grout, Casting cut 4 feet below grade

Water Level Observation
Groundwater encountered at 13.5 feet during drilling. The use of drilling fluids precluded further groundwater measurement below 59.5 feet

Notes:
** - Pocket Penetrometer Value*
WOH - Weight of Hammer
8-inch Inside diameter steel casting installed/grouted to 59.5 feet

Figure No. 3

LOG OF TEST BORING NO: TB-101

NTH Consultants, Ltd.

Project Name: *DRIC Preliminary Foundation Investigation*

NTH Proj. No. *15-050014.00*

Project Location: *Detroit, MI*

Checked By

SUBSURFACE PROFILE

SURFACE ELEVATION: 581.2

ELEV (FT)	ELEV		DEPTH
540			
535			
530	530.2		51.0
		Medium Compact Gray SILTY SAND	
525	525.7		55.5
520			
515			
		Soft to Medium Gray SILTY CLAY with Trace Sand and Gravel	
510			
505			
500			

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/6 INCHES	STD. PEN. RESIST. (N)	REC. (in)	FIELD TEST	MOIST. CONTENT (%)	DRY DENSITY (PCF)	UNCONF. COMPRESS. (PSF)
	LS-10	2 6 7	13	13				
40								
	LS-11	5 7 10	17	18		19.3	109.1	
45								
	LS-12	3 5 5	10	15				
50								
	LS-13	3 6 13	19	18				
55								
	LS-14	2 4 6	10	18		21.0	107.3	1363
60								
	LS-15	1 3 5	8	18				750*
65	LS-16	WOH 1 2	3	15				<600*
70	LS-17	WOH 1 3	4	18				<500*
75	LS-18	WOH 3 5	8	18		35.8	86.5	873
80	LS-19	WOH WOH 7	7	18				<500*

LOG OF TEST BORING NO: TB-101

NTH Consultants, Ltd.

Project Name *DRC Preliminary Foundation Investigation*

NTH Proj. No.: *15-050014-00*

Project Location: *Detroit, MI*

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SUBSURFACE PROFILE

SOIL SAMPLE DATA

ELEV (FT)	PROFILE	ELEV	SURFACE ELEVATION 581.2	DEPTH	DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS PER 6 IN.	STD PEN RESIST (N)	REC (IN)	FIELD TEST	MOIST CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (PSF)
		497.2		84.0									
					85	LS-20	20 28 50/4"	78/10"	16				>9000
495													
			HARDPAN Very Hard Dark Gray SANDY CLAY with Trace Silt and Some Gravel		90	LS-21	23 23 31	54	15		14.0	120.3	14053
490													
						LS-22	49 50/3"	--	6	----			>9000
					95	LS-23	50/2"	--	0.1				
		485.4	TOP OF BEDROCK (Boring Continued as Rock Core)	95.8									
485													
					100								
480													
					105								
475													
					110								
470													
					115								
465													
					120								
460													
					125								
455													

LOG OF TEST BORING NO: TB-102

NTH Consultants, Ltd.

Project Name: *DRIC Preliminary Foundation Investigation*

NTH Proj. No.: *13-080014-00*

Project Location: *Detroit, MI*

Checked By:

SUBSURFACE PROFILE / SOIL SAMPLE DATA

ELEV (FT)	ELEV	SURFACE ELEVATION 583.6	DEPTH	DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/6 INCHES	STD PEN RESIST (N)	REC (in)	FIELD TEST	MOIST CONTENT (%)	DRY DENSITY (PCF)	UNCONF COMPR ST (PSF)
	582.8	FILL: Brown GRAVELLY SAND	0.8									
		FILL: Compact Black ASH with Little Sand and Trace Gravel			LS-1	4 18 16	34	0				
	580.6		3.0									
580												
		FILL: Compact Brown SAND with Little Ash and Trace Gravel		5	S-1	25 14 30	44	10				
	578.1		5.5									
					LS-2	3 6 4	10	10				
575		FILL: Loose to Medium Compact Brown SILTY SAND with Little Ash and Trace Gravel										
				10	LS-3	8 10 14	24	12				
	571.6		12.0									
570												
		FILL: Loose Black SAND with Trace Gravel		15	S-2	5 3 2	5	4				
	566.6		17.0									
565												
		Soft Black Organic CLAY with Trace Silt		20	LS-4	1 2 2	4	9		40.8	70.0	1000*
	561.6		22.0									
560												
		Compact Brown SILTY SAND with Trace Clay		25	LS-5	6 14 19	33	13				
	556.6		27.0									
555												
				30	LS-6	12 8 10	18	9				
550												
				35	LS-7	15 10 13	23	12		10.4	110.7	
		Medium Compact Brown SILTY SAND										

Total Depth: *112.0 FT*
Drilling Date: *5/1/08 to 5/8/08*
Inspector: *N. Emery*
Contractor: *DLZ*
Driller: *D. Haskins*

Water Level Observation:
Groundwater encountered at 12.5 feet during drilling. The use of fluids precluded further groundwater measurement below 19 feet

Drilling Method:
CME 750 ATV Mounted Drill Rig with a combination of 3 1/4 inch Inside Diameter Hollow Stem Auger, 12 1/4-inch Tri-Cone Roller Bit, NQ NX Wireline tooling, and NWJ Rods

Plugging Procedure:
Grout. Casing cut 4 feet below grade

Notes:
** - Pocket Penetrometer Value*
WOH = Weight of Hammer
8-inch Inside diameter steel casing installed/grouted to 61.5 feet

Figure No. 4

LOG OF TEST BORING NO: TB-102

Project Name: *DRIC Preliminary Foundation Investigation*

Project Location: *Detroit, MI*

NTH Consultants, Ltd.

NTH Proj. No: [illegible]

Checked By:

SUBSURFACE PROFILE

SURFACE ELEVATION 583.0

ELEV (FT)	ELEV	[illegible]	DEPTH
545			
540			
	537.0		46.0
535			
		Very Compact Brown SILTY SAND with Trace Clay	
530			
	527.0		56.0
525			
520		Soft to Medium Gray SILTY CLAY with Trace Sand	
515			
	511.0		72.0
510			
		Very Soft to Soft Gray CLAY with Trace Silt and Trace Sand	
505			
	501.0		82.0

SOIL SAMPLE DATA

DEPTH (FT)	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
40	LS-8	4 6 9	15	17				
45	LS-9	6 7 14	21	18				
50	LS-10	9 50/6"		14		10.9		
55	LS-11	4 20 40	60	16				
60	LS-12	WOH 3 6	9	17				1000*
65	LS-13	3 3 5	8	15		23.0	100.4	872
	LS-14	1 3 5	8	12				1000*
70	ST-1	Pushed	--	24		25.0	101.3	
75	LS-15	WOH 1 3	4	13				500*
80	LS-16	WOH 4 3	7	16				<500*

LOG OF TEST BORING NO: TB-102

NTH Consultants, Ltd.

Project Name: *DRIC Preliminary Foundation Investigation*

Project Location: *Detroit, MI*

NTH Proj. No.: *13-050014-00*

Checked By:

SUBSURFACE PROFILE					SOIL SAMPLE DATA								
ELEV (FT)	PROFILE	ELEV	SURFACE ELEVATION: 583 ft	DEPTH	DEPTH (FT)	SAMPLE NUMBER	BLOWS/6 INCHES	RECOVERY (IN)	REC (%)	FIELD TEST	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONF. STRENGTH (PSF)
500			HARDPAN: Very Stiff Gray SILTY CLAY with Trace Sand		85	LS-17	8 8 10	18	12				4000*
		496.0		87.0									
495			HARDPAN: Very Hard Gray SILTY CLAY with Trace Sand		90	LS-18	40 50/4"	--	4		9.1	168.1	20388
490													
		488.0	TOP OF BEDROCK (Boring Continued as Rock Core)	95.0	95	LS-19	50/6"	--	4				9000*
485					100								
480					105								
475					110								
					115								
					120								
					125								

LOG OF TEST BORING NO: TB-103

NTH Consultants, Ltd.

Project Name DRIC Preliminary Foundation Investigation

NTH Proj. No. 13-050014-00

Project Location: Detroit, MI

Checked By:

SUBSURFACE PROFILE						SOIL SAMPLE DATA							
ELEV (FT)	PROFILE	ELEV	SURFACE ELEVATION 585.2	DEPTH	DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/6 INCHES	STD. PEN. RESIST. (N)	REC (IN)	FIELD TEST	MOIST CONTENT (%)	DRY DENSITY (PCF)	UNCONF. COMPRESS. (PSF)
		584.7	FILL: Dark Brown SAND with Trace Organic Matter	0.5									
		584.3	FILL: Brown SAND	0.9									
						LS-1	4 13 10	23	10				
580					5	LS-2	4 4 4	8	9	PID = 1.0 ppm			
			FILL: Loose to Medium Compact Black SILTY SAND with Some Ash										
						LS-3	4 8 13	21	10	PID = 1.0 ppm			
575					10	LS-4	3 7 6	13	11	PID = 1.0 ppm			
		573.2		12.0									
			FILL: Very Loose Black SILTY SAND with Trace Gravel										
570					15	S-1	2 1 2	3	4	PID = 0.5 ppm			
		568.2		17.0									
565			Medium Gray SILTY CLAY with Trace Sand		20	LS-5	WOH 1 2	3	18				2000*
		563.2		22.0									
560			Very Loose Gray SILTY SAND		25	S-2	1 1 1	2	3				
		557.2		28.0									
555					30	LS-6	14 17 17	34	18		14.2	106.9	
			Compact Gray SAND with Some Silt										
		552.2		33.0									
			Very Loose Gray SILTY SAND			LS-7	2 WOH 1	1	18				
550		550.2		35.0	35								

Total Depth: 175.0 FT

Drilling Date: 5-8-08 to 5-14-08

Inspector: N. Emery/M. Flanagan

Contractor: DLZ

Driller: D. Haskins

Water Level Observation:

Groundwater encountered at 14 feet during drilling. The use of drilling fluids precluded further groundwater measurement below 25 feet.

Drilling Method:

CME 750 ATV Mounted Drill Rig with a combination of 3 1/4 inch Inside Diameter Hollow Stem Auger, 12 1/4 inch Tri-Cone Rotary Bit, NQ/NX Wireline Tooling, and NWJ Rods

Plugging Procedure:

Grout. Casing cut 4 feet below grade

Notes:

* - Pocket Penetrometer Value

WOH - Weight of Hammer

8 inch Inside diameter steel casing installed/grouted to 61.5 feet

PID - Photo Ionization Detector

Figure No. 5

LOG OF TEST BORING NO: TB-103

NTH Consultants, Ltd.

Project Name: *DRIC Preliminary Foundation Investigation*

Project Location: *Detroit, MI*

NTH Proj. No.: *15-050014 00*

Checked By

SUBSURFACE PROFILE

ELEV (FT)	ELEV	SURFACE ELEVATION: 585.7	DEPTH
		Medium Gray SILTY CLAY	
545	544.7		41.0
540			
535			
530			
525			
520		Soft to Medium Gray SILTY CLAY with Sand and Gravel	
515			
510			
505			

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/6 INCHES	PENETRATION (N)	REC (IN)	FIELD TEST	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONF. COMP. STRENGTH (PSF)
40	LS-8	2 1 2	3	10				1500*
45	LS-9	WOH 1 3	4	18		–		1000*
50	LS-10	WOH WOH WOH	0	18	–			<500*
55	LS-11	WOH WOH 2	2	18		22.1	106.7	552
60	LS-12	WOH WOH 2	2	18				<500*
	ST-1	Pushed	--	24		21.5	106.6	0.25
65	LS-13	WOH WOH 1	1	10		22.7	106.3	1101
70	LS-14	WOH 1 3	4	18				<500*
75	LS-15	WOH WOH 2	2	18				<500*
80	LS-16	WOH WOH 1	1	18				<500*

LOG OF TEST BORING NO: TB-103

NTH Consultants, Ltd.

Project Name: *DRIC Preliminary Foundation Investigation*

NTH Proj. No.: *15 050014-00*

Project Location: *Detroit, MI*

Checked By:

SUBSURFACE PROFILE

SURFACE ELEVATION 585.2

ELEV (FT)	ELEV	DESCRIPTION	DEPTH	DEPTH (FT)
500				85
	497.2		88.0	
495		HARDPAN Very Hard Damp Brown to Black SILTY CLAY with Some Fine Gravels		90
490	490.7	TOP OF BEDROCK (Boring continued as Rock Core)	94.5	95
485				100
480				105
475				110
				115
				120
				125

SOIL SAMPLE DATA

SAMPLE TYPE/NO	BLOWS ON SAMPLER	BLOWS PER FOOT (N)	REC (in)	FIELD TEST	MOIST CONTENT (%)	DRY DENSITY (PCF)	UNCONF COMP ST (PSF)
LS-17	WOH WOH 1	1	18				1000*
LS-18	15 26 40	74	15				>9000*
LS-19	28 50/3"	--	19				>9000*
LS-20	50/0"		0				

LOG OF TEST BORING NO: TB-104E

NTH Consultants, Ltd.

Project Name: *DRIC Preliminary Foundation Investigation*

Project Location: *Detroit, MI*

NTH Proj. No.: *13-050014.00*

Checked By

SUBSURFACE PROFILE

ELEV	SURFACE ELEVATION: 587.6	DEPTH
587.4	FILL: Black SILTY SAND with Trace Organic Matter and Roots	0.2
584.6	FILL: Medium Compact Black SAND with Trace Gravel and Brick Fragments	3.0
582.1	FILL: Very Compact Brown SAND with Trace Gravel	5.5
580.1	FILL: Compact Black SILTY SAND with Trace Gravel	7.5
573.1	FILL: Medium Compact Brown GRAVELLY SAND	14.5
565.6	Very Soft Black ORGANIC CLAY with Trace Sand and Gravel	22.0
560.6	Loose Brown SILTY SAND with Trace Gravel	27.0

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE TYPE/NO	BLOWS/6 INCHES	STD. PEN. RESIST. (N)	REC. (%)	FIELD TEST	MOIST. CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. ST. (PSF)
	S-1	12 13 14	27	8				
5	LS-1	32 50/3"	--	5				
	LS-2	13 15 24	30	10				
10	LS-3	7 8 10	18	8				
15	S-2	WOH WOH WOH	0	16				<500*
20	S-3	WOH WOH WOH	0	16				<500*
25	LS-5	WOH 2 7	9	12		25.4	100.4	-
30	LS-6	2 3 4	7	8		20.8	95.6	770
35	LS-7	WOH WOH WOH	0	15		44.1	77.1	412

Total Depth: *152.5 FT*
Drilling Date: *5-15-08 to 5-21-08*
Inspector: *N. Emery/M. Flanagan*
Contractor: *DLZ*
Driller: *D. Hacklin*

Drilling Method:
CME 750 ATV Mounted Drill Rig with a combination of 3 1/4 inch Inside Diameter Hollow Stem Auger, 12 1/4 inch Tri-Cone Rotary Bit, NQ/NX Wireline Tooling, and NWJ Rods

Plugging Procedure:
Grout. Casing cut 4 feet below grade

Water Level Observation:
Groundwater encountered at 19.5 feet during drilling. The use of drilling fluids precluded further groundwater measurement below 17 feet

Notes:
- * *Pocket Penetrometer Value*
- *WOH = Weight of Hammer*
- *8 inch Inside diameter steel casing installed/grouted to 17 feet*
- *TB-104 through D encountered demolition debris at 5 feet below ground surface and were terminated*

Figure No. 6

LOG OF TEST BORING NO: TB-104E

Project Name: *DRIC Preliminary Foundation Investigation*

Project Location: *Detroit, MI*

NTH Consultants, Ltd.

NTH Proj. No.: *13-050014-00*

Checked By

SUBSURFACE PROFILE

ELEV (FT) | PROFILE | ELEV | SURFACE ELEVATION 587.6 | DEPTH

550
545
540
535
530
525
520
515
510
505

Very Soft to Soft Gray SILTY CLAY with Trace Sand and Gravel

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/6 INCHES	STD. PEN. RESIST (N)	REC. (in)	FIELD TEST	MOIST. CONTENT (%)	DRY DENSITY (PCF)	UNCONF. COMP. ST. (PSF)
40	LS-8	WOH WOH WOH	0	10				<500*
45	LS-9	WOH WOH WOH	0	18				<500*
50	LS-10	WOH WOH WOH	0	17		22.4	110.4	500*
55	LS-11	WOH WOH WOH	0	15				500*
60	LS-12	WOH WOH WOH	0	16				<500*
65	LS-13	WOH WOH WOH	0	18				1000*
70	LS-14	WOH WOH WOH	0	18		24.6		1000*
75	LS-15	WOH WOH WOH	0	16				500*
80	LS-16	WOH WOH WOH	0	17				500*

LOG OF TEST BORING NO: TB-104E

NTH Consultants, Ltd.

Project Name: *DRIC Preliminary Foundation Investigation*

NTH Proj. No.: *[illegible]*

Project Location: *Detroit, MI*

Checked By:

ELEV (FT)	PROFILE	ELEV	SUBSURFACE PROFILE SURFACE ELEVATION: 587.0	DEPTH	DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/6 INCHES	STD PEN RESIST (N)	REC (in)	FIELD TEST	MOIST CONTENT (%)	DRY DENSITY (PCF)	UNC COMP ST (PSF)
					85	LS-17	WOH WOH WOH	0	16				500*
500		500.0		87.0									
			Medium Grey SILTY CLAY with Trace Sand and Trace Gravel		90	LS-18	WOH WOH WOH	0	17				1000*
495		495.0		92.0									
			HARDPAN - Hard Grey SILTY CLAY with Trace Sand and Gravel		95	LS-19	75/6"	--	9		11.3	127.1	>9000*
490		488.0		99.0									
			TOP OF BEDROCK (Boring Continued as Rock Core)		100								
485													
					105								
480													
					110								
475													
					115								
470													
					120								
465													
					125								
460													

LOG OF TEST BORING NO: TB-105A

NTH Consultants, Ltd.

Project Name DRIC Preliminary Foundation Investigation

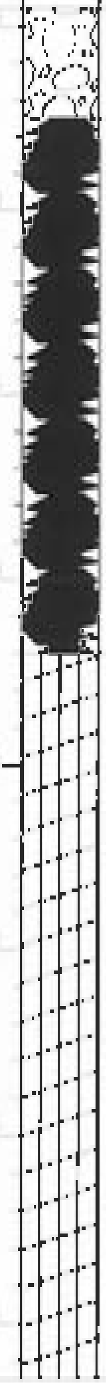
NTH Proj. No.: 13-050014-00

Project Location Detroit, MI

Checked By

SUBSURFACE PROFILE

SURFACE ELEVATION: 585.5



ELEV (FT)	ELEV		DEPTH
585		FILL, Boulders, Cobbles, Building Demolition Debris, Some Sand	
	582.5		3.0
580			
575		FILL, COAL/COKE Fragments, Some Coke Tar, Trace Sand, Strong Chemical Odor	
570			
	568.0		17.5
565			
560			
555			
550			

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE TYPE/NO	BLOWS / 6 INCHES	STD PEN RESIST (N)	REC (in)	FIELD TEST	MOIST CONTENT (%)	DRY DENSITY (PCF)	UNCONF COMP ST (PSF)
	LS-1	4 50/1"	--	6		--		
5	LS-2	5 5 4	9	12				
	LS-3	5 7 9	16	15				--
10	LS-4	2 3 6	9	12				5000*
15	LS-5	3 1 2	3	9		137.2	32.0	1000*
20	LS-6	2 3 3	6	9				1000*
	LS-7	1 2 2	4	12				1500*
25	LS-8	7 1 1	2	2				
30	LS-9	WOH WOH WOH	0	10		52.5	69.2	243
35	LS-10	WOH WOH WOH	0	18				

Total Depth: 117.0 FT
Drilling Date: 5-22-08 to 6-12-08
Inspector: M. Flanagan/N. Emery
Contractor: DLZ
Driller:

Drilling Method
CME 750 ATV Mounted Drill Rig with a combination of 3 1/4 inch Inside Diameter Hollow Stem Auger, 12 1/4 inch Tri Cone Rotary Bit, NQ/NX Wireline Tooling, and NWJ Rods

Plugging Procedure:
Grout. Casing cut 4 feet below grade

Water Level Observation
Groundwater encountered at 19.5 feet during drilling. The use of drilling fluids precluded further groundwater measurement below 23 feet

Notes:
* - Pocket Penetrometer Value
WOH = Weight of Hammer
8-inch inside diameter steel casing installed grouted to 17 feet
TB 105 encountered demolition debris at 3 feet below ground surface and was terminated

Figure No. 7

LOG OF TEST BORING NO: TB-105A

Project Name *DRIC Preliminary Foundation Investigation*

Project Location *Detroit, MI*

NTH Consultants, Ltd.

NTH Proj. No.: *13-050914.00*

Checked By

SUBSURFACE PROFILE

SOIL SAMPLE DATA

ELEV (FT)	PROFILE	ELEV	SURFACE ELEVATION: 585.5	DEPTH	DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS ON SAMPLER	STD PEN RESIST (N)	REC (IN)	[illegible]	MOIST CONTENT (%)	DRY DENSITY (PCF)	UNCONF COMPR (PSF)
545			Soft to Medium Gray SILTY CLAY		40	LS-11	WOH WOH 1	1	18				
540					45	LS-12	WOH WOH 1	1	18				
535					50	LS-13	WOH WOH 2	2	18				
530					55	LS-14	WOH 1 3	4	18				<500*
525					60	LS-15	WOH 1 4	5	18				<500*
520					65	LS-16	WOH 4 4	8	18				500*
515					70	LS-17	WOH 1 3	4	18		25.7	102.7	877
510					75	LS-18	WOH WOH 2	2	18				<500*
505					80	LS-19	WOH 3 4	7	18				<500*

LOG OF TEST BORING NO: TB-105A

NTH Consultants, Ltd.

Project Name *DRC Preliminary Foundation Investigation*

Project Location *Detroit, MI*

NTH Proj. No : *15-050014-00*

Checked By

SUBSURFACE PROFILE

ELEV (FT)	ELEV	SURFACE ELEVATION 585.5	DEPTH
500	497.5		88.0
495		HARDPAN Dark Gray GRAVELLY CLAY, Some Silt and Sand	
	493.0		92.5
		Very Compact Gray GRAVEL and SAND with Trace Silt	
490	490.5		95.0
		TOP OF BEDROCK Advanced Augers to 97.0 (Boring Continued as Rock Core)	
485			
480			
475			
470			

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE TYPE/NO	BLOWS / 6 INCHES	STD PEN RESIST (N)	REC (%)	FIELD TEST	MOIST CONTENT (%)	DRY DENSITY (PCF)	UNCONF COMPR ST (PSF)
85	LS-20	WOH WOH 4	4	18	--			600 100%
90	LS-21	28 37 50/5"	87/11"	17		9.3		*9000*
	LS-22	50/1"	--	1		13.2		
95	LS-23	50/1"	--	1				
100								
105								
110								
115								
120								
125								

LOG OF TEST BORING NO: TB-106

NTH Consultants, Ltd.

Project Name: *DRIC Preliminary Foundation Investigation*
Project Location: *Detroit, MI*
NTH Proj. No. *13-050014-00*
Checked By:

SUBSURFACE PROFILE

SURFACE ELEVATION: 587.3

ELEV (FT)	ELEV	DESCRIPTION	DEPTH
585		FILL: Boulders, Cobbles, Bricks and Other Demolition Debris (Excavated to 3.5 feet)	
	583.8		3.5
580			
575		FILL: Medium Compact Black Coke and Coal Fragments	
570			
	569.3		18.0
565			
560			
555			

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE TYPE/NO	BLOWS/6 INCHES	STD PEN RESIST (N)	REC (in)	FIELD TEST	MOIST CONTENT (%)	DRY DENSITY (PCF)	UNCONF COMPRESSIVE (PSF)
5	LS-1	5 10 10	20	4				
	LS-2	8 7 4	11	12				
10	LS-3	2 4 12	16	12				
15	LS-4	5 4 7	11	15				
20	LS-5	2 2 3	5	15		25.3	101.0	1971
	LS-6	WOH 1 1	2	18				1000*
25								
30	LS-7	2 1 1	2	15				<500*
35	LS-8	WOH WOH WOH	0	18				<500*

Total Depth: *145.0 FT*
Drilling Date: *5-28-08 to 6-4-08*
Inspector: *M. Flanagan/N. Emery*
Contractor: *DLZ*
Driller: *D. Haskins*

Drilling Method
CME 750 ATV Mounted Drill Rig with a combination of 3 1/4 inch Inside Diameter Hollow Stem Auger, 12 1/4 inch Tri-Cone Rotary Bit, NQ/NX Wireline Tooling, and NWJ Rods

Plugging Procedure:
Grout. Casing cut 4 feet below grade

Water Level Observation
Groundwater encountered at 12.5 feet during drilling. The use of drilling fluids precluded further groundwater measurement below 23 feet.

Notes:
* Pocket Penetrometer Value
WOH = Weight of Hammer
8-inch inside diameter steel casting installed/grouted to 23 feet

Figure No. 8

LOG OF TEST BORING NO: TB-106

NTH Consultants, Ltd.

Project Name: *DRIC Preliminary Foundation Investigation*

NTH Proj. No.: *15 050014.00*

Project Location: *Detroit, MI*

Checked By

SUBSURFACE PROFILE

SURFACE ELEVATION 587.3

ELEV (FT): 545, 540, 535, 530, 525, 520, 515, 510, 505

Soft To Medium Gray SILTY CLAY with Trace Sand and Gravel

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS PER 6 INCHES	BLOWS PER FOOT (N)	REC. (IN)	FIELD TEST	MOIST CONTENT (%)	DRY DENSITY (PCF)	UNCONF COMPRESS (PSF)
40	LS-9	WOH WOH WOH	0	18		20.9	-	-
45	LS-10	WOH WOH WOH	0	18	-			
50	LS-11	WOH WOH 2	2	18				<500*
55	LS-12	WOH WOH WOH	0	18				500*
60	LS-13	WOH 1 2	3	18				500*
65	LS-14	WOH WOH 1	1	18				1000*
70	LS-15	WOH WOH 4	4	18				500*
75	LS-16	WOH WOH WOH	0	18		21.5		<500*
80	LS-17	WOH 1 4	5	18	-			600*

LOG OF TEST BORING NO: TB-106

NTH Consultants, Ltd.

Project Name: *DRIC Preliminary Foundation Investigation*

NTH Proj. No.: *15-050014.00*

Project Location: *Detroit, MI*

Checked By:

SUBSURFACE PROFILE

SURFACE ELEVATION 587.3

ELEV (FT)	PRO FILE	ELEV		DEPTH
500		500.3		87.0
			Medium Gray SILTY CLAY	
495		495.3		92.0
			HARDPAN: Hard Gray SILTY CLAY with Trace Sand and Gravel	
		492.3		95.0
			TOP OF BEDROCK (Boring Continued as Rock Core)	
490				
485				
480				
475				
470				
465				
460				

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE NUMBER	BLOWS/6 INCHES	STD PEN INDEX (N)	REC (%)	FIELD TEST	MOIST CONTENT (%)	DRY DENSITY (PCF)	UNCONF COMP ST (PSF)
85	LS-10	WOH WOH 3	3	18				600*
90	LS-10	WOH WOH 2	2	10				2000*
95	S-1	50/3"	--	2				*9000*
100								
105								
110								
115								
120								
125								

LOG OF TEST BORING NO: TB-107

NTH Consultants, Ltd.

Project Name: *DRIC Preliminary Foundation Investigation*

Project Location: *Detroit, MI*

NTH Proj. No.: *13-050014.00*

Checked By

SUBSURFACE PROFILE

SURFACE ELEVATION 588.5

ELEV (FT)	ELEV	DESCRIPTION	DEPTH
		FILL: Dark Brown GRAVELLY SAND (Excavated to 2.0 feet)	
	586.5		2.0
585		FILL: Very Compact Black GRAVELLY SAND	
	583.0		5.5
		FILL: Medium Compact Black GRAVELLY SAND	
580	580.5		8.0
		FILL: Very Loose Dark Brown CLAYEY SAND with Trace Silt	
	576.5		12.0
575		FILL: Very Loose Dark Brown SILTY SAND with Trace Gravel	
	571.0		17.5
570			
565			
560			
555			

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE [illegible]	BLOWS [illegible]	[illegible]	REC (IN)	FIELD [illegible]	MOIST CONTENT (%)	DRY DENSITY (PCF)	[illegible] (PSF)
5	LS-1	4 10 50	60	15				
	S-1	5 5 5 7	10	9				
10	S-2	1 1 1	2	7				
15	S-3	3 2 2	4	4				
20	LS-2	WOH 1 2	3	18		28.0	99.2	698
	LS-3	WOH 1 1	2	18				500*
25								
30	LS-4	2 WOH WOH	0	10		46.1	71.6	349
35	LS-5	WOH WOH WOH	0	18				

Total Depth: *143.2 FT*
Drilling Date: *6-5-08 to 6-12-08*
Inspector: *M. Flanagan/N. Emery*
Contractor: *DLZ*
Driller: *D. Haskins*

Drilling Method:
CME 750 ATV Mounted Drill Rig with a combination of 3 1/4 inch Inside Diameter Hollow Stem Auger, 12 1/4 inch Tri-Cone Rotary Bit, NQ/NX Wireline Tooling, and NWJ Rods

Plugging Procedure
Grout. Casing cut 4 feet below grade

Water Level Observation
Groundwater encountered at 13 feet during drilling. The use of drilling fluids precluded further groundwater measurement below 23 feet

Notes
* *Pocket Penetrometer Value*
WOH = Weight of Hammer
8-inch Inside diameter steel casing installed/grouted to 23 feet

Figure No. 9

LOG OF TEST BORING NO: TB-107

Project Name: *DRIC Preliminary Foundation Investigation*

Project Location: *Detroit, MI*

NTH Consultants, Ltd.

NTH Proj. No. *13-050014-00*

Checked By:

SUBSURFACE PROFILE

SURFACE ELEVATION: 588.6

Soft to Medium Gray SILTY CLAY with Trace Sand and Gravel

SOIL SAMPLE DATA

ELEV (FT)	DEPTH (FT)	SAMPLE [illegible]	BLOWS [illegible]	[illegible]	REC [illegible]	[illegible]	MOIST [illegible] (%)	DRY [illegible]	[illegible]
550									
	40	LS-6	WOH WOH WOH	0	17				
545									
	45	LS-7	WOH WOH 1	1	18				
540									
	50	LS-8	WOH WOH 2	2	18				
535									
	55	LS-9	WOH WOH 2	2	17				500*
530									
	60	LS-10	WOH 2 3	5	18		22.6	105.4	
525									
	65	LS-11	WOH 1 3	4	18				1000*
520									
	70	LS-12	WOH WOH 2	2	18				500*
515									
	75	LS-13	WOH WOH 1	1	18				1000*
510									
	80	LS-14	WOH 1 3	4	18				1000*

LOG OF TEST BORING NO: TB-107

NTH Consultants, Ltd

Project Name: *DRIC Preliminary Foundation Investigation*

NTH Proj. No. *15-050014-00*

Project Location: *Detroit, MI*

Checked By:

SUBSURFACE PROFILE

ELEV (FT)	PROFILE	ELEV	SURFACE ELEVATION 588.5	DEPTH
505				
500		499.5		89.0
495			Medium Compact Gray CLAYEY SILT with Trace Sand	
		493.5	TOP OF BEDROCK (Boring Continued as Rock Core)	95.0
490				
485				
480				
475				
470				
465				
460				

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/6 INCHES	STD PEN RESIST (N)	REC (IN)	FIELD TEST	MOIST CONTENT (%)	DRY DENSITY (PCF)	UNCONF COMPR (PSF)
85	LS-15	WOH WOH WOH	0	17		42.3	80.2	
90	LS-16	WOH WOH 17	17	18		19.9	107.3	
	S-4	50/1"	--	1				
95								
100								
105								
110								
115								
120								
125								

LOG OF TEST BORING NO: TB-108

NTH Consultants, Ltd.

Project Name: *DRIC Preliminary Foundation Investigation*

NTH Proj. No. *13-050014.00*

Project Location: *Detroit, MI*

Checked By:

SUBSURFACE PROFILE

SURFACE ELEVATION 577.0

ELEV (FT)	ELEV		DEPTH
	576.8	FILL: Dark Brown SILTY SAND with Trace Organic Material (Roots)	0.2
575	574.0	FILL: Medium Compact Brown SILTY SAND with Brick Fragments	3.0
	571.5	FILL: Very Stiff Gray SANDY CLAY with Trace Gravel	5.5
570	569.0	FILL: Loose Gray SILTY SAND	8.0
	565.0	FILL: Medium Compact Black GRAVEL with Trace Silt and Sand	12.0
565			
560			
555			
550		Very Soft to Soft Gray SILTY CLAY with Trace Sand and Gravel	
545			
540			

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/6 INCHES	STD. PEN. INDEX (N)	REC. (IN)	LIQUID LIMIT	MOIST. CONTENT (%)	DRY DENSITY (PCF)	UNCONF. COMP. ST. (PSF)
	LS-1	3 10 13	23	9	—		—	—
5	LS-2	3 6 8	14	12		—		
	S-1	5 5 5	10	9			—	
10	S-2	5 8 3	11	4				
15	LS-3	WOH WOH WOH	0	18		40.8	81.8	283
20	LS-4	WOH WOH WOH	0	18				500*
25	LS-5	WOH WOH WOH	0	18				500*
30	LS-6	WOH WOH WOH	0	18				<500*
35	LS-7	WOH WOH WOH	0	18				<500*

Total Depth: *139.0 FT*
Drilling Date: *4-24-08 to 4-30-08*
Inspector: *N. Emery*
Contractor: *DLZ*
Driller: *D. Haskins*

Drilling Method:
CME 750 ATV Mounted Drill Rig with a combination of 3 1/4 inch Inside Diameter Hollow Stem Auger, 12 1/4 inch Tri-Cone Rotary Bit, NQ/NX Wireline Tooling, and NWJ Rods

Plugging Procedure:
Grout. Casing cut 4 feet below grade

Water Level Observation:
Groundwater encountered at 6.5 feet during drilling. The use of drilling fluids precluded further groundwater measurement below 25 feet

Notes:
* Pocket Penetrometer Value
WOH = Weight of Hammer
8 inch Inside diameter steel casing installed/grouted to 20 feet

Figure No. 10

LOG OF TEST BORING NO: TB-108

NTH Consultants, Ltd.

Project Name *DRIC Preliminary Foundation Investigation*

Project Location *Detroit, MI*

NTH Proj. No.: *13-050014-00*

Checked By:

SUBSURFACE PROFILE

SURFACE ELEVATION 577.0

ELEV (FT)	ELEV	DESCRIPTION	DEPTH
535	535.0		42.0
530			
525			
520			
515			
510		Soft to Medium Gray SILTY CLAY with Trace Sand and Gravel	
505			
500			
495			

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/6 INCHES	[illegible] (N)	REC (in)	FIELD TEST	MOIST CONTENT (%)	DRY DENSITY (PCF)	UNCONF COMP ST (PSF)
	ST-1	Pushed		24		21.2	107.4	878
40	LS-8	WOH WOH WOH	0			21.0	108	554
45	LS-9	WOH 2 3	5	17		20.9	110.5	
50	LS-10	1 3 3	6	10				1000*
55	LS-11	WOH 2 4	6	18				1000*
60	LS-12	1 3 4	7	13				<0.25*
65	LS-13	WOH 4 5	9	10				1000
70	LS-14	WOH 3 5	8	15				1000
75	LS-15	WOH 2 5	7	18				1000
80	LS-16	WOH 1 3	4	18				500*

LOG OF TEST BORING NO: TB-108

NTH Consultants, Ltd.

Project Name: *DRIC Preliminary Foundation Investigation*

NTH Proj. No.: *13-050014-00*

Project Location: *Detroit, MI*

Checked By:

SUBSURFACE PROFILE

SURFACE ELEVATION: 577.0

ELEV (FT)	ELEV		DEPTH
490	490.0		87.0
		Stiff Gray SILTY CLAY with Trace Sand and Gravel	
485	485.0		92.0
480		HARDPAN - Very Compact Gray SILTY SAND with Trace Gravel	
	476.0		101.0
475		TOP OF BEDROCK (Boring Continued as Rock Core)	
470			
465			
460			
455			
450			

SOIL SAMPLE DATA

DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/6 INCHES	PENETRATION RESIST (N)	REC. (IN)	FIELD TEST	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONF. COMP. ST. (PSF)
85	LS-17	WOH WOH 2	2	10				
90	LS-18	2 3 4	7	18				4000
95	LS-19	12 28 28	56	12		7.0	142.1	
100	S-3	50/3"	--	3				
	S-4	50/1"		1				
105								
110								
115								
120								
125								

LOG OF CORE BORING NO: TB-101

NTH CONSULTANTS, LTD.

Project Name. DRIC - Preliminary Foundation Investigation
Project Location. Detroit, Michigan
NTH Proj. No. 15-050014-00
Drilling Date. 5/19/2008 - 5/30/2008
Core Size. NQ
Inspector N. Emery/M. Flanagan
Contractor: DLZ
Driller: J. Pailucci
Checked By: C. Johnson

Depth (ft)	Run No.	Depth (ft)	Recovery (%)	RQD	Fractures/Foot	Hardness	Weathering	Unc Comp (ksi)	Elev (ft)
									485
	2	98	98.4	88.0	1.5	MH	SW		
100									
									480
		102	93.0	18	2.0	MH	SW		
105									
		106						13.9	475
110									
									470
	3	112	100.0	40	1.1	MH	F		
115									
									465
120									
									460
	4	122	100.0	86	0.6	MH	F		
125									

Subsurface Profile

Ground Surface Elevation 581.4

DUNDEE LIMESTONE

Light Grey Amorphous LIMESTONE, Laminated Bedding
Roll Auger advanced to 98 feet.

Light Grey Amorphous LIMESTONE, Laminated Bedding. Stylolites at 101.15 feet and 101.88 feet.

Light Grey Amorphous to Fine Grained LIMESTONE. Laminated Bedding, Occasional Fossils and Pits. Numerous Stylolites and Healed Cracks along core length. Chert Bands and Nodules also along core length. Intensely Fractured between 111.4 feet and 112 feet.

Light Grey Amorphous LIMESTONE. Laminated Bedding, Occasional Fossils and Pits. Stylolites along core length, especially between 116.1 feet and 120.3 feet. Chert Banding and Nodules along core length, especially between 118 feet and 120.3 feet. Numerous Healed Cracks, between 116.17 feet and 120.0 feet. Petroleum Stained.

Discontinuities

Depth (ft)	Type	Dip	Roughness	Inf
98.3	J	M	SR	
98.8	J	H	SR	
101.1	J	H	SR	
101.6	J	H	SR	.
102.2	J	H	SR	.
103.3	J	L	R	
104.1	J	L	R	.
105.1	J	H	SR	.
105.8	J	H	SR	
106.4	J	H	SR	.
107.7	J	L	SR	.
108.8	J	H	R	
109.3	J	H	VR	SP
111.1	J	V	SR	
111.4	J	M	SR	.
116.7	J	H	SR	
116.9	J	H	SR	
117.4	J	H	SR	
118.0	J	H	SR	.
118.8	J	L	R	
119.4	J	H	R	
120.0	J	L	SR	.
121.9	J	H	SR	.

NOTES

LOG OF CORE BORING NO: TB-101

NTH CONSULTANTS, LTD.

Project Name: DRIC - Preliminary Foundation Investigation

Project Location: Detroit, Michigan

Checked By: C. Johnson

NTH Proj. No: 15-050014-00

Ground Surface Elevation: 581.2

Depth (ft)	Run No	Depth (ft)	Recovery (%)	RQD	Fractures/Foot	Hardness	Weathering	Unc Comp (tsf)	Elev (ft)
									455
130									450
	5	132	100.0	95	0.4	MH	1	--	
135									445
140									440
	6	142	97.0	85	1.0	MH	1		
145									435
150									430
	7	152	100.0	83	0.8	MH	1		
155									

Subsurface Profile

Light Gray Amorphous LIMESTONE, Laminated to Thinly Bedded, Occasional Pits and Fossils. A Vug is noted at 128.7 feet. A Stylolite is noted at 103.13 feet. Chert Nodule is noted at 128.83 feet. Petroleum Staining throughout

DETROIT RIVER GR (LUCAS FORMATION)

Light Gray Amorphous LIMESTONE, Laminated Bedding, Occasional Fossils and Vugs, numerous Pits. Stylolite noted at 134.6 feet. Chert noted between 138.35 feet and 138.8 feet. Numerous healed Cracks noted between 140.75 feet and 142 feet. 1/4-inch Clay or Ash Interbed (possible Kawkawlin Bentonite) noted at 137.77 feet. Petroleum Staining throughout

Gray Amorphous LIMESTONE with Laminated Bedding becoming Dolomitic below 146.67 feet. Occasional Fossils. Stylolites noted at 134.6 feet, 140 feet and 150.7 feet. Vertical Crack noted between 146.67 and 149.19 feet with a midpoint at 147.93 feet. A Vertical Fracture is noted between 149.6 and 150.10 feet with a midpoint at 149.8 feet. Unconformity noted at 149.96 feet. Turbidite Bedding noted between 149.96 and 150.6 feet.

End of Boring at 152 feet

Discontinuities

Depth (ft)	Type	Dip	Roughness	Fill
125.4	J	H	SR	-
127.6	J	H	S	
127.8	J	H	VR	-
128.0	J	L	VR	
132.6	J	H	R	
134.7	J	H	R	NP
135.5	J	H	SR	
135.8	J	H	SR	
136.6	J	H	SR	-
137.3	J	H	SR	-
137.5	J	H	SR	
137.6	J	H	SR	
137.8	J	H	SR	
138.4	J	H	VR	
138.8	J	L	R	
140.0	J	H	SR	
141.2	J	H	SR	-
141.4	J	H	SR	
141.5	J	H	SR	-
142.0	J	H	SR	-
143.4	J	H	SR	
143.8	J	H	SR	
144.0	J	L	R	-
147.9	C	V	SR	
149.5	J	V	SR	
150.2	J	H	SR	
150.7	J	H	VR	-
151.7	J	L	S	

NOTES:

LOG OF CORE BORING NO: TB-102

NTH CONSULTANTS, LTD.

Project Name: DRIC - Preliminary Foundation Investigation
Project Location: Detroit, MI
NTH Proj. No: 15-050014-00
Drilling Date: 5/1/2008 - 5/8/2008
Core Size: NQ
Inspector: N. Emery
Contractor: DLZ
Driller: D. Hoskins
Checked By: C. Johnson

Depth (ft)	Run #	Depth (ft)	Recovery (%)	RQD	Fractures/Foot	Hardness	Weathering	Unc Comp (ksi)
100								
	1	101	91.0	46	0.63	MH	SW	
		102						11.6
105								
	2	106	95.0	78	1.4	MH	SW	
110								
	3	111	100.0	70	1	MH	SW	
	4	113.5	49.0	48	0.2	MH	SW	
115								
120								
125								

Subsurface Profile

Ground Surface Elevation: 583.5

Elev (ft)	
	DUNDEE LIMESTONE
485	Light Gray Amorphous LIMESTONE, Laminated Bedding, Occasional Pits and Fossils, Styolite at 95.7 feet, Petroleum Odor noted throughout
480	Light Gray Amorphous LIMESTONE, Laminated Bedding. Styolites located 101.4 feet, 101.5 feet, 104.3 feet, and between 105.1 and 105.6 feet. Chert noted between 103.3 and 103.4 feet. Intensely Fractured between 101 to 101.3 feet and between 102.25 to 102.5 feet. Intensely Cracked between 102.5 feet and 102.7 feet.
475	Light Gray Amorphous LIMESTONE, Laminated Bedding, Occasional Pits, Vugs, and Fossils. Numerous Styolites noted along length. A nearly vertical fracture is noted between 108.3 and 109.4 feet, with a midpoint at 108.9 feet. Intensely Fractured between 106 and 106.3 feet. Petroleum Stained
	Light Gray Amorphous LIMESTONE, Laminated Bedding, Occasional Pits and Vugs. Styolites are present between 112.1 feet and 113.4 feet. Petroleum Stained
	End of Boring at 113.5 feet

Discontinuities

Depth (ft)	Type	Dip	Roughness	Infill
96.5	J	H	SR	ST
97.4	J	H	SR	SP
97.6	J	H	SR	ST
98.7	J	H	SR	-
98.3	J	H	SR	ST
99.4	J	H	SR	SP
101.3	J	H	SR	SP
102.3	J	I	SR	ST
102.7	J	H	SR	ST
103.0	J	H	SR	SP
103.6	J	H	SR	SP
103.7	J	H	SR	ST
104.6	J	H	R	ST
106.7	J	H	SR	ST
106.3	J	H	SR	
106.6	J	H	SR	P
106.6	J	L	R	
108.3	J	H	SR	
108.6	J	I	R	
108.9	J	V	SR	-
109.0	J	H	SR	
111.6	J	H	SR	SP

NOTES:

Sheet 1 of 1

Figure No. 12

LOG OF CORE BORING NO: TB-103

NTH CONSULTANTS, LTD.

Project Name. DRIC - Preliminary Foundation Investigation
Project Location. Detroit, MI
NTH Proj. No: 15-050014-00
Drilling Date: 5/8/2008 - 5/14/2008
Core Size. NQ
Inspector N. Emery
Contractor. DLZ
Driller: D. Hoskins
Checked By. C Johnson

Subsurface Profile

Ground Surface Elevation 585.2

Depth (ft)	Run No	Depth (ft)	Recovery %	RQD	Fractures/Foot	Hardness	Weathering	Unc Comp (ksi)	Elev (ft)	Subsurface Profile
95									490	**DUNDEE LIMESTONE**
100	1	100	81.0	4	>12	MH	F		485	Light Gray Amorphous LIMESTONE, Laminated Bedding, Occasional Pits. Intensely Fractured between 99.05 and 100 feet. Petroleum Staining throughout
105	2	105	30.0	35	1.6	MH	F		480	Light Gray Amorphous LIMESTONE, Laminated Bedding, Chert banding noted between 103.58 and 104.3 feet. Styolites noted between 103.7 and 104.1 feet
		106						10.6		
110	3	110	100.0	95	0.6	MH	F	10.9	475	Light Gray Amorphous LIMESTONE, Laminated Bedding, Occasional Pits, especially between 105 and 106.25 feet. Styolites at 107.25 feet, 107.75 feet, and 109.15 feet. Petroleum staining throughout
115	4	115	100.0	55	2.2	MH	F			Light Gray Amorphous LIMESTONE, Laminated Bedding, Occasional Pits, especially between 111.3 and 112.75 feet. Calcite Infilling located at 114.08 feet. Petroleum Staining throughout End of Boring at 115 feet
120										

Discontinuities

Depth (ft)	Type	Dip	Roughness	Fill
99.1	J	H	VR	
103.1	J	H	SR	-
103.6	J	H	R	SP
104.2	J	H	R	SP
106.3	J	H	SR	-
107.6	J	H	SR	SP
108.9	J	H	SR	-
110.1	J	M	SR	
110.3	J	H	SR	
110.6	J	H	SR	
111.0	J	H	SR	SP
112.1	J	S	VR	
112.3	J	H	R	-
112.8	J	H	SR	SP
112.9	J	H	SR	SP
114.0	J	H	SR	SP
114.5	J	S	R	-
114.8	J	S	R	

NOTES

LOG OF CORE BORING NO: TB-104E

NTH CONSULTANTS, LTD.

Project Name: DRIC - Preliminary Foundation Investigation
Project Location: Detroit, Michigan
NTH Proj. No: 15-050014-00
Drilling Date: 5/15/2008 5/21/2008
Core Size: NQ
Inspector: N. Emery
Contractor: DLZ
Driller: J. Poilucci
Checked By: C. Johnson

Depth (ft)	Run No	Depth (ft)	Recovery (%)	RQD	Fractures/Foot	Hardness	Weathering	Unc Comp (ksi)	Elev (ft)
100									
	1	102.5 103	25.0	33	2.07	MH	SW	22.4	485
105									
									480
110									
	2	112.5	90.0	91	0.9	MH	F		475
115									
									470
120									
	3	122.5	96.0	90	0.9	MH	F	15.4	465
125									
									460

Subsurface Profile

Ground Surface Elevation: 587.6

DUNDEE LIMESTONE

Light Gray Amorphous to Fine Grained LIMESTONE, Laminated Bedding, Occasional Pits and Fossils. Stylolites noted between 101.1 and 101.75 feet. Intensely Fractured 99 to 99.5 feet and between 101.7 to 102.3 feet.

Light Gray Amorphous to Fine Grained LIMESTONE, Laminated Bedding, Occasional Fossils. Pits and Vugs noted from 107.1 to 108 feet and 111.1 to 112.3 feet. Numerous stylolites noted along core length, especially between 103.3 and 103.5 feet. Chert Bands noted at 104 feet, 104.5 feet, and 109.2 feet. Petroleum Stained.

Light Gray Amorphous to Fine Grained LIMESTONE, Laminated Bedding, Occasional Fossils, Occasional Pits and Vugs, especially between 112.6 and 117.3 feet. Numerous Stylolites noted along core length. Occasional Chert Bands and Nodules noted along core length. Petroleum Stained, especially between 112.5 feet and 117.3 feet.

Discontinuities

Depth (ft)	Type	Dip	Roughness	Infill
99.5	J	H	SR	
100.5	J	M	SR	SP
100.9		-	-	-
101.0	J	H	SR	ST
101.6	J	H	SR	ST
101.8	J	H	SR	SP
101.8	J	H	SR	-
101.9	J	V	SR	-
102.3	J	L	SR	ST
103.2	J	H	SR	SP
103.3	J	H	SR	ST
103.6	J	H	R	ST
104.6	J	H	SR	ST
105.0	-	-	-	
107.5	J	H	SR	
108.1	I	H	R	ST
108.6	I	H	SR	
110.0	J	H	SR	SP
111.6	J	H	R	ST
115.5	J	H	SR	-
116.0	J	H	SR	
116.0	J	H	SR	-
116.2	J	H	SR	
117.0	J	H	S	
119.0	J	H	S	SP
119.8	J	H	SR	SP
121.1	J	H	S	
121.4	J	H	SR	
122.0	J	H	SR	SP
122.3	J	H	SR	SP
122.8	J	H	S	SP
123.8	J	H	SR	SP
126.0	J	H	SR	SP
126.3	J	H	SR	SP
126.9	J	H	SR	ST
127.3	J	H	SR	
127.7	J	H	SR	ST
128.0	J	H	SR	SP

NOTES:

LOG OF CORE BORING NO: **TB-104E**

NTH CONSULTANTS, LTD.

Project Name: DRIC - Preliminary Foundation Investigation
Project Location: Detroit, Michigan
Checked By: C. Johnson
NTH Proj. No: 15 050014-00

Depth (ft)	Run No.	Depth (ft)	Recovery (%)	RQD	Fractures/Foot	Hardness	Weathering	Unc. Comp. (tsi)	Elev (ft)	Subsurface Profile
										Ground Surface Elevation: 587.0
130										Light Gray Amorphous to Fine Grained LIMESTONE, Laminated Bedding. Occasional Fossils and Pits. Stylolites noted at 127.8 feet, 125.25 feet, and from 127.4 to 130.85 feet. Occasional Chert nodules. Petroleum Stained
	4	132.5	100.0	58	1.9	MH	F		455	
135										
										DETROIT RIVER GR (LUCAS FORMATION)
									450	Light Gray Coming to Gray Amorphous to Fine Grained LIMESTONE, Laminated Bedding. Occasional Fossils and Pits noted along core. Intensely Fractured between 140.4 and 140.9 feet. Vertical Crack noted between 140.94 and 141.75 feet, with a midpoint at 141.35 feet. Horizontal Cracks noted between 130.58 and 138.9 feet. Occasional Chert Bands and Nodules noted along core length. Petroleum Stained
140										
	5	142.5	100.0	78	1.2	MH	F		445	
145										Light Gray Amorphous to Fine Grained LIMESTONE, Laminated Bedding. Limestone grading to Dolomite. Occasional Fossils and Pits, noted between 142.5 and 147.5 feet. Stylolite noted at 145.67 feet. A nearly Vertical Crack extends below the steep Fracture between 145 feet and 145.4 feet, with a midpoint at 145.2 feet. Another nearly Vertical Crack is located between 150.5 feet and 152.5 feet, with a midpoint at 151.5 feet. Numerous filled and healed Cracks are noted along core length, especially between 143.5 to 146 feet, and 152.25 to 152.5 feet. Turbidite Zone is noted between 146.88 and 148 feet. An Unconformity exists at 148 feet. The Limestone is grading to Dolomite below 150 feet.
									440	
150										
	6	152.5	99.0	82	1.2	MH	F			End of Boring at 152.5 feet
155										
160										

Discontinuities

Depth (ft)	Type	Dip	Roughness	Infill
128.9	J	H	R	SP
129.0	J	H	SR	SP
129.7	J	H	R	SP
130.3	J	H	SR	-
130.6	J	H	SR	
130.7	J	H	SR	SP
131.4	J	H	SR	SP
133.1	J	H	S	ST
134.6	J	H	SR	SP
135.7	J	L	SR	SP
136.3	J	H	SR	SP
137.6	J	H	SR	SP
138.6	J	H	SR	
139.5	J	H	R	SP
140.2	J	H	SR	ST
140.5	J	H	SR	SP
140.9	J	H	R	SP
142.0	J	H	S	
142.9	J	H	SR	ST
143.8	J	H	S	
144.2	J	H	SR	ST
144.7	J	H	S	
145.0	J	V	SR	-
145.5	J	H	SR	ST
146.0	J	H	SR	ST
146.9	J	H	SR	
147.6	J	H	S	
149.5	J	H	S	
149.9	J	H	SR	SP
150.8	J	H	SR	
151.1	J	H	S	

NOTES

LOG OF CORE BORING NO: **TB-105A**

NTH CONSULTANTS, LTD.

Project Name: DRIC - Preliminary Foundation Investigation
Project Location: Detroit, Michigan
NTH Proj. No: 15-050014-00
Drilling Date: 5/22/2008 - 6/12/2008
Core Size: NQ

Inspector: N. Emery
Contractor: DLZ
Driller: D. Hoskins
Checked By: C. Johnson

Ground Surface Elevation: 585.5

Depth ft	Run No	Depth ft	Recovery (%)	RQD	Fractures/Foot	Hardness	Weathering	Unc Comp (tsi)	Elev ft	Subsurface Profile
										DUNDEE LIMESTONE
100	1	100	76.0	12	2.7	MH	SW		485	Light Gray Amorphous LIMESTONE, Laminated Bedding. Stylolites noted between 98.17 and 98.92 feet. Intense Fracturing between 98.70 and 99.3 feet.
105	2	105	90.0	60	2.2	MH	F		480	Light Gray Amorphous LIMESTONE, Laminated Bedding, Occasional Pitting between 102 and 102.55 feet. Stylolites at 100.4 feet, 100.85 feet, 101.05 feet and 101.92 feet. Petroleum Staining noted between 100.5 and 104.5 feet.
110	3	111	100.0	82	1.6	MH	F		475	Light Gray Amorphous LIMESTONE, Laminated Bedding, Occasional Pits and Vugs noted from 106.3 to 107.5 feet and 107.6 feet to 108.1 feet, and 109.48 feet. Petroleum Staining throughout.
115	4	117	100.0	90	0.83	MH	F		470	Light Gray Amorphous LIMESTONE, Laminated Bedding, Occasional Pitting, noted between 111 and 112 feet. Chert Banding 0.25 and 2 inches thick present between 113.90 and 117 feet. Stylolites noted at 112.75 feet, 113.4 feet, 113.73 feet, 114.3 feet, and 114.6 to 116.7 feet. Petroleum staining predominately between 111 feet and 112.0 feet. End of Boring at 117 feet
120										
125										

Discontinuities				
Depth ft	Type	Dip	Roughness	Infill
97.3	J	H	SR	
97.6	J	H	SR	-
98.0	J	I	SR	
98.2	J	M	SR	-
98.6	J	H	SR	
98.7	J	H	SR	-
98.8	J	H	SR	
100.0	J	H	R	ST
100.3	J	M	SR	ST
100.6	J	M	SR	ST
101.7	J	H	R	-
102.6	J	M	SR	
102.8	J	H	R	SP
103.1	J	I	SR	
103.2	J	H	SR	-
103.7	J	H	SR	
103.9	J	L	SR	-
104.2	J	H	SR	
105.3	J	H	SR	
105.6	J	H	SR	
106.1	J	H	SR	
107.0	J	H	R	SP
108.3	J	H	SR	
108.8	J	H	SR	
109.3	J	H	SR	-
110.1	J	H	SR	
110.7	J	H	SR	
113.9	J	H	SR	
114.6	J	H	SR	-
115.0	J	H	SR	
115.6	J	H	SR	-
116.8	J	H	SR	

NOTES:

Sheet 1 of 1

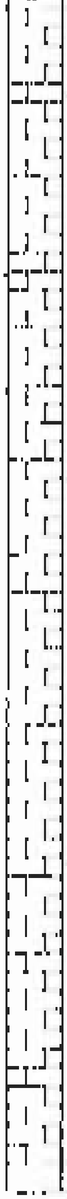
Figure No. 15

LOG OF CORE BORING NO: TB-106

NTH CONSULTANTS, LTD.

Project Name: DRIC - Preliminary Foundation Investigation
Project Location: Detroit, Michigan
NTH Proj. No: 15-050014-00
Drilling Date: 5/28/2008 - 6/4/2008
Core Size: NQ
Inspector: N. Emery
Contractor: DLZ
Driller: D. Hoskins
Checked By: C. Johnson

Depth ft	Run No	Depth ft	Recovery (%)	RQD	Fractures/Foot	Hardness	Weathering	Unc Comp (ksi)
100								
	1	102	100.0	92	2.5	MH	SW	
105	2	105	100.0	88	1.5	MH	SW	
110								
115	3	115	97.0	91	1.2	MH	F	
120								
125	4	125	97.0	31	1.3	MH	I	



Subsurface Profile

Ground Surface Elevation 587.3

DUNDEE LIMESTONE

Light Gray Amorphous LIMESTONE, Laminated Bedding. Chert Banding noted between 99.1 and 100 feet. Occasional Pits and one 1/4-inch Vug also noted. Fossils visible in some joints. Intense Cracking between 98.5 and 99.73 feet. Stylolites noted at 98.92 to 99.73 feet. Vertical Joint noted between 95 and 95.75 feet with a midpoint at 95.38 feet. Petroleum Staining between 95.5 and 96.94 feet.

Light Gray Amorphous LIMESTONE, Laminated Bedding. Occasional Pits and Fossils noted. Stylolites noted at 104.1 and 104.58 feet. Petroleum Staining between 100.5 and 105 feet.

Light to Dark Gray Amorphous LIMESTONE, Laminated Bedding. Pits and Vugs noted throughout, but most prevalent between 105 and 110 feet. Chert bands between 114.04 to 114.17 feet and from 114.4 to 114.5 feet. Stylolites noted between 105.3 and 106.21 feet, at 107.1 feet, 107.4 feet, between 108.04 and 108.6 feet, at 110.2 feet, 112.35 feet, and between 112.60 and 114.67 feet. Petroleum Staining present between 105 and 109.5 feet.

Light Gray Amorphous LIMESTONE, Laminated Bedding. Pits and Fossils noted throughout. Chert Banding noted between 115 and 117.6 feet. Stylolites noted between 115 and 118 feet, at 119 feet, at 122.17 feet, and at 122.7 feet. A steep (>60 degree) Angle Joint is located between 118.1 and 118.7 feet with a midpoint at 118.4 feet. Wavy Carbonaceous Banding noted between 119.27 and 123.35 feet with a midpoint at 121.31 feet. Two nearly Vertical Joints noted at 118.4 and 121.31 feet. Intense Fracturing noted between 115.54 and 117.5 feet, and between 120.4 and 121.13 feet. Petroleum Staining noted between 118.2 and 125 feet.

Discontinuities

Depth ft	Type	Dip	Roughness	[illegible]
94.4	I	H	SR	
95.4	J	V	SR	
96.7	J	H	SR	
96.1	J	S	SR	·
96.6	I	H	R	·
97.0	J	H	R	
97.1	J	H	SR	·
97.5	J	H	R	
98.6	J	H	R	
98.7	J	H	SR	
99.7	J	H	R	
99.8	J	H	SR	
99.9	J	H	R	·
100.0	J	H	SR	
100.8	J	H	SR	·
101.8	I	H	R	
102.0	J	H	SR	·
102.2	C	H		
102.3	J	H	SR	·
104.0	J	H	SR	
104.4	J	H	SR	·
104.7	J	L	SR	
106.3	J	H	SR	·
107.0	J	H	R	
107.4	J	H	R	·
108.4	J	H	SR	
109.1	J	H	SR	
109.8	J	H	SR	·
110.2	J	H	SR	
111.1	J	H	SR	·
111.6	J	H	SR	·
111.9	J	S	SR	
112.0	J	H	SR	·
112.8	J	H	SR	
113.3	J	H	SR	·
113.7	J	H	SR	
115.8	J	L	R	·
115.9	J	I	R	
116.1	J	L	R	·
116.8	J	M	R	
117.0	J	H	R	·
117.2	J	H	R	
117.5	J	H	SR	·
117.7	J	H	SR	
118.0	J	H	SR	·
118.3	J	I	R	
118.4	J	S	SR	·
118.7	J	H	R	
119.5	J	H	VR	·
119.8	J	H	SR	
120.0	J	H	SR	·
120.4	J	H	SR	·
120.8	J	H	SR	
120.8	J	L	SR	·

NOTES:

LOG OF CORE BORING NO: TB-106

NTH CONSULTANTS, LTD.

Project Name: DRIC - Preliminary Foundation Investigation
Project Location: Detroit, Michigan
Checked By: C. Johnson
NTH Proj No: 15-050014-00

Depth ft.	Run No.	Depth ft.	Recovery (%)	RQD	Fractures/Foot	Hardness	Weathering	Unc Comp (tsf)
130	5	130	100.0	90	1	MH	F	
135	6	135	100.0	90	0.4	MH	F	
140	7	140	82.0	61	1.75	MH	F	
145	8	145	82.0	69	1.6	MH	F	
150								
155								

Subsurface Profile

Ground Surface Elevation 587.3

Elev. ft.	Description
460	Light Gray Amorphous LIMESTONE, Laminated Bedding. Pits and Fossils noted throughout. Chert noted at 125.3 feet. Stylolites at 125.27 feet, 127.87 feet, from 128.58 to 128.7 feet, 129.29 feet, and 129.71 feet. Petroleum Staining throughout, but most prevalent between 127.7 and 129.8 feet.
455	Gray Amorphous LIMESTONE, Laminated Bedding. Pits are present throughout with Vugs at 132 feet and 134.75 feet. Stylolites noted between 130.4 feet and 130.5 feet, 131.5 feet, and 131.8 feet. Petroleum Stained from 131.26 to 132.5 feet and 133.4 to 134.5 feet.
450	Light Gray Amorphous LIMESTONE, Laminated Bedding. Pitting thoroughout, but most prevalent between 136.7 and 137.5 feet. Chert noted between 135 and 135.4 feet. A steep (~75 degree) to nearly Vertical Joint is noted between 138 and 139 feet with a midpoint at 138.5 feet. Intense Fracturing noted between 136.2 and 136.66 feet. Stylolites noted between 137.2 to 137.63 feet and from 138.08 to 139 feet. Petroleum Staining throughout, but most prevalent between 136.17 feet and 137.5 feet.
445	Light Gray Amorphous Interbedded LIMESTONE and DOLOMITIC LIMESTONE, Laminated Bedding. Pitting noted but most prevalent between 140 and 140.63 feet. A near Vertical Crack is noted between 141 and 142.38 feet with midpoint at 141.69 feet. A Vertical Crack is noted between 144.38 and 144.9 feet with a midpoint at 144.64 feet. Intense Cracking along laminations between 143.85 and 144.25 feet. Petroleum Staining, especially between 140 and 140.82 feet.
	End of Boring at 145 feet

Discontinuities

Depth (ft)	Type	Dip	Roughness	Infill
121.1	J	H	SR	-
122.8	J	H	SR	
124.3	J	H	SR	-
124.8	J	L	SR	
126.1	J	M	SR	-
126.2	J	M	SR	
126.4	J	H	SR	
128.5	J	H	SR	
128.9	J	H	SR	
129.8	J	H	SR	
134.5	J	H	SR	
134.8	J	H	SR	
135.4	J	L	SR	
136.5	J	H	SR	-
137.1	J	H	SR	-
137.6	J	H	S	
138.1	J	H	S	
138.4	J	H	S	-
139.0	J	H	SR	-
140.1	J	H	SR	
141.0	J	H	S	
141.5	J	H	S	
143.9	J	H	S	-
144.3	J	H	S	-
144.8	J	H	S	
144.8	C	V	-	-

NOTES:

LOG OF CORE BORING NO: TB-107

NTH CONSULTANTS, LTD.

Project Name: DRIC - Preliminary Foundation Investigation
Project Location: Detroit, Michigan
NTH Proj. No: 15-050014-00
Drilling Date: 6/5/2008 - 6/12/2008
Core Size: NQ
Inspector: H. Audet
Contractor: DLZ
Driller: D. Hoskins
Checked By: C. Johnson

Depth ft	Run No	Depth ft	Recovery (%)	RQD	Fractures Per Ft	Hardness	Weathering	Unc Comp (ksi)	Elev ft	Subsurface Profile
										Ground Surface Elevation - 588.5
	1	95.7	93.0	93.8	1.2	MH	SW	11.5		Amorphous to Fine Grained Grey LIMESTONE, Laminated Bedding, Fossiliferous Zones, Carbonaceous Banding and Oil Staining throughout run.
100	2	100.7	80.0	60	2.0	MH	SW	-	490	Fine Grained Grey LIMESTONE, Laminated Bedding, Fossiliferous and Carbonaceous Banding throughout, Chert from 98.7-99.0 foot (Extremely Fractured), Stylolite at 99.2 feet, Oil Staining Top 6 Inches
105	3	105.7	96.0	62.8	22	MH	SW		485	Fine Grained Grey LIMESTONE, Laminated Bedding, Stylolites, Fossils, and Oil Staining throughout run
110	4	110.7	96.0	85	1.4	MH	SW		480	Fine Grained Grey LIMESTONE, Laminated Bedding, Pits, Vugs, Fossils, Stylolites, and Slight Oil Stains throughout run
115	5	115.7	100.0	100	0.6	MH	F		475	Fine Grained Grey LIMESTONE, Laminated Bedding, Trace Fossils and Chert Inclusions noted throughout run
120	6	120.7	100.0	100	1.2	H	SW		470	Fine Grained Grey LIMESTONE, Laminated Bedding, Pits, Fossils, Chert Inclusions and Nodules noted throughout run, Oil Staining from 115.7 to 118 Feet also noted
125									465	Fine Grained Grey LIMESTONE, Laminated Bedding.

Discontinuities

Depth ft	Type	Dip	Roughness	Inf
95.6	J	I	SR	
95.7	J	H	SR	
96.7	J	H	SR	P
97.1	J	H	SR	
97.7	J	I	SR	
98.0	J	H	SR	
98.3	J	H	S	
98.7	J	V	SR	
99.3	J	H	SR	
99.4	J	H	R	
99.5	J	I	SR	
99.7	J	M	SR	
100.8	J	I	R	
101.1	J	L	SR	
101.2	J	I	SR	
101.3	J	L	SR	
101.8	J	L	SR	
102.5	J	L	SR	
103.8	J	M	SR	
104.0	J	M	SR	
105.1	J	M	SR	-
105.3	J	L	SR	-
106.7	J	H	SR	-
107.0	J	H	SR	
108.2	J	H	R	
108.3	J	H	SR	
109.0	J	H	SR	
109.2	J	H	SR	
110.5	J	L	SR	
112.2	J	L	SR	
113.0	J	L	SR	
114.8	J	L	SR	
116.7	J	H	SR	
117.1	J	L	SR	
118.3	J	H	SR	
118.7	J	H	SR	
119.3	J	H	S	
120.1	J	H	SR	
121.1	J	H	SR	
121.2	J	H	SR	
122.4	J	H	SR	
123.9	J	I	SR	
124.1	J	L	SR	

NOTES: DUNDEE LIMESTONE

LOG OF CORE BORING NO: TB-107

NTH CONSULTANTS, LTD.

Project Name: DRIC - Preliminary Foundation Investigation

Project Location: Detroit, Michigan

Checked By: C. Johnson

NTH Proj. No: 15-050014-00

Ground Surface Elevation 588.5

Depth (ft)	Run No	Depth (ft)	Recovery (%)	RQD	Fractures/Foot	Hardness	Weathering	Unc Comp (tsf)	Elev (ft)	Subsurface Profile
	7	125.7	102.0	6	1.4	MH	SW			Fossils and Styolites noted throughout run.
									460	
130	8	130.7	100.0	95	0.6	MH	SW			Fine Grained Gray LIMESTONE, Laminated Bedding, Fossils and Styolites noted throughout run
									455	
135	9	135.2	100.0	80	0.6	MH	SW			Fine Grained LIMESTONE, Laminated Bedding with Fossils noted throughout run. Pitting noted from 131 to 132 feet
									450	
140	10	140.7	100.0	89	1.6	MH	SW			Fine Grained Gray LIMESTONE, Laminated Bedding
	11	143.2	87.0	87	1.2	MH	SW			Fine Grained Light Gray LIMESTONE, Laminated Bedding End of Boring at 143.2 feet
145										
150										
155										

Discontinuities

Depth (ft)	Type	Dip	Roughness	Infill
125.7	J	H	SR	
126.2	J	L	R	
126.7	J	H	SR	
126.8	J	L	SR	
128.1	J	H	SR	
129.0	J	H	SR	
134.2	J	L	SR	-
134.4	J	H	SR	
134.6	J	H	SR	-
134.7	J	H	SR	
137.0	J	H	S	-
137.7	J	H	SR	
137.9	J	H	S	
138.3	J	H	SR	
138.9	J	H	S	-
139.3	J	H	S	
140.1	J	H	S	-
140.7	J	L	SR	
141.2	J	H	SR	
141.8	J	H	SR	
142.0	J	H	SR	-

NOTES: DUNDEE LIMESTONE

LOG OF CORE BORING NO: TB-108

NTH CONSULTANTS, LTD.

Project Name: DRIC - Preliminary Foundation Investigation
Project Location: Detroit, MI
NTH Proj. No: 15 050014-00
Drilling Date: 4/24/2008 - 4/30/2008
Core Size: NQ
Inspector: N. Emory/D. Adler
Contractor: DLZ
Driller: D. Hoskins
Checked By: C. Johnson

Depth (ft)	Run No	Depth (ft)	Recovery (%)	RQD	Fractures/Foot	Hardness	Weathering	Unc Comp (ksi)	Elev (ft)	Subsurface Profile
										Ground Surface Elevation: 577.6
									475	
105										**DUNDEE LIMESTONE**
									470	Gray Fine Grained LIMESTONE, Laminated Bedding, Horizontal Carbonaceous Banding from 103.1 to 105.1 feet, Possible Clay Seam (White Clay) 4 inches to 6 inches thick (Shelf Rock)
	1	108	82.5	78	1.2	H	F	14.0		
	2	109	20.0	0	0.7	H	F			Gray Fine Grained LIMESTONE, Laminated Bedding, Horizontal Carbonaceous Bedding throughout run.
110										
									465	
										Clay Infilling at 113.3 feet
115										
										Gray Fine Grained Amorphous LIMESTONE, Laminated Bedding, Petroleum Odor and Staining throughout run. Occasional Vuggy sections
	3	117	89.0	63	0.6	MH	SW			
									460	
										Gray Fine Grained FOSSILIFEROUS LIMESTONE, Laminated Bedding, No Pieces Greater Than 4 inches. (All Mechanical Breaks) Petroleum Odor noted throughout run.
120										
	4	121	95.0	95	0	M	SW			
									455	
125										
									450	
										Gray Fine Grained FOSSILIFEROUS LIMESTONE, Laminated Bedding, Petroleum Odor noted throughout run.
	5	129.0	100.0	100	0.1	M	SW			
130										

Discontinuities

Depth (ft)	Type	Dip	Roughness	Infill
101.6	J	-	-	
102.1	C	-		
102.7	J		-	-
103.5	C			-
108.5	C	-	-	-
109.6	J	M	R	
110.2	J	H	SR	-
110.6	J	V	SR	
113.3	J	H	SR	F
115.3	J	H	SR	
115.6	J	H	SR	
116.2	J	V	SR	-
126.6	C			
126.9	C			-
127.7	J	H	SR	

NOTES

Sheet 1 of 2

Figure No. 18

LOG OF CORE BORING NO: TB-108

NTH CONSULTANTS, LTD.

Project Name: DRIC - Preliminary Foundation Investigation
Project Location: Detroit, MI
Checked By: C. Johnson
NTH Proj. No. 15-050014-00

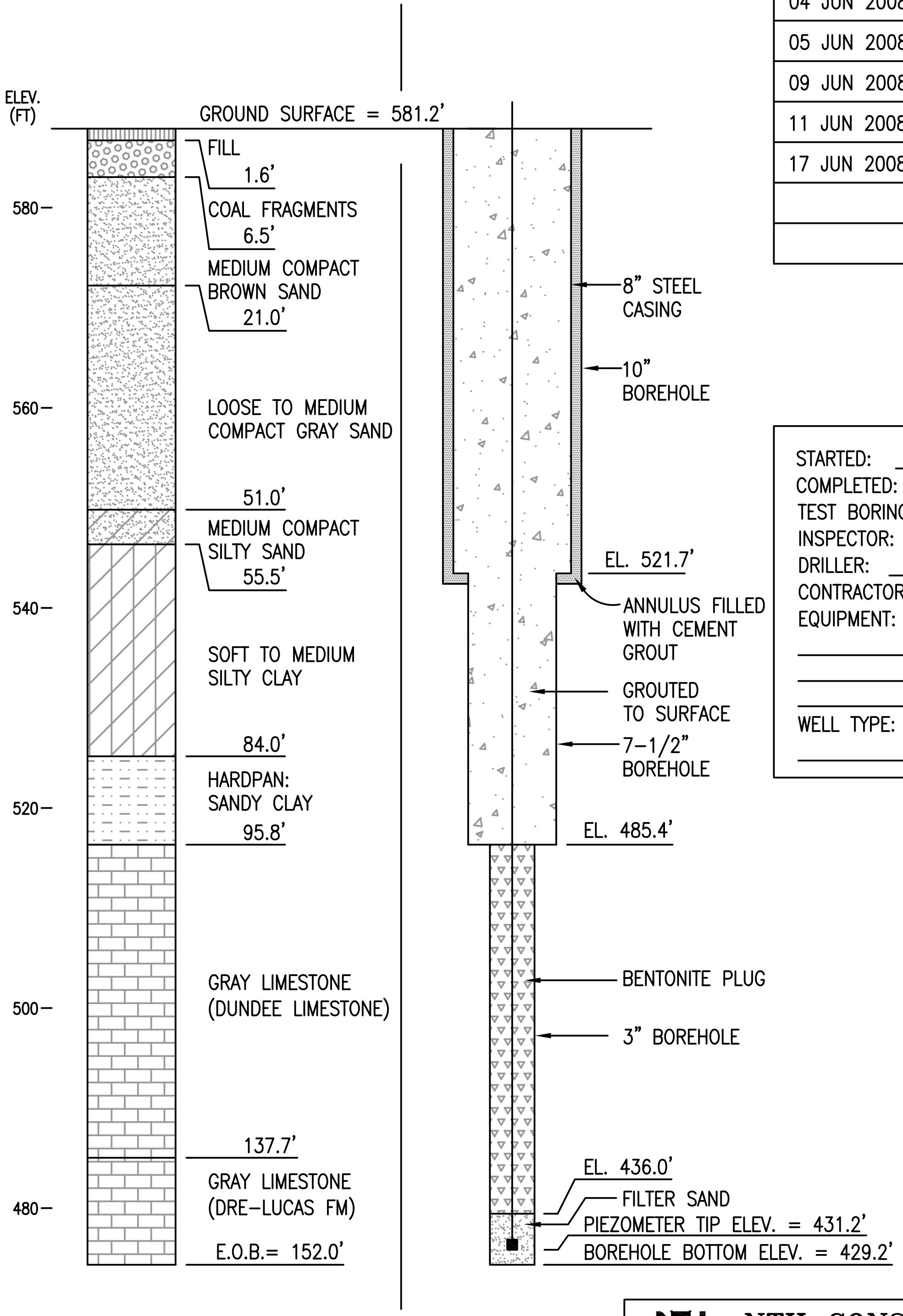
Depth (ft)	Run No.	Depth (ft)	Recovery (%)	RQD	Fractures/Foot	Hardness	Weathering	Unc Comp (ksi)	Profile	Elev. (ft)	Subsurface Profile Ground Surface Elevation: 577.6	Discontinuities: Depth (ft)	Type	Dip	Roughness	Infill
										445						
135																
		136						17.4								
										440	Gray Fine Grained FOSSILIFEROUS LIMESTONE, Laminated Bedding, Petroleum Odor and Occasional Pits noted throughout run End of Boring at 139 feet					
	6	139	97.0	97	0	M	SW									
140																
145																
150																
155																
160																
165																

NOTES:

LOG PIEZOMETER No. PZ-101

CLASSIFIED BY:
NTH CONSULTANTS, LTD.

GENERALIZED SUBSURFACE PROFILE	SCHEMATIC



DATE	ELEVATION (FEET)
02 JUN 2008	592.7 (70 psi)
03 JUN 2008	592.7 (70 psi)
04 JUN 2008	592.7 (70 psi)
05 JUN 2008	592.7 (70 psi)
09 JUN 2008	592.7 (70 psi)
11 JUN 2008	592.7 (70 psi)
17 JUN 2008	592.7 (70 psi)

STARTED: 19 MAY 2008
COMPLETED: 30 MAY 2008
TEST BORING: TB-101
INSPECTOR: N. EMERY/R. BUTLER
DRILLER: D. HOSKINS
CONTRACTOR: DLZ, INC.
EQUIPMENT: CME-750

WELL TYPE: PNEUMATIC PIEZOMETER

NTH CONSULTANTS, LTD.
Professional Engineering & Environmental Services
Detroit, Michigan

PIEZOMETER No. PZ-101

DRIC PRELIMINARY FOUNDATION INVESTIGATION
DETROIT, MICHIGAN

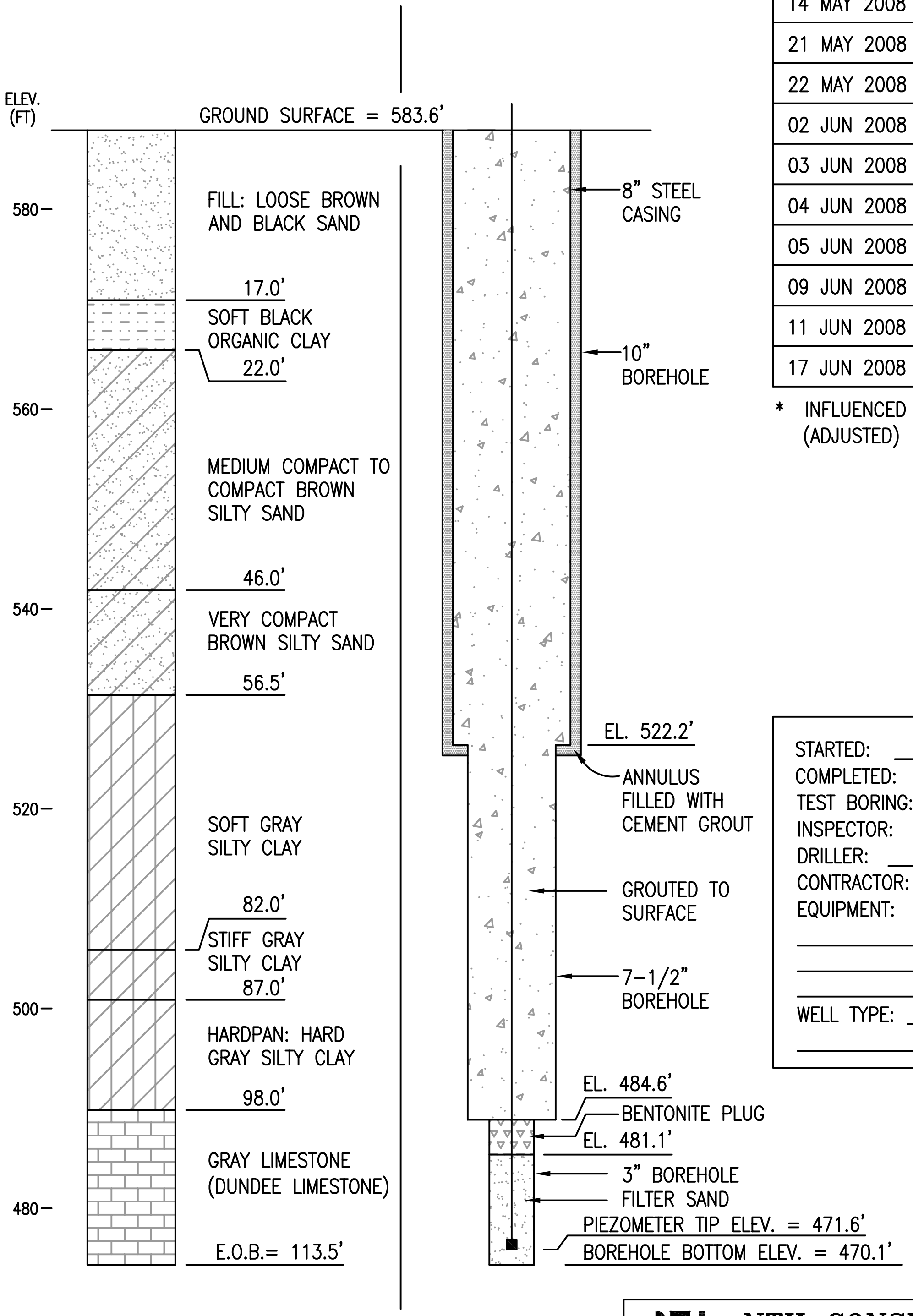
PROJECT NO.	DRAWN BY:	DATE:	FIGURE NO.
15-050014-00	SHB	12 AUG 2008	19
SCALE: NONE	CHECKED BY: CRJ	SHEET 1 OF 1	

D050014PZ101 12 AUG 2008

LOG PIEZOMETER No. PZ-102

CLASSIFIED BY:
NTH CONSULTANTS, LTD.

GENERALIZED SUBSURFACE PROFILE	SCHEMATIC



DATE	ELEVATION (FEET)
08 MAY 2008	591.6 (52.0 psi) *
13 MAY 2008	593.9 (53.0 psi)
14 MAY 2008	591.6 (52.0 psi)
21 MAY 2008	590.4 (51.5 psi)
22 MAY 2008	591.6 (52.0 psi)
02 JUN 2008	592.7 (52.5 psi)
03 JUN 2008	592.7 (52.5 psi)
04 JUN 2008	592.7 (52.5 psi)
05 JUN 2008	592.7 (52.5 psi)
09 JUN 2008	592.7 (52.5 psi)
11 JUN 2008	592.7 (52.5 psi)
17 JUN 2008	592.7 (52.5 psi)

* INFLUENCED BY BRINE IN FILTER SAND (ADJUSTED)

STARTED: 01 MAY 2008
COMPLETED: 07 MAY 2008
TEST BORING: TB-102
INSPECTOR: N. EMERY
DRILLER: D. HOSKINS
CONTRACTOR: DLZ, INC.
EQUIPMENT: CME-750

WELL TYPE: PNEUMATIC PIEZOMETER

NTH CONSULTANTS, LTD.
Professional Engineering & Environmental Services
Detroit, Michigan

PIEZOMETER No. PZ-102

DRIC PRELIMINARY FOUNDATION INVESTIGATION
DETROIT, MICHIGAN

PROJECT NO.	DRAWN BY:	DATE:	FIGURE NO.
15-050014-00	SHB	12 AUG 2008	20
SCALE: NONE	CHECKED BY: CRJ	SHEET 1 OF 1	

D050014PZ102 12 AUG 2008

Project No. 15-050014-00	NTH Consultants, Ltd.	DRIC Preliminary Foundation Investigation

TABULATION OF LABORATORY TEST DATA

Boring / Designation	Sample Number	Depth of Sample (ft)	Elevation of Sample Tip (ft)	Unconfined Compressive Strength (PSF)	Failure Strain (%)	Natural Water Content (% of dry weight)	In-Place Dry Density (PCF)	Permeability (cm/sec)	PARTICLE SIZE DISTRIBUTION (%)							ATTERBERG LIMITS (%)			Apparent Specific Gravity	Loss on Ignition (%)	Unified Soil Classification
									Colloids	Clay (passing 200)	Silt	Fine Sand	Medium Sand	Coarse Sand	Gravel	Liquid Limit	Plastic Limit	Plasticity Index			
TB-101	LS-11	44	537			19.3	109.1			3	23	74	0	0	0						
	LS-14	59	522	1350	14.7	21.6	107.3			47	33	12	6	1	1						
	LS-18	75	506	820	14.5	35.9	86.5			72	26	1	1	0	0						
	LS-21	90	491	14050	8.8	14.0	120.3			39	36	14	8	3	0						
TB-102	LS-4	20	561			40.9	78.0													5.4	
	LS-7	34.5	546.5			19.4	110.7			2	3	90	5	0	0						
	LS-10	49.5	531.5			19.9				2	23	75	0	0	0						
	LS-14	68	513	870	14.5	25.9	100.4														
	ST-1	70	511			25.0	101.3														
	LS-18	90	491	20370	11.7	9.1	136.1														
TB-103	LS-6	29.5	552			14.2	106.9			1	1	16	46	17	19						
	LS-11	54.5	527	550	14.4	22.1	106.7														
	ST-1	63	518.5			21.5	106.8														
	LS-13	64.5	517	1160	14.5	22.7	106.3														
TB-104	LS-5	25	568			25.4	100.4			3	8	61	27	1	0						

FIGURE NO. 21

Project No. 15-050014-00

NTH Consultants, Ltd.

DRIC Preliminary Foundation Investigation

TABULATION OF LABORATORY TEST DATA

Boring / Designation	Sample Number	Depth of Sample (ft)	Elevation of Sample Tip (ft)	Unconfined Compressive Strength (PSF)	Failure Strain (%)	Natural Water Content (% of dry weight)	In-Place Dry Density (PCF)	Permeability (cm/sec)	PARTICLE SIZE DISTRIBUTION (%)							ATTERBERG LIMITS (%)			Apparent Specific Gravity	Loss on Ignition (%)	Unified Soil Classification
									Colloids	Clay (passing 200)	Silt	Fine Sand	Medium Sand	Coarse Sand	Gravel	Liquid Limit	Plastic Limit	Plasticity Index			
TB-104	LS-6	30	551	770	14.7	29.8	95.6														
	LS-7	35	546	410	14.6	44.1	77.1			61	38	1	0	0	0						
	LS-10	50	531			22.4	110.4														
	LS-14	70	511			24.6															
	LS-19	95	486			11.3	127.1														
TB-105	LS-5	15	578.5			137.2	32.0													32.2	
	LS-9	30	563.5	240	14.6	52.5	69.2														
	LS-17	70	523.5	880	14.7	25.7	102.7														
	LS-21	90	503.5			9.3				33	34	16	9	3	5						
	LS-22	93.5	500			13.2				6	13	38	21	10	12						
TB-106	LS-5	20		1970	14.7	25.3	101.0														
	LS-9	40				29.9															
	LS-16	75				21.5															
	LS-18	85		920	14.8	27.0	98.1														
TB-107	LS-2	20	566	600	14.9	26.9	99.2			35	64	1	0	0	0						

FIGURE NO. 21

Project No. 15-050014-00

NTH Consultants, Ltd.

DRIC Preliminary Foundation Investigation

TABULATION OF LABORATORY TEST DATA

Boring / Designation	Sample Number	Depth of Sample (ft)	Elevation of Sample Tip (ft)	Unconfined Compressive Strength (PSF)	Failure Strain (%)	Natural Water Content (% of dry weight)	In-Place Dry Density (PCF)	Permeability (cm/sec)	PARTICLE SIZE DISTRIBUTION (%)							ATTERBERG LIMITS (%)			Apparent Specific Gravity	Loss on Ignition (%)	Unified Soil Classification
									Colloids	Clay (passing 200)	Silt	Fine Sand	Medium Sand	Coarse Sand	Gravel	Liquid Limit	Plastic Limit	Plasticity Index			
TB-107	LS-4	30	556	350	14.6	46.1	71.6														
	LS-10	60	526			22.6	105.4														
	LS-15	85	501			42.3	80.2														
	LS-16	90	496			19.9	107.3			9	69	22	0	0	0						
TB-108	LS-3	15	562	280	14.7	40.8	81.8														
	ST-1	38	539			21.2	107.4									26	16	10			
	LS-8	40	537	550	14.6	21.0	108.0														
	LS-9	45	532			20.9	110.5														
	LS-19	95.0	482			7.9	142.1			8	19	22	12	10	29						

NTH Consultants, Ltd.
Infrastructure Engineering
and Environmental Services

NTH Consultants, Ltd.
Southeast Michigan Laboratory

Telephone: 248.553.6300
Fax: 248.324.5179

Aggregate/Soil Test Report

Report No: MAT:15-050014-00-S035
Issue No: 1

Client:	The Corradino Group
Project:	Detroit River Inter. Crossing Geotechnical Engineering
Job No:	15-050014-00

This laboratory is accredited by American Association of State Highway and Transportation Officials (AASHTO). The test(s) reported have been performed in accordance with the terms of the accreditation.

Zeerak Paydawy

Date of Issue: 6/19/2008
Approved Signatory: Zeerak Paydawy

Sample Details

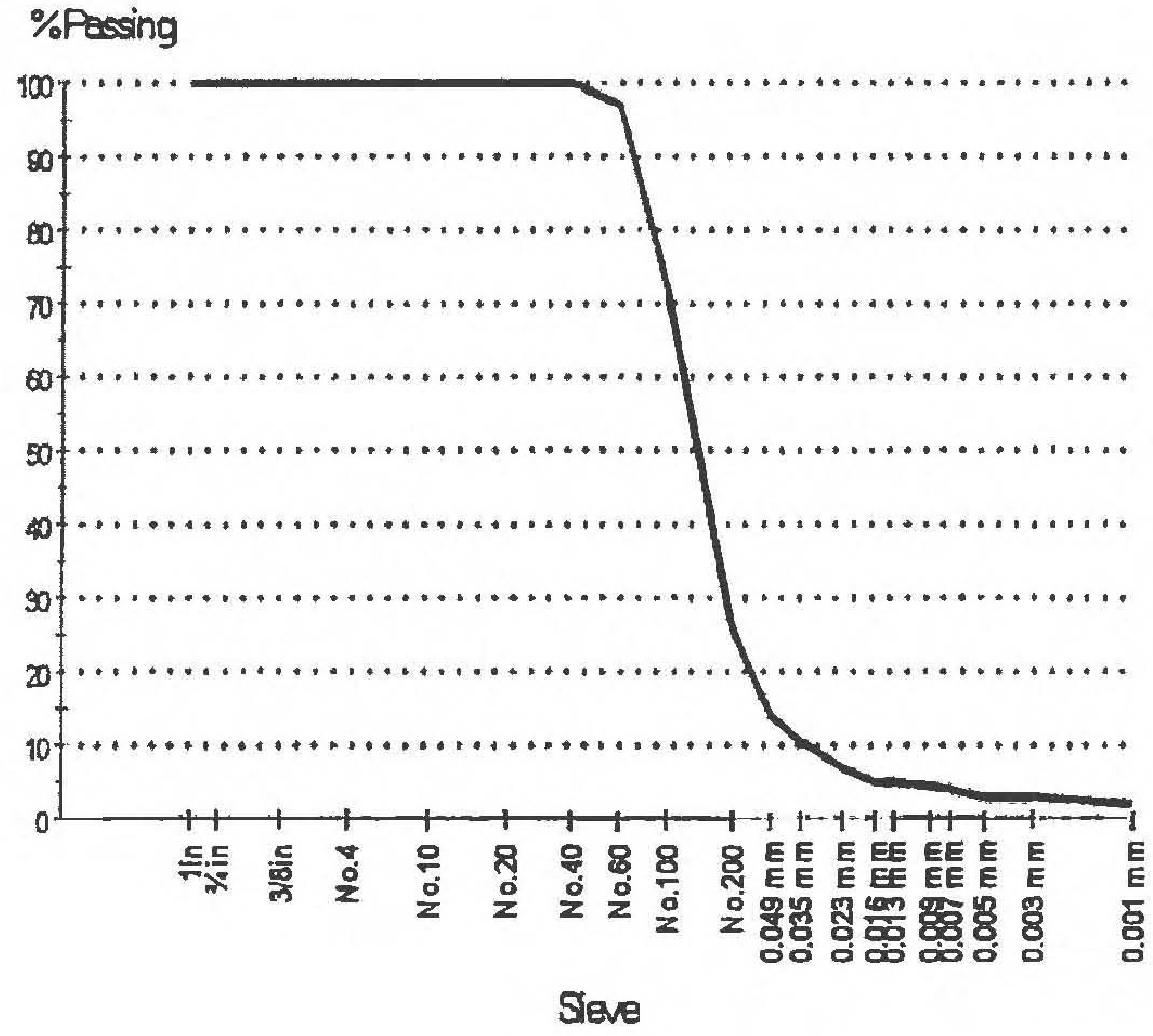
Boring No:	TB-101
Field Sample No:	LS-11
Sample Depth:	44
Date Sampled:	
Sampled By:	Nathan Emery
LWO No:	000307
Sample Location:	Detroit River International Crossing

Other Test Results

Description	Method	Result	Limits
Sand Gravel Description	ASTM D 422		
Shape			
Hardness			
Dispersion Device			
Dispersion Period		1	
Moisture Content (%)	ASTM D 2216	19.3	
Wet Density (lb/ft³)		130.2	
Dry Density (lb/ft³)		109.1	

Particle Size Distribution

Method: ASTM D 422
Drying by: Oven



Sieve Size	% Passing	Limits
1in (25.0mm)	100	
¾in (19.0mm)	100	
3/8in (9.5mm)	100	
No.4 (4.75mm)	100	
No.10 (2.0mm)	100	
No.20 (850µm)	100	
No.40 (425µm)	100	
No.60 (250µm)	97	
No.100 (150µm)	72	
No.200 (75µm)	26	
0.049 mm	14	
0.035 mm	11	
0.023 mm	7	
0.016 mm	5	
0.013 mm	5	
0.009 mm	5	
0.007 mm	4	
0.005 mm	3	
0.003 mm	3	
0.001 mm	2	

Comments
N/A

NTH Consultants, Ltd.
Infrastructure Engineering
and Environmental Services

NTH Consultants, Ltd.
Southeast Michigan Laboratory

Telephone: 248. 553.6300
Fax: 248.324.5179

Aggregate/Soil Test Report

Report No: MAT:15-050014-00-S036
Issue No: 1

Client:	The Corradino Group
Project:	Detroit River Inter. Crossing Geotechnical Engineering
Job No:	15-050014-00

This laboratory is accredited by American Association of State Highway and Transportation Officials (AASHTO). The test(s) reported have been performed in accordance with the terms of the accreditation.

Date of Issue: 6/19/2008
Approved Signatory: Zeerak Paydawy

Sample Details

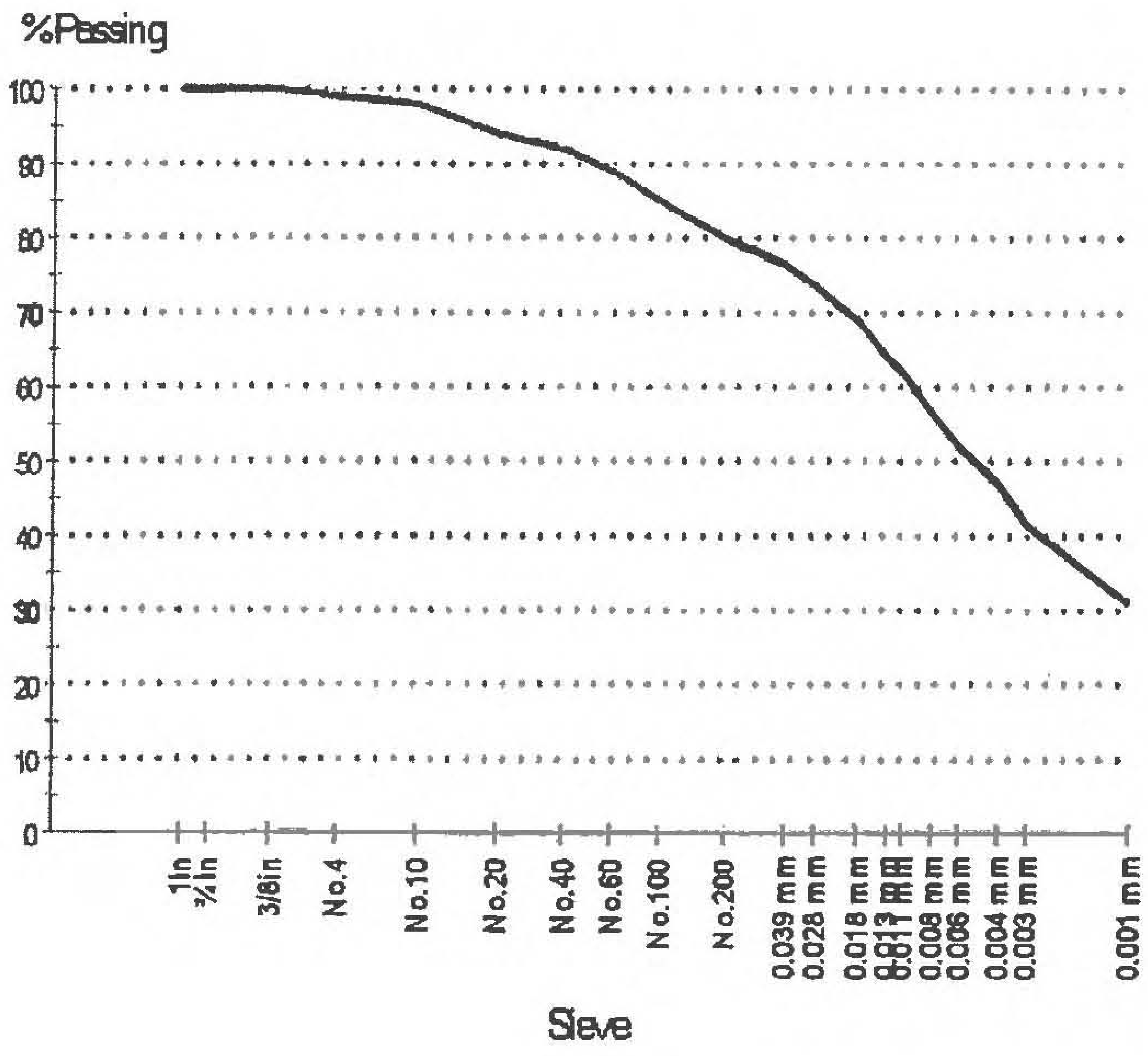
Boring No:	TB-101
Field Sample No:	LS-14
Sample Depth:	59
Date Sampled:	
Sampled By:	Nathan Emery
LWO No:	000307
Sample Location:	Detroit River International Crossing

Other Test Results

Description	Method	Result	Limits
Unconfined Compressive Strength (lb/ft²)	ASTM D 2166	1353	
Shear Strength (lb/ft²)		677	
Ave. Rate Strain to Failure(%)		0.9	
Strain at Failure(%)		14.7	
Average Height (in.)		2.728	
Average Diameter (in.)		1.339	
Height-Diameter Ratio		2.0	
Init. Dry Dens.			
Init. Water Content (%)			
Liquid Limit			
Plastic Limit			

Particle Size Distribution

Method: ASTM D 422
Drying by: Oven



Sieve Size	% Passing	Limits
1in (25.0mm)	100	
¾in (19.0mm)	100	
3/8in (9.5mm)	100	
No.4 (4.75mm)	99	
No.10 (2.0mm)	98	
No.20 (850µm)	94	
No.40 (425µm)	92	
No.60 (250µm)	89	
No.100 (150µm)	85	
No.200 (75µm)	80	
0.039 mm	76	
0.028 mm	74	
0.018 mm	69	
0.013 mm	64	
0.011 mm	62	
0.008 mm	57	
0.006 mm	52	
0.004 mm	47	
0.003 mm	42	
0.001 mm	31	

Comments
N/A

NTH Consultants, Ltd.
Infrastructure Engineering
and Environmental Services

NTH Consultants, Ltd.
Southeast Michigan Laboratory

Telephone: 248. 553.6300
Fax: 248.324.5179

Aggregate/Soil Test Report

Report No: MAT:15-050014-00-S037
Issue No: 1

Client: The Corradino Group

Project: Detroit River Inter. Crossing
Geotechnical Engineering

Job No: 15-050014-00

This laboratory is accredited by American Association of State Highway and Transportation Officials (AASHTO). The test(s) reported have been performed in accordance with the terms of the accreditation.

Date of Issue: 6/19/2008
Approved Signatory: Zeerak Paydawy

Sample Details

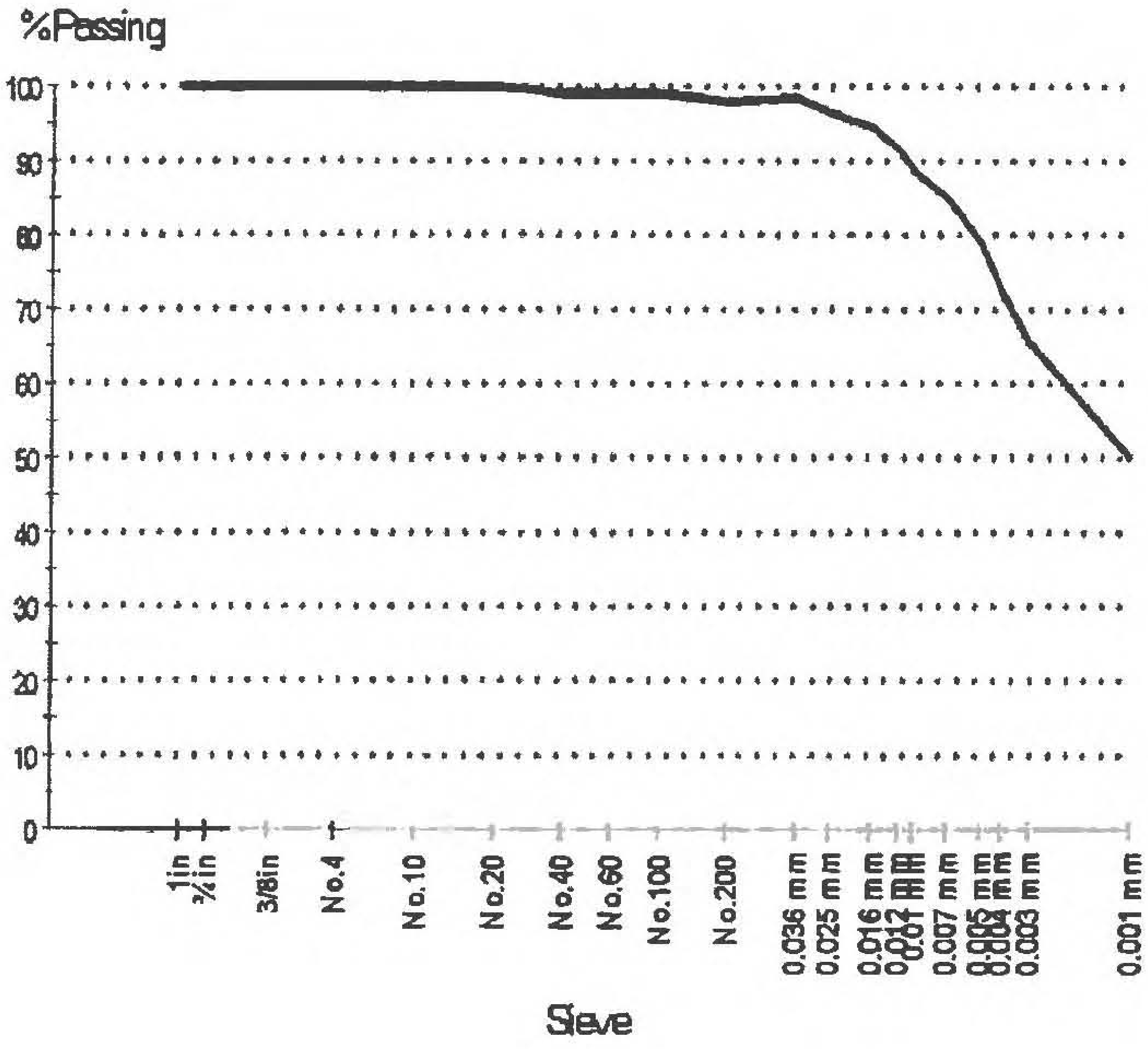
Boring No: TB-101
Field Sample No: LS-18
Sample Depth: 75
Date Sampled:
Sampled By: Nathan Emery
LWO No: 000307
Sample Location: Detroit River International Crossing

Other Test Results

Description	Method	Result	Limits
Unconfined Compressive Strength (lb/ft²)	ASTM D 2166	823	
Shear Strength (lb/ft²)		412	
Ave. Rate Strain to Failure(%)		0.9	
Strain at Failure(%)		14.5	
Average Height (in.)		2.751	
Average Diameter (in.)		1.347	
Height-Diameter Ratio		2.0	
Init. Dry Dens.			
Init. Water Content (%)			
Liquid Limit			
Plastic Limit			

Particle Size Distribution

Method: ASTM D 422
Drying by: Oven



Sieve Size	% Passing	Limits
1in (25.0mm)	100	
¾in (19.0mm)	100	
3/8in (9.5mm)	100	
No.4 (4.75mm)	100	
No.10 (2.0mm)	100	
No.20 (850μm)	100	
No.40 (425μm)	99	
No.60 (250μm)	99	
No.100 (150μm)	99	
No.200 (75μm)	98	
0.036 mm	98	
0.025 mm	96	
0.016 mm	94	
0.012 mm	92	
0.010 mm	89	
0.007 mm	85	
0.005 mm	79	
0.004 mm	72	
0.003 mm	66	
0.001 mm	50	

Comments
N/A

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Aggregate/Soil Test Report

Report No: MAT:15-050014-00-S038
Issue No: 1

Client:	The Corradino Group
Project:	Detroit River Inter. Crossing Geotechnical Engineering
Job No:	15-050014-00

This laboratory is accredited by American Association of State Highway and Transportation Officials (AASHTO). The test(s) reported have been performed in accordance with the terms of the accreditation.

Zerak Paydawy

Date of Issue: 6/19/2008
Approved Signatory: Zeerak Paydawy

Sample Details

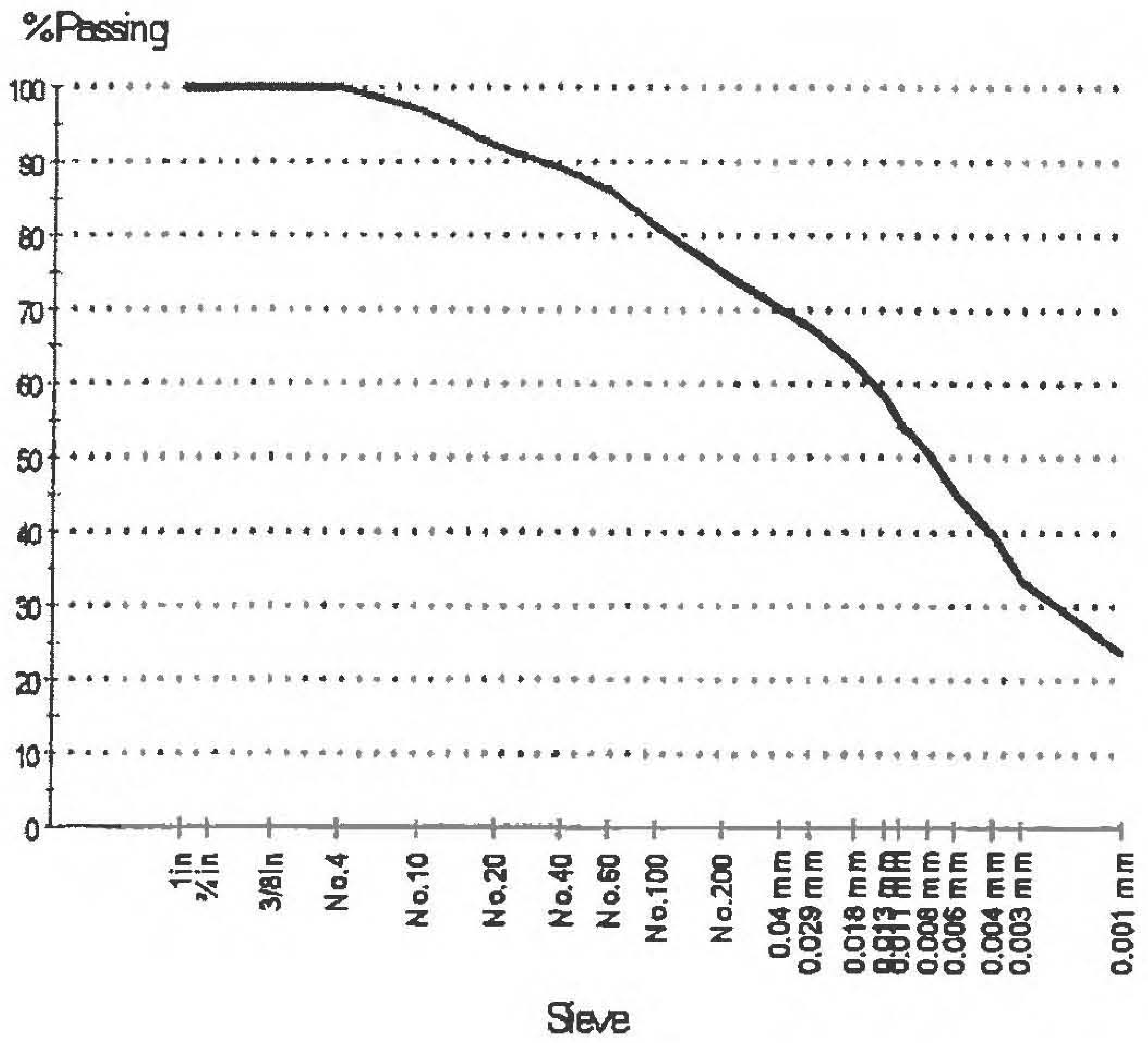
Boring No:	TB-101
Field Sample No:	LS-21
Sample Depth:	90
Date Sampled:	
Sampled By:	Nathan Emery
LWO No:	000307
Sample Location:	Detroit River International Crossing

Other Test Results

Description	Method	Result	Limits
Unconfined Compressive Strength (lb/ft²)	ASTM D 2166	14053	
Shear Strength (lb/ft²)		7027	
Ave. Rate Strain to Failure(%)		0.9	
Strain at Failure(%)		8.8	
Average Height (in.)		2.827	
Average Diameter (in.)		1.362	
Height-Diameter Ratio		2.1	
Init. Dry Dens.			
Init. Water Content (%)			
Liquid Limit			
Plastic Limit			

Particle Size Distribution

Method: ASTM D 422
Drying by: Oven



Sieve Size	% Passing	Limits
1in (25.0mm)	100	
¾in (19.0mm)	100	
3/8in (9.5mm)	100	
No.4 (4.75mm)	100	
No.10 (2.0mm)	97	
No.20 (850μm)	92	
No.40 (425μm)	89	
No.60 (250μm)	86	
No.100 (150μm)	81	
No.200 (75μm)	75	
0.040 mm	70	
0.029 mm	67	
0.018 mm	63	
0.013 mm	58	
0.011 mm	54	
0.008 mm	51	
0.006 mm	45	
0.004 mm	39	
0.003 mm	34	
0.001 mm	23	

Comments
N/A

Figure No. 25

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Aggregate/Soil Test Report

Report No: MAT:15-050014-00-S017
Issue No: 1

Client:	The Corradino Group
Project:	Detroit River Inter. Crossing Geotechnical Engineering
Job No:	15-050014-00

This laboratory is accredited by American Association of State Highway and Transportation Officials (AASHTO) The test(s) reported have been performed in accordance with the terms of the accreditation.

Date of Issue: 5/21/2008
Approved Signatory: Zeerak Paydawy

Sample Details

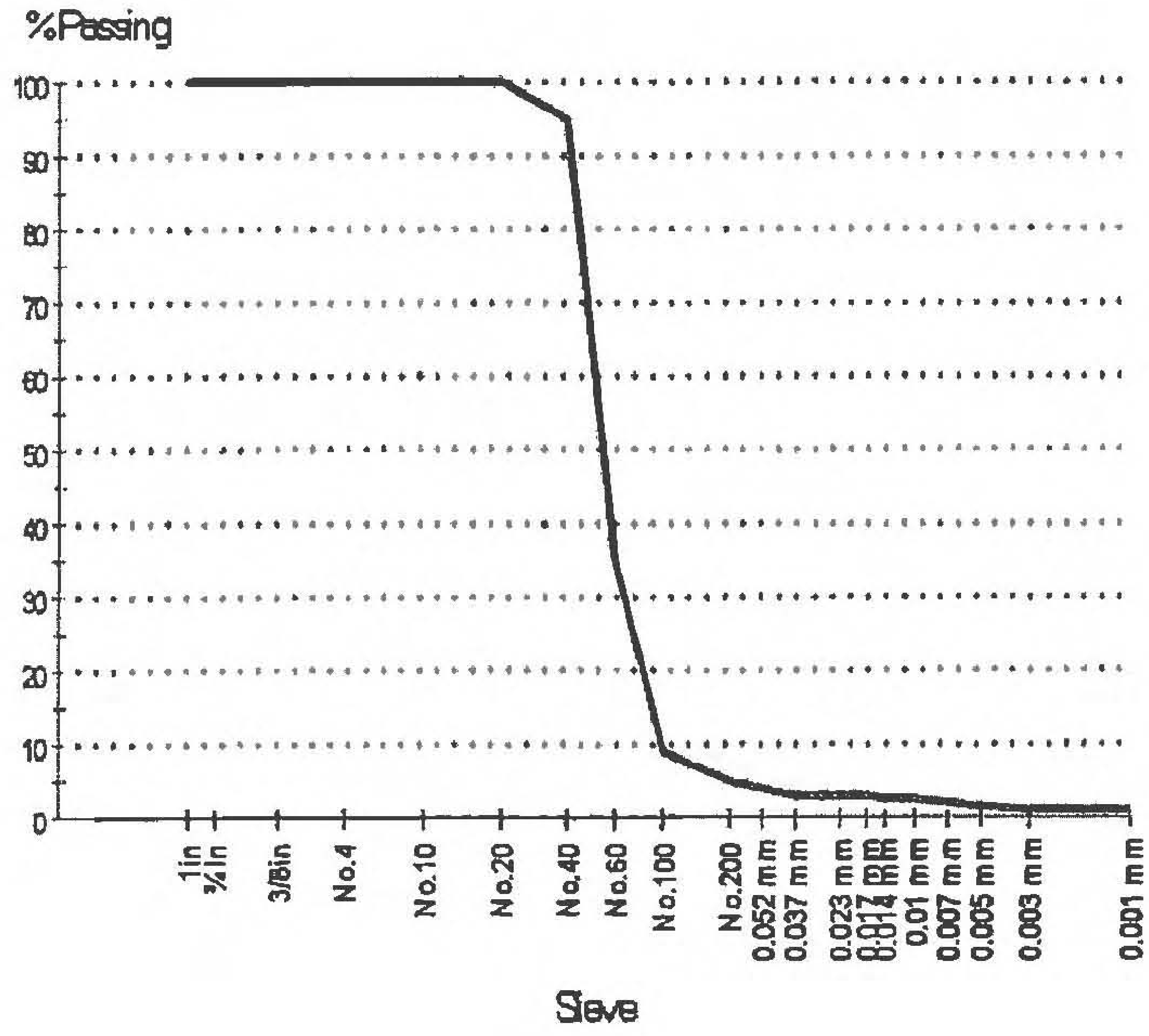
Boring No:	TB-102
Field Sample No:	LS-7
Sample Depth:	34.5
Date Sampled:	
Sampled By:	
LWO No:	000289
Sample Location:	Detroit River International Crossing

Other Test Results

Description	Method	Result	Limits
Sand Gravel Description	ASTM D 422		
Shape			
Hardness			
Dispersion Device			
Dispersion Period		1	
Moisture Content (%)	ASTM D 2216	19.4	
Wet Density (lb/ft³)		132.2	
Dry Density (lb/ft³)		110.7	

Particle Size Distribution

Method: ASTM D 422
Drying by: Oven



Sieve Size	% Passing	Limits
1in (25.0mm)	100	
¾in (19.0mm)	100	
3/8in (9.5mm)	100	
No.4 (4.75mm)	100	
No.10 (2.0mm)	100	
No.20 (850µm)	100	
No.40 (425µm)	95	
No.60 (250µm)	35	
No.100 (150µm)	9	
No.200 (75µm)	5	
0.052 mm	4	
0.037 mm	3	
0.023 mm	3	
0.017 mm	3	
0.014 mm	3	
0.010 mm	3	
0.007 mm	2	
0.005 mm	2	
0.003 mm	1	
0.001 mm	1	

Comments
N/A

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Aggregate/Soil Test Report

Report No: MAT:15-050014-00-S018
Issue No: 1

Client:	The Corradino Group
Project:	Detroit River Inter. Crossing Geotechnical Engineering
Job No:	15-050014-00

This laboratory is accredited by American Association of State Highway and Transportation Officials (AASHTO). The test(s) reported have been performed in accordance with the terms of the accreditation.

Date of Issue: 5/21/2008
Approved Signatory: Zeerak Paydawy

Sample Details

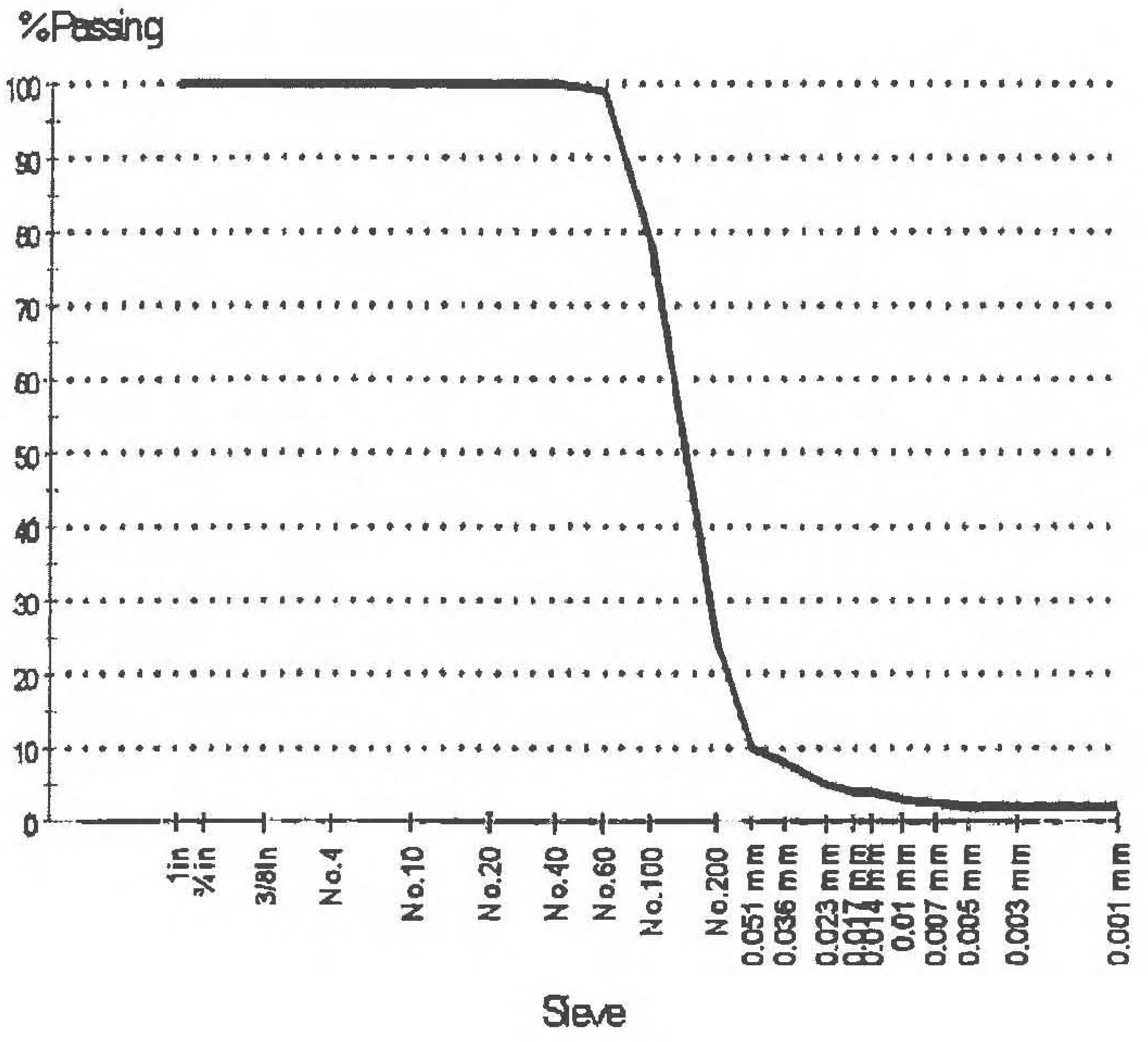
Boring No:	TB-102
Field Sample No:	LS-10
Sample Depth:	49.5
Date Sampled:	
Sampled By:	
LWO No:	000289
Sample Location:	Detroit River International Crossing

Other Test Results

Description	Method	Result	Limits
Sand Gravel Description	ASTM D 422		
Shape			
Hardness			
Dispersion Device			
Dispersion Period		1	
Moisture Content (%)	ASTM D 2216	19.9	
Wet Density (lb/ft³)			
Dry Density (lb/ft³)			

Particle Size Distribution

Method: ASTM D 422
Drying by: Oven



Sieve Size	% Passing	Limits
1in (25.0mm)	100	
¾in (19.0mm)	100	
3/8in (9.5mm)	100	
No.4 (4.75mm)	100	
No.10 (2.0mm)	100	
No.20 (850µm)	100	
No.40 (425µm)	100	
No.60 (250µm)	99	
No.100 (150µm)	78	
No.200 (75µm)	25	
0.051 mm	10	
0.036 mm	8	
0.023 mm	5	
0.017 mm	4	
0.014 mm	4	
0.010 mm	3	
0.007 mm	3	
0.005 mm	2	
0.003 mm	2	
0.001 mm	2	

Comments
N/A

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Aggregate/Soil Test Report

Report No: MAT:15-050014-00-S022
Issue No: 1

Client:	The Corradino Group
Project:	Detroit River Inter. Crossing Geotechnical Engineering
Job No:	15-050014-00

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Date of Issue: 5/21/2008
Approved Signatory: Zeerak Paydawy

Sample Details

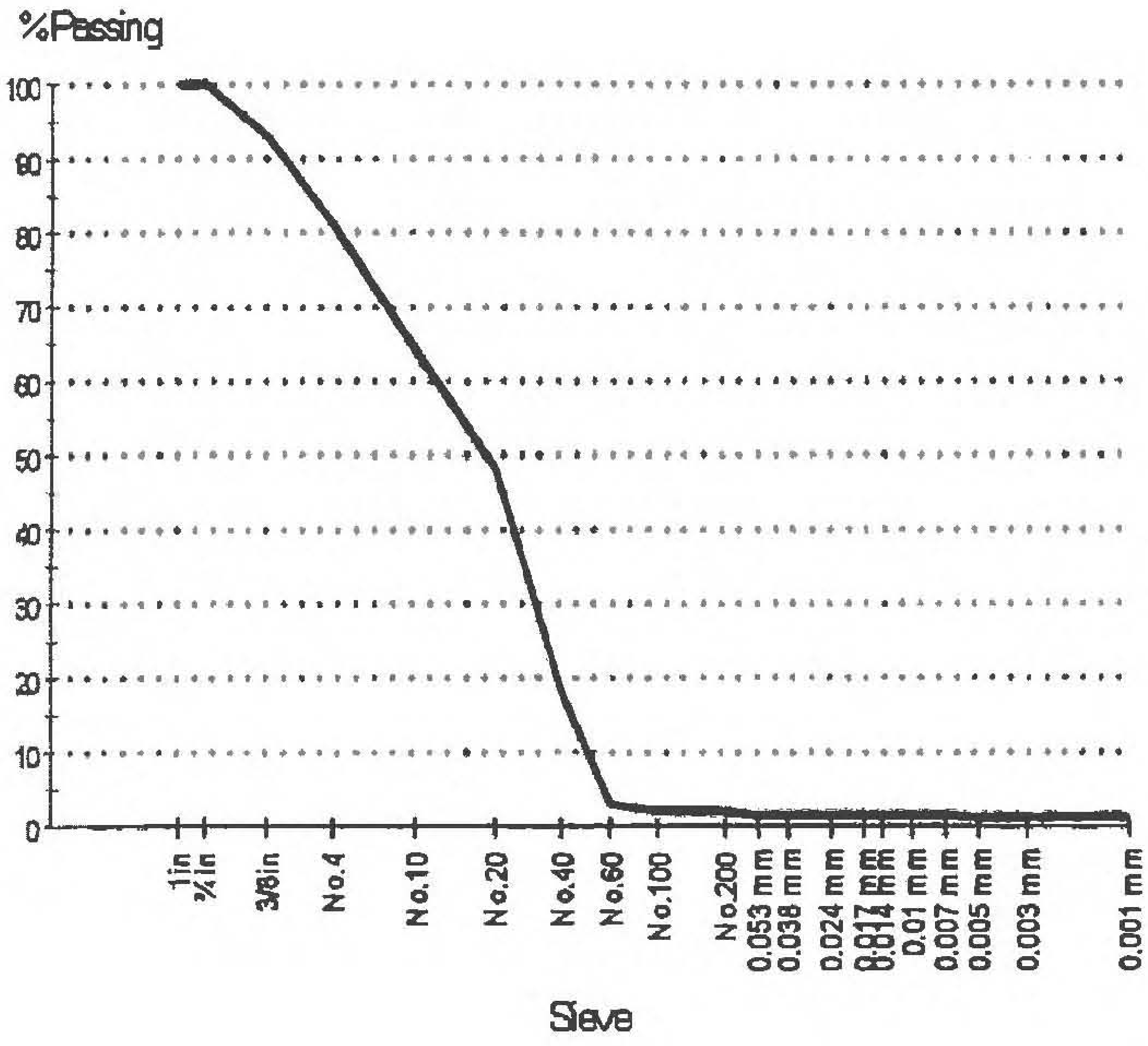
Boring No:	TB-103
Field Sample No:	LS-6
Sample Depth:	29.5
Date Sampled:	
Sampled By:	
LWO No:	000289
Sample Location:	Detroit River International Crossing

Other Test Results

Description	Method	Result	Limits
Sand Gravel Description	ASTM D 422		
Shape			
Hardness			
Dispersion Device			
Dispersion Period		1	
Moisture Content (%)	ASTM D 2216	14.2	
Wet Density (lb/ft³)		122.1	
Dry Density (lb/ft³)		106.9	

Particle Size Distribution

Method: ASTM D 422
Drying by: Oven



Sieve Size	% Passing	Limits
1in (25.0mm)	100	
¾in (19.0mm)	100	
3/8in (9.5mm)	93	
No.4 (4.75mm)	81	
No.10 (2.0mm)	64	
No.20 (850µm)	48	
No.40 (425µm)	18	
No.60 (250µm)	3	
No.100 (150µm)	2	
No.200 (75µm)	2	
0.053 mm	1	
0.038 mm	1	
0.024 mm	1	
0.017 mm	1	
0.014 mm	1	
0.010 mm	1	
0.007 mm	1	
0.005 mm	1	
0.003 mm	1	
0.001 mm	1	

Comments
N/A

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Aggregate/Soil Test Report

Report No: MAT:15-050014-00-S039
Issue No: 1

Client:	The Corradino Group
Project:	Detroit River Inter. Crossing Geotechnical Engineering
Job No:	15-050014-00

This laboratory is accredited by American Association of State Highway and Transportation Officials (AASHTO). The test(s) reported have been performed in accordance with the terms of the accreditation.

Date of Issue: 6/19/2008
Approved Signatory: Zeerak Paydawy

Sample Details

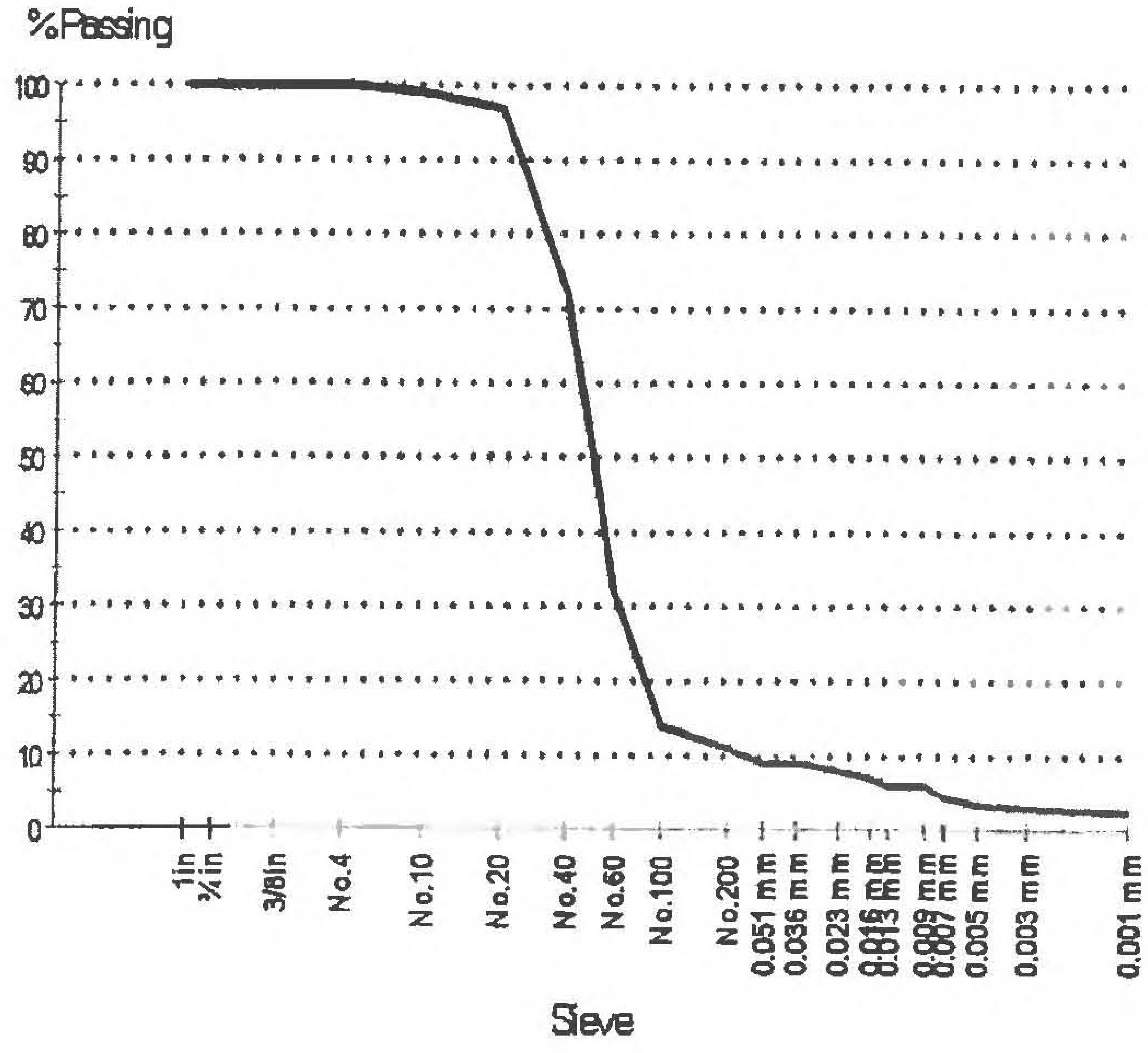
Boring No:	TB-104
Field Sample No:	LS-5
Sample Depth:	25
Date Sampled:	
Sampled By:	Nathan Emery
LWO No:	000307
Sample Location:	Detroit River International Crossing

Other Test Results

Description	Method	Result	Limits
Sand Gravel Description	ASTM D 422		
Shape			
Hardness			
Dispersion Device			
Dispersion Period		1	
Moisture Content (%)	ASTM D 2216	25.4	
Wet Density (lb/ft³)		125.9	
Dry Density (lb/ft³)		100.4	

Particle Size Distribution

Method: ASTM D 422
Drying by: Oven



Sieve Size	% Passing	Limits
1in (25.0mm)	100	
¾in (19.0mm)	100	
3/8in (9.5mm)	100	
No.4 (4.75mm)	100	
No.10 (2.0mm)	99	
No.20 (850µm)	97	
No.40 (425µm)	72	
No.60 (250µm)	32	
No.100 (150µm)	14	
No.200 (75µm)	11	
0.051 mm	9	
0.036 mm	9	
0.023 mm	8	
0.016 mm	7	
0.013 mm	6	
0.009 mm	6	
0.007 mm	4	
0.005 mm	3	
0.003 mm	3	
0.001 mm	2	

Comments
N/A

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Aggregate/Soil Test Report

Report No: MAT:15-050014-00-S041
Issue No: 1

Client:	The Corradino Group
Project:	Detroit River Inter. Crossing Geotechnical Engineering
Job No:	15-050014-00

This laboratory is accredited by American Association of State Highway and Transportation Officials (AASHTO). The test(s) reported have been performed in accordance with the terms of the accreditation.

Date of Issue: 6/19/2008
Approved Signatory: Zeerak Paydawy

Sample Details

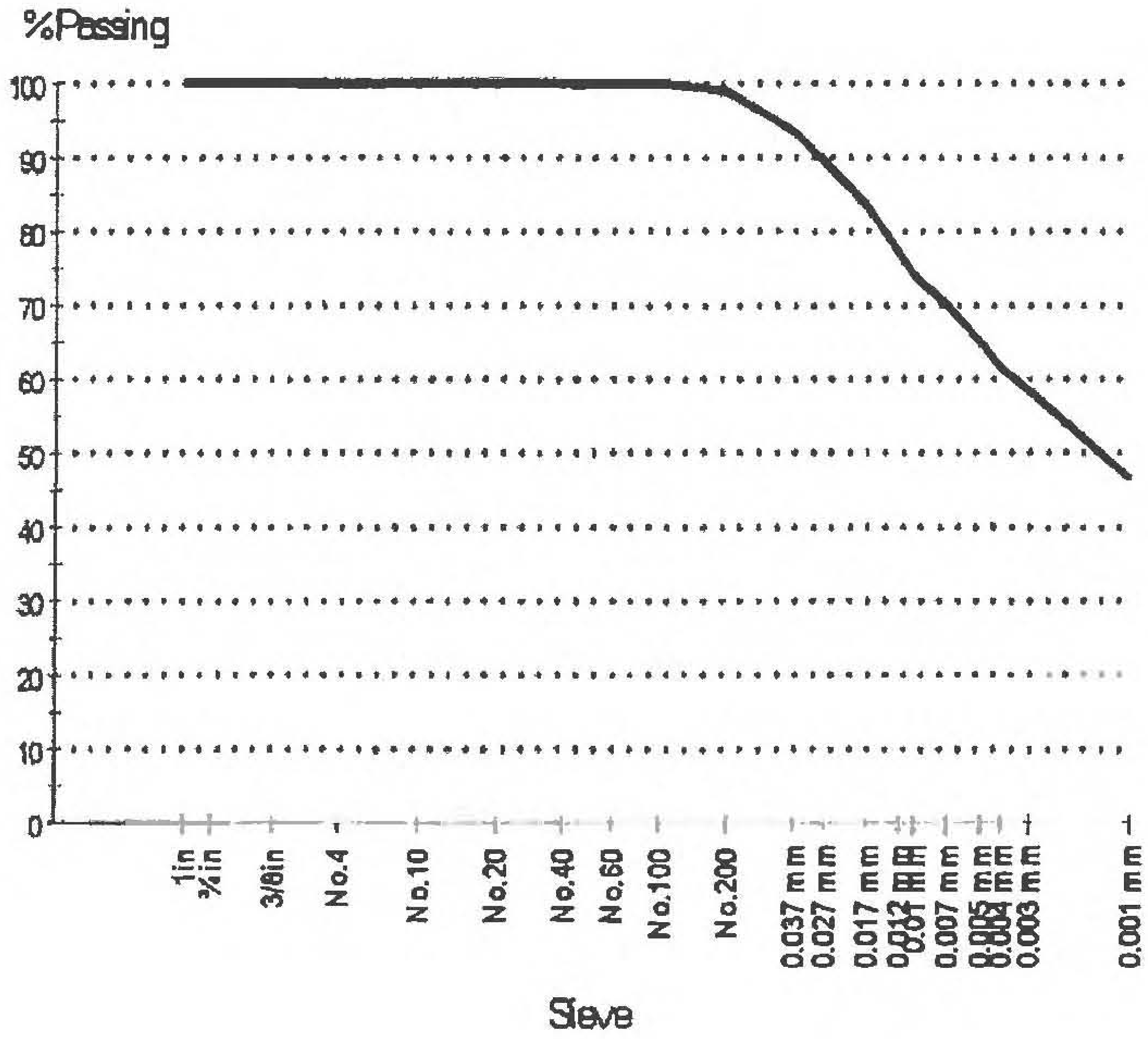
Boring No:	TB-104
Field Sample No:	LS-7
Sample Depth:	35
Date Sampled:	
Sampled By:	Nathan Emery
LWO No:	000307
Sample Location:	Detroit River International Crossing

Other Test Results

Description	Method	Result	Limits
Unconfined Compressive Strength (lb/ft²)	ASTM D 2166	412	
Shear Strength (lb/ft²)		206	
Ave. Rate Strain to Failure(%)		0.9	
Strain at Failure(%)		14.6	
Average Height (in.)		2.748	
Average Diameter (in.)		1.353	
Height-Diameter Ratio		2.0	
Init. Dry Dens.			
Init. Water Content (%)			
Liquid Limit			
Plastic Limit			

Particle Size Distribution

Method: ASTM D 422
Drying by: Oven



Sieve Size	% Passing	Limits
1in (25.0mm)	100	
¾in (19.0mm)	100	
3/8in (9.5mm)	100	
No.4 (4.75mm)	100	
No.10 (2.0mm)	100	
No.20 (850µm)	100	
No.40 (425µm)	100	
No.60 (250µm)	100	
No.100 (150µm)	100	
No.200 (75µm)	99	
0.037 mm	93	
0.027 mm	90	
0.017 mm	84	
0.012 mm	77	
0.010 mm	74	
0.007 mm	70	
0.005 mm	65	
0.004 mm	61	
0.003 mm	58	
0.001 mm	47	

Comments
N/A

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Aggregate/Soil Test Report

Report No: MAT:15-050014-00-S029
Issue No: 1

Client:	The Corradino Group
Project:	Detroit River Inter. Crossing Geotechnical Engineering
Job No:	15-050014-00

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Date of Issue: 6/19/2008
Approved Signatory: Zeerak Paydawy

Sample Details

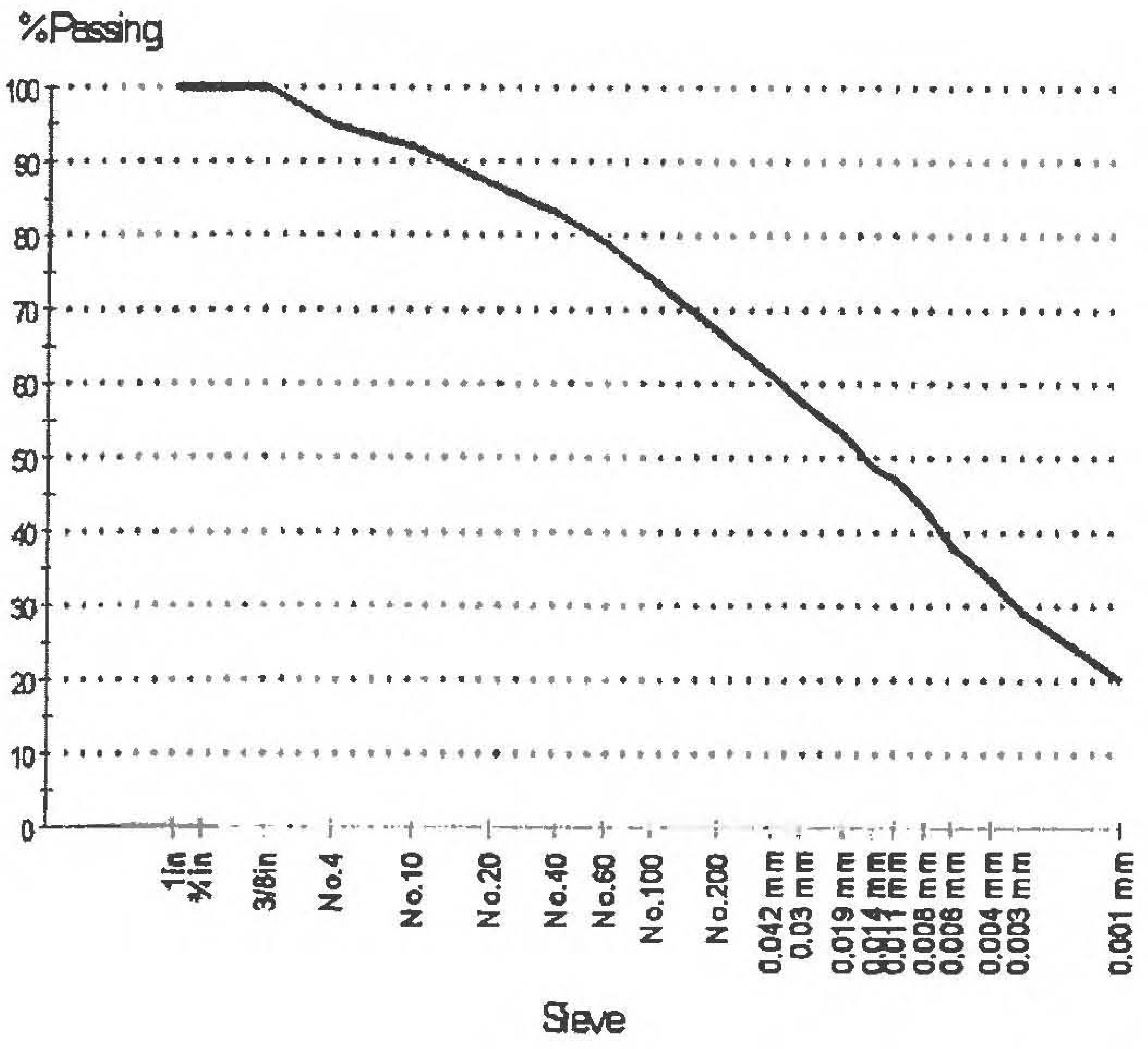
Boring No:	TB-105
Field Sample No:	LS-21
Sample Depth:	90
Date Sampled:	
Sampled By:	Nathan Emery
LWO No:	000307
Sample Location:	Detroit River International Crossing

Other Test Results

Description	Method	Result	Limits
Unconfined Compressive Strength (lb/ft²)	ASTM D 2166	N/O	
Shear Strength (lb/ft²)		N/O	
Ave. Rate Strain to Failure(%)		N/O	
Strain at Failure(%)		N/O	
Average Height (in.)		N/O	
Average Diameter (in.)		N/O	
Height-Diameter Ratio		N/O	
Init. Dry Dens.		N/O	
Init. Water Content (%)		N/O	
Liquid Limit		N/O	
Plastic Limit		N/O	

Particle Size Distribution

Method: ASTM D 422
Drying by: Oven



Sieve Size	% Passing	Limits
1in (25.0mm)	100	
¾in (19.0mm)	100	
3/8in (9.5mm)	100	
No.4 (4.75mm)	95	
No.10 (2.0mm)	92	
No.20 (850μm)	87	
No.40 (425μm)	83	
No.60 (250μm)	79	
No.100 (150μm)	74	
No.200 (75μm)	67	
0.042 mm	61	
0.030 mm	58	
0.019 mm	53	
0.014 mm	49	
0.011 mm	47	
0.008 mm	43	
0.006 mm	38	
0.004 mm	33	
0.003 mm	30	
0.001 mm	20	

Comments
Disturbed sample
N/O = Not Obtainable

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Aggregate/Soil Test Report

Report No: MAT:15-050014-00-S030
Issue No: 1

Client:	The Corradino Group
Project:	Detroit River Inter. Crossing Geotechnical Engineering
Job No:	15-050014-00

This laboratory is accredited by American Association of State Highway and Transportation Officials (AASHTO). The test(s) reported have been performed in accordance with the terms of the accreditation.

Date of Issue: 6/19/2008
Approved Signatory: Zeerak Paydawy

Sample Details

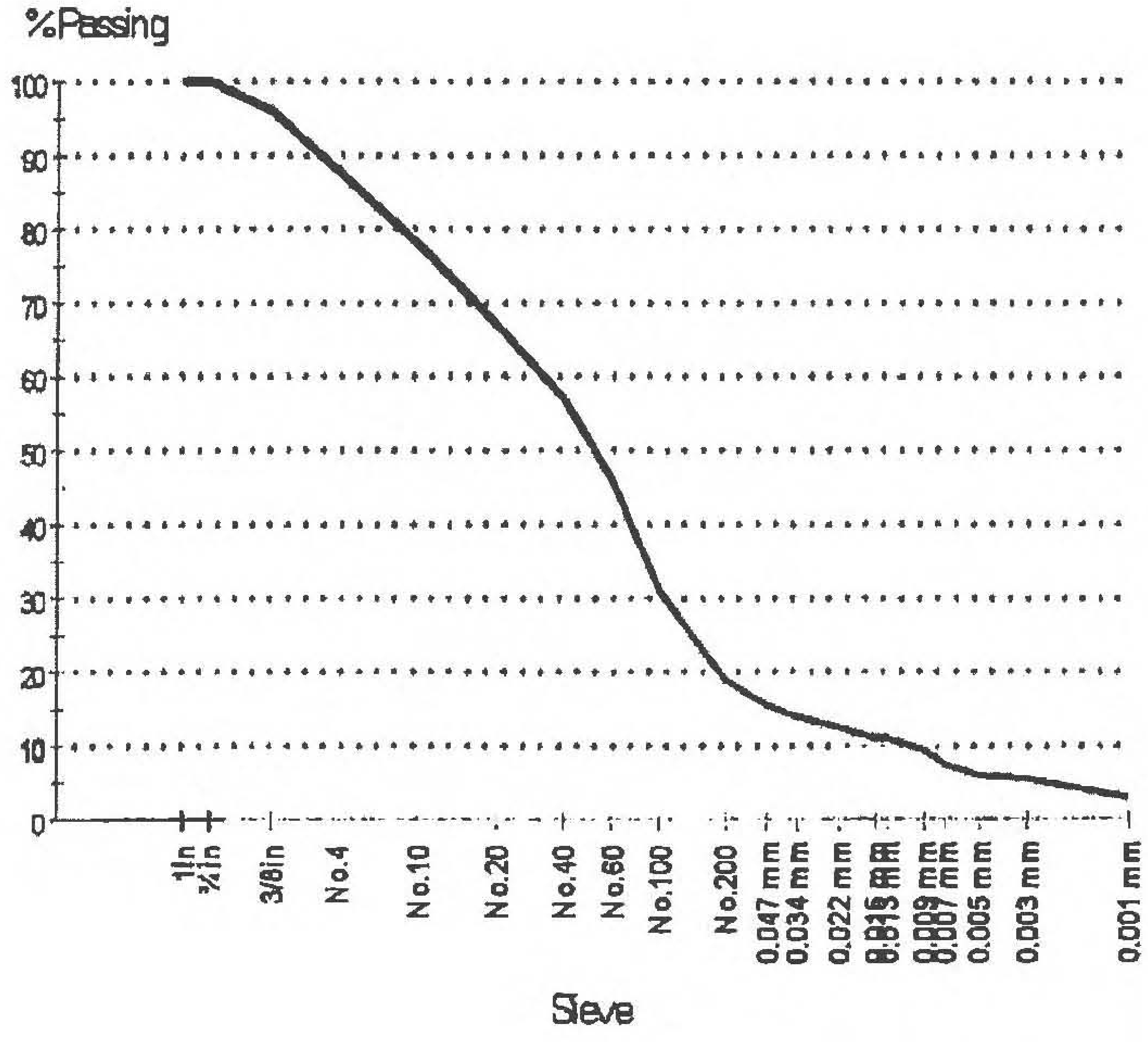
Boring No:	TB-105
Field Sample No:	LS-22
Sample Depth:	93.5
Date Sampled:	
Sampled By:	Nathan Emery
LWO No:	000307
Sample Location:	Detroit River International Crossing

Other Test Results

Description	Method	Result	Limits
Sand Gravel Description	ASTM D 422		
Shape			
Hardness			
Dispersion Device			
Dispersion Period		1	
Moisture Content (%)	ASTM D 2216	13.2	
Wet Density (lb/ft³)			
Dry Density (lb/ft³)			

Particle Size Distribution

Method: ASTM D 422
Drying by: Oven



Sieve Size	% Passing	Limits
1in (25.0mm)	100	
¾in (19.0mm)	100	
3/8in (9.5mm)	96	
No.4 (4.75mm)	88	
No.10 (2.0mm)	78	
No.20 (850µm)	67	
No.40 (425µm)	57	
No.60 (250µm)	46	
No.100 (150µm)	31	
No.200 (75µm)	19	
0.047 mm	15	
0.034 mm	14	
0.022 mm	13	
0.015 mm	11	
0.013 mm	11	
0.009 mm	10	
0.007 mm	8	
0.005 mm	6	
0.003 mm	5	
0.001 mm	3	

Comments
N/A

Figure No. 32

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Aggregate/Soil Test Report

Report No: MAT:15-050014-00-S045
Issue No: 1

Client: The Corradino Group

Project: Detroit River Inter. Crossing
Geotechnical Engineering

Job No: 15-050014-00

This laboratory is accredited by American Association of State Highway and Transportation Officials (AASHTO). The test(s) reported have been performed in accordance with the terms of the accreditation.

Date of Issue: 6/19/2008
Approved Signatory: Zeerak Paydawy

Sample Details

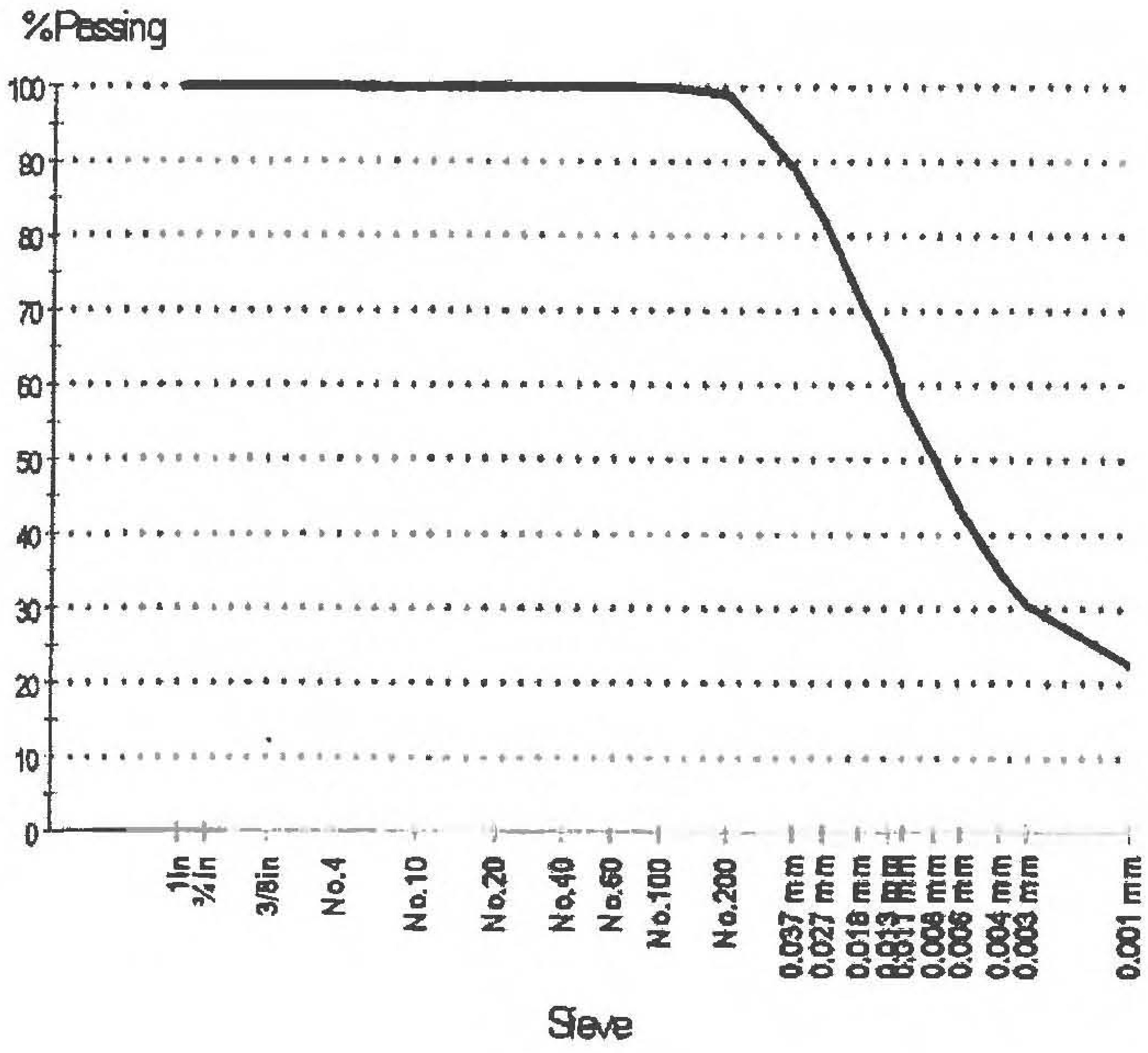
Boring No: TB-107
Field Sample No: LS-2
Sample Depth: 20
Date Sampled:
Sampled By: Nathan Emery
LWO No: 000308
Sample Location: Detroit River International Crossing

Other Test Results

Description	Method	Result	Limits
Unconfined Compressive Strength (lb/ft²)	ASTM D 2166	598	
Shear Strength (lb/ft²)		299	
Ave. Rate Strain to Failure(%)		0.9	
Strain at Failure(%)		14.9	
Average Height (in.)		2.688	
Average Diameter (in.)		1.361	
Height-Diameter Ratio		1.9	
Init. Dry Dens.			
Init. Water Content (%)			
Liquid Limit			
Plastic Limit			

Particle Size Distribution

Method: ASTM D 422
Drying by: Oven



Sieve Size	% Passing	Limits
1in (25.0mm)	100	
¾in (19.0mm)	100	
3/8in (9.5mm)	100	
No.4 (4.75mm)	100	
No.10 (2.0mm)	100	
No.20 (850µm)	100	
No.40 (425µm)	100	
No.60 (250µm)	100	
No.100 (150µm)	100	
No.200 (75µm)	99	
0.037 mm	89	
0.027 mm	82	
0.018 mm	71	
0.013 mm	64	
0.011 mm	58	
0.008 mm	50	
0.006 mm	43	
0.004 mm	35	
0.003 mm	30	
0.001 mm	23	

Comments
N/A

Figure No. 33

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Aggregate/Soil Test Report

Report No: MAT:15-050014-00-S049
Issue No: 1

Client: The Corradino Group

Project: Detroit River Inter. Crossing
Geotechnical Engineering

Job No: 15-050014-00

This laboratory is accredited by American Association of State Highway and Transportation Officials (AASHTO). The test(s) reported have been performed in accordance with the terms of the accreditation.

Date of Issue: 6/19/2008
Approved Signatory: Zeerak Paydawy

Sample Details

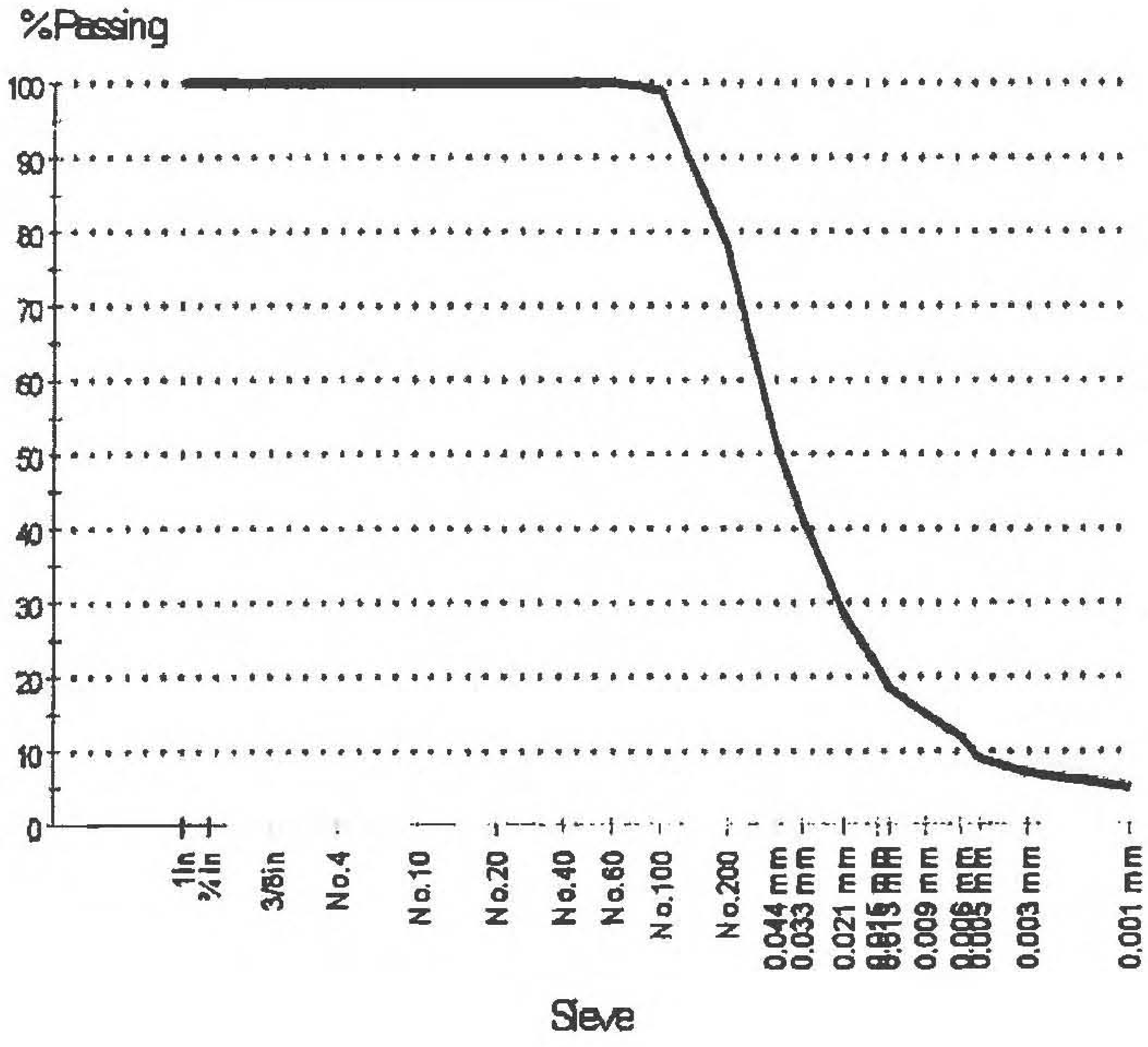
Boring No: TB-107
Field Sample No: LS-16
Sample Depth: 90
Date Sampled:
Sampled By: Nathan Emery
LWO No: 000308
Sample Location: Detroit River International Crossing

Other Test Results

Description	Method	Result	Limits
Sand Gravel Description	ASTM D 422		
Shape			
Hardness			
Dispersion Device			
Dispersion Period		1	
Moisture Content (%)	ASTM D 2216	19.9	
Wet Density (lb/ft³)		128.7	
Dry Density (lb/ft³)		107.3	

Particle Size Distribution

Method: ASTM D 422
Drying by: Oven



Sieve Size	% Passing	Limits
1in (25.0mm)	100	
¾in (19.0mm)	100	
3/8in (9.5mm)	100	
No.4 (4.75mm)	100	
No.10 (2.0mm)	100	
No.20 (850µm)	100	
No.40 (425µm)	100	
No.60 (250µm)	100	
No.100 (150µm)	99	
No.200 (75µm)	78	
0.044 mm	52	
0.033 mm	41	
0.021 mm	29	
0.015 mm	22	
0.013 mm	19	
0.009 mm	15	
0.006 mm	12	
0.005 mm	9	
0.003 mm	7	
0.001 mm	5	

Comments
N/A

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Aggregate/Soil Test Report

Report No: MAT:15-050014-00-S005
Issue No: 1

Client: The Corradino Group

Project: Detroit River Inter. Crossing
Geotechnical Engineering

Job No: 15-050014-00

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Date of Issue: 5/15/2008
Approved Signatory: Zeerak Paydawy

Sample Details

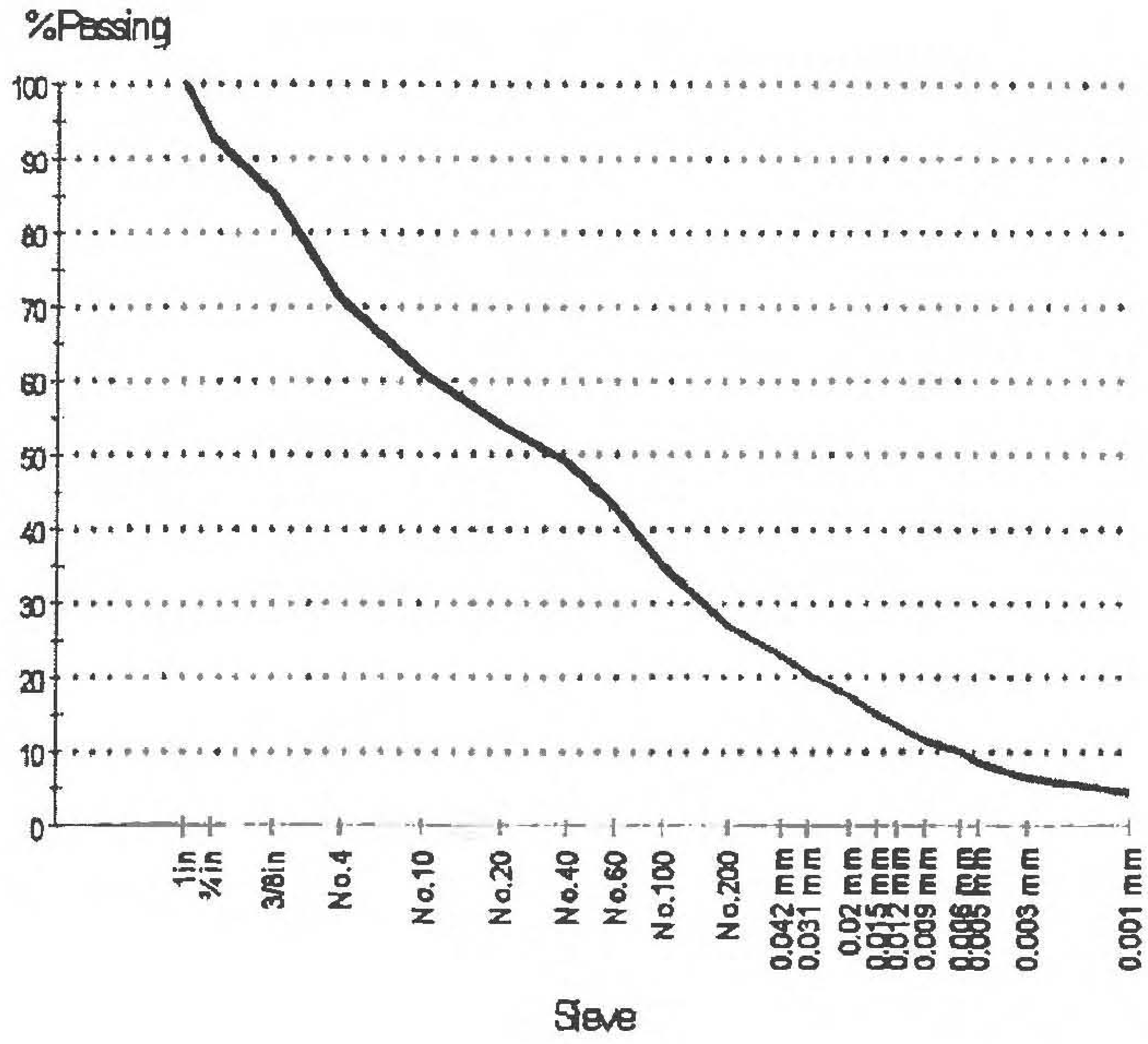
Boring No: TB-108
Field Sample No: LS-19
Sample Depth: 95
Date Sampled:
Sampled By: Nathan Emery
LWO No: 000286
Sample Location: Detroit River International Crossing

Other Test Results

Description	Method	Result	Limits
Sand Gravel Description	ASTM D 422		
Shape			
Hardness			
Dispersion Device			
Dispersion Period		1	
Moisture Content (%)	ASTM D 2216	7.9	
Wet Density (lb/ft³)		153.3	
Dry Density (lb/ft³)		142.1	

Particle Size Distribution

Method: ASTM D 422
Drying by: Oven



Sieve Size	% Passing	Limits
1in (25.0mm)	100	
¾in (19.0mm)	93	
3/8in (9.5mm)	85	
No.4 (4.75mm)	71	
No.10 (2.0mm)	61	
No.20 (850µm)	54	
No.40 (425µm)	49	
No.60 (250µm)	43	
No.100 (150µm)	35	
No.200 (75µm)	27	
0.042 mm	23	
0.031 mm	21	
0.020 mm	18	
0.015 mm	15	
0.012 mm	14	
0.009 mm	12	
0.006 mm	10	
0.005 mm	8	
0.003 mm	6	
0.001 mm	5	

Comments
N/A

Earth Mechanics Institute

Colorado School of Mines

Mining Engineering Department

Project Name: n/a

Location: n/a

Client: NTH Consultants

Date: 7/2/2008 Sample	Rock Type	Average Length	Average Diameter	Length to Diameter Ratio	Density	Failure Load	Failure Stress	Uniaxial Compressive Strength Corrected (2:1)		Notes
ID		(in)	(in)		(lbs/ft^3)	(lbs)	σ_c (psi)	(psi)	(MPa)	*(Failure type)*
TB-101@138.85-139.50	Sedimentary	4.121	1.975	2.09	155	33,993	11,101	11,255	78	Non-Structural
TB-102@106.2-106.8	Sedimentary	4.199	1.954	2.15	159	27,893	9,301	9,459	65	Non-Structural
TB-103@106.25-106.89	Sedimentary	4.161	1.988	2.09	161	32,148	10,362	10,509	72	Non-Structural
TB-103@110.6-111.34	Sedimentary	4.191	1.985	2.11	157	33,142	10,715	10,877	75	Non-Structural
TB-104@103.55-104.0	Sedimentary	4.184	1.946	2.15	169	65,594	22,054	22,430	155	Structural
TB-104@122.45-123.0	Sedimentary	4.167	1.938	2.15	162	44,576	15,111	15,369	106	Non-Structural
TB-105@102.8-103.4	Sedimentary	4.124	1.960	2.10	161	39,490	13,095	13,288	92	Non-Structural
TB-105@115.09-115.64	Sedimentary	4.070	1.964	2.07	153	33,466	11,047	11,191	77	Non-Structural
TB-106@101.5-102.05	Sedimentary	4.145	1.968	2.11	162	40,170	13,212	13,408	92	Non-Structural
TB-106@127.91-128.45	Sedimentary	4.101	1.973	2.08	154	27,027	8,844	8,963	62	Structural
TB-107@95.25-95.7	Sedimentary	4.111	1.960	2.10	160	34,209	11,338	11,501	79	Non-Structural
TB-107@120.15-120.7	Sedimentary	4.143	1.980	2.09	156	30,237	9,825	9,964	69	Structural
TB-108@101.15-101.87	Sedimentary	4.150	2.003	2.07	162	48,655	15,441	15,642	108	Non-Structural
TB-108@135.95-136.4	Sedimentary	4.202	1.987	2.12	162	38,459	12,409	12,598	87	Non-Structural

$$UCS_{2:1correction} = \frac{\sigma_c}{0.88 + 0.222(\frac{d}{l})}$$

Figure No. 36

Earth Mechanics Institute

Colorado School of Mines

Mining Engineering Department

Project Name: n/a

Location: n/a

Client: NTH Consultants

Date: 7/16/2008								Dynamic Elastic Constants		
Sample ID	Rock Type	Average Length	Average Diameter	Sample Weight	Density	P-Wave Velocity	S-Wave Velocity	Young's Modulus		Poisson's Ratio
		(in)	(in)	(g)	(lb/ft3)	(ft/sec)	(ft/sec)	(ksi)	(GPa)	
TB-101@105.83-106.4	Sedimentary	4.215	1.974	538.69	159	16,726	9,123	7,364	51	0.29
TB-101@138.85-139.5	Sedimentary	4.121	1.975	514.80	155	14,309	8,177	5,641	39	0.26

Figure No. 36

Earth Mechanics Institute

Project Name: Chile

Location: n/a

Client: NTH Consultants

Colorado School of Mines

Mining Engineering Department

8/5/2008	Rock Type	Avgerage Diameter	Avgerage Length	Confining Pressure	Triaxial Compressive Strength		Notes
Sample					Peak Stress		
ID		(in)	(in)	(psi)	(psi)	(MPa)	*(Failure type)*
TB-101@105.83-106.4_T400	Sedimentary	1.975	4.215	400	13,922	96.0	Non-Structural
TB-102@101.6-107.25_T200	Sedimentary	1.949	4.103	200	13,626	93.9	Structural

Figure No. 36

Earth Mechanics Institute

Project Name: n/a

Location: n/a

Client: NTH Consultants

Colorado School of Mines

Mining Engineering Department

Date: 7/1/2008						
Sample ID	**Rock Type**	**Average Length**	**Average Diameter**	**Indirect (Brazilian) Tensile Strength**		**Notes** *(Failure type)*
		(in)	**(in)**	**(psi)**	**(MPa)**	
TB-101@105.83-106.4	Sedimentary	1.39	1.975	669	4.6	Non-Structural
TB-101@138.85-139.5	Sedimentary	1.42	1.978	494	3.4	Non-Structural
TB-102@101.6-107.25	Sedimentary	1.47	1.947	877	6.0	Non-Structural
TB-102@106.2-106.8	Sedimentary	1.39	1.953	890	6.1	Non-Structural
TB-105@102.8-103.4	Sedimentary	1.32	1.961	751	5.2	Non-Structural
TB-106@101.5-102.05	Sedimentary	1.45	1.968	975	6.7	Non-Structural

Figure No. 36

Earth Mechanics Institute

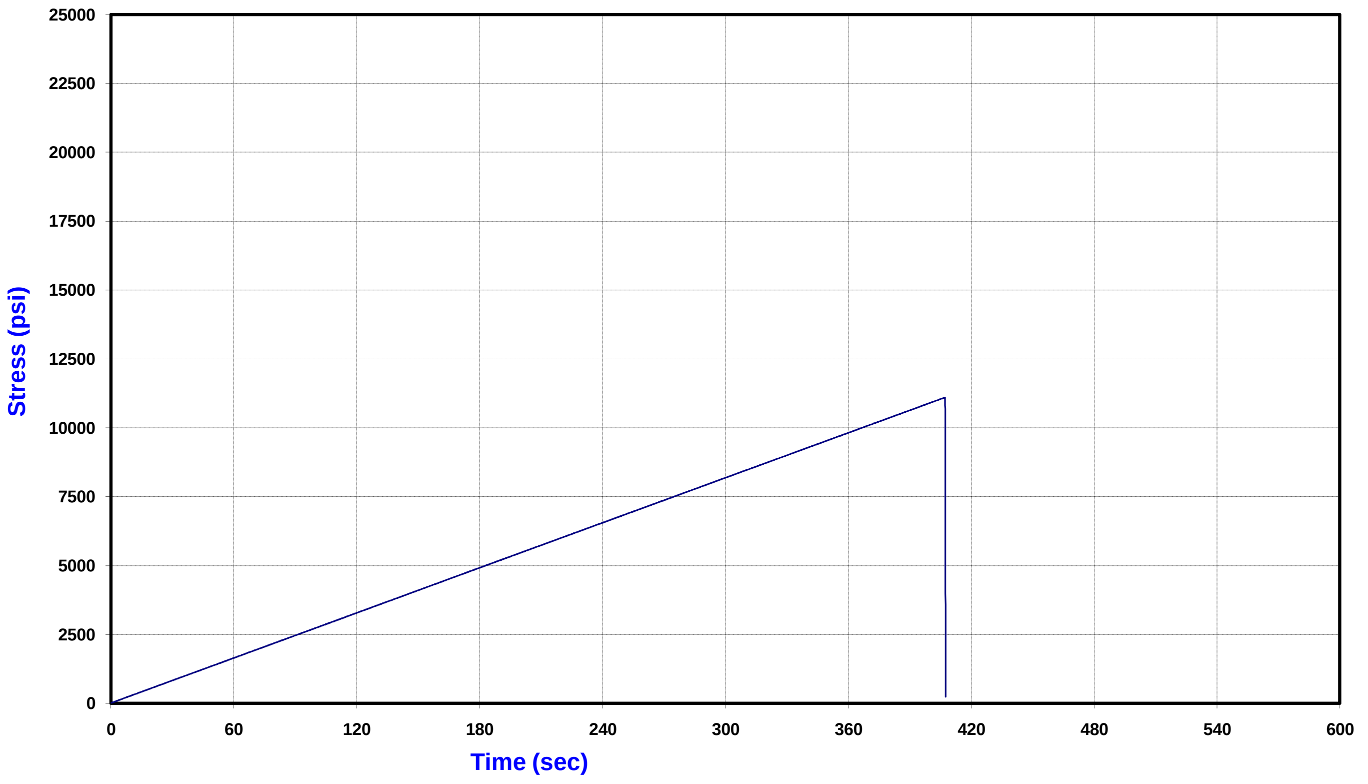
Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-101@138.85-139.50
File Name: TB-101@138.85-139.50_USG
Test Performed by: BM
Date Tested: 8/12/2008
Data Reduced by: BM
Date Reduced: 8/12/2008

Core Length		Diameter		L/D	Failure Load	Failure Stress	UCS 2:1		Failure
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	Mode
4.121	10.47	1.975	5.02	2.09	33,993	11,101	11,255	78	Non-Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	155	2.49

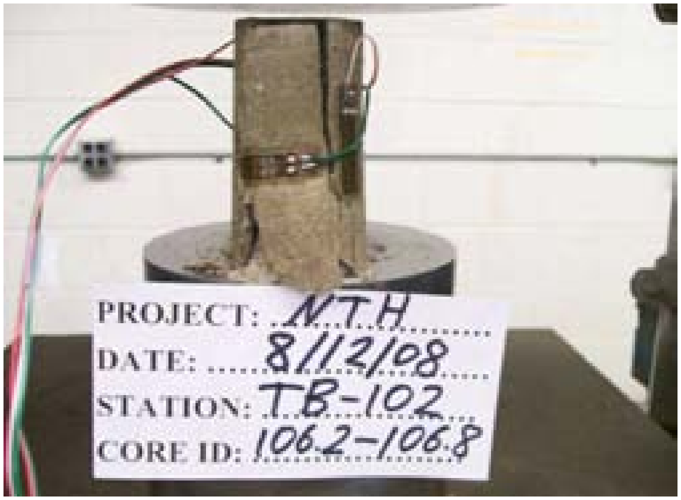


Earth Mechanics Institute

Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-102@106.2-106.8
File Name: TB-102@106.2-106.8_USG
Test Performed by: BM
Date Tested: 8/12/2008
Data Reduced by: BM
Date Reduced: 8/12/2008



Core Length		Diameter		L/D	Failure Load	Failure Stress	UCS 2:1		Failure
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	Mode
4.199	10.67	1.954	4.96	2.15	27,893	9,301	9,459	65	Non-Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	159	2.55

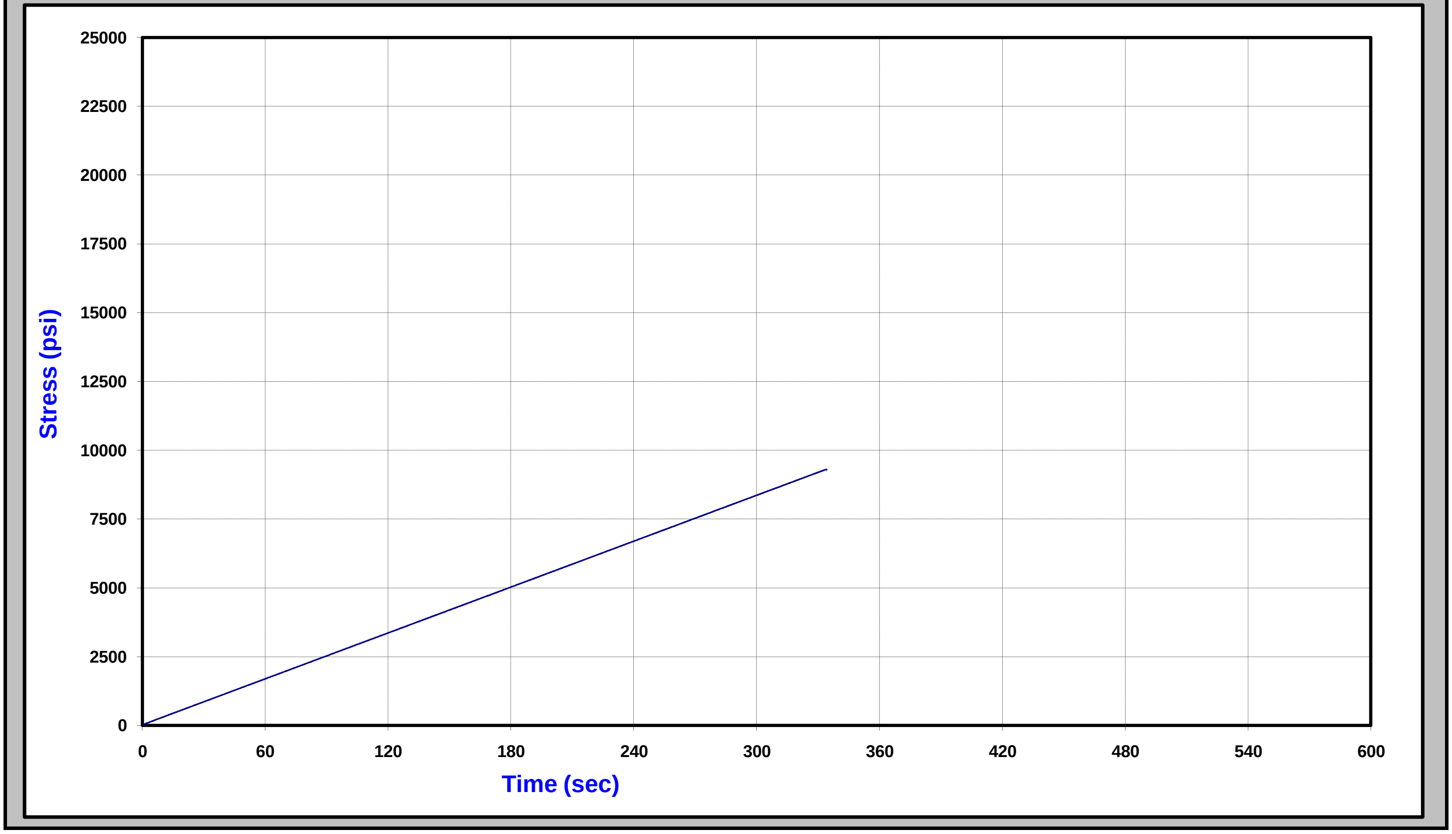


Figure No. 36

Earth Mechanics Institute

Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-103@106.25-106.89
File Name: TB-103@106.25-106.89_UCS
Test Performed by: Jake
Date Tested: 7/1/2008
Data Reduced by: BM
Date Reduced: 7/2/2008

Core Length		Diameter		L/D	Failure Load	Failure Stress	UCS 2:1		Failure
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	Mode
4.161	10.57	1.988	5.05	2.09	32,148	10,362	10,509	72	Non-Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	161	2.57

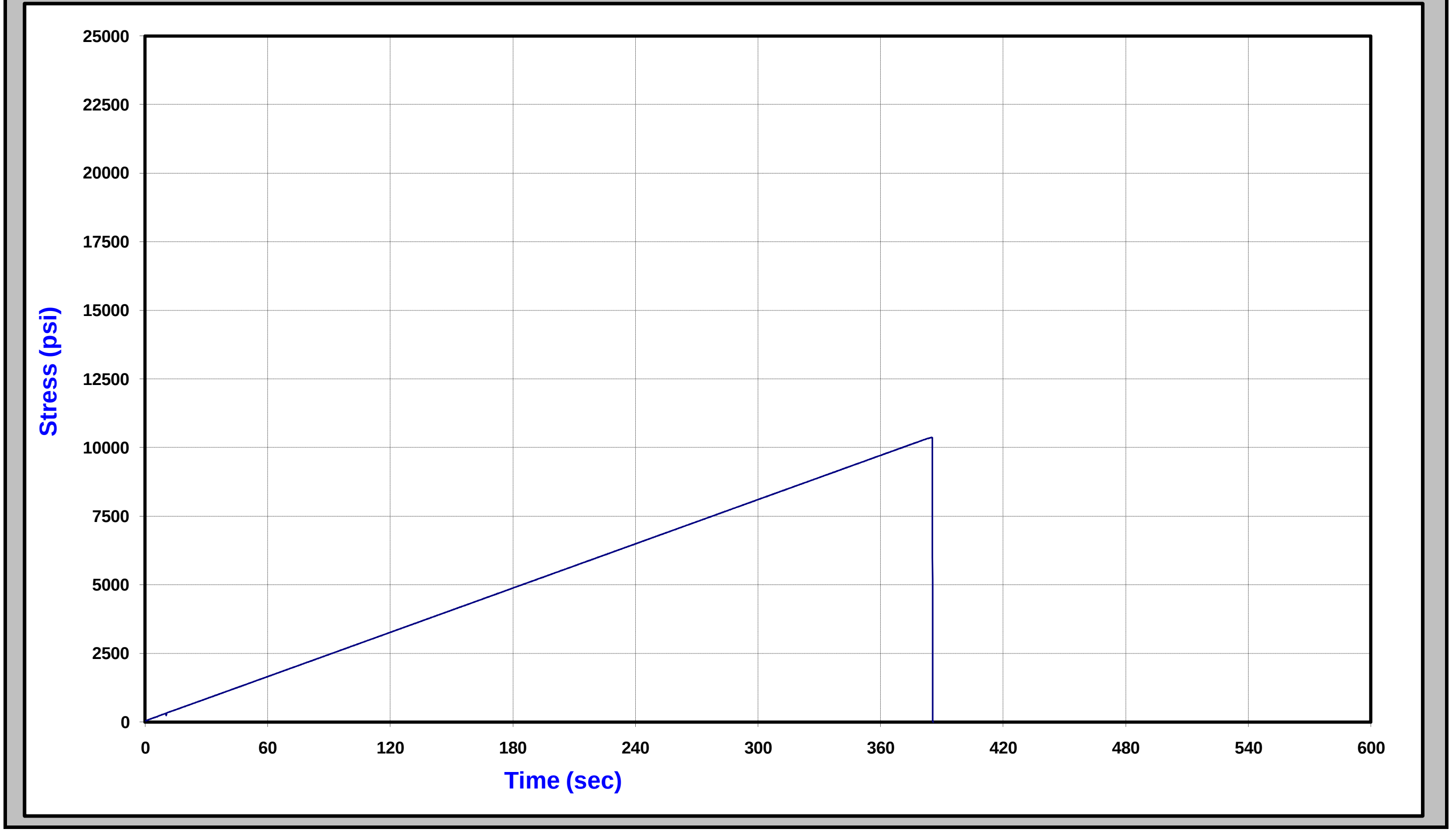


Figure No. 36

Earth Mechanics Institute

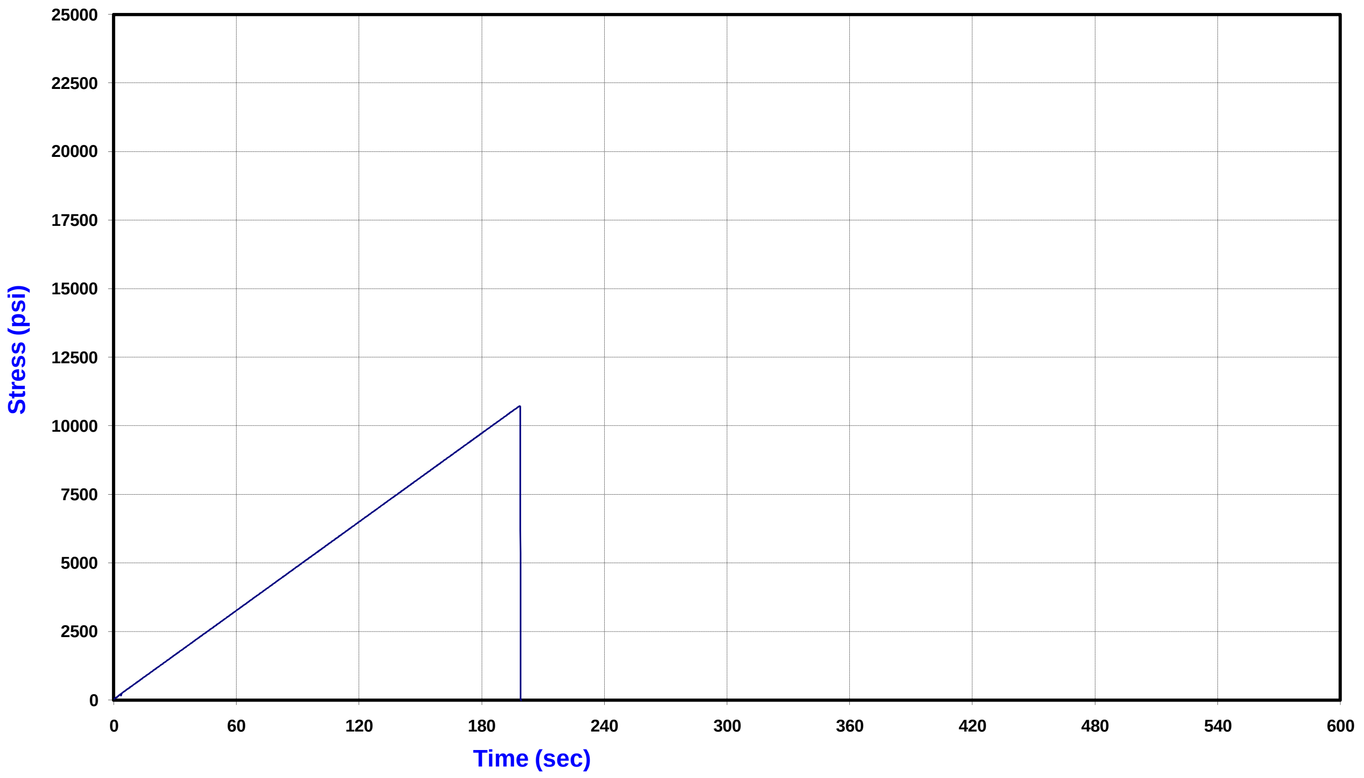
Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-103@110.6-111.34
File Name: TB-103@110.6-111.34_UCS
Test Performed by: Jake
Date Tested: 7/1/2008
Data Reduced by: BM
Date Reduced: 7/2/2008

Core Length		Diameter		L/D	Failure Load	Failure Stress	UCS 2:1		Failure
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	Mode
4.191	10.65	1.985	5.04	2.11	33,142	10,715	10,877	75	Non-Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	157	2.51



Earth Mechanics Institute

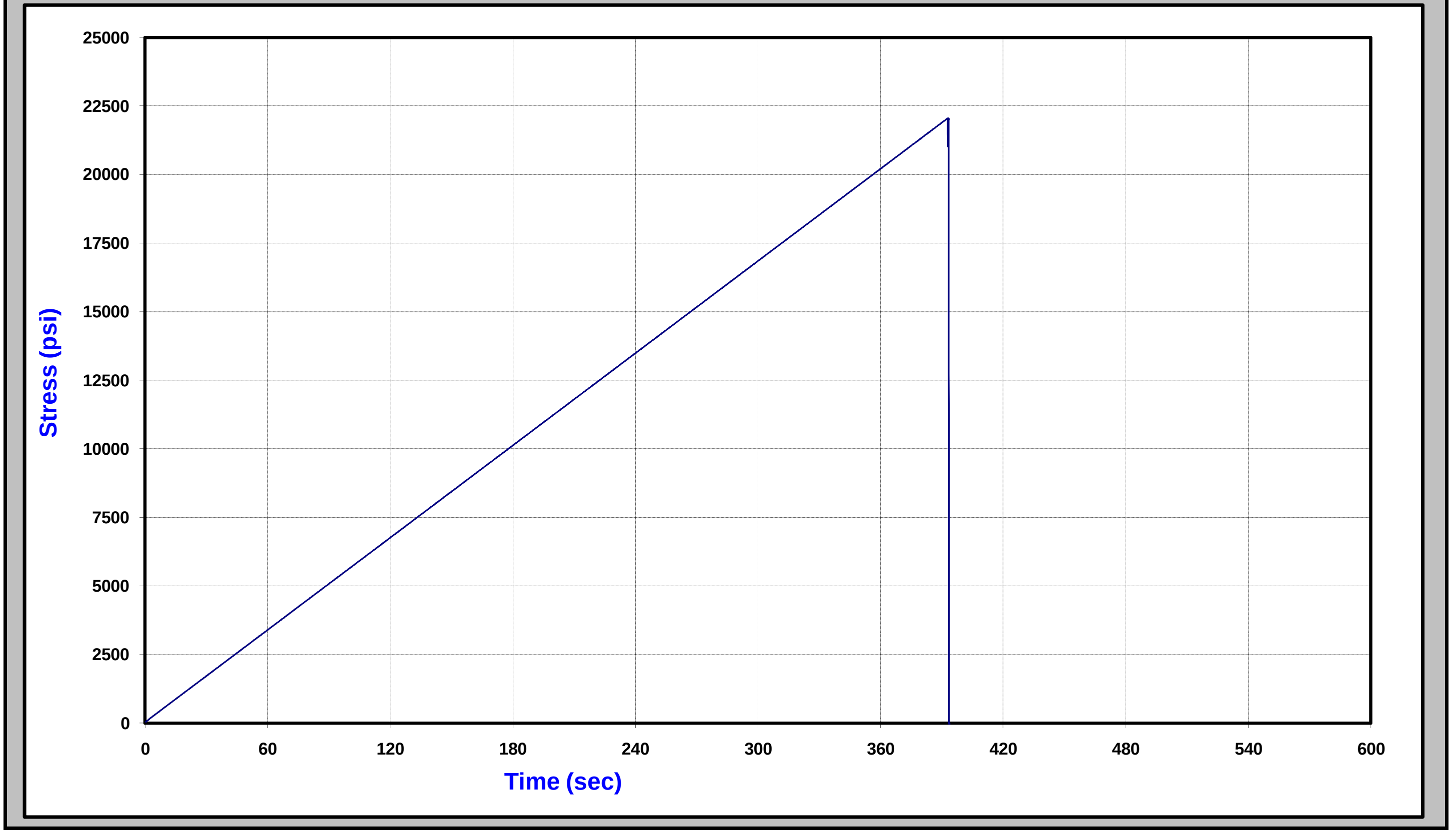
Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-104@103.55-104.0
File Name: TB-104@103.55-104.0_UCS
Test Performed by: Jake
Date Tested: 7/1/2008
Data Reduced by: BM
Date Reduced: 7/2/2008

Core Length		Diameter		L/D	Failure Load	Failure Stress	UCS 2:1		Failure
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	Mode
4.184	10.63	1.946	4.94	2.15	65,594	22,054	22,430	155	Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	169	2.71



Earth Mechanics Institute
Mining Engineering Department, CSM
Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-104@122.45-123.0
File Name: TB-104@122.45-123.0_UCS
Test Performed by: Jake
Date Tested: 7/1/2008
Data Reduced by: BM
Date Reduced: 7/2/2008

Core Length		Diameter		L/D	Failure Load	Failure Stress	UCS 2:1		Failure
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	Mode
4.167	10.58	1.938	4.92	2.15	44,576	15,111	15,369	106	Non-Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	162	2.59

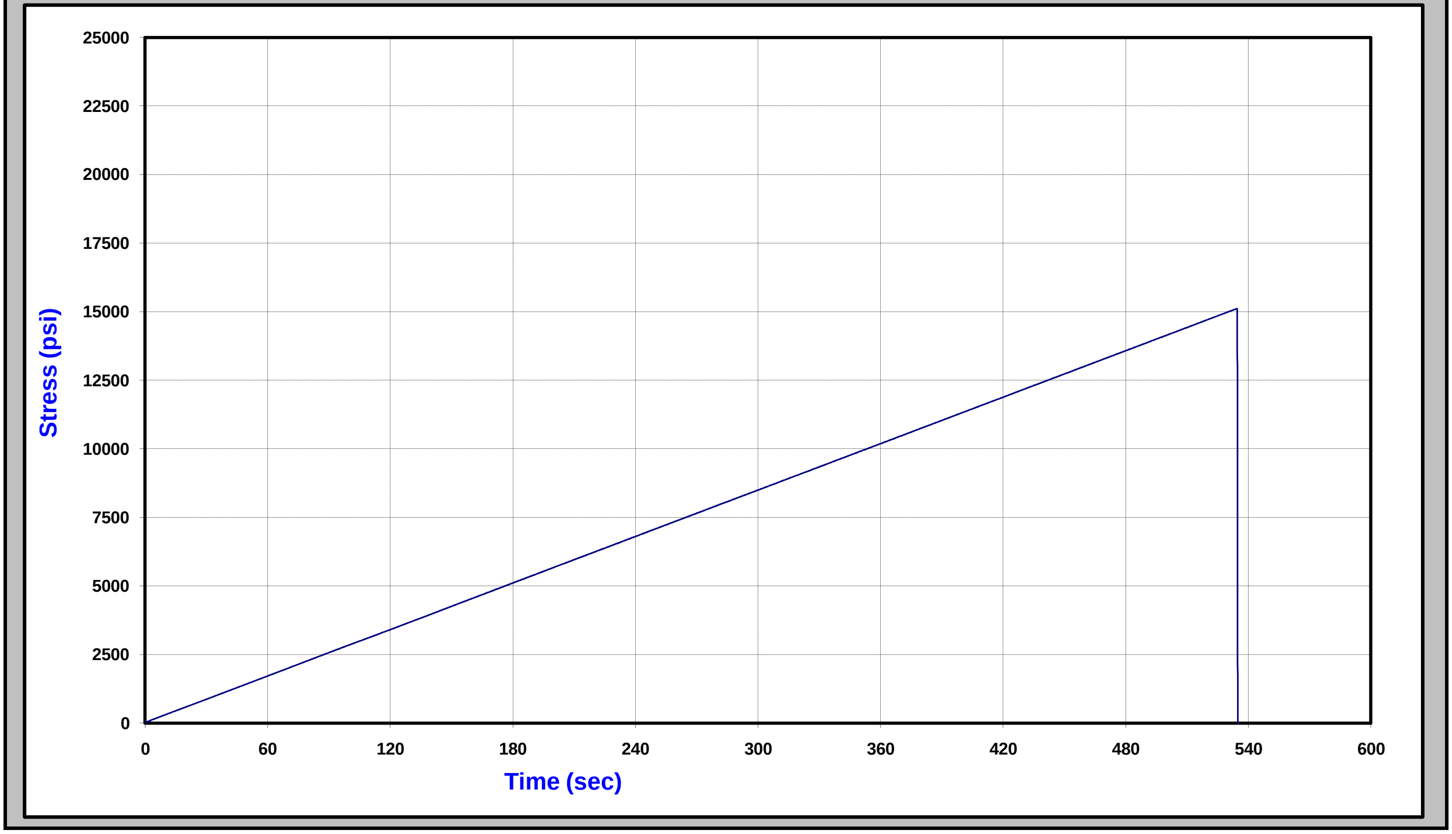


Figure No. 36

Earth Mechanics Institute

Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-105@102.8-103.4
File Name: TB-105@102.8-103.4_USG
Test Performed by: BM
Date Tested: 8/12/2008
Data Reduced by: BM
Date Reduced: 8/12/2008


PROJECT: NTH
DATE: 8/12/08
STATION: TB-105
CORE ID: 102.8-1034

Core Length		Diameter		L/D	Failure Load	Failure Stress	UCS 2:1		Failure Mode
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	Mode
4.124	10.47	1.960	4.98	2.10	39,490	13,095	13,288	92	Non-Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	161	2.58

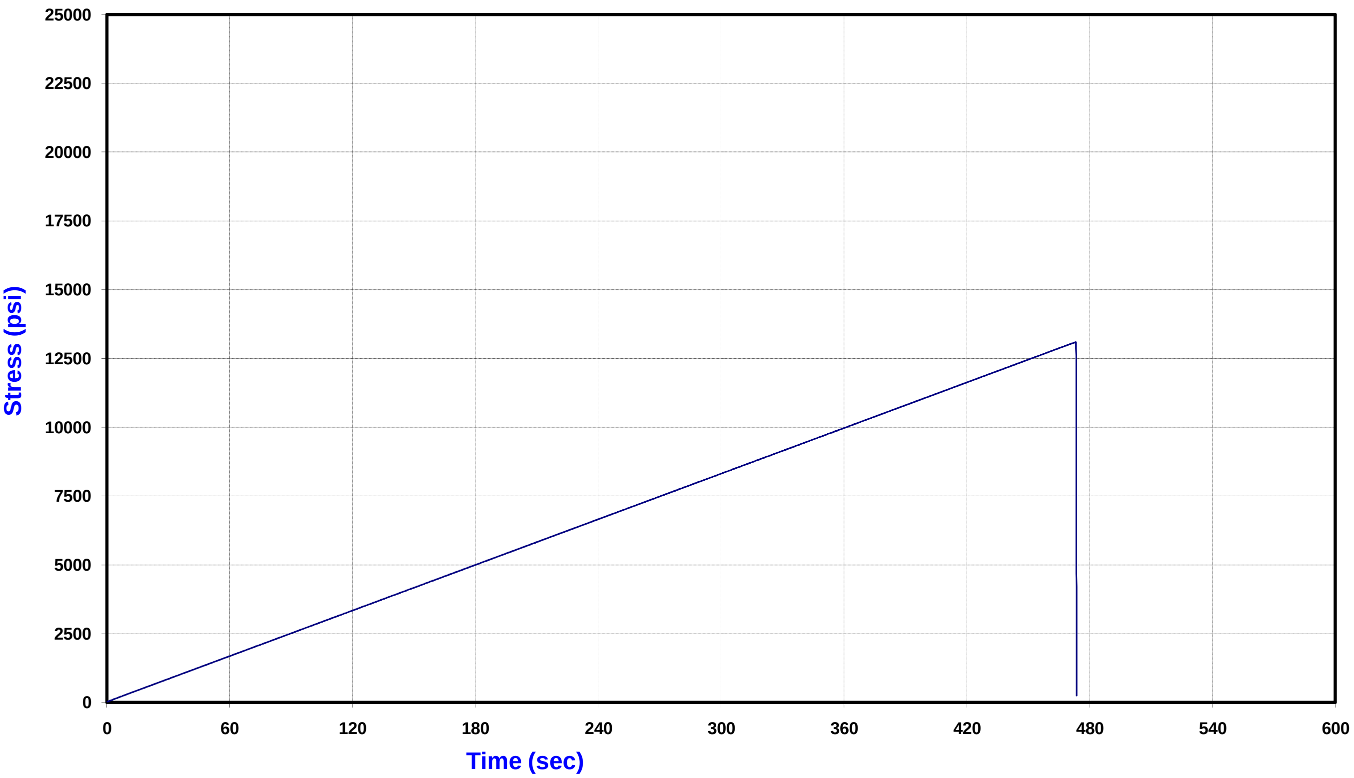


Figure No. 36

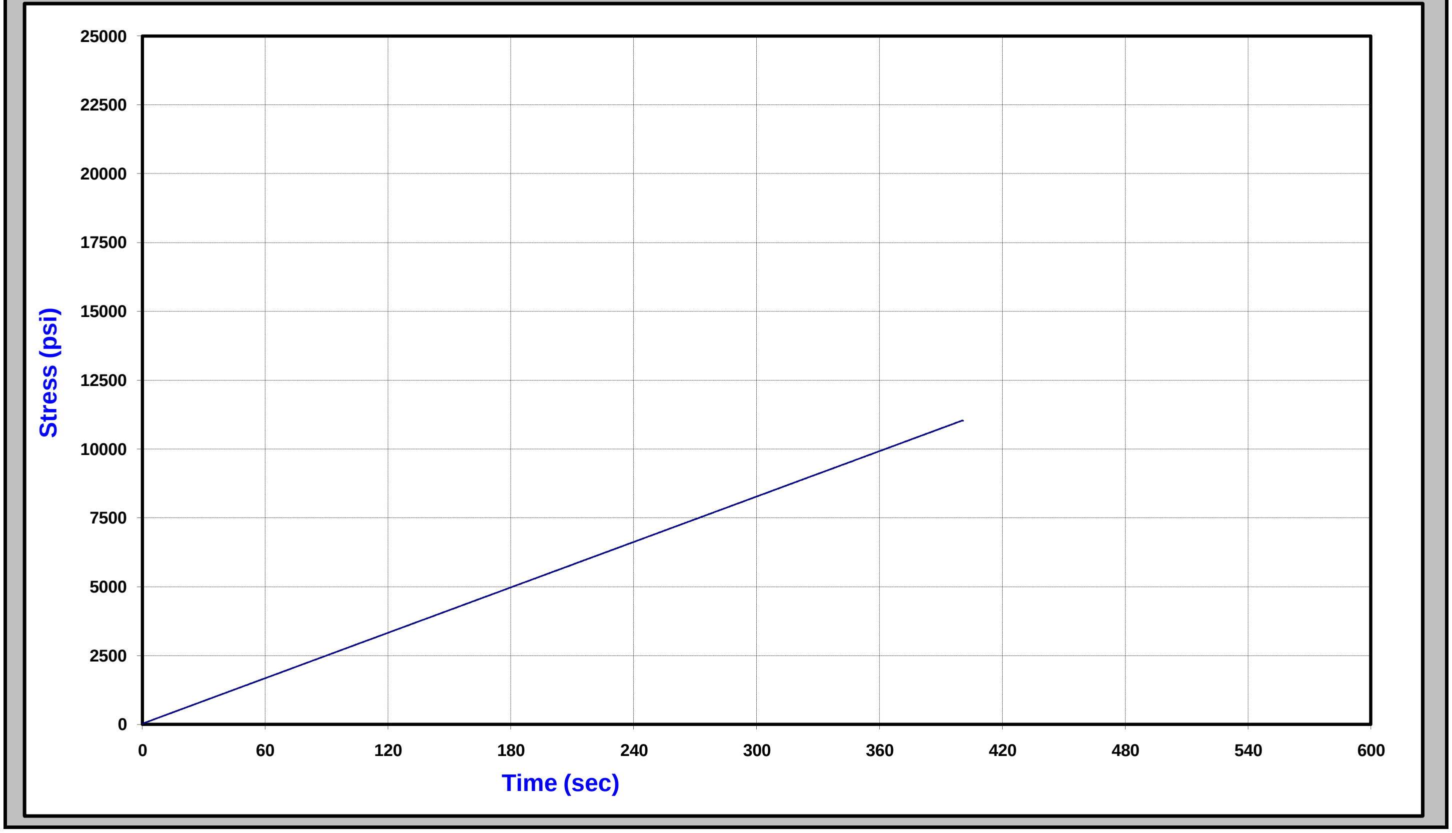
Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-105@115.09-115.64
File Name: TB-105@115.09-115.64_USG
Test Performed by: BM
Date Tested: 8/12/2008
Data Reduced by: BM
Date Reduced: 8/12/2008


PROJECT: NTH
DATE: 8/12/08
STATION: TB-105
CORE ID: 115.09-115.64

Core Length		Diameter		L/D	Failure Load	Failure Stress	UCS 2:1		Failure Mode
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	
4.070	10.34	1.964	4.99	2.07	33,466	11,047	11,191	77	Non-Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	153	2.45



Earth Mechanics Institute

Mining Engineering Department, CSM

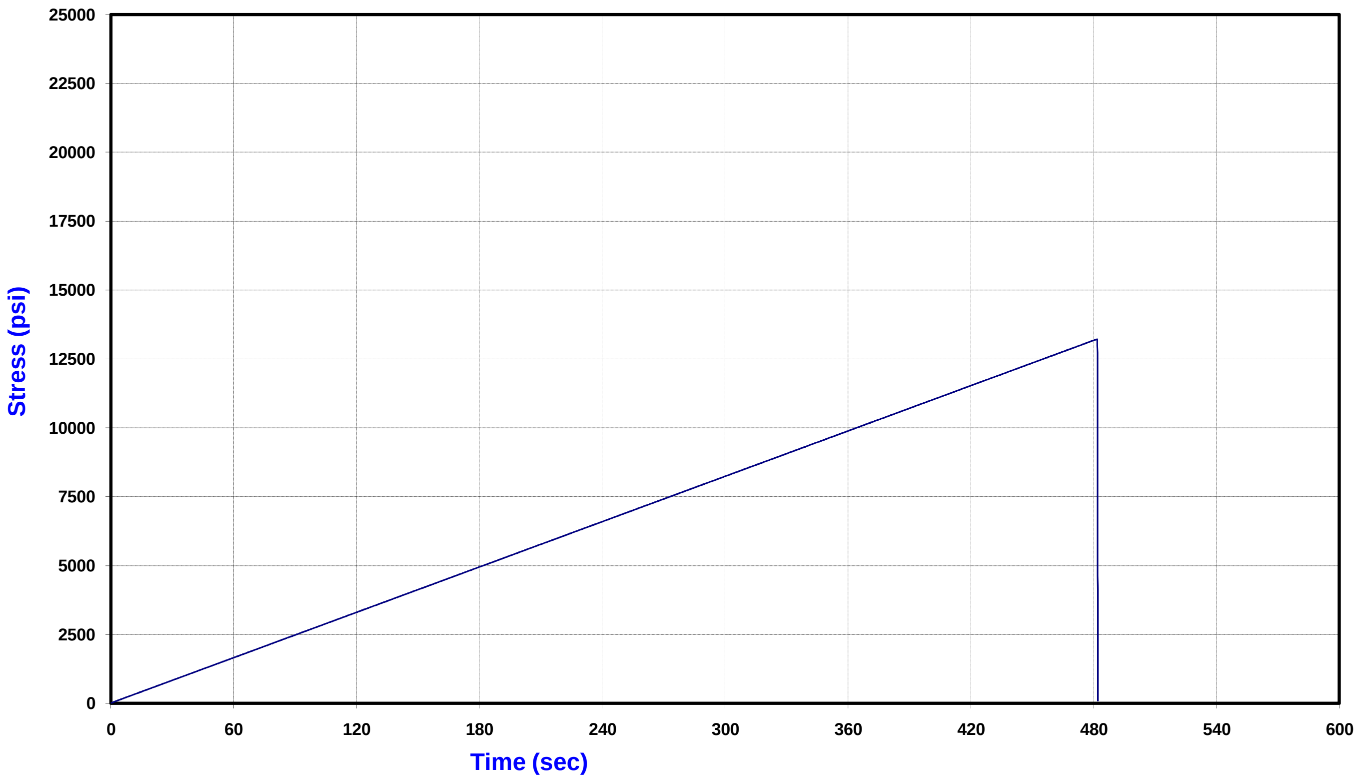
Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-106@101.5-102.05
File Name: TB-106@101.5-102.05_USG
Test Performed by: BM
Date Tested: 8/12/2008
Data Reduced by: BM
Date Reduced: 8/12/2008


PROJECT: NTH
DATE: 8/12/08
STATION: TB-106
CORE ID: @101.5-102.05

Core Length		Diameter		L/D	Failure Load	Failure Stress	UCS 2:1		Failure Mode
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	
4.145	10.53	1.968	5.00	2.11	40,170	13,212	13,408	92	Non-Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	162	2.59



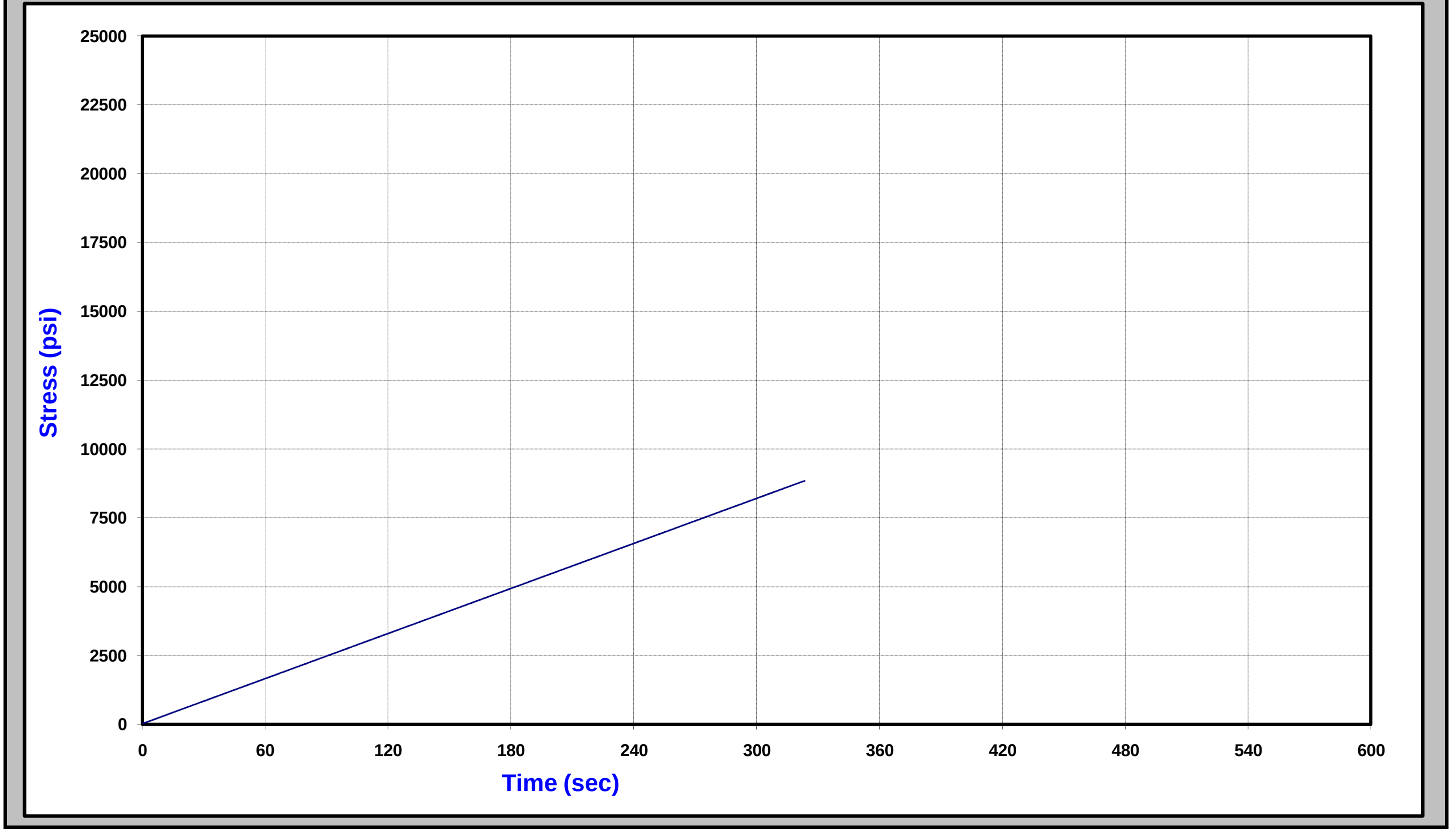
Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-106@127.91-128.45
File Name: TB-106@127.91-128.45_USG
Test Performed by: BM
Date Tested: 8/12/2008
Data Reduced by: BM
Date Reduced: 8/12/2008

PROJECT: NTH
DATE: 8/12/08
STATION: TB-106
CORE ID: 127.91-128.45

Core Length		Diameter		L/D	Failure Load	Failure Stress	UCS 2:1		Failure Mode
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	
4.101	10.42	1.973	5.01	2.08	27,027	8,844	8,963	62	Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	154	2.47

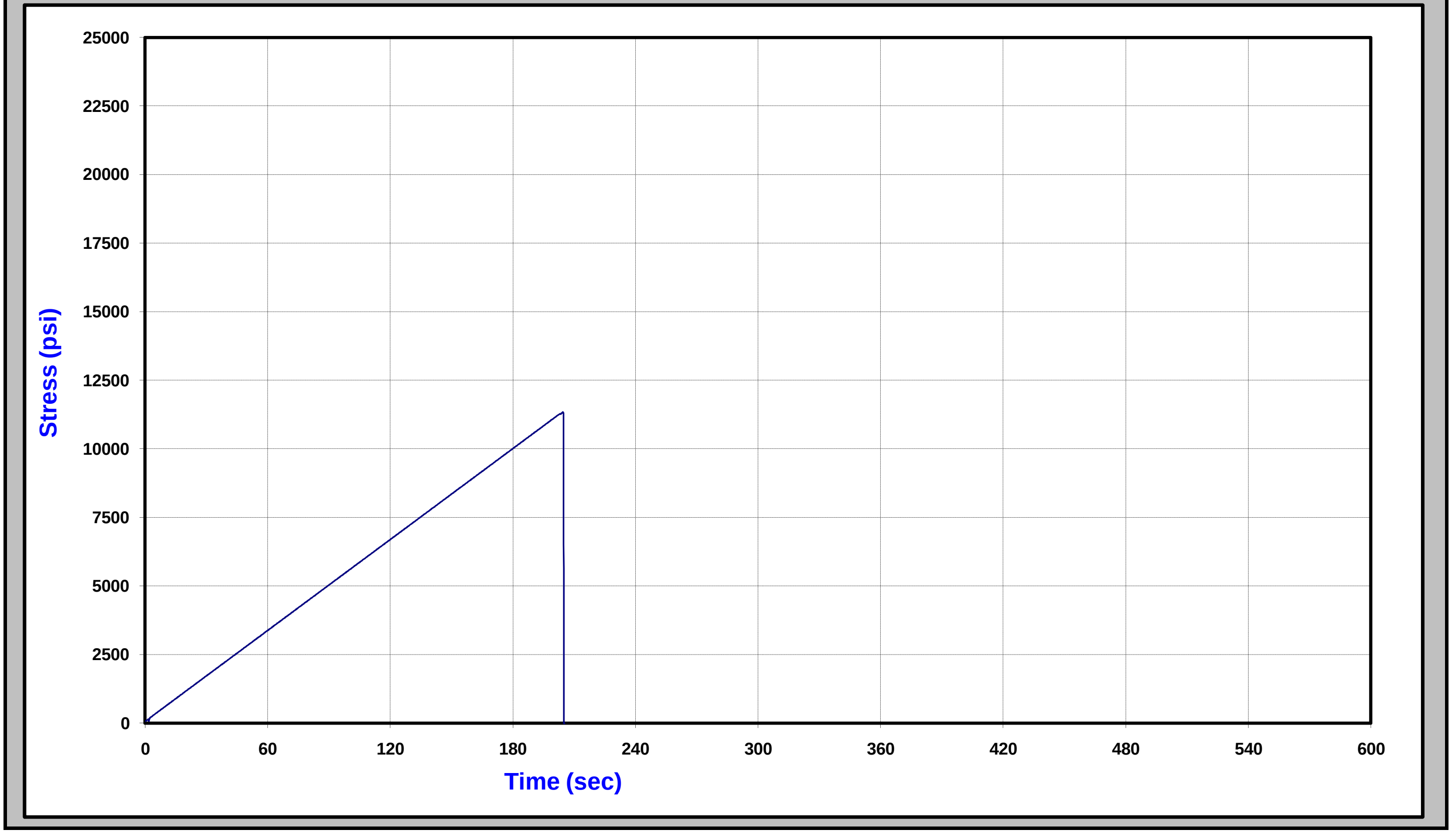


Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-107@95.25-95.7
File Name: TB-107@95.25-95.7_UCS
Test Performed by: Jake
Date Tested: 7/1/2008
Data Reduced by: BM
Date Reduced: 7/2/2008

Core Length		Diameter		L/D	Failure Load	Failure Stress	UCS 2:1		Failure Mode
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	
4.111	10.44	1.960	4.98	2.10	34,209	11,338	11,501	79	Non-Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	160	2.56



Earth Mechanics Institute

Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-107@120.15-120.7
File Name: TB-107@120.15-120.7_UCS
Test Performed by: Jake
Date Tested: 7/1/2008
Data Reduced by: BM
Date Reduced: 7/2/2008

Core Length		Diameter		L/D	Failure Load	Failure Stress	UCS 2:1		Failure
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	Mode
4.143	10.52	1.980	5.03	2.09	30,237	9,825	9,964	69	Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	156	2.50

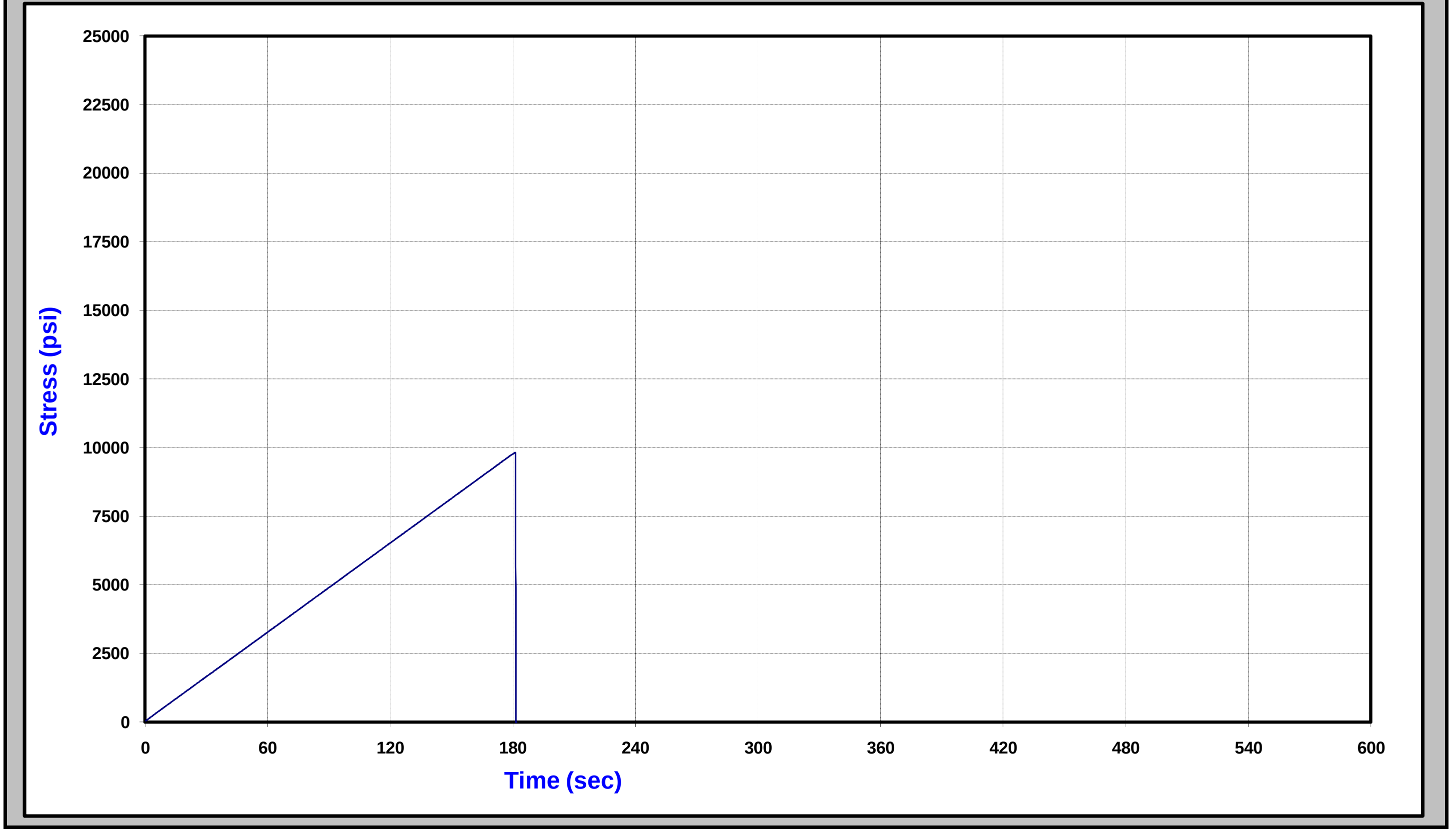


Figure No. 36

Earth Mechanics Institute

Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-108@101.15-101.87
File Name: TB-108@101.15-101.87_UCS
Test Performed by: Jake
Date Tested: 7/1/2008
Data Reduced by: BM
Date Reduced: 7/2/2008

PROJECT: NTH
DATE: 7/1/08
STATION: TB-108
CORE ID: @101.15-101.87

Core Length		Diameter		L/D	Failure Load	Failure Stress	UCS 2:1		Failure Mode
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	
4.150	10.54	2.003	5.09	2.07	48,655	15,441	15,642	108	Non-Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	162	2.59

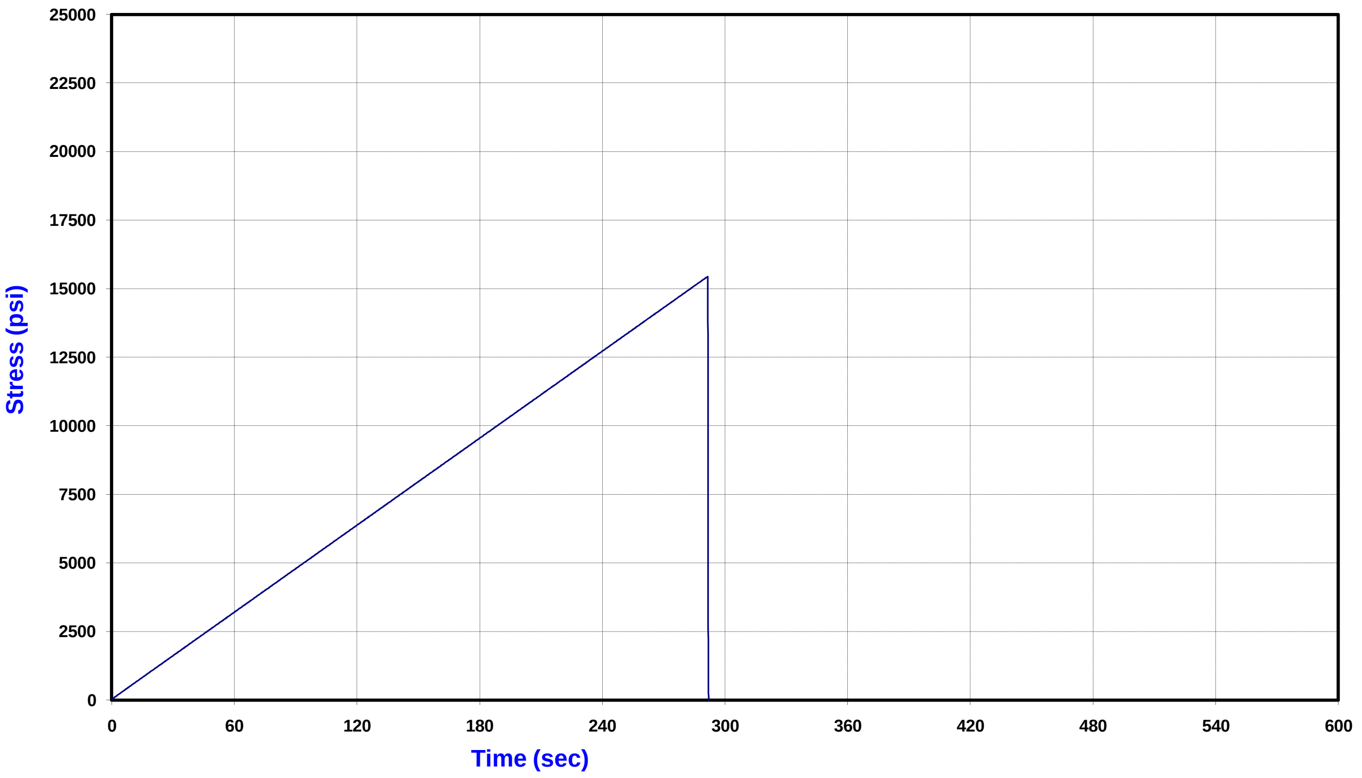


Figure No. 36

Earth Mechanics Institute

Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-108@135.95-136.4
File Name: TB-108@135.95-136.4_UCS
Test Performed by: Jake
Date Tested: 7/1/2008
Data Reduced by: BM
Date Reduced: 7/2/2008

Core Length		Diameter		L/D	Failure Load	Failure Stress	UCS 2:1		Failure
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	Mode
4.202	10.67	1.987	5.05	2.12	38,459	12,409	12,598	87	Non-Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	162	2.59

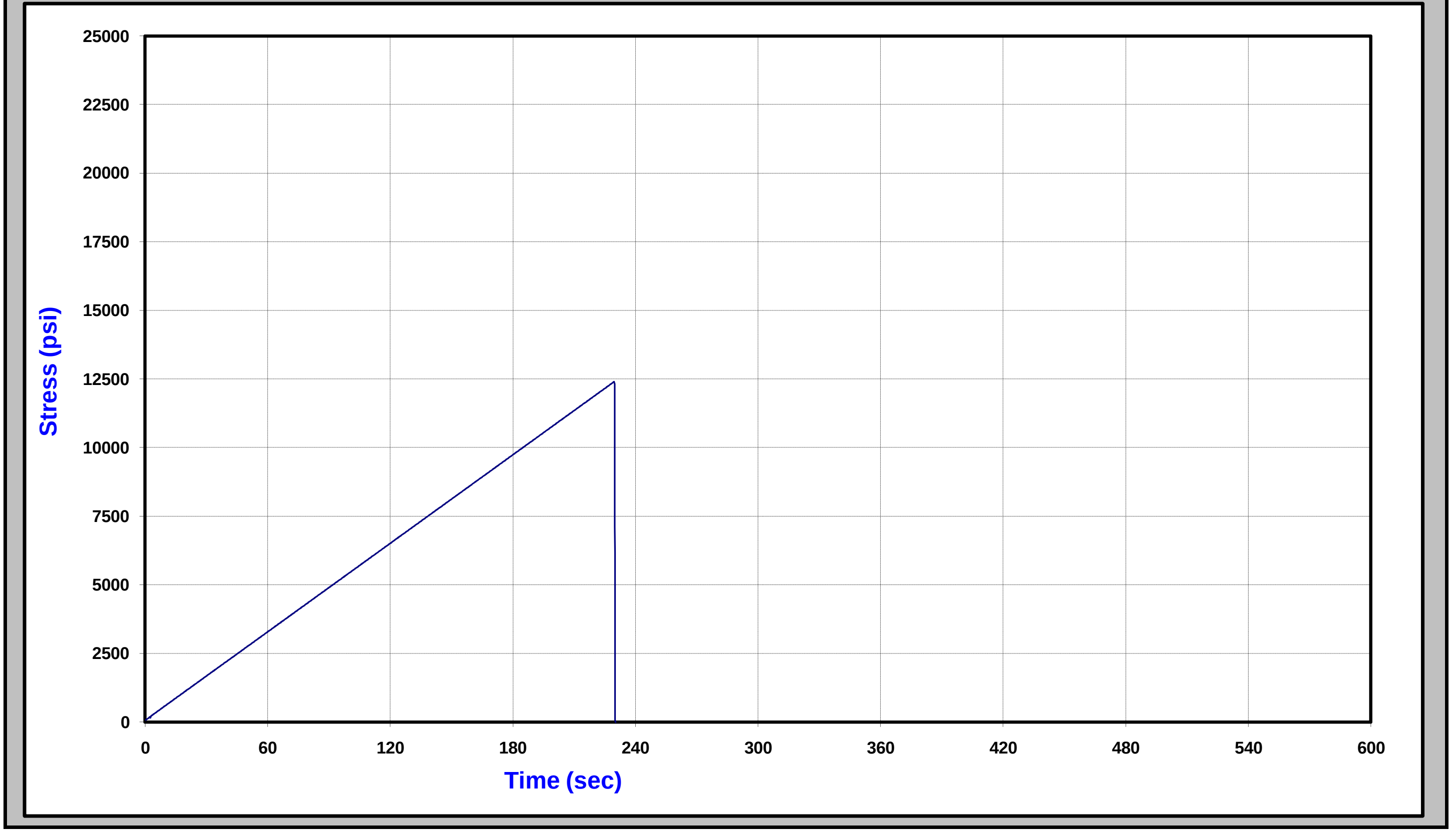


Figure No. 36

Earth Mechanics Institute

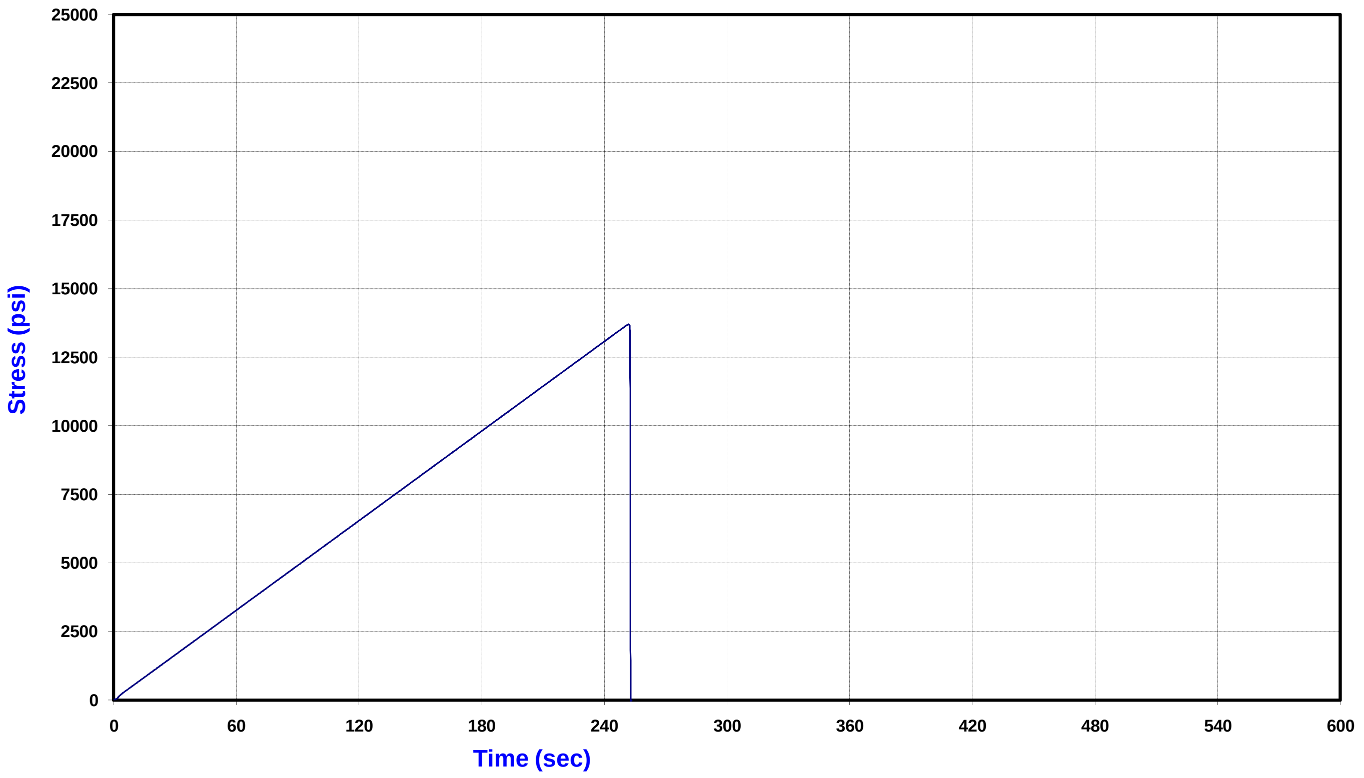
Mining Engineering Department, CSM

Tri-Axial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-101@105.83-106.4_T400
File Name: TB-101@105.83-106.4_T400
Test Performed by: Jake
Date Tested: 8/5/2008
Data Reduced by: BM
Date Reduced: 8/5/2008
Confining Pressure 400 psi

Core Length		Diameter		L/D	Failure Load	Failure Stress	Tri-Ax 2:1		Failure
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	Mode
4.215	10.71	1.975	5.02	2.13	41,946	13,699	13,922	96	Non-Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	*v*	ksi	GPa	*v*	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	159	2.55



Earth Mechanics Institute

Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results (V 3.0)

Project: NTH
Location: n/a
Rock Type: Sedimentary
Rock Name: n/a
Characteristics:
Core ID: TB-102@101.6-107.25_T200
File Name: TB-102@101.6-107.25_T200
Test Performed by: Jake
Date Tested: 8/5/2008
Data Reduced by: BM
Date Reduced: 8/5/2008
Confining Pressure 200 psi

Core Length		Diameter		L/D	Failure Load	Failure Stress	Tri-Ax 2:1		Failure
in	cm	in	cm	Ratio	lbs	psi	psi	MPa	Mode
4.103	10.42	1.949	4.95	2.11	40,061	13,428	13,626	94	Structural

P-wave		S-wave		Dynamic E		Dynamic	Static E		Static	Density, ρ	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa	v	ksi	GPa	v	lb/ft3	g/cm^3
NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	165	2.64

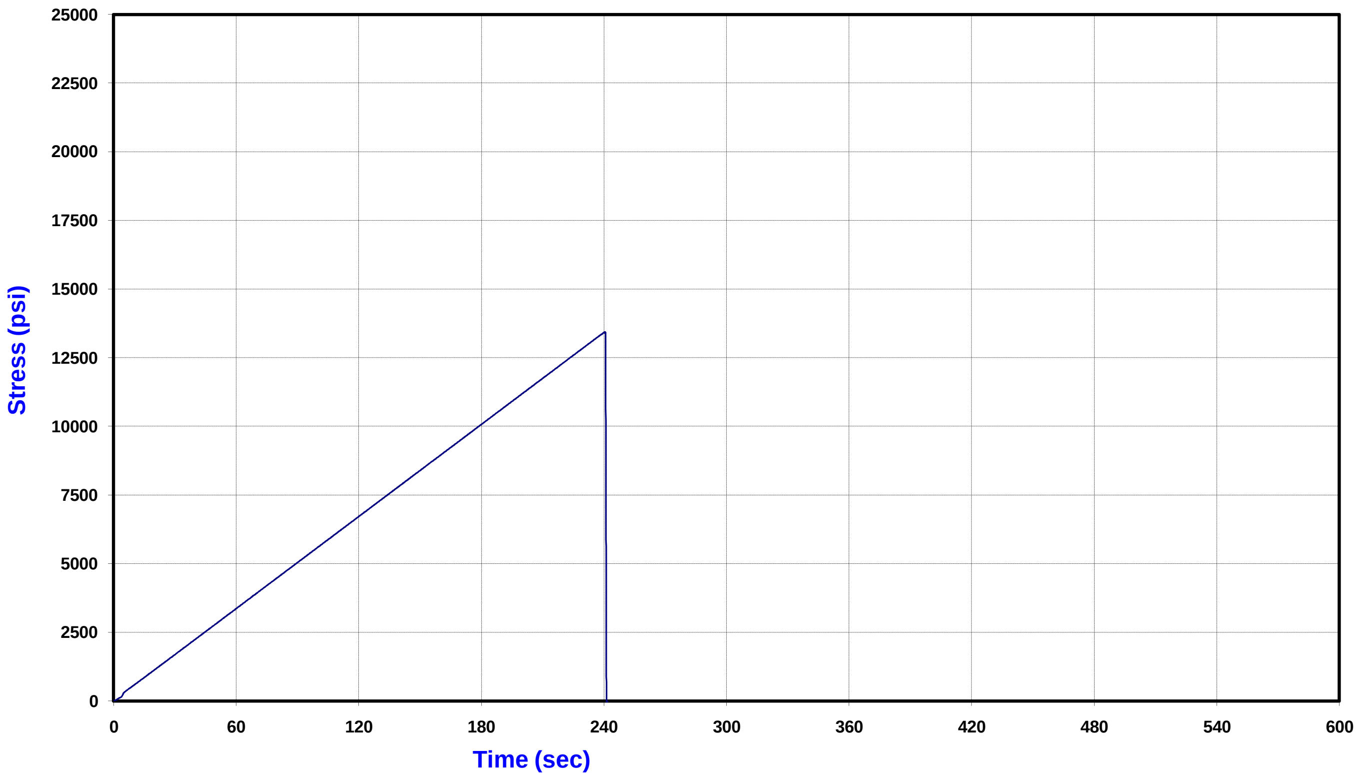


Figure No. 36

Earth Mechanics Institute
Mining Engineering Department, CSM
BTS Reduction Program (Ver. 2.0)

Project Name : NTH
Location : n/a
Rock Type : Sedimentary
Rock Name : n/a
Characteristics :
Test Performed by : BM
Date Tested : 6/30/2008
Data Reduced by : BM
Date Reduced : 7/1/2008
Rock Source : Drill core
Core ID : TB-101@105.83-106.4
File Name : TB-101@105.83-106.4_BTS

Disc Thickness		Disc Diameter		L/D	Failure Load		BTS		Failure
in	cm	in	cm	Ratio	lbs	N	psi	MPa	Mode
1.39	3.53	1.975	5.02	0.70	2,886	12,836	669	4.6	Non-Structural

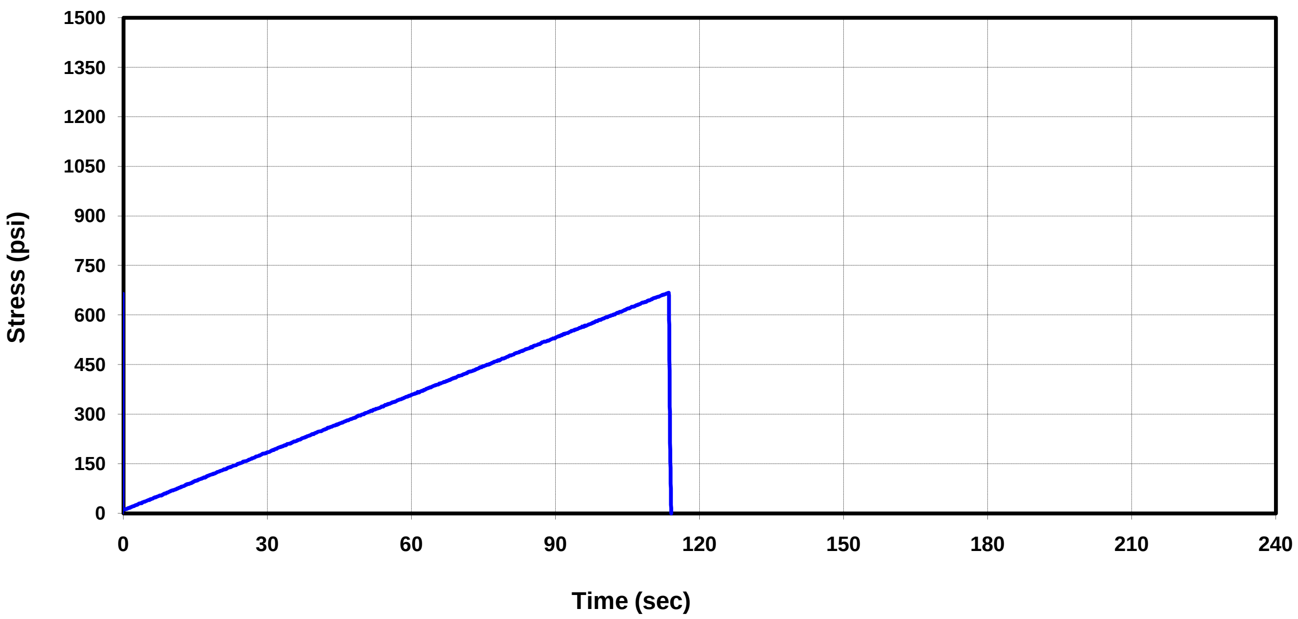


Figure No. 36

Earth Mechanics Institute
Mining Engineering Department, CSM
BTS Reduction Program (Ver. 2.0)

Project Name : NTH
Location : n/a
Rock Type : Sedimentary
Rock Name : n/a
Characteristics :
Test Performed by : BM
Date Tested : 7/1/2008
Data Reduced by : BM
Date Reduced : 7/1/2008
Rock Source : Drill core
Core ID : TB-101@138.85-139.5
File Name : TB-101@138.85-139.5_BTS

Disc Thickness		Disc Diameter		L/D	Failure Load		BTS		Failure
in	cm	in	cm	Ratio	lbs	N	psi	MPa	Mode
1.42	3.60	1.978	5.02	0.72	2,172	9,662	494	3.4	Non-Structural

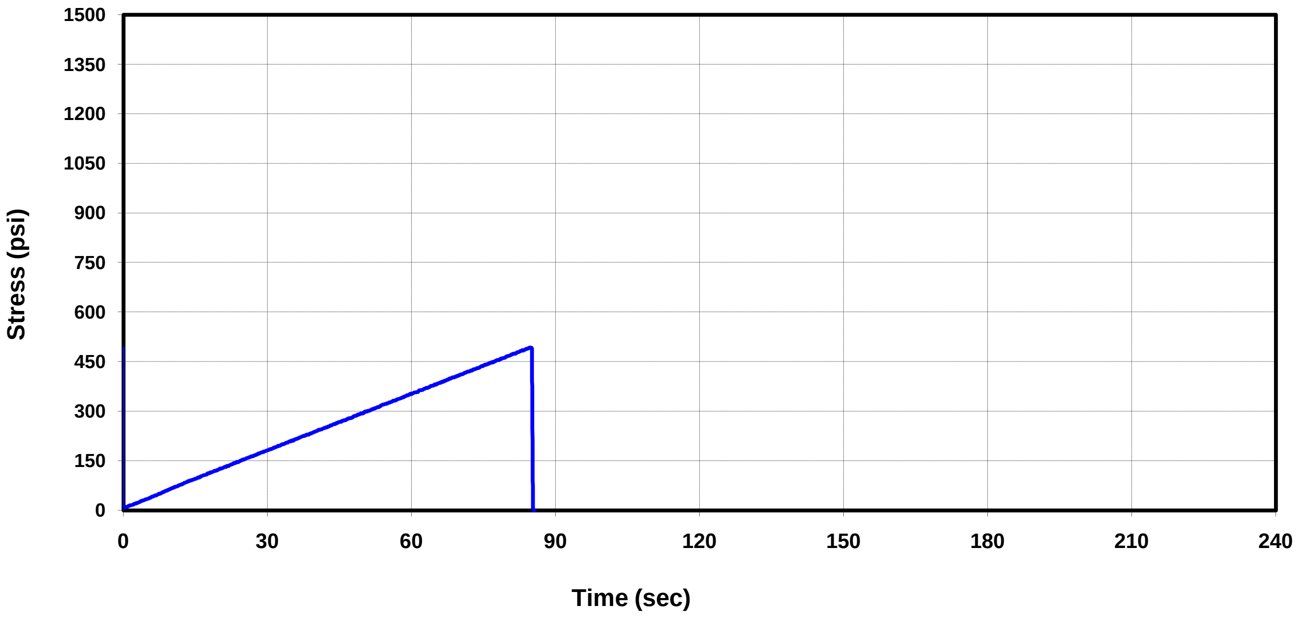


Figure No. 36

Earth Mechanics Institute

Mining Engineering Department, CSM

BTS Reduction Program (Ver. 2.0)

Project Name : NTH
Location : n/a
Rock Type : Sedimentary
Rock Name : n/a
Characteristics :
Test Performed by : BM
Date Tested : 7/1/2008
Data Reduced by : BM
Date Reduced : 7/1/2008
Rock Source : Drill core
Core ID : TB-102@101.6-107.25
File Name : TB-102@101.6-107.25_BTS

PROJECT: NTH
DATE: 7/1/08
STATION: TB-102
CORE ID: @101.6-107.25

Disc Thickness		Disc Diameter		L/D	Failure Load		BTS		Failure
in	cm	in	cm	Ratio	lbs	N	psi	MPa	Mode
1.47	3.74	1.947	4.95	0.76	3,951	17,575	877	6.0	Non-Structural

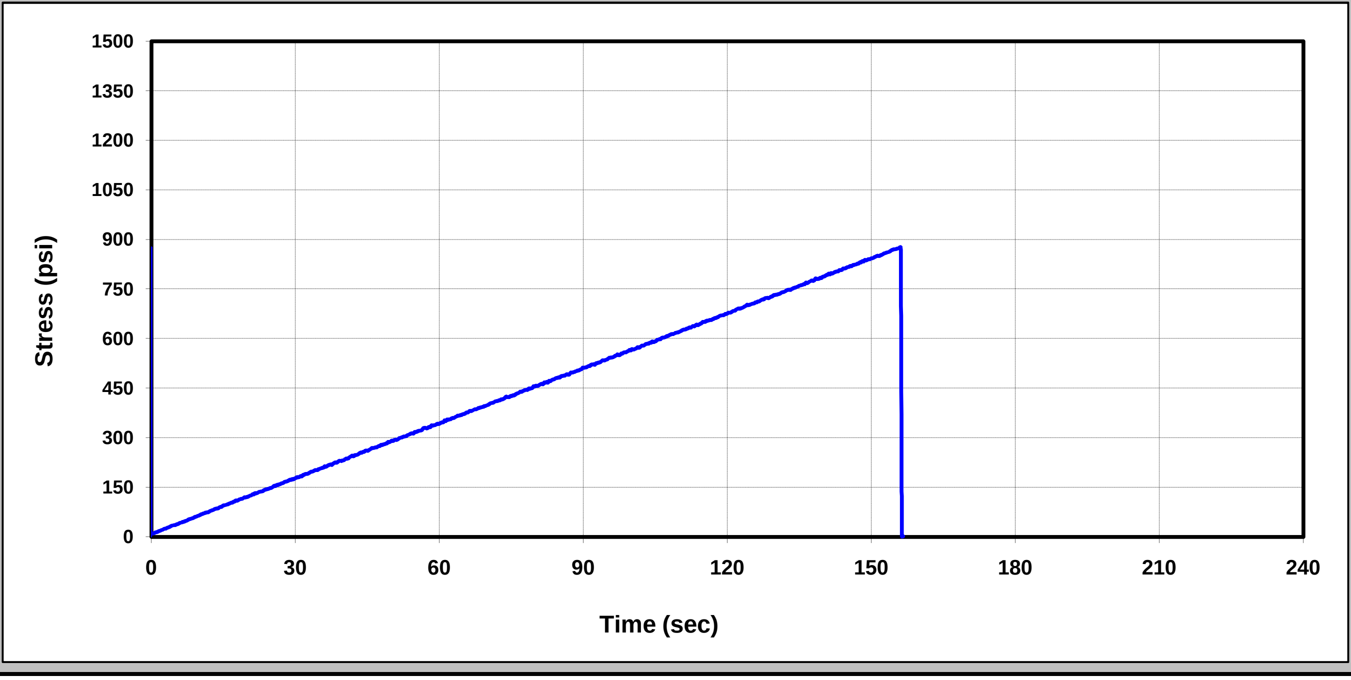
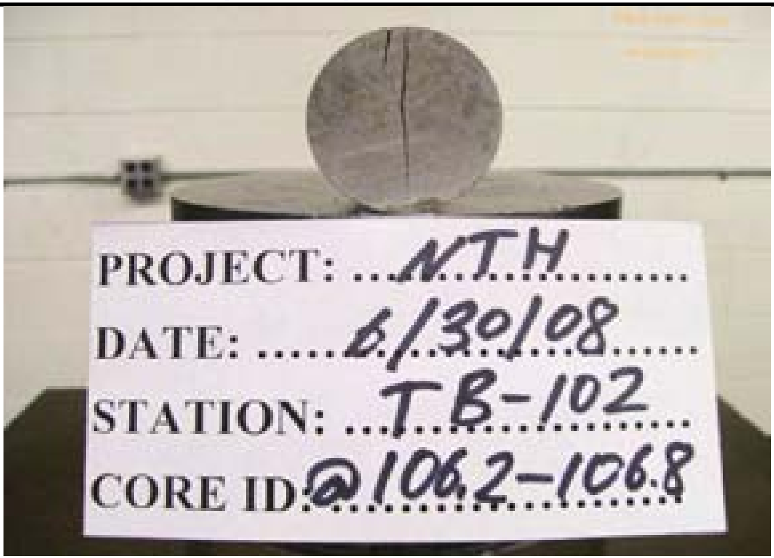


Figure No. 36

Earth Mechanics Institute
Mining Engineering Department, CSM
BTS Reduction Program (Ver. 2.0)

Project Name :	NTH
Location :	n/a
Rock Type :	Sedimentary
Rock Name :	n/a
Characteristics :	
Test Performed by :	BM
Date Tested :	6/30/2008
Data Reduced by :	BM
Date Reduced :	7/1/2008
Rock Source :	Drill core
Core ID :	TB-102@106.2-106.8
File Name :	TB-102@106.2-106.8_BTS


PROJECT: NTH
DATE: 6/30/08
STATION: TB-102
CORE ID: @106.2-106.8

Disc Thickness		Disc Diameter		L/D	Failure Load		BTS		Failure
in	cm	in	cm	Ratio	lbs	N	psi	MPa	Mode
1.39	3.52	1.953	4.96	0.71	3,783	16,825	890	6.1	Non-Structural

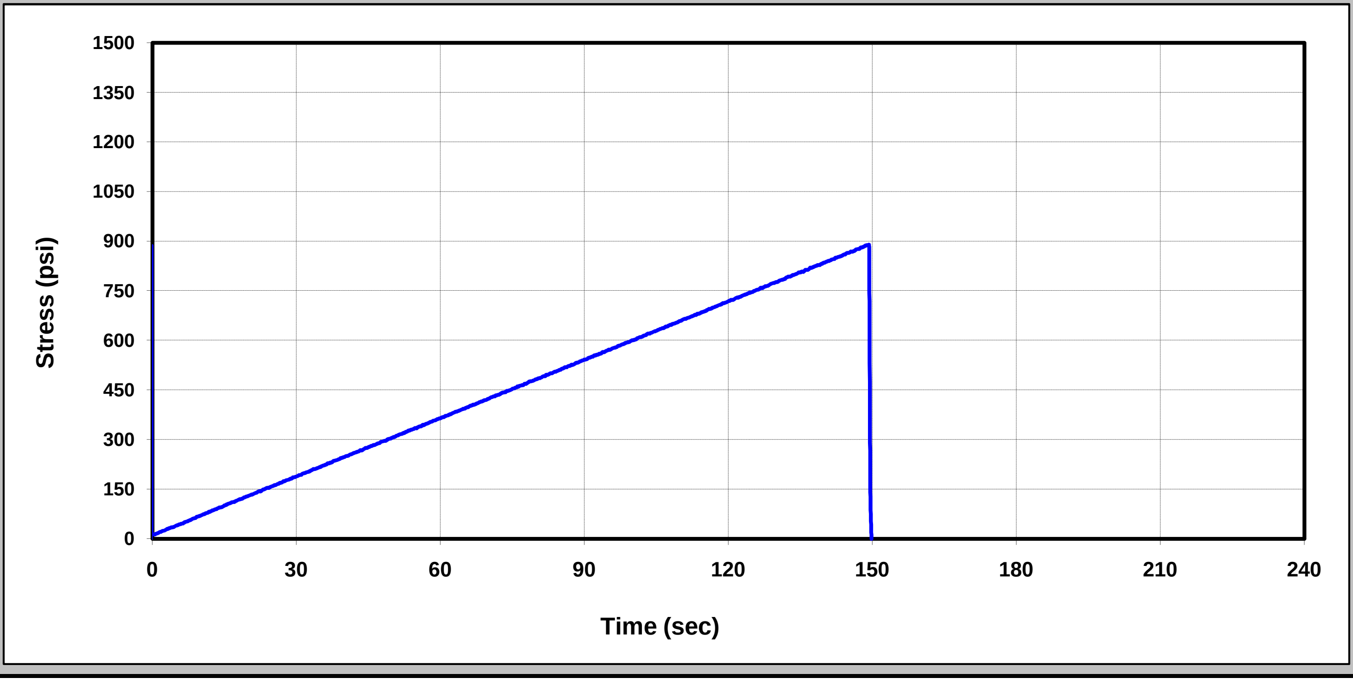


Figure No. 36

Earth Mechanics Institute

Mining Engineering Department, CSM

BTS Reduction Program (Ver. 2.0)

Project Name : NTH
Location : n/a
Rock Type : Sedimentary
Rock Name : n/a
Characteristics :
Test Performed by : BM
Date Tested : 7/1/2008
Data Reduced by : BM
Date Reduced : 7/1/2008
Rock Source : Drill core
Core ID : TB-105@102.8-103.4
File Name : TB-105@102.8-103.4_BTS

PROJECT: NTH
DATE: 7/ 1 /08
STATION: TB-105
CORE ID: @102.8-103.4

Disc Thickness		Disc Diameter		L/D	Failure Load		BTS		Failure
in	cm	in	cm	Ratio	lbs	N	psi	MPa	Mode
1.32	3.35	1.961	4.98	0.67	3,055	13,587	751	5.2	Non-Structural

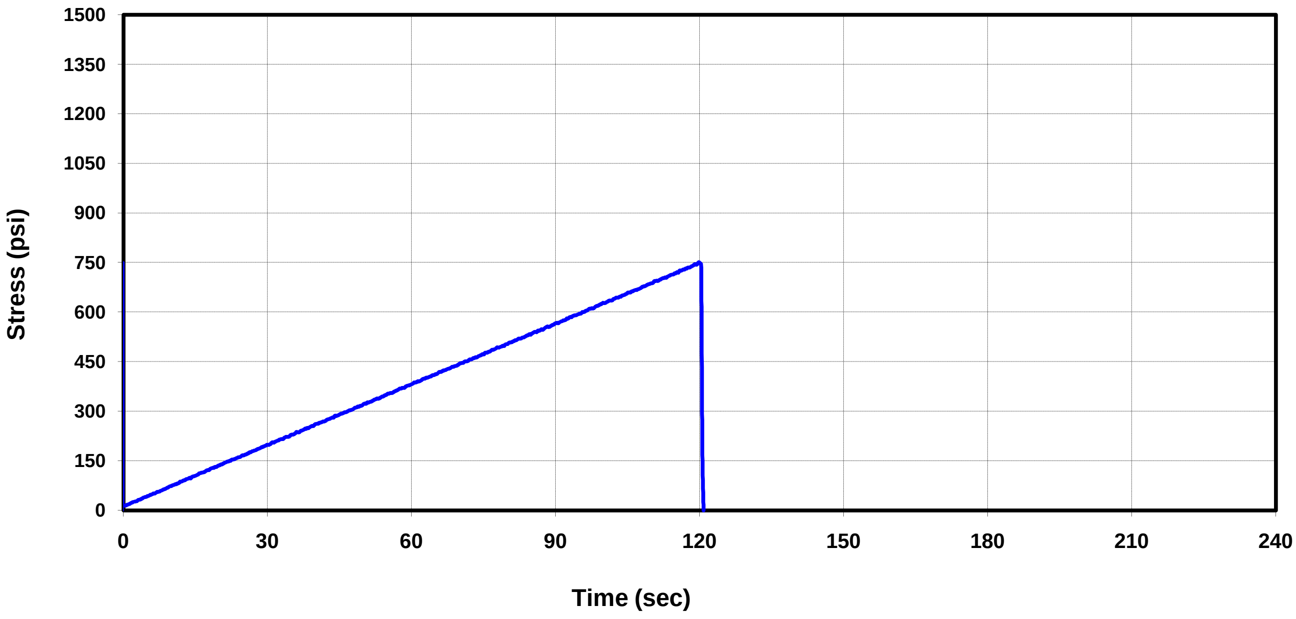


Figure No. 36

Earth Mechanics Institute
Mining Engineering Department, CSM
BTS Reduction Program (Ver. 2.0)

Project Name :	NTH
Location :	n/a
Rock Type :	Sedimentary
Rock Name :	n/a
Characteristics :	
Test Performed by :	BM
Date Tested :	6/30/2008
Data Reduced by :	BM
Date Reduced :	7/1/2008
Rock Source :	Drill core
Core ID :	TB-106@101.5-102.05
File Name :	TB-106@101.5-102.05_BTS

Disc Thickness		Disc Diameter		L/D	Failure Load		BTS		Failure
in	cm	in	cm	Ratio	lbs	N	psi	MPa	Mode
1.45	3.68	1.968	5.00	0.74	4,367	19,424	975	6.7	Non-Structural

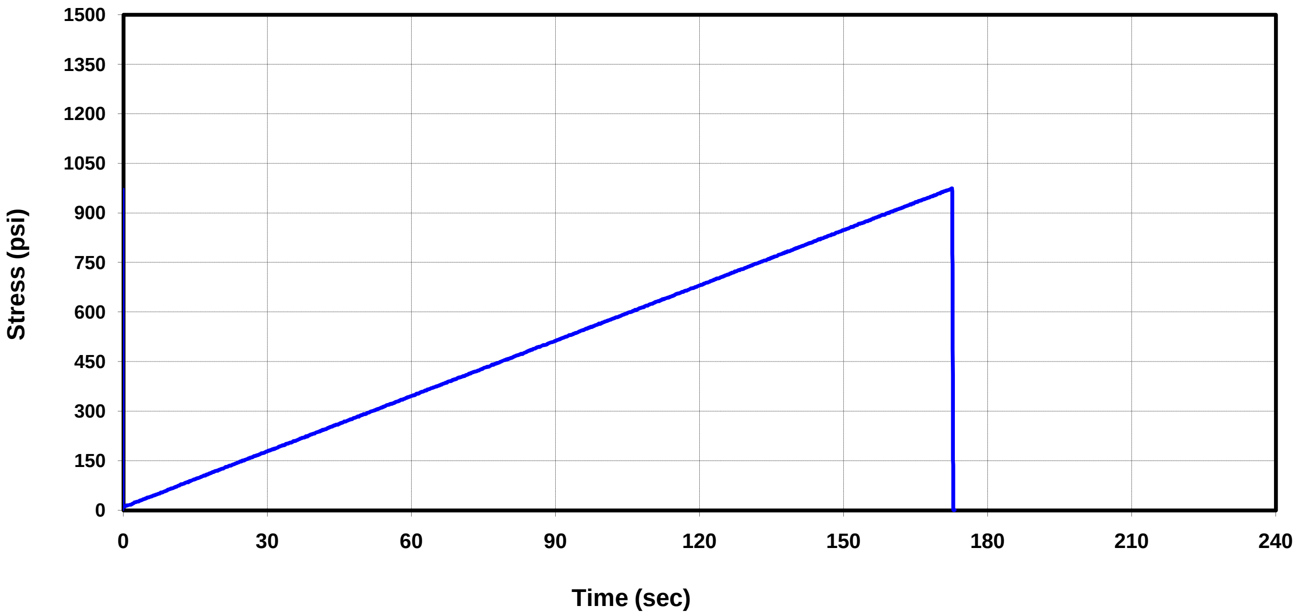


Figure No. 36

Detroit River International Crossing - Cable-Stayed Bridge Option

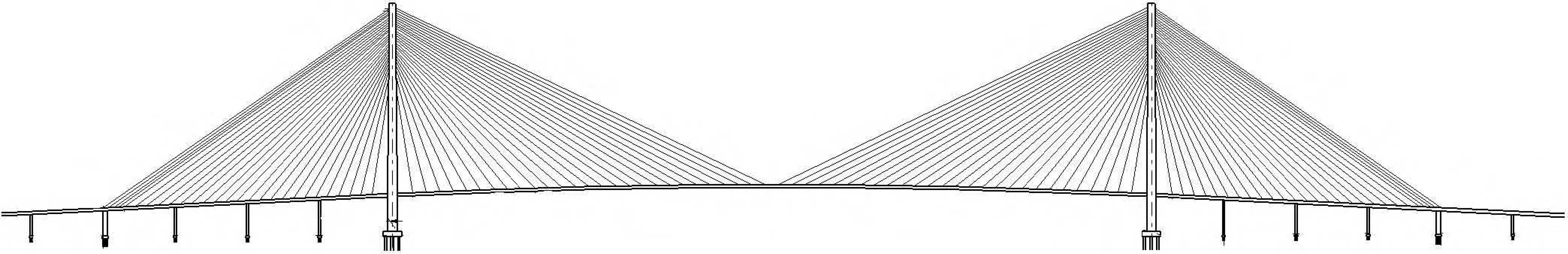


Figure Modified from Detroit River International Crossing, Bridge Conceptual Engineering Report, November 2007, by Parsons

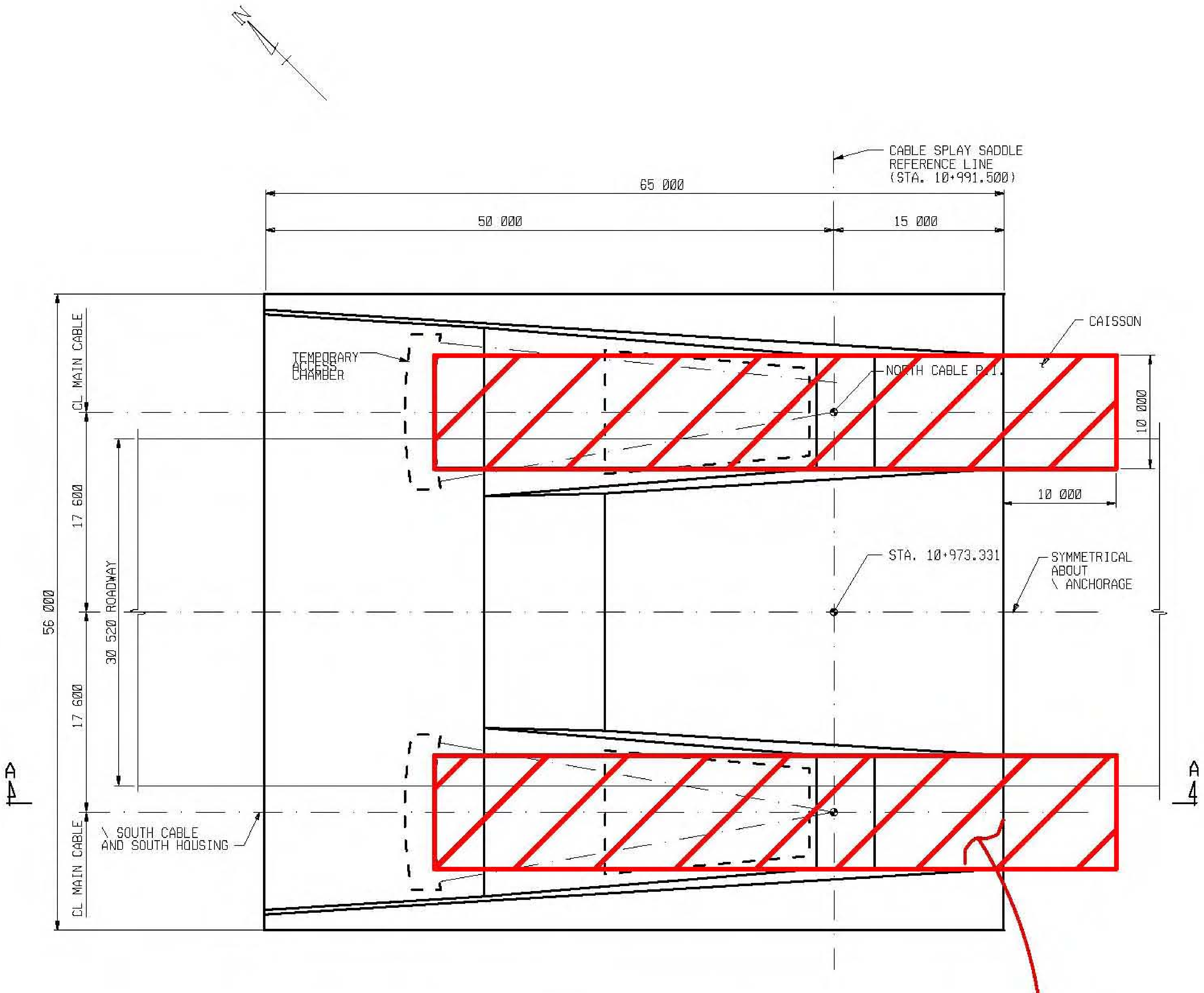
Figure No. 37

Detroit River International Crossing - Suspension Bridge Option

Figure Modified from Detroit River International Crossing, Bridge
Conceptual Engineering Report, November 2007, by Parsons

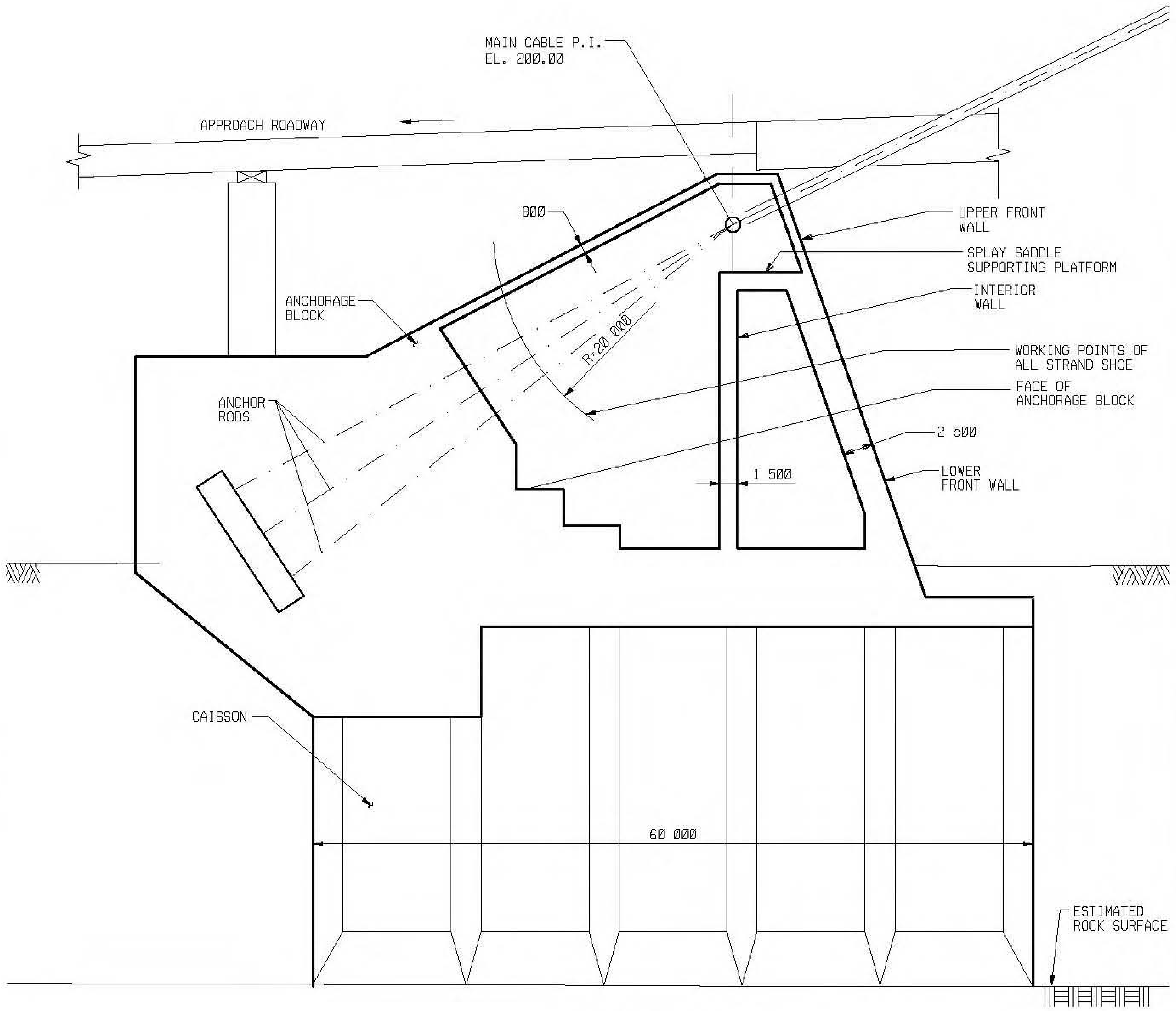
Figure No. 38

REVISIONS			
NO.	DESCRIPTION	DATE	BY



PLAN

(U.S. ANCHORAGE SHOWN
CANADA ANCHORAGE SIMILAR)



SECTION A-A

OPTION 1

TWO RECTANGULAR CAISSONS
10M x 60M

NTH MODIFICATIONS BASED ON ORIGINAL DRAWINGS

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

ANCHORAGE
CROSSING X10 (B) OPTION 7

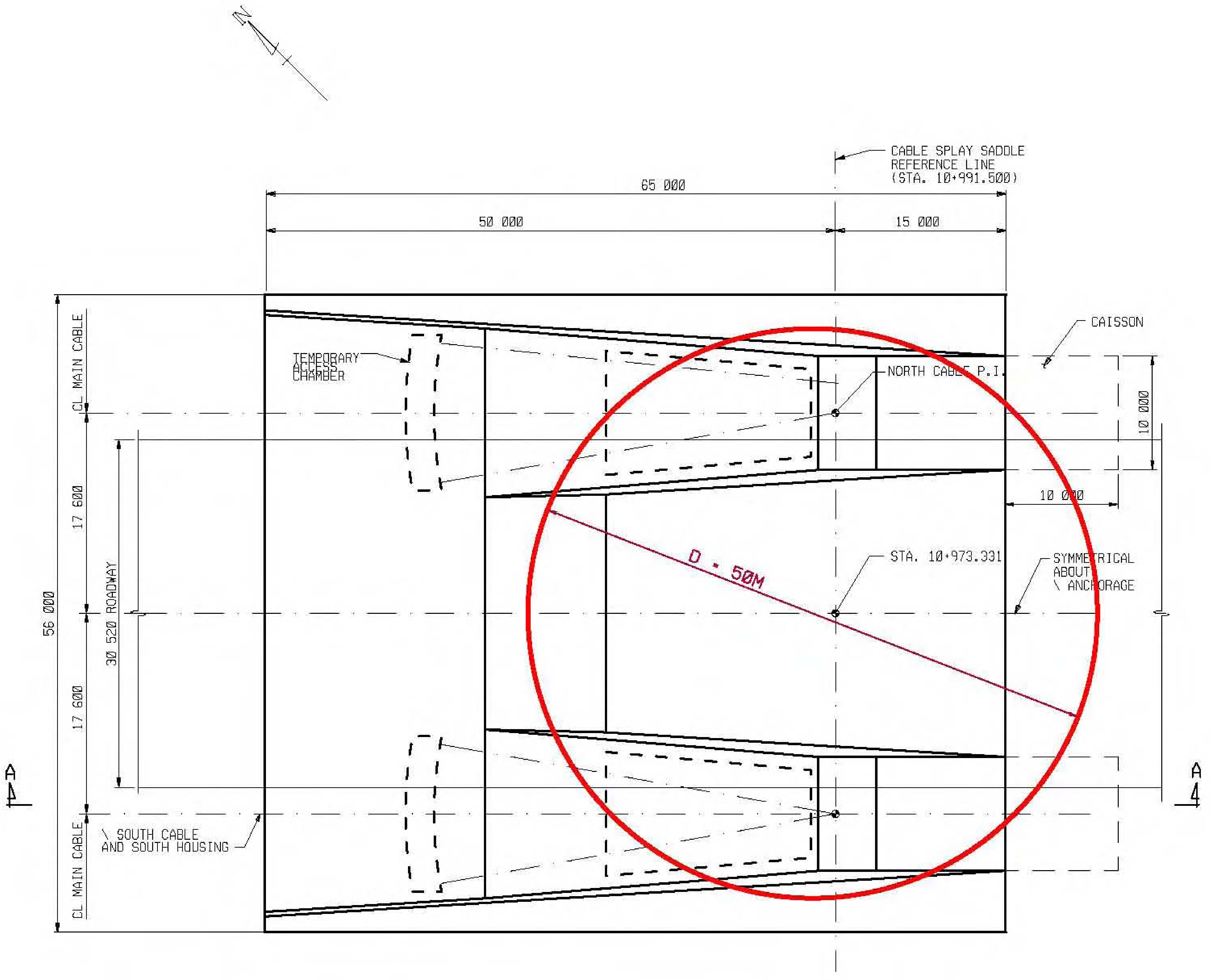
SCALE 1:250
DATE $DATE$

DATE:
CORRECTED BY:
DATE:
CHECKED BY:
DATE:
DRAWN BY:
FILE NAME:

$TIME$
$DATE$
$FILE$

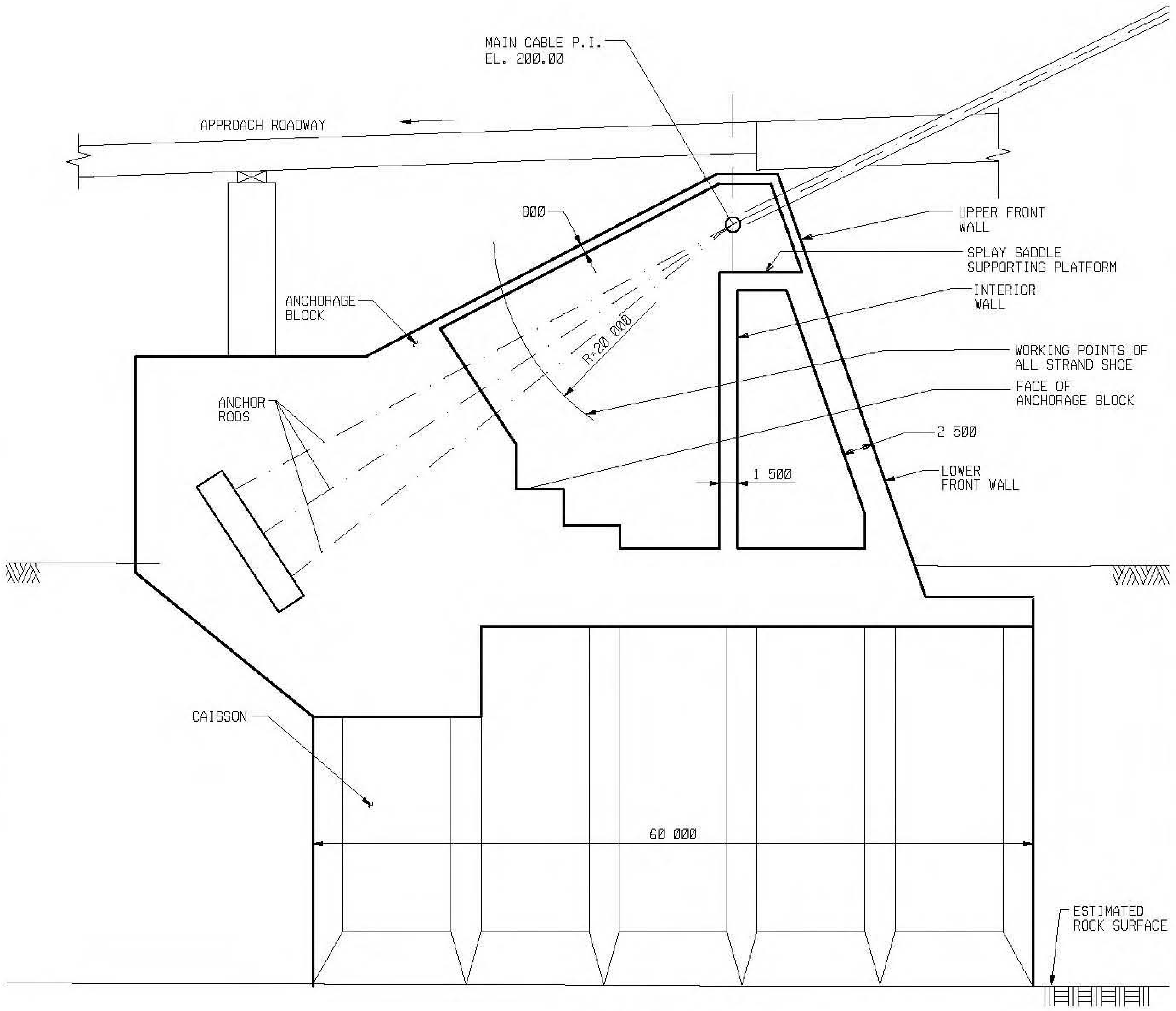
Figure No. 39

REVISIONS			
NO.	DESCRIPTION	DATE	BY



PLAN

(U.S. ANCHORAGE SHOWN CANADA ANCHORAGE SIMILAR)



SECTION A-A

OPTION 2

SINGLE 50M Ø CAISSON

NTH MODIFICATIONS BASED ON ORIGINAL DRAWINGS

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

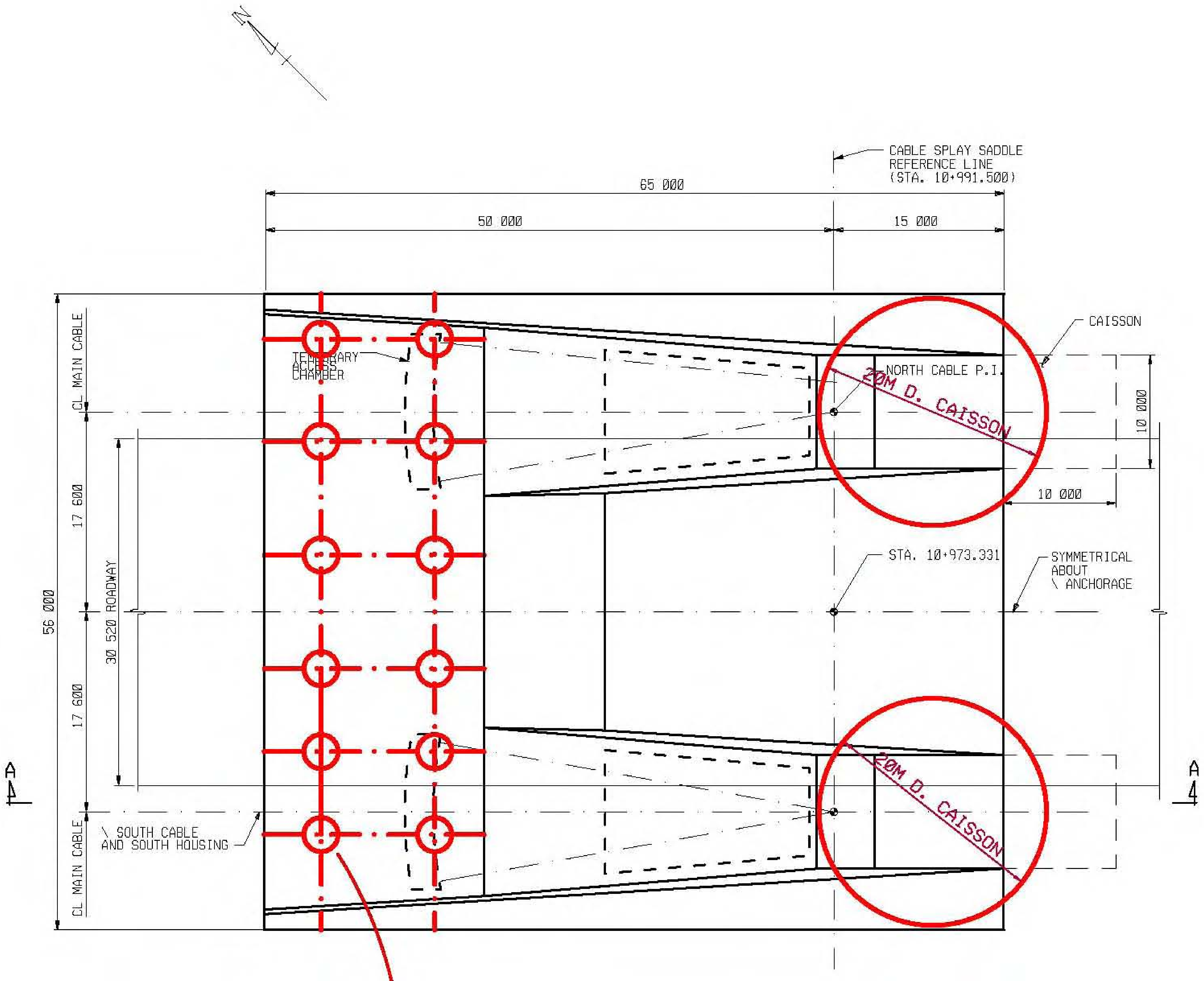
ANCHORAGE
CROSSING X10 (B) OPTION 7

SCALE 1:250
DATE $DATE$

DATE:
CORRECTED BY:
DATE:
CHECKED BY:
DATE:
DRAWN BY:
FILE NAME:

$TIME$ $DATE$ $FILE$

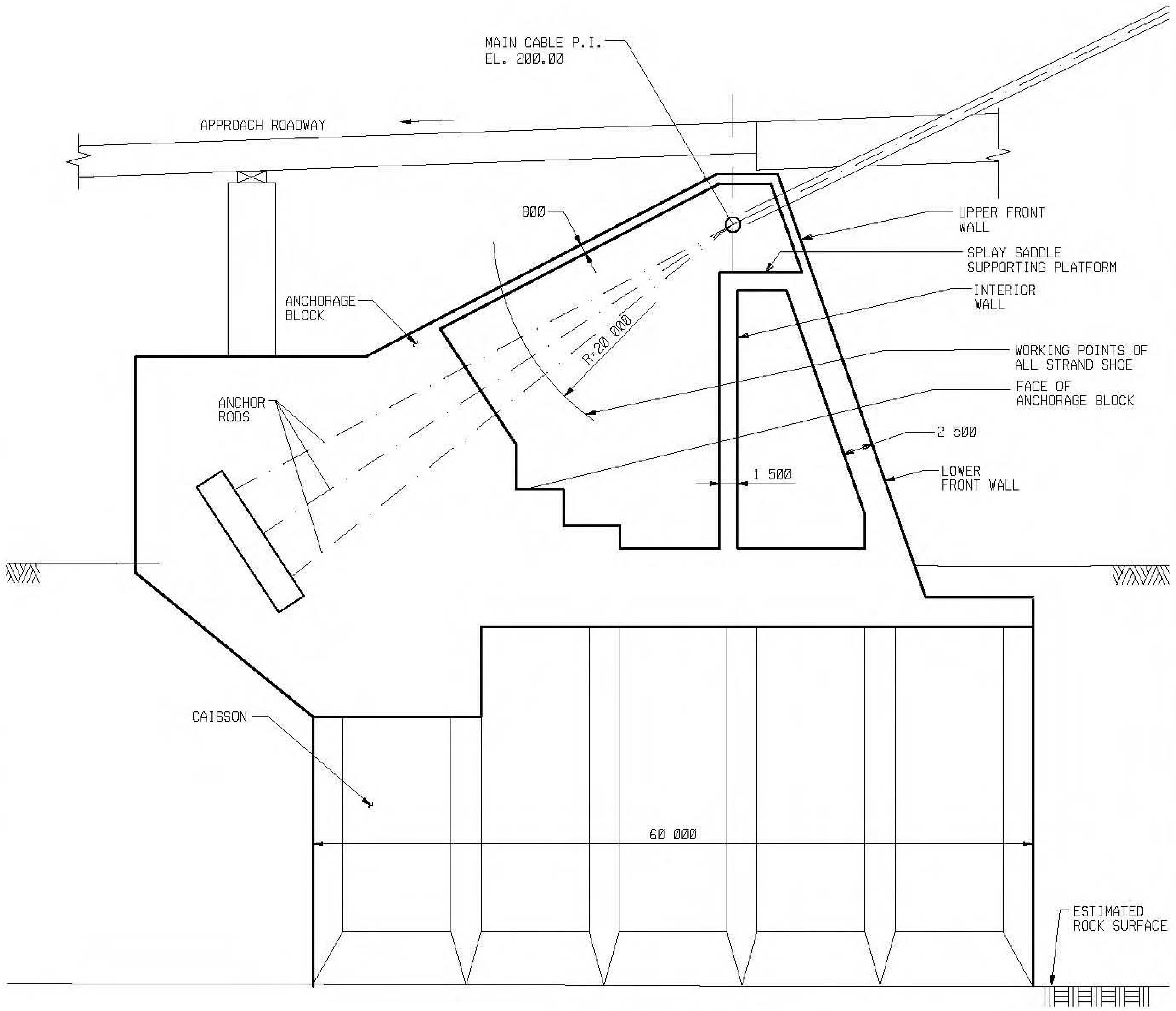
Figure No. 40



PLAN

(U.S. ANCHORAGE SHOWN
CANADA ANCHORAGE SIMILAR)

3M Ø DRILLED SHAFT - 12 TOTAL



SECTION A-A

OPTION 3

NTH MODIFICATIONS BASED ON ORIGINAL DRAWINGS

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario

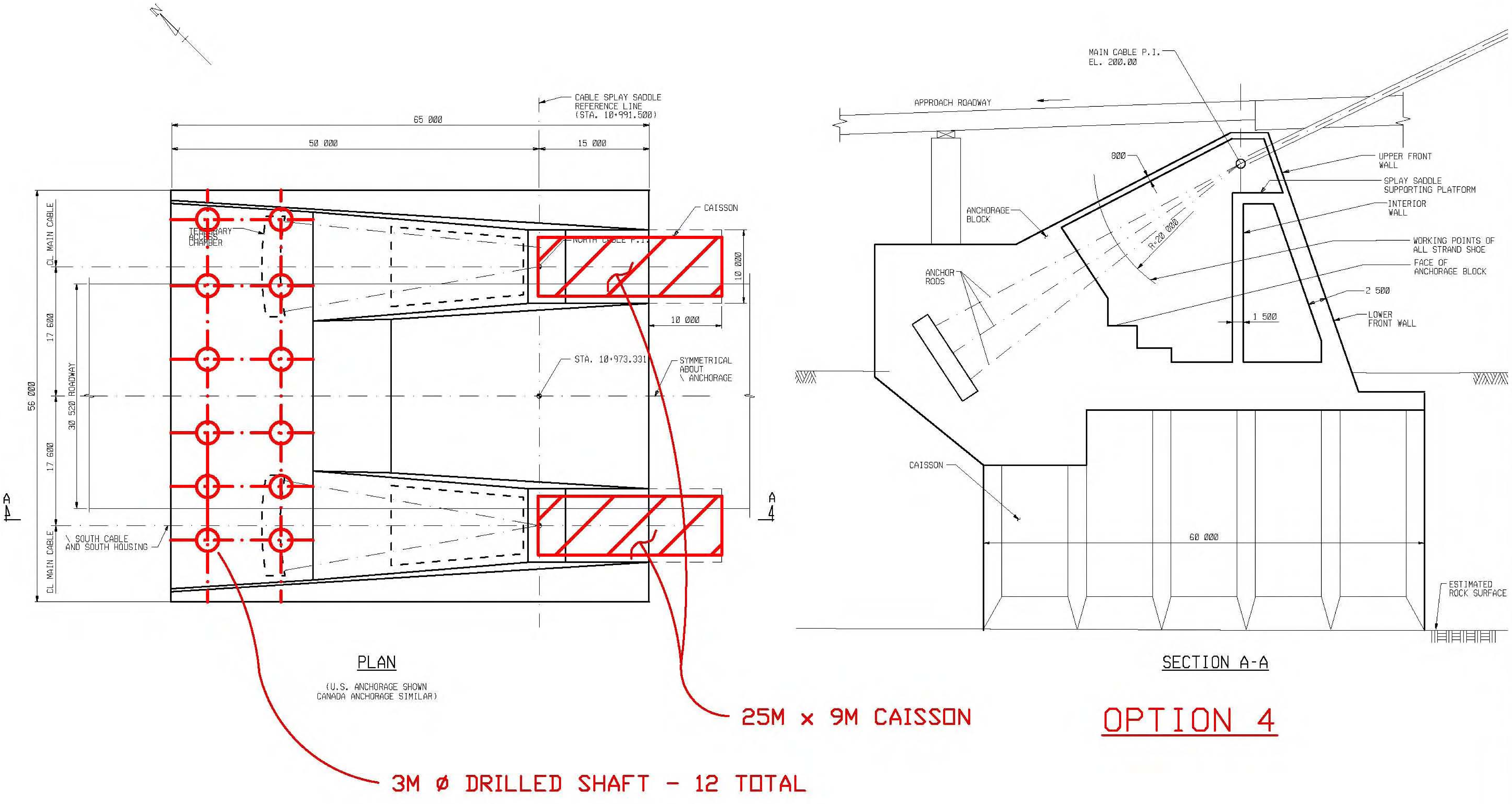
DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

ANCHORAGE
CROSSING X10 (B) OPTION 7

SCALE 1:250
DATE $DATE$

Figure No. 41



NTH MODIFICATIONS BASED ON ORIGINAL DRAWINGS

REVISIONS			
NO.	DESCRIPTION	DATE	BY

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

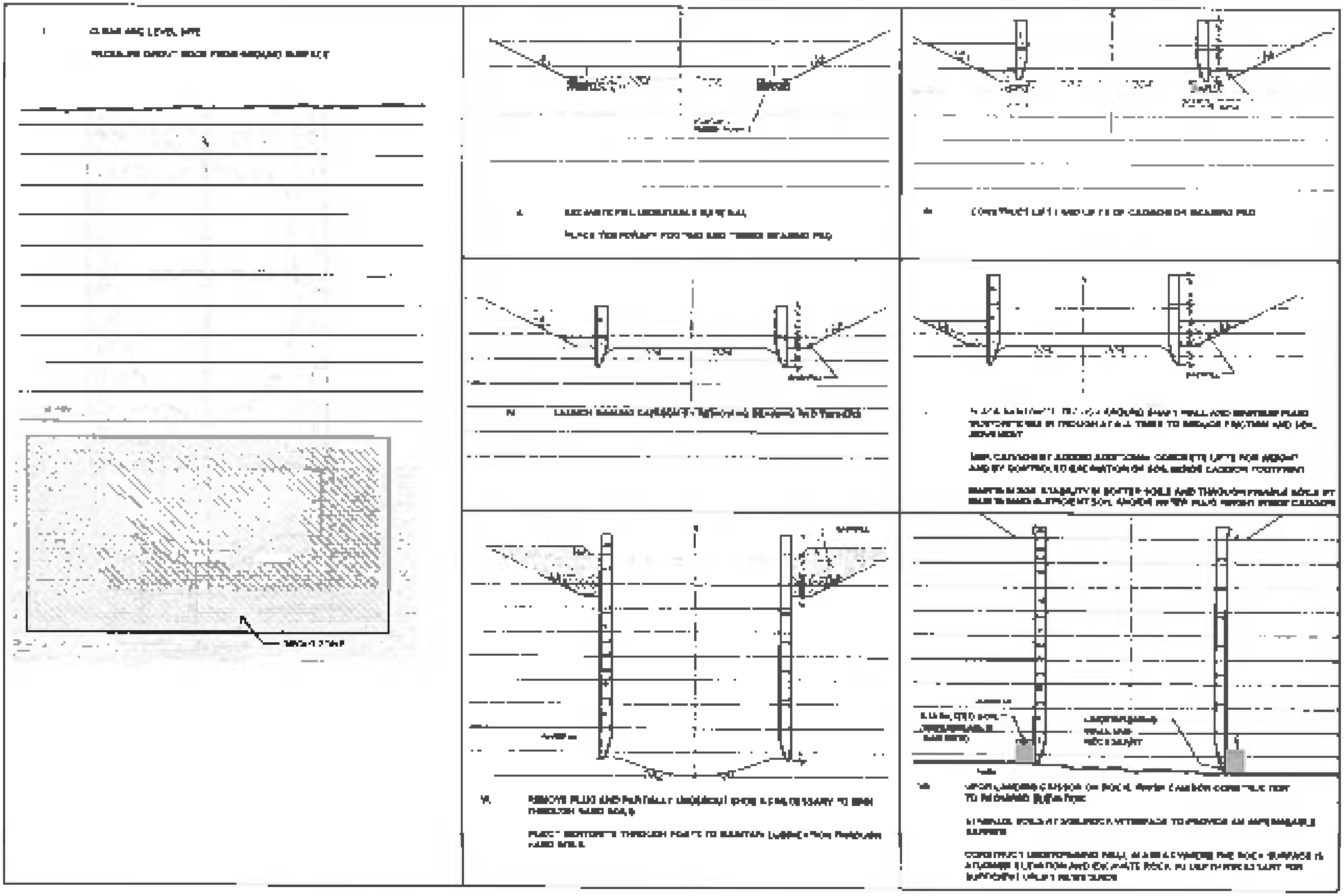
PARSONS

ANCHORAGE
CROSSING X10 (B) OPTION 7

SCALE 1:250
DATE $DATE$

DRAWN BY: DATE:
CHECKED BY: DATE:
CORRECTED BY: DATE:
FILE NAME:
$FILE$ $DATE$ $TIME$

Figure No. 42



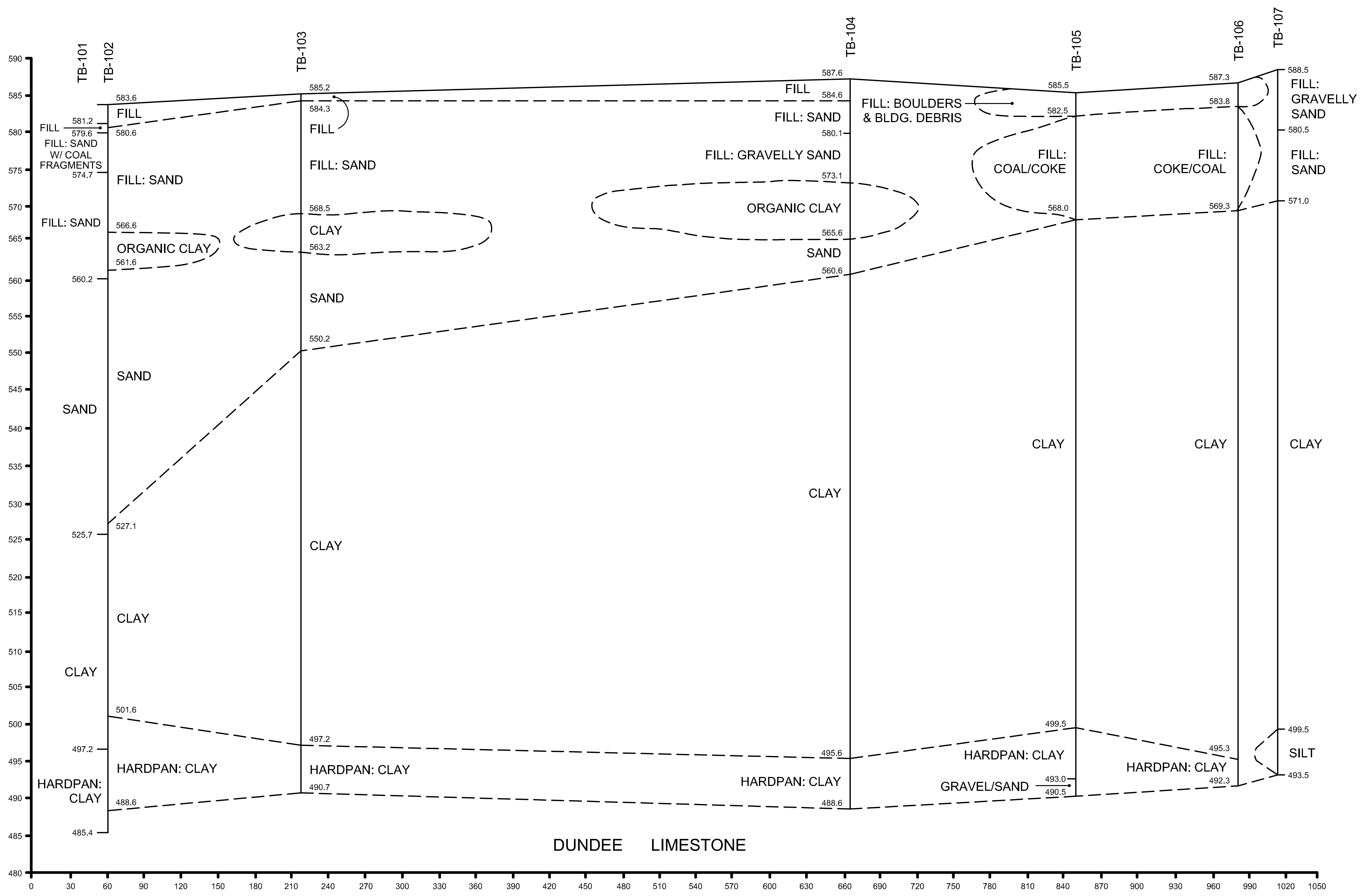
NTH Consultants, Ltd.
Infrastructure Engineering and Environmental Services

13-080714-00
CRJ
SEP 2008
NONE
SEP 2008
CRJ

CONCEPTUAL DIAGRAM OF CAISSON CONSTRUCTION & SINKING

DRIC PRELIMINARY FOUNDATION INVESTIGATION
DETROIT, MICHIGAN

FIGURE No.
43



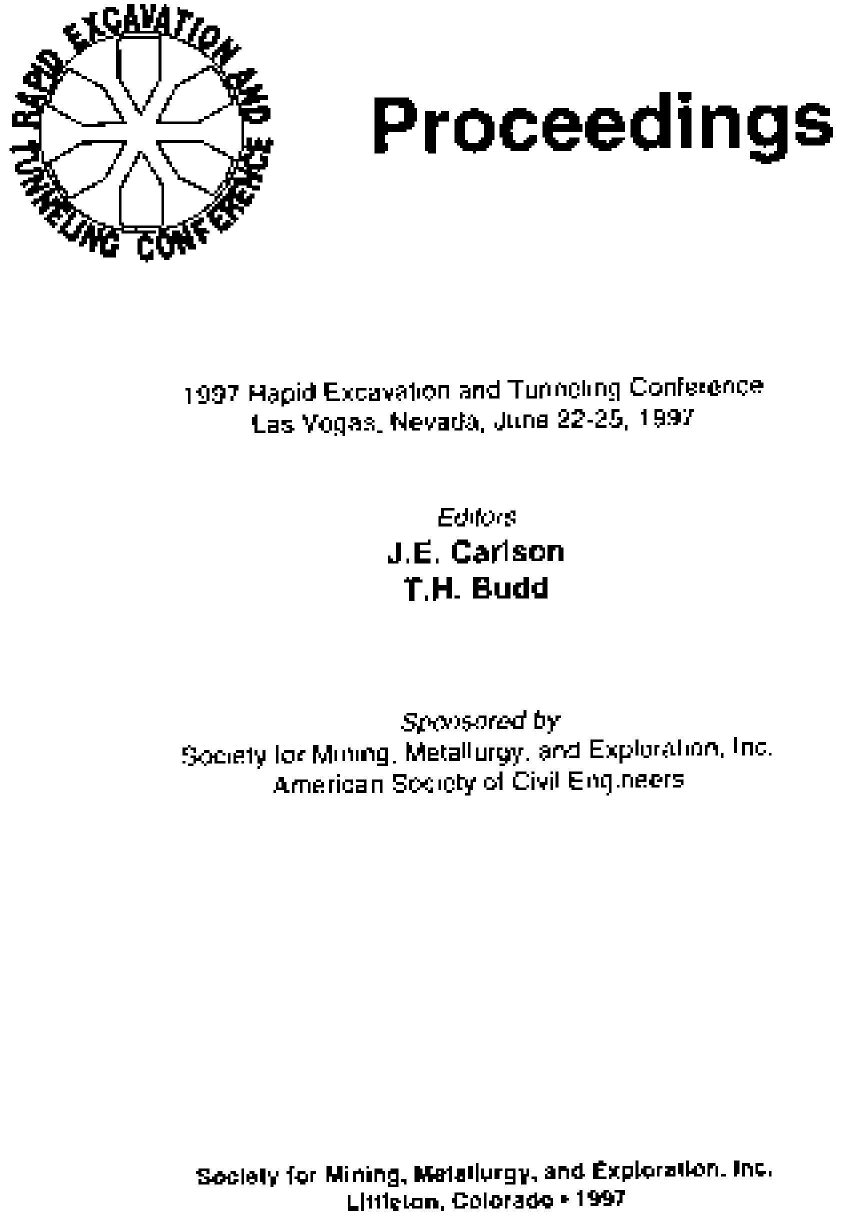
NTH Consultants, Ltd.
Infrastructure Engineering
and Environmental Services

NTH PROJECT No: 15-050014	CAD FILE NAME: 050014001
DESIGNED BY: P.N.S.	PLOT DATE: 9-16-08
DRAWN BY: J.M.R.	DRAWING SCALE: AS SHOWN
CHECKED BY: C.R.J.	INCEPTION DATE: 9-15-08

SITE X-10 FOUNDATION BORINGS
CROSS SECTION

DETROIT INTERNATIONAL CROSSING
DETROIT, MICHIGAN

FIGURE
44



Proceedings

1997 Rapid Excavation and Tunneling Conference
Las Vegas, Nevada, June 22-25, 1997

Editors
J.E. Carlson
T.H. Budd

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American Society of Civil Engineers

Society for Mining, Metallurgy, and Exploration, Inc.
Littleton, Colorado • 1997

Figure No. 45

Chapter 29

SINKING CAISSONS AS AN EFFECTIVE MEANS OF CONSTRUCTING SHAFTS

F.J. Klingler,[1] K.M. Swaffar,[1] and J.C. Neyer,[1] and R. Hausmann[2]
[1]NTH Consultants, Ltd., [2]Walbridge Aldinger Co.

ABSTRACT

This paper presents the results of a study of case histories of projects on which sinking caissons were used to construct shafts for tunnel construction and other underground works. Design methodologies are reviewed with respect to the impact of soil and groundwater conditions on the performance and constructability of sinking caissons. Factors which have led to success or failure are reviewed and evaluated, and recommendations for future investigation, design and construction activities are presented. Cost data for five recent projects are reviewed and compared to the cost of shaft installation by other means.

INTRODUCTION

A sinking caisson consists of a circular, square, or rectangular concrete wall structure that is initially formed at the ground surface and then sunk into place through the underlying soils, using a systematic approach of adding successive lifts of concrete to the top of the structure walls, while removing soil from the interior of the caisson. This process is continued until the required depth of excavation is achieved, after which a structural base slab is typically installed. Once sunk into position, the caisson serves the dual purposes of providing the necessary earth support during construction, as well as providing the walls of the final structure.

The technology of using sinking caissons was originally developed in Europe and brought to North America in the mid 1800's. The technology was first used in North America for the construction of bridge foundations for crossings of the Mississippi River in St. Louis and the East River in New York. These early caissons involved the use of compressed air to mitigate ground water infiltration at the excavation bottom. Excavation proceeded with small construction equipment or by hand, and muck was evacuated from the compressed caisson chamber through pressure relief pipes. The use of more modern construction equipment and materials, as well as better knowledge of soil conditions and their behavior, has enabled the sinking caisson to become an important option for making excavations into and through difficult ground conditions.

The authors combined experience in the last decade includes design and construction of more than two dozen sinking caissons. These have typically included projects where sinking caissons were judged to be the most practical means of construction because of concerns related to either bottom stability or potential lateral ground movements. Ground

Figure No. 45

1997 RETC

SINKING CAISSONS AS AN EFFECTIVE MEANS OF CONSTRUCTING SHAFTS

F. J. Klingler, K. M. Swaffar, and J.C. Neyer, NTH Consultants, Ltd.
and
R. Hausmann, Walbridge Aldinger Co.

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Figure No. 45

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The authors combined experience in the last decade includes design and construction of more than two dozen sinking caissons. These have typically included projects where sinking caissons were judged to be the most practical means of construction because of concerns related to either bottom stability or potential lateral ground movements. Ground conditions through which these caissons have been constructed include both predominantly granular as well as predominantly cohesive deposits. This study examines five recent projects where sinking caissons were used in favor of other systems to construct shafts through soft and very soft clay strata.

DESIGN METHODOLOGY

The phrase "difficult ground conditions" can mean many things to many people. With respect to shaft construction, difficult ground conditions are typically those in which conventional means of earth support such as the use of internally braced soldier piles and lagging or steel sheet piling are not appropriate. Factors which often contribute to conventional systems or approaches not being feasible within predominantly clay soils include: high lateral earth pressures; strict ground movement limitations; instability of the excavation bottom as a result of weak clayey soils; the potential for hydrostatic uplift of the shaft bottom due to underlying artesian pressures; or the potential for ground water inflows from isolated granular layers. Factors which sometimes contribute to conventional systems or approaches not being feasible in predominantly granular soils include strict ground movement limitations and the potential for uncontrollable groundwater inflows into the shaft.

For many of these situations, a number of available construction techniques are suitable and have proven to be effective. These techniques include the use of diaphragm wall and tangent pile support wall systems, in-place soil mixing, or soil freezing. With such earth retention systems, the purpose of the system is only to support the earth and adjacent facilities until the final structure is built inside. The sinking caisson method of construction, on the other hand, will provide temporary earth support during construction, and can also be used as the permanent structure.

Sinking Caissons to Support High Lateral Earth Pressures

Many shaft construction methods exist which are appropriate for supporting high lateral earth loads. However, where lateral loads become very high, internal bracing may become large and difficult to install, and may obstruct the contractor's access to the shaft. In addition, where overload factors exceed eight, clay soils

Figure No. 45

may be subject to squeezing or sudden collapse (Broms and Bennermark, 1967). In these cases, sinking caissons may be an effective alternative means of ground support. The caisson wall is typically sized to insure sufficient weight to sink the structure, which usually results in a wall thickness greater than necessary to support lateral earth loads.

Sinking Caissons to Reduce Ground Movements

Limitations on ground movements may also preclude the use of conventional means of earth support for shafts. Work by D'Appolonia (1971) suggests that for clay with shear strength less than 2000 psf, lateral movement of sheet pile shaft walls can be expected to be about 2 percent of the height of the wall, while lateral movement of rigid wall systems can be expected to be about 0.2 percent of the height of the wall. Data from studies by Peck (1969) suggests that vertical settlement adjacent to shafts may range from one-half to twice the magnitude of lateral wall movement. These correlations compare well with ground movement data observed by the authors for flexible braced shafts and rigid shafts in the Detroit area. As a result of concern regarding ground movements, rigid ground support systems (such as sinking caissons) have been specified for several projects in the Detroit area.

Sinking Caissons to Prevent Bottom Instability

Initial shaft design efforts in clay soils typically include analysis to determine the factor of safety for bottom stability using the given shaft depth and diameter, either by methods after Terzaghi (1943) or Bjerrum and Eide (1956). Current industry practice requires that where the factor of safety against bottom instability is less than 1.3, measures must be taken to resist ground movement below the base of the excavation. Conventional means of earth support such as internally braced steel sheet pile shafts can be designed to extend below the base of excavation, but are generally not capable of providing sufficient resistance to below-invert lateral loads where the factor of safety against bottom instability approaches 1.0. In these cases, use of a rigid earth support system (such as a sinking caisson) extending below the base of the excavation is often necessary to provide the required factor of safety against bottom instability.

Sinking Caissons to Control Hydrostatic Uplift And/or Groundwater Inflow

Sinking caissons may also be suitable for conditions where high hydrostatic pressures exist in formations below the base of the shaft, or where the shaft is to be installed through granular soils below the water table. In each case, the caisson can be sunk while maintaining the water level inside the caisson to balance outside or underlying hydrostatic forces. Soil is excavated from inside the caisson using a clamshell, and the base slab is either placed under water, or placed after lowering the groundwater pressures below the base by means of dewatering wells or other methods.

SINKING CAISSON CASE HISTORIES

Five case histories of recent projects in which sinking caissons were used to construct shafts through difficult ground conditions were reviewed by the authors with respect to: geotechnical and hydrogeologic conditions; unique project requirements; other shaft construction systems considered; why the sinking caisson method of construction was chosen, and the outcome of the construction effort. The following case histories demonstrate the ability to successfully use the sinking caisson method of construction for varied types of ground conditions where conventional earth support systems were not considered to be practical and/or economical.

Detroit Wastewater Treatment Plant Pumping Station 2A Project

The Detroit Water and Sewerage Department's Pumping Station 2A project in southwest Detroit included construction of two sinking caisson shaft structures. The shaft required for the main pumping station was 38.4 meters (126 feet) inside diameter and approximately 21 meters (70 feet) deep, while the nearby connecting shaft was 9.1 meters (30 feet) inside diameter and approximately 15.2 meters (50 feet) deep. The structures were designed by Metcalf and Eddy, Inc. and constructed by Walbridge-Aldinger, Inc. Construction on the project was begun in 1990 and completed in 1994.

The Pump Station 2A project was constructed on the grounds of Detroit's Wastewater Treatment Plant, within close proximity to numerous existing structures and underground facilities. Protection of these facilities, many of which were critical to the operation of the existing plant, was dependent on controlling construction-related ground movements.

Ground conditions at the site consisted of surficial fill and a relatively thin desiccated clay layer, underlain by thick deposits of very soft silty clay with shear strength as low as 17.2 ka (360 psf). The clay deposits were underlain by a relatively thin layer of very dense glacial till (locally termed hardpan), which was further underlain by fractured dolomitic limestone containing artesian groundwater.

Several methods of ground support for construction of the pump station and connecting structure were considered during the design process, including circular freeze walls, diaphragm walls, and sinking caissons. Ground freezing was considered undesirable due to local contractors' unfamiliarity with this method, its expected expense, and questions regarding potential for ground movements and associated utility disruption occurring during thawing. Diaphragm walls were also considered to be undesirable due to lack of local experience. The size and depth of the structures, combined with poor soil conditions, very low ground movement tolerances, and much local experience with the sinking caisson technique, made sinking caissons the method of choice for installing the pump station and nearby junction chamber structures.

Preparations for installation of both caissons included pre-excavating the ground surface approximately 3 meters (10 feet). In addition, a three stage rock grouting program was performed for the pumping station caisson to prevent hydrogen sulfide laden groundwater from entering the excavation as the caisson approached the hardpan and bedrock layers. Due to the presence of artesian water, these procedures would have been required for the pumping station regardless of the system used. At the connecting shaft location, an approximately 3.6 meter (12 foot) clay separation was expected between the caisson bottom and the hardpan aquifer, and as such, artesian water infiltration was not expected and rock grouting was not specified.

The pumping station caisson was constructed with seven foot thick concrete walls which were placed in five lifts (including the caisson shoe). Using mass placement methods, the contractor placed each lift as a single pour, without vertical construction joints. Type II cement was used to combat the potential effects of dissolved sulfides and sulfates in the artesian groundwater. During excavation and sinking, a soil plug ranging in height from 3 to 6.4 meters (10 to 21 feet) above the tip of the shoe was required in order to maintain a stable excavation bottom through the very soft clay soils. Two levels of bentonite slurry injection pipes were cast into the caisson walls to allow for lubrication of the exterior during sinking. However, the contractor was able to control the sinking adequately by discrete excavation of the interior of the caisson, and bentonite injection was not necessary. Prior to placement of the last lift of the caisson walls, steel pipe landing piles were driven around the perimeter of the caisson to ensure that it would be plumb and level when it landed. After landing the caisson with the shoe approximately 0.6 to 1.8 meters (2 to 6 feet) above the sloping rock surface, the caisson shoe was underpinned to the bedrock surface. Because of overload factors as high as 16, the soft clay below the shoe could not be safely exposed. As such, a frozen soil wall was installed along the outside of the shoe to prevent soil squeezing and/or soil flow below the shoe, while the caisson was underpinned by excavating approximately 3 meter (10 foot) long zones of soft clay and hardpan, and filling with concrete. The rock grouting program for the pumping station caisson proved to be a success, with one leaky area of the bedrock producing only about one gallon per minute of seepage. Construction time for the pumping station caisson, from shoe construction to landing the caisson, was approximately 32 weeks. Underpinning the shoe and placement of the base slab took approximately 8 additional weeks. An independent estimate by the contractor indicated that a braced steel sheeting shaft would have taken twice as long to construct.

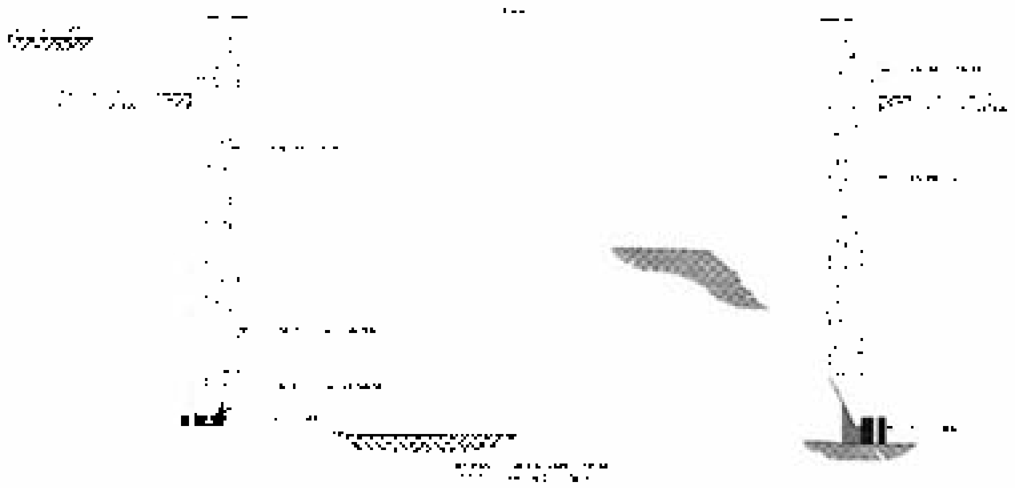

Figure No. 1 - Pumping Station 2A Caisson

The connecting shaft caisson was constructed with 1.5 meter (5 foot) thick walls which were placed in five lifts including the shoe. Type II concrete was also used for the junction chamber caisson. As with the pumping station caisson, excavation and sinking were completed by maintaining a soil plug within the caisson walls to maintain bottom stability, and steel pipe landing piles were driven around the perimeter of the caisson to ensure that it would be plumb and level when it landed. After sinking the caisson to the required depth, an artesian spring developed at the shaft invert. This flow was apparently the result of the landing piles penetrating the clay separation barrier and the underlying ungrouted artesian aquifer. In an effort to maintain a stable excavation bottom, the shaft was flooded. The caisson base slab was later installed underwater, using tremie concrete placement methods. Construction time for sinking this caisson, from shoe construction to

Figure No. 45

landing, was approximately 22 weeks, with approximately 2 additional weeks required to install the base slab. However, several months elapsed in between the caisson landing and the base slab installation, due to the artesian flow into the shaft and the subsequent shaft flooding required to maintain a stable bottom.

Rouge Steel Scale Pit

Rouge Steel Company, located in Dearborn, Michigan, began work on a new continuous casting operation in 1987. The new facility required construction of a rectangular mill scale pit, 37.2 meters (122 feet) long, 14 meters (46 feet) wide, and 15.2 meters (50 feet) deep. The structure was designed by Lockwood Greene Engineers and NTH Consultants, Ltd. and constructed by Walbridge-Aldinger Company.

Ground conditions at the site of the new scale pit were very similar to those at the nearby Detroit Wastewater Treatment Plant. Factors important for the performance of the earth retention system included controlling bottom stability, and minimizing lateral movements and their impacts on adjacent structures. Several methods of installing the scale pit were considered, including an internally braced steel sheet pile cofferdam, soldier pile and lagging systems, and frozen ground techniques. The poor ground conditions, together with the use of the caisson walls as part of the permanent structure, made the sinking caisson option the least time consuming and the most cost effective in the contractor's view.

Prior to constructing the shoe for the scale pit caisson, the area was precut approximately 4.6 meters (15 feet) below the prevailing ground surface, and steel pipe landing piles were driven. The excavation bottom was expected to be within thick clay deposits approximately 6.1 meters (20 feet) above the water bearing hardpan and rock layers. As such, groundwater control measures such as rock grouting were not required.

The 0.9 meter (3 foot) thick caisson walls were placed in 4 lifts including the shoe. Type III concrete was used, since groundwater (and dissolved hydrogen sulfide) infiltration was not expected to be a significant problem. Bottom instability was prevented during sinking the caisson by maintaining a soil plug within the caisson interior. A 1.8 meter (6 foot) thick concrete base slab was installed after the caisson was sunk to design elevation.

Construction time for sinking the caisson and installing the base slab was approximately 11 weeks. This compared to an estimate of 20 weeks to install an internally braced cofferdam and construct the scale pit walls and slab within.

Redford CSO Pumping Station and Swirl Concentrator Caissons

The Redford Combined Sewage Overflow (CSO) retention facility includes a 18.3 meter (60 foot) inside diameter reinforced concrete pump station and a 10.7 meter (35 foot) inside diameter reinforced concrete swirl concentrator, constructed within 2.4 meters (8 feet) of each other. Both of these facilities were constructed as sinking caissons with completed base slabs requiring excavation to depths on the order of 16.8 meters (55 feet). These structures were designed by Wade-Trim/ Associates, Inc., for the owner, Charter Township of Redford. The project was constructed by Walbridge Aldinger Company.

The original contract documents as well as subsequent contractor test borings indicated soil conditions consisting of approximately 6 feet of surficial sand underlain by thick deposits of medium consistency silty clay.

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further underlain by compact predominantly granular layers, then shale bedrock. The soils below the base of the caisson excavations were expected to consist of silt, sand, and hardpan layers under artesian groundwater pressure. Because these layers were expected to be very difficult to dewater, the contract documents required that hydrostatic uplifts forces be balanced during construction by flooding the caissons during excavation. In addition, the design called for dome shaped caisson base slabs to be placed under water using tremie concrete placement methods. The purpose of the domed base slab was to carry permanent hydrostatic loads and eliminate the need for rock anchors within the base slab

Recognizing the difficulty and expense which would be associated with performing the excavation and base slab construction under water, the contractor proposed to develop a dewatering plan which would allow construction in the dry. A supplemental geotechnical investigation at the site helped define the presence and extent of the aquifer below the caissons. Based on grain size analysis of the soil samples, it was determined that the aquifer was dewaterable. A dewatering plan was then developed which was subsequently accepted by the owner's engineer.

Figure No. 2 - Redford CSO Shafts

In order to minimize the number and size of wells, a groundwater pumping test was conducted to further define the hydrogeologic parameters of the aquifer and possibly allow a reduction in the number of dewatering wells estimated to be required based on the grain size analysis. The results of this test allowed modification of the dewatering plan to include four, 0.9 meter (3 foot) diameter gravel packed wells.

The walls of the caissons were constructed in four lifts, including the shoe. Using the modified dewatering plan, the excavation was completed in the dry and the domed base slabs were placed without incident. The caissons were grouted in place by injecting grout through the bentonite injection pipes. Construction time was approximately 20 weeks for the 18.3 meter (60 foot) diameter caisson, and 15 weeks for the 10.7 meter (35 foot) diameter structure, not including the base slab construction. In each case, these slabs required approximately 3 additional weeks to construct.

River Rouge CSO Basin Demonstration Project

The City of River Rouge, Michigan has undertaken a major construction project to control the release of combined sewage into the Rouge River. The major component of the project is a large diameter concrete reinforced sinking caisson which will serve as the main combined sewage detention facility. The caisson has seven foot thick walls, with an outside diameter of 45.4 meters (149 feet) and a depth of approximately 22.9 meters (75 feet). This project was begun in the spring of 1996, and is planned to be completed in winter 1999. The project was designed by Sigma Associates, Inc., and is being constructed by Walbridge - Aldinger Company.

Soil conditions at the site consisted of approximately 2.4 meters (8 feet) of surficial sand and debris fill, underlain by an approximately 3 meter (10 foot) thick desiccated clay layer. The fill and sand layers typically contained perched groundwater. The desiccated clay is underlain by an extensive deposit of soft silty clay, which is further underlain by the hardpan till layer at a depth of about 20.7 meters (68 feet). The bedrock underlying the hardpan consists of dolomitic limestone, which is typically highly fractured within the upper 1 to 3 meters (3 to 10 feet). Artesian pressure within the hardpan and bedrock was measured at approximately 1.5 meters (5 feet) above the ground surface. The hardpan at this site is considered relatively impermeable, although the fractured bedrock is expected to produce high volumes of groundwater if it is exposed. Where encountered, artesian groundwater is expected to contain high concentrations of dissolved hydrogen sulfide, and moderate concentrations of methane gas.

Prior to constructing the caisson, the project site was pre-cut approximately 2.1 to 3 meters (7 to 10 feet) in order to prevent difficulties in sinking the caisson through the upper fill and desiccated clay layers. Perched groundwater was controlled by pumping from open sumps. A four stage rock grouting program was then undertaken to seal the fractured bedrock and to minimize infiltration of groundwater during excavation. Additionally, a geotechnical instrumentation program was undertaken with the purpose of protecting existing adjacent structures.

The caisson wall construction is planned to be competed in six lifts including the shoe. The excavation from the center of the caisson has been conducted in the dry, using a clamshell and smaller earthmoving equipment working within the caisson. At this writing, the caisson is close to its final elevation, and is planned to be completed within five weeks. Following sinking of the caisson walls to the desired depth, the clay zone below and around the perimeter of the shoe is planned to be subject to ground freezing to prevent soil squeezing below the shoe prior to base slab placement. Rock anchors will then be installed from the rock surface, and post tensioned in place following base slab placement.

The caisson construction is planned to be completed in approximately 28 weeks, not including the base slab construction. The base slab construction (including rock anchor installation) is expected to be completed in an additional 8 weeks.

Northwest Lateral Pipeline

Construction for the Costal Water Authority and City of Houston's Northwest Lateral Pipeline began in 1988 and represented an integral step in the expansion of the city's water supply system. The original design called for approximately 3350 meters (11,000 feet) of tunnel crossing two major bodies of water: the Houston Ship Channel and Greens Bayou. After award of the contract to the Greenfield Merriman Joint Venture, the joint venture retained NTH Consultants to redesign the water conveyance system under a value engineering provision in the contract. As part of the redesign, the tunnel depth was decreased and sinking caissons were used to construct "siphon" type extensions below the Houston Ship Channel and Greens Bayou.

At the Houston Ship Channel crossing, octagonal shaped shafts which were specified in the contract documents to be either diaphragm walls or freeze walls, were redesigned to be round sinking caissons. This modification allowed a reduction in the wall thickness and reinforcement required, and made sinking caissons the economical choice. At the Greens Bayou crossing, round sinking caissons were designed to replace long sloping tunnel sections which extended below the Bayou. As a result of this design, the majority of the tunnel was installed at relatively shallow depth while only the portions of tunnel directly below the bodies of water were mined at the deeper depth. The decreased tunnel depth through most of the tunnel alignment allowed the use of steel rib and lagging primary tunnel liner rather than steel liner plate as called for in the original design. In addition, steep tunnel grades which were called for in the original design were eliminated.

The four shafts used to construct the siphon extensions were each constructed in similar ground conditions. Subsurface soils of each shaft locations generally consisted of medium to stiff clay with interbedded water bearing zones of fine sand and silt, underlain at depths of 21 to 30 meters (70 to 100 feet) by generally soft clay. Where groundwater was present within the water bearing layers, the potentiometric level was about 3 to 4.6 meters (10 to 15 feet) below ground surface.

Figure No. 3 - Greens Bayou Shaft

Figure No. 45

The shaft designs at the Greens Bayou and Houston Ship Channel crossings called for two stage shaft construction, consisting of an upper sheet pile supported shaft and a lower concrete sinking caisson shaft. The upper steel sheet pile shafts were 15.3 meters (50 feet) deep, and were supported internally by concrete ring wales. The lower sinking caisson portions of each shaft were 7.6 meters (25 feet) in internal diameter with 1.2 meter (4 foot) thick walls. The shafts extended to overall depths of approximately 33.5 meters (110 feet) below ground surface. It should be noted that only one of the four sinking caisson structures was designed to be incorporated into the permanent structure. The remaining three caissons were used only as temporary structures, and were abandoned in place.

Control of ground movements during construction was critical due to the presence of an active railroad line which was located within 6.1 meters (20 feet) of the edge of one of the shafts. Borehole extensometers were used to measure ground movement in this area. Within the sinking caissons, bottom instability was typically prevented by maintaining a soil "plug" within the shaft bottom. Within one of the Greens Bayou shafts however, flooding the caisson was also required during sinking to balance hydrostatic pressures and groundwater seepage into the shaft. Typically, bentonite was injected at the shoe during excavation to reduce friction and adhesion on the outside of the caisson, and to aid in sinking. Once the caissons were advanced to the required depth, they were grouted in place by injecting grout through the bentonite injection pipes. The average time to construct each of the shafts was approximately 3 to 4 weeks for the upper sheet pile stage and 10 to 12 weeks for the lower sinking caisson (including the base slab).

COST COMPARISONS

Costs for constructing the sinking caissons discussed above were summarized and compared to other construction methods which were considered. All costs were adjusted to 1996 dollars. Based on our review of cost data for these projects, the overall cost for sinking caissons in place (excluding base slab construction) was found to be approximately $795 per cubic meter ($610 per cubic yard), which can be equated to approximately $1450 per square meter ($135 per square foot) of exposed shaft wall. However, it should be noted that unit area costs varied more widely than unit volume costs due primarily to differences in the wall thicknesses. Other methods which were considered for the above projects ranged from $1940 to $2150 per square meter ($180 to $200 per square foot) of wall, including the cost of the final concrete shaft walls in place. A summary of cost data for the above discussed projects is presented in Table 1.

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TABLE 1 - Cost Comparisons for Sinking Caissons Vs. Other Shaft Const. Techniques

Project/Shaft	**Total Cost (Sinking Caisson Method)**	**Cost per Unit Volume for Caisson Method**	**Comments**
PS2A - Pumping Station	\$5,080,000 (caisson walls) \$2,213,000 (base slab w/ freeze & underpin)	$\$842/\text{m}^3$ (\$643/cy) $\$816/\text{m}^3$ (\$624/cy)	1. Costs for steel sheeting and bracing were quoted at \$2 million higher than sinking caisson. 2. Costs for freeze wall were quoted at \$3 million higher than sinking caisson. 3. At least 6 months were saved over alternative methods
PS2A - Conn. Shaft	Not Available	Not Available	
Rouge Steel Scale Pit	\$2,065,000 (caisson walls) \$301,000 (base slab)	$\$943/\text{m}^3$ (\$721/cy) $\$322/\text{m}^3$ (\$246/cy)	1. Savings of \$0.7 million gained over use of braced sheet pile shaft construction.
Redford CSO (60'dia)	\$337,000 (caisson walls) \$152,000 (base slab)	$\$388/\text{m}^3$ (\$297/cy) $\$387/\text{m}^3$ (\$296/cy)	
Redford CSO (35'dia)	\$173,000 (caisson walls) \$49,000 (base slab)	$\$322/\text{m}^3$ (\$246/cy) $\$323/\text{m}^3$ (\$247/cy)	
Rouge CSO Basin	\$4,717,000 (caisson walls) \$635,000 (base slab)	$\$801/\text{m}^3$ (\$612/cy) $\$177/\text{m}^3$ (\$135/cy)	1. Savings of \$2.5 million gained over use of braced sheet pile shaft construction. 2. Construction time for caisson estimated to be half that of braced sheeting
NW Lat Pipe (all shafts)	Not Available	Not Available	

Figure No. 45

CONSIDERATIONS FOR DESIGNERS

Several considerations are apparent based on the author's experience in the design and construction of shafts using sinking caissons. These are discussed below:

1) Sinking caissons are often cost effective when utilized as both the construction shaft and final structure. The volume of concrete used for the caisson walls will typically be greater than the volume of concrete required for the walls of the final structure, as a consequence of the weight required to sink the caisson walls. As a rough comparison of construction costs between sinking caisson construction and temporary earth support with an internally constructed structure, the preliminary design may compare the cost of the additional concrete required in the caisson walls (above that amount which would be required for the final structure), to the cost of a temporary earth support system. The designer should be aware, however, that the majority of caisson wall construction costs are from the labor for vertical form work, rather than the bulk materials. As such, the costs for thickened caisson walls do not increase the overall shaft costs incrementally.

2) Sinking caissons are highly effective in controlling ground movements adjacent to constructed shafts. Based on empirical studies of ground movement by Peck and D'Appolonia, as well as project experience from the above case histories, settlement adjacent to sinking caisson shafts is typically on the order of 0.2 percent of the depth of the shaft. This compares to estimates by Peck of 1 percent to 2 percent of the shaft depth for flexible earth support systems such as internally braced steel sheet piles.

3) The design of sinking caissons in clay soils is highly sensitive to adhesion and friction of the soil surrounding the caisson walls, as well as the shear strength of soils below the base. Methods of evaluation of basal stability (Bjerrum and Eide, and Terzaghi), suggest that failure zones will extend to a depth below the caisson shoe approximately equal to 0.7 times the shaft width. As such, geotechnical investigations should be planned to gather data to depths equal to a minimum depth corresponding to one shaft width below the planned final shaft invert. Where bottom stability concerns require utilization of a soil plug within the caisson to control bottom heave during excavation, geotechnical investigations should extend to a depth below the caisson shoe of at least 0.7 times the width of the shaft. Soil testing should include unconfined compression strength evaluations or unconsolidated undrained triaxial tests of relatively undisturbed Shelby tube samples. For soft and very soft clay, in-situ vane shear testing should also be considered. Such samples and/or in-situ testing should be procured and/or performed at minimum intervals of approximately 15 feet. Where artesian groundwater is encountered within test borings, a piezometer should be installed to allow for design provisions to prevent hydrostatic uplift of the shaft base.

4) The use of sinking caisson techniques is often most successful and most cost effective where ground conditions are considered very poor. In the words of a contractor's tradesman "The worse the clay is, the better these sinking caissons work!" It is significant that in most of the above cited cases, sinking of the caisson was controlled by discrete excavation, without using the bentonite lubrication systems which were installed.

5) Sinking caissons have proved to be an effective means of construction for both owner-designed and contractor-designed shafts. Advantages to owner-designed shafts include: ability to control ground movements (by specifying construction means such as sinking caissons), ability to incorporate the sinking caisson into the permanent structure; speed of construction in appropriate conditions; and the ability to obtain competitive bids for the specific type of shaft construction desired. The primary advantage of allowing the contractor to design

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shafts is that creative and innovative solutions can be proposed by the contractor, and the owner can share in the cost savings which result from contractor ingenuity.

BIBLIOGRAPHY

Bjerrum, L. And Eide, O., 1956, "Stability of Strutted Excavation in Clay," Geotechnique, Vol. 6, pp. 32-47.

Broms, B.B. and Bennermark, H., 1967, "Stability of Clay at Vertical Openings," Journal of the Soil Mechanics and Foundations Division, Proceedings, American Society of Civil Engineers, Vol. 93, No. 1 Jan., pp 71-94

D'Appolonia, D.J., 1971, "Effects of Foundation Construction on Nearby Structures," Proceedings of the 4th Panamerican Conference on Soil Mechanics and Foundation Engineering, Vol. 1, pp. 189-236.

Ivor-Smith, D.J., et al., 1989, "Deep Soft Ground Tunneling Under Houston Ship Channel and Greens Bayou," Proceedings, Rapid Excavation and Tunneling Conference, June, 1989, Pond, R.A., and Kenny, P.B., ed., pp. 230-243.

Olden, J.M., and DiPonio, J.J., 1989, "Northwest Lateral Progress," World Tunneling and Subsurface Excavation, Vol. 2, No. 6, Aug., pp. 310-316

Peck, R.B., 1969, "Deep Excavations and Tunneling in Soft Ground, State-of-the-Art Report," Proceedings of the 7th ICSMFE, Mexico City, State-of-the-Art Volume, pp 225-290

Porter, T.G., et al., 1993, "Shaft Excavation in Soft Clay by Caisson Construction," Retaining Structures, Clayton, C.R.I., ed., Thomas Telford, London, pp 816-825.

Terzaghi, K., 1943, Theoretical Soil Mechanics, John Wiley & Sons, Inc., New York, New York

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