City of Flint

Department of Purchases & Supplies

Sheldon A. Neeley

TO: All Proposers

FROM: Lauren Rowley
Purchasing Manager

DATE: **July 13, 2021**

SUBJECT: **Addendum #03 –** Bid# 22000048 – WPC Third Avenue Pumping Station Improvements

This addendum has been issued because of the following:

1. Exhibit 4 – Additional Drawings
2. Exhibit 5 – Report on: Geotechnical Exploration Proposed Dewatering Building Flint Wastewater Treatment Plant, Flint, Michigan

All other bidding terms, requirements, and conditions continue as indicated in the remaining original bid documents.

The Purchasing Manager, Lauren Rowley, is an officer for the City of Flint with respect to this RFP.

In the submission of their proposal, Proposer must acknowledge receipt of this addendum. Proposer shall acknowledge this addendum by signing and returning one copy of this notice with their submission.

Company Name: ____

Address: ____

City / State / Zip: ____

Telephone: ________________Fax: ________________ Email: ______________________

Print Name: _________________________Title: __________________________

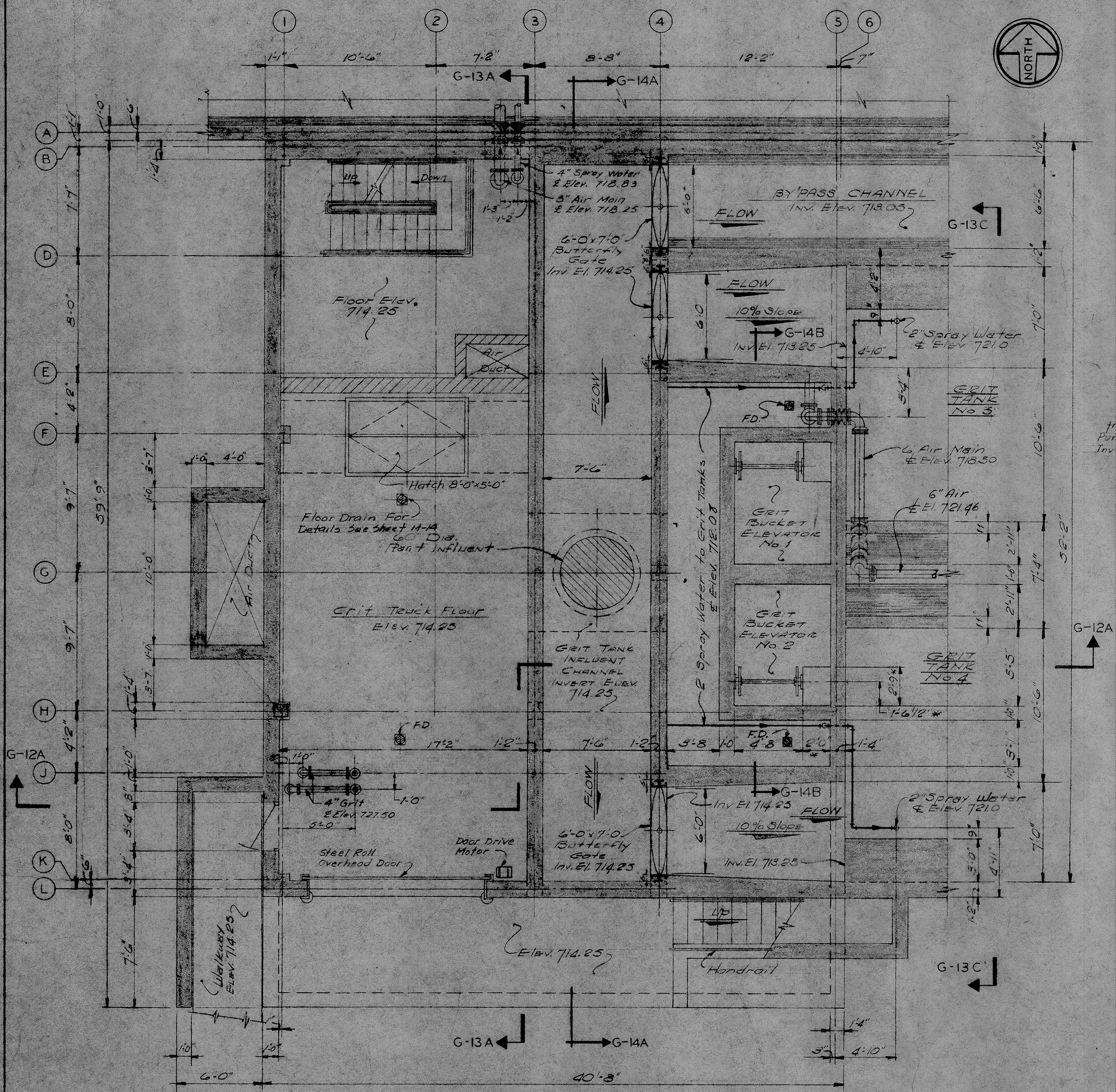
Signature: _____________________________Date: ______________________

Thank you,

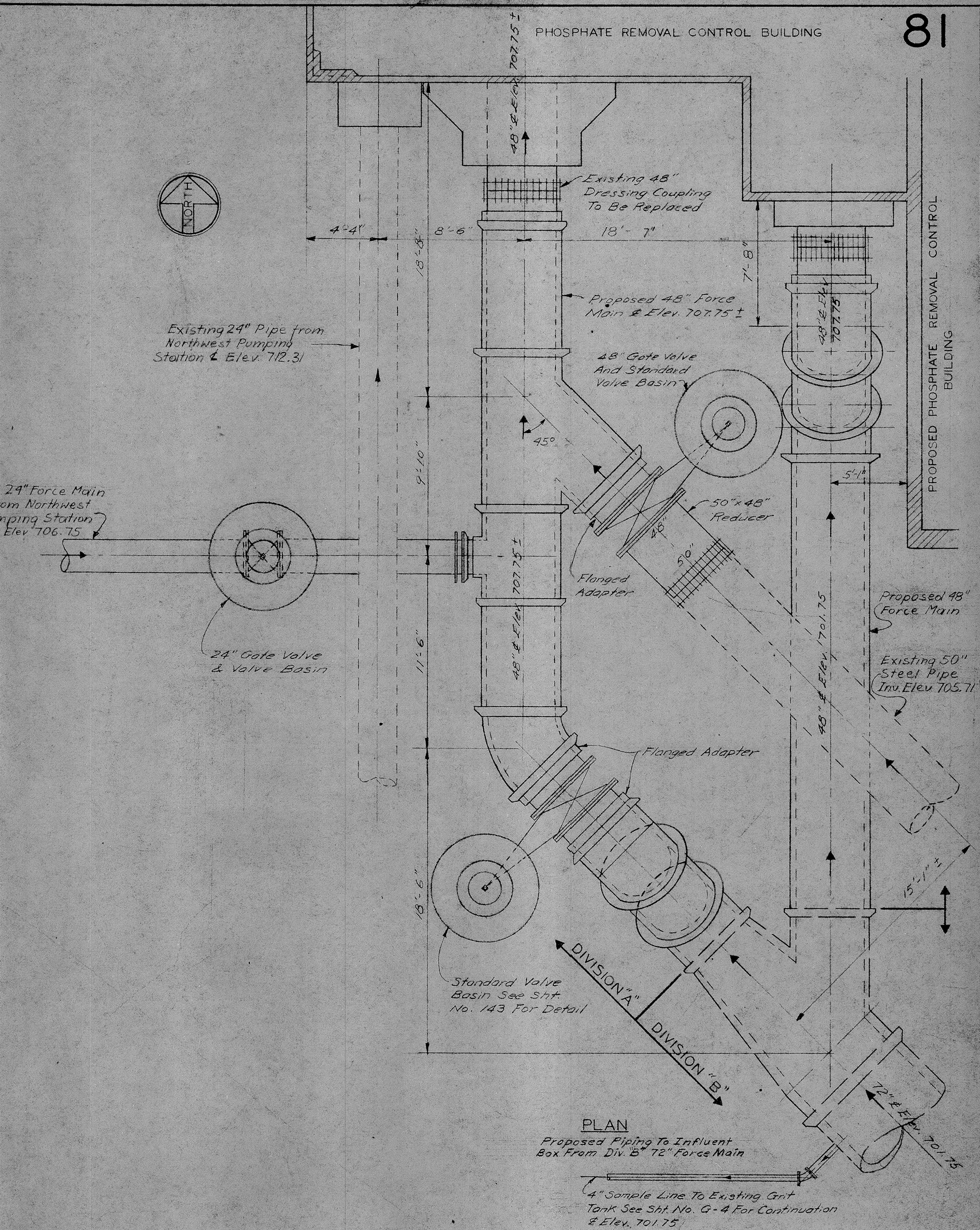
Lauren Rowley

Lauren Rowley
Purchasing Manager

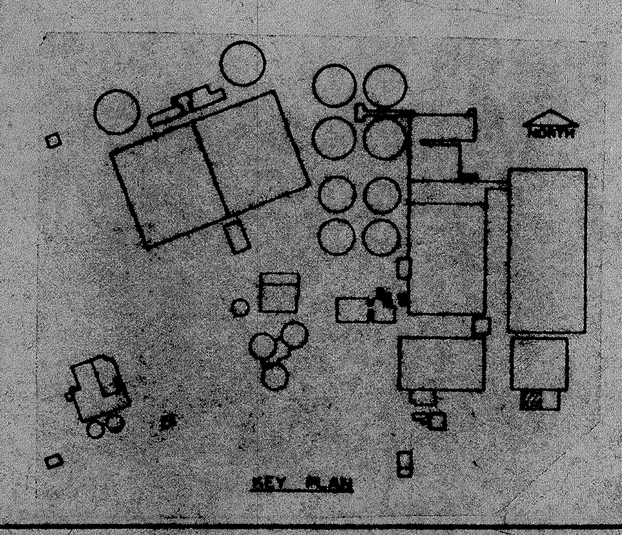
Exhibit 4



PLAN
Grit Dewatering Building



PLAN
Proposed Piping To Influent Box From Div. B 72" Force Main



CITY OF FLINT, MICHIGAN
WATER POLLUTION CONTROL PLANT
IMPROVEMENTS AND MODIFICATIONS
PROPOSED GRIT DEWATERING BUILDING
PLAN — ELEV 714.25

DRAWN BY: M.E.J.	CONSOER, TOWNSEND & ASSOCIATES IN MICHIGAN CONSULTING ENGINEERS	SCALE: 1/4"=1'-0"
DATE: Mar. 1973		REVISED:
CHECKED BY: G. Y. K.		SHEET NO. G-10
APPROVED BY: W. J. B.		OF G-163 SHEETS

70295

82

CITY OF FLINT, MICHIGAN

WATER POLLUTION CONTROL PLANT
IMPROVEMENTS AND MODIFICATIONS

PROPOSED GRIT TANKS & GRIT DEWATERING BLDG
PLAN — ELEV. 702.25

DRAWN BY: M.B.J.	CONSOER, TOWNSEND & ASSOCIATES IN MICHIGAN	SCALE: 1/4" = 1'-0"
DATE: Mar. 1973		REVISED:
CHECKED BY: G. Y. K.		
APPROVED BY: W. J. B.	CONSULTING ENGINEERS	SHEET NO. G-11 OF G-163 SHEETS

70295

Exhibit 5

GEOTECHNICAL EXPLORATION

Report on:

Geotechnical Exploration
Proposed Dewatering Building
Flint Wastewater Treatment Plant
Flint, Michigan

Prepared For:
Wade Trim Inc.
555 S. Saginaw Street, Suite 201
Flint, Michigan 48502

GeoTran Project No.: 17-09002G-10
February 8, 2018

GeoTran

10315 E. Grand River
Suite 201
Brighton, Michigan 48116

GeoTran
Consultants, LLC

10315 E. Grand River, Suite 201
Brighton, MI 48116
P: 810.229.6805; F: 810.229.6561

Mr. Jeremy Schrot, PE
Wade Trim Inc.
555 S. Saginaw Street, Suite 201
Flint, Michigan 48502

February 8, 2018
GeoTran Project No. 17-09002G-10

RE: Report on Geotechnical Exploration
Proposed Dewatering Building
Flint Wastewater Treatment Plant
Flint, Michigan

Dear Mr. Schrot:

We are pleased to submit this report of our geotechnical exploration completed for the proposed dewatering building project at the City of Flint Wastewater Treatment Plant in Flint, Michigan. The investigation was performed in accordance with the scope of services outlined in our September 21, 2017 E-mail correspondence to you and Subconsultant Agreement between Wade Trim Inc. and GeoTran Consultants, LLC dated September 26, 2017.

We appreciate the opportunity to be of service to you on this project. If you have any questions regarding this report or if we can be of further assistance, please feel free to contact us at (810) 229-6805.

Sincerely,
GeoTran Consultants, LLC

Danny Yip
Danny Yip, P.E.
Project Engineer

Tanweer Shah
Tanweer Shah, P.E.
Senior Project Manager

DY/TS/dy
Attachments

TABLE OF CONTENTS

APPENDIX

1.0 INTRODUCTION

This report presents the results of a geotechnical exploration completed for the proposed dewatering building at the City of Flint Wastewater Treatment Plant in Flint, Genesee County, Michigan. The site location, relative to existing streets and topographic features, is shown on the Site Location Map, Figure No. 1 of the Appendix.

The purpose of the investigation was to explore and evaluate the general subsurface conditions at the site and to develop foundation and related site preparation recommendations for the proposed project. Conclusions and recommendations presented in this report are based on the subsurface conditions encountered at the locations of our explorations and our current understanding of the proposed project. Conditions may vary between boring locations, and should not be extrapolated to other areas without our prior review.

2.0 SITE AND PROJECT CHARACTERISTICS

The site of the proposed project is located on the premises of the City of Flint Wastewater Treatment Plant (WWTP) located at 4652 Beecher Road in Flint. This location is near the northeast corner of the intersection of North Linden and Beecher Roads. The approximate layout of the site is presented on the Boring Location Plan, Figure No. 2 of the Appendix. As indicated on Figure No. 2, the site is located south of the existing Primary Settling Tanks and east of the Grit Tanks Nos. 1 and 2. Currently, the site is grass covered, while concrete sidewalks run along the north and west sides of the site. The topography at the site is relatively flat with site ground surface elevation presumed to be on the order of about 717 feet. Flint River bounds the WWTP approximately 800 feet northeast of the site.

Based on our visual field observations, review of yard piping drawing and utility staking markings, underground utilities such as plant influent piping, drains, tunnels, forced mains, sewers, water or electric lines and/or other unknown below-grade utilities are assumed to exist near the site, but were not encountered during this investigation.

We understand that the City of Flint has contracted with Wade Trim, Inc. (Wade Trim) to prepare design plans for the project. Based on the information provided, it is our understanding that the project calls for construction of a one-story slab-on-grade dewatering building with plan dimensions anticipated on the order of 20 feet by 30 feet. The proposed building is presumed to house presses, pumps, piping and other equipment. The structural loads associated with the new building are not currently known to us; however, for purposes of this report, we have assumed the loads will be light to moderate.

3.0 CURRENT FIELD EXPLORATION

Subsurface conditions at the site were explored by drilling two exploratory soil borings, designated as B-1 and B-2, at the approximate locations shown on the Boring Location Plan, Figure No. 2 of the Appendix. The soil borings were located in the field by GeoTran representative by referencing to existing site features. B-1 was located near the northeast corner and B-2 near the southwest corner of the proposed building footprint. The borings were not surveyed by a licensed land surveyor; therefore, the locations shown on Figure No. 2 are considered to be approximate. Likewise, site ground surface elevations at the boring locations were estimated from the topographic information provided to us by Wade Trim and should be considered approximate as well.

The soil borings were machine-drilled by DLZ/American Drilling under the full-time technical observation of a field engineer with our firm. The borings were drilled using a CME 75 truck-mounted drilling rig and completed to a depth of about 39 feet each below existing ground surface. Continuous flight, hollow-stem augers having an inside diameter of 2¼ inches were used to advance the borings to the explored depth. It is noted that Boring B-2 was offset from its initial location due to a concrete obstruction encountered at the initial location at a depth of about 5½ feet (initial B-2 location approximately 10 feet southwest of its final as-drilled location).

Within the borings, soil samples were obtained at intervals of 2½ feet within the upper 10 feet and at intervals of 5 feet below that depth. In general, the samples were obtained using a 1⅜-inch inside-diameter split-barrel sampler and the Standard Penetration Test (SPT) method ASTM D 1586, described on the attached General Notes, Figure No. 3. Soil samples obtained from the borings were visually classified in the field by our representative using Unified Soil Classification System (USCS), sealed in containers and transported to the laboratory for further classification and testing. We will retain these samples for 60 days after the date of this report. At that time, we will dispose of the samples unless we are otherwise instructed.

Upon the completion of drilling and sampling operations, and following subsequent groundwater observations, the soil borings were backfilled using excavated materials.

4.0 LABORATORY TESTING

Representative soil samples obtained from the borings were subjected to limited laboratory testing to determine the pertinent engineering characteristics of the site soils. The laboratory testing included a grain size analysis of a selected soil sample using hydrometer. The results are included as Summary of Laboratory Test Data, Figure No. 6 and Grain Size Analysis, Figure No. 7 in the Appendix.

In addition to laboratory testing, field pocket penetrometer measurements were made as appropriate on representative cohesive soil samples obtained from the borings as an aid in evaluating their unconfined compressive strengths. The pocket penetrometer values are indicated on the boring logs.

5.0 GENERAL SUBSURFACE CONDITIONS

We have evaluated the soil and groundwater conditions encountered in the borings and have presented these conditions in the form of individual Logs of Soil Boring, Figure Nos. 4 and 5 in the Appendix. In addition to subsoil stratification, the boring logs present SPT results or N-values, observed groundwater levels, drilling and sampling information and other pertinent data. General notes defining the nomenclature used on the logs and within the text of this report are presented on Figure No. 3. We have prepared the boring logs on the basis of visual classification and limited laboratory testing.

The stratification indicated on the boring logs represents the subsurface conditions at the actual explored locations. Variations in subsurface conditions may occur between these locations. In addition, the stratigraphic lines represent the approximate boundary between material types. The transition from one material type to another may be more gradual than indicated.

Subsoil Conditions and Evaluations

The results of this investigation indicate that the proposed dewatering building site is generally underlain by existing fill materials overlying native cohesive and granular soils. The ground surface at the boring locations was covered with about 6 to 12 inches thick topsoil, consisting of dark brown clayey sand with organic matter. Directly underlying the topsoil cover, existing fill materials were encountered to an approximate depth of 12 feet below ground surface within the borings. The fills consist of granular or cohesive materials including brown sand with trace amounts of clay and gravel and dark brown sandy clay with trace amounts of gravel. N-values for the granular fills ranged from 9 to 36 blows per foot (bpf), indicating loose to dense conditions.

Underlying the existing fill materials, the borings encountered native cohesive and granular soil deposits. The native cohesive soils consist of gray silty clay with trace amounts of sand, gravel and occasional silty sand layers to a depth of about 32 feet. Pocket Penetrometer (PP) unconfined compressive strength measurements for the native silty clays ranged from 4,500 to in excess of 9,000 pounds per square foot (psf). The PP measurements for the native cohesive deposits indicate the presence of very stiff to hard consistency soils. The cohesive soils are, in turn, underlain by native granular soils to the explored depth of the borings (about 39 feet). The granular soils consist of gray silty sand with little clay and gravel. N-values for these native granular soils ranged from 50 blows for 5 inches to 100 blows for 5 inches, indicating very dense conditions. Results of the grain size analysis for the granular soil sample SS-10 obtained from about 39 feet depth within boring location B-1 revealed 12 percent gravel, 52 percent sand, about 23 percent silt and 13 percent clay content.

As mentioned above, existing fill materials were encountered in the soil borings to an approximate depth of 12 feet and these materials vary in strength and composition. We have not reviewed any documentation confirming that the existing fills were placed in an engineered manner. Due to the variability and unreliability of the existing fills, these materials are prone to settlement and are not considered suitable for support of building foundations loads, and will require removal prior to new structural fill placement or installation of building foundations. However, following suitable site conditioning in accordance with procedures outlined in the Site Preparation section of this report, and provided some settlement can be tolerated and existing fills do not contain appreciable amounts of non-soil debris or other undesirable materials, we expect the fill materials can largely remain in place beneath floor slabs.

The native very stiff to hard silty clay soils or very dense silty sand soils underlying the existing fill materials are considered suitable for the direct support of the foundation loads of the type anticipated for this project.

Groundwater Conditions

Groundwater level observations were made at each boring location during and at the completion of drilling (end of drilling) operations. Groundwater was observed at depths of about 28½ feet (Elevation 688½ feet) and 29½ feet (Elevation 687 feet) during drilling within borings B-1 and B-2, respectively. Upon completion of drilling, groundwater was observed within these borings at approximate depths of 7 feet (Elevation 710 feet) and 6 feet (Elevation 710½ feet). In addition, caving was observed within the boreholes at approximate depths of 28 feet (Elevation 689 feet) in B-1 and at 26 feet (Elevation 690½ feet) in B-2.

Fluctuations in the groundwater levels at the site should be anticipated with seasonal variations and following periods of prolonged precipitation. In cohesive or other fine-grained soils encountered at the site, groundwater observations are not necessarily indicative of the hydrostatic or long-term water levels due to the low permeability of such soils and their tendency to seal off natural pathways of groundwater flow during drilling operations. The actual hydrostatic water levels or the presence of perched groundwater should be anticipated to fluctuate depending on variations in precipitation, surface runoff, infiltration, surrounding topography, drainage and nearby Flint River channel. Long-term groundwater levels including fluctuations in groundwater levels can best be determined through observations made in cased boreholes or observation wells over a prolonged period of time. The installation of cased boreholes was beyond the scope of the current investigation.

6.0 SITE PREPARATION

Based on the nature and type of anticipated construction, we have assumed that the finished grades will be close to existing ground surface. If shallow foundations are used, site preparation will require a significant amount of over excavation and backfilling due to the presence of existing fill materials, in order to bring the grades to desired elevations for construction purposes. Regardless of the amount of earthwork required to achieve final grades, it is recommended that all site and subgrade preparation activities be performed under adequate specifications and be properly observed in the field.
All areas intended to support new building foundations or grade raise fill must be properly prepared before proceeding with new construction. At the start of earthwork operations, existing grass cover,

topsoil and/or other unsuitable materials including organic matter, refuse and deleterious non-soil debris, as well as any other exposed soil containing obvious amounts of organic matter should be stripped in their entirety from within the proposed construction areas. The stripping should typically extend a horizontal distance of at least 5 feet beyond planned construction lines. All underlying existing fill materials or other unsuitable materials such as yielding soils should be removed in their entirety, where they exist below the proposed footing locations. All debris and materials resulting from the stripping operations and removal of existing fill from footing areas should be disposed of outside the proposed construction limits.

It is noted that a concrete obstruction, presumably construction debris, was encountered at an approximate depth of 5½ feet during drilling of boring B-2 at its initial location (initial B-2 location approximately 10 feet southwest of its final as-drilled location). Therefore, it is recommended that probes or test pits be performed prior to actual construction to evaluate the existence and lateral extent of the buried obstruction within the site soils. This will help the prospective bidders to reduce the potential of claims and construction delays.

All active and in-active utilities within or adjacent to the construction area should be identified for protection, relocation or abandonment prior to grading. Utilities that are to be left in place should be evaluated for their effect on the proposed project and vice versa. Existing backfill around utilities that are to remain should be checked for compaction and suitability to meet the project requirements and should be improved, if necessary. Excavations or voids resulting from site stripping, clearing and removal of existing fill materials and/or buried obstructions should be backfilled to surrounding grade or design subgrade level with approved and compacted granular engineered fill.

As mentioned earlier, within the shallow footings excavations, we recommend that all of the existing fill materials or any other yielding or unstable soils be removed in their entirety, where they exist at the proposed footing locations, and replaced with well-compacted granular engineered fill. Based on the soil boring data, it is anticipated that removal of existing fills from footing locations for the new building will extend to a depth of about 12 feet below ground surface. The exact depth and lateral extent of the fill materials within the building area should be expected to vary. Evaluation of the required depth of removal must be performed by a qualified person at the time of construction.

After rough grade has been established in cut areas and prior to placement of new grade raise fill in all fill areas, the exposed subgrade should be carefully observed by probing and testing as needed. All organic material (if any) still in place, frozen, wet, soft or loose soils and other unsuitable materials should be removed. The subgrade resulting from the removal of surficial materials is expected to consist primarily of existing granular and cohesive fills or native cohesive soils. Therefore, areas of exposed granular subgrade soils including those underlying the proposed building floor slab area should be thoroughly proof-compacted using a medium weight, smooth drum vibratory roller making a sufficient number of passes in each of two perpendicular directions. This is intended to densify any loose granular soils or granular soils that have been disturbed by site clearing and grading operations, thereby improving their load supporting capability. If the operation of the vibratory roller is observed to decrease the stability of the subgrade soils by drawing water towards the subgrade surface, vibration should be discontinued and the roller should be operated in the static mode. The smooth drum roller should be kept a minimum distance of 10 feet from any existing structures and only light-weight compaction equipment such as a plate compactor or hoe-pak should be used to achieve the required compaction in these areas. The use of light weight compaction equipment will be more practical than use of a vibratory roller in areas adjacent to the existing facilities or in areas with limited space available for the heavier equipment to perform proof-compaction operations.

Areas that exhibit excessive movement or pumping during proof-compaction operations should be re-compacted or undercut and replaced with engineered granular fill or improved by using other methods depending upon site conditions at the time of construction. If undercutting is used, the undercut should be a minimum depth of 12 inches and the resulting excavation properly backfilled with engineered MDOT Class II or 21AA materials.

In addition to proof-compaction operations, areas where the exposed subgrade consists of cohesive soils, the subgrade should be thoroughly proof-rolled using heavy rubber tired roller or earthmoving equipment such as a loaded dump truck or loaded scarper. Any areas of cohesive subgrade soils that exhibit excessive movement or instability during proof-rolling operation should be stabilized by aeration, drying and re-compaction, if weather conditions are favorable, or by removal of the yielding soils and their replacement with engineered granular fill.

As mentioned above, within the footprint of the proposed building, a concrete obstruction was encountered at a depth of about 5½ feet below ground surface. Where old construction debris (i.e., concrete, rubble, abandoned utility lines, etc.) is encountered during excavations, it must be removed in its entirety or at least where it is encountered below the new footings and replaced with compacted granular engineered fill. If old construction debris is encountered within the proposed on-grade slab area, it should be removed to a depth of at least 18 inches below the final subgrade elevation; any slabs or pads encountered below this depth should be thoroughly broken up prior to the placement of new engineered fill to allow for passage of water.

Material for backfill or engineered fill required to achieve design grades should preferably consist of free-draining and well-graded non-organic granular soils, such as soils meeting the requirements of Michigan Department of Transportation (MDOT) Class II or equivalent granular material. The on-site granular soils that are free of organic matter and other deleterious materials may be used for engineered fill materials provided they are approved by a qualified representative of the project owner and placed under favorable weather conditions to control moisture.

Engineered fill should be placed in uniform horizontal lifts, the thickness of which is compatible with the type and condition of material being placed, area of placement and type of compaction equipment being used. In general, we recommend that lifts be placed in 12 inches (or less) in loose thickness for materials being compacted with a medium smooth non-vibratory roller for granular soils. Other types of compaction equipment may require reducing lift thickness in order to achieve suitable compaction. Within structural areas, the fill should be compacted to achieve a density of at least 95 percent of Maximum Dry Density (MDD) as determined by the Modified Proctor compaction test (ASTM D 1557). All fill material should be placed and compacted at or near optimum moisture content. In-situ density tests should be performed to verify that proper compaction is achieved. Frozen material should not be used as fill, nor should fill be placed on a frozen subgrade.

Extreme care must be exercised when making excavations close to any existing facilities include below grade utilities, vaults, tunnels, conduits, drains, influent or other piping and/or other nearby facilities to prevent undermining or damage to the supported facilities. Open excavations for new footings or pipe trenches should not extend below the bearing level of any adjacent footings or pipe inverts. If excavations must be extended deeper than any existing footings or pipe/tunnel inverts, provisions should be made either to underpin the existing footings and inverts or to provide lateral support system to prevent movement of existing structures during the time the nearby excavations are open. For conventional footings, the support measures may include temporary support systems or underpinning of any existing footings or inverts adjacent to open excavations. Furthermore, if the new footings are located within the zone of influence of the existing footings or other below grade structural elements such as pipe/tunnel inverts and floor slabs, it will also be necessary to account for the loading of the existing footings or these other below grade structures on the support systems as well as the new foundations. The zone of influence of a footing may be considered to extend from the edge of the footing bottom in a downward direction away from the footing at a slope of 1 unit horizontal to 1 unit vertical (1H: 1V).

The subgrade resulting from the satisfactory completion of site and subgrade preparation operations can be used for supporting on-grade concrete floor slabs. However, if the existing fill materials are allowed to remain below the floor slab, some settlement of the floor slab cannot be precluded. To virtually eliminate settlement of the floor slabs, the existing fill materials would need to be removed in their entirety and be replaced with engineered fill. In addition, we recommend that all floor slabs be suitably reinforced and separated from the building foundation system to allow for independent movement. All ground-supported floor slabs placed beneath or around the building should be underlain by a base course layer consisting of a minimum of 6 inches of crushed limestone aggregate material such as MDOT 6AA compacted to at least 95 percent of the Maximum Dry Density (MDD) as determined by the ASTM D 1557 Modified Proctor test. Prior to the placement of base course layer and concrete, the upper 12 inches of surficial subgrade soils anticipated below the slabs should be scarified, moisture-conditioned and re-compacted to at least 95 percent of MDD as determined by

ASTM D 1557. The final site grades should be oriented to drain storm water and/or other surface runoff away from the floor slabs and any nearby existing structures.

7.0 FOUNDATION RECOMMENDATIONS AND SITE CLASS

The following recommendations have been developed on the basis of the previously described project characteristics and our evaluation of the subsurface conditions encountered during the current investigation. If there is a change in the project characteristics, including anticipated structure loads and building location at the site, a review should be made by our office.

Shallow Foundations

Soil conditions encountered within the borings completed at the location of the proposed dewatering building consist of about 12 feet deep existing fill materials over very stiff to hard native cohesive soil deposits which, in turn, are underlain by very dense granular soils. The existing fill materials are prone to settlement upon application of additional loads and, therefore, are not considered suitable for support of conventional shallow foundations. We recommend that building foundations not be supported on this existing fill. It is recommended the existing fill materials be undercut in their entirety where they are encountered at the building footing locations and replaced with structural or engineered fill consisting of new compacted granular materials placed over suitable native soils. The intent of undercutting is to minimize the risk of settlements and to provide a uniform bearing surface for foundation support via a uniform thickness of compacted fill beneath load bearing elements. Engineered fill placement should be performed in strict accordance with the Site Preparation recommendations discussed above in Section 6.0 of this report.

Following undercutting and replacement with compacted engineered fill, the proposed building may be supported on conventional shallow foundations bearing in the new compacted engineered fill materials. A net allowable soil bearing pressure of 2,500 pounds per square foot (psf) may be used for the design of shallow building footings that bear upon approved compacted engineered fill placed over suitable native soils. Lean concrete (2,000 pounds per square inch mix or better) may be used as an alternative to compacted granular fill if approved by the structural engineer. The allowable bearing pressure may be increased by 33 percent for short term loading due to wind or seismic forces. Due to the presence of existing undocumented fills at the site, shallow foundations will require substantial amounts of undercutting and additional measures to protect any nearby above or below-grade existing facilities or structures. Further, it is noted that the depth of existing fill materials may vary significantly between or away from the borings completed for the current investigation.

All strip footings should be at least 18 inches in width and all isolated spread footings should be at least 30 inches in their least dimension regardless of the resultant bearing pressure. The footings should be established at a depth of at least 3½ feet below exposed finished grade for protection against frost penetration. The determination of the required depth of excavation at each footing location should be performed by a qualified representative of the project owner. We recommend that all foundation excavations should be checked and tested in the field to verify that adequate in-situ soil bearing pressures, compatible with the recommendations outlined in this report, are achieved. Should loose soils be present at the base of footing excavations, these soils should be densified in place using a plate compactor or hoe-pak in order to compact the soils and improve their bearing capability, or the loose soils must be removed until suitable bearing soils are achieved or should be replaced with engineered fill.

It is imperative that the sidewalls of the building footing be maintained vertical during the concrete pour. If footings for the new structure are constructed by directly placing concrete in unformed excavations (trench footings), the footing may become wider at the top as sloughing of upper surrounding granular fill occurs and will develop a “lip” or flare outward. Such condition may cause the footing to heave despite the bottom of the footing being below the frost depth, due to the frozen soils lifting the upper (wider) portions of the footing. As such, if the footing excavations cannot be maintained with vertical sides, we recommend the use of formwork to construct the footing under these conditions.

Soils exposed in the bases of all satisfactory foundation excavations should be protected against detrimental change in condition such as from disturbance, precipitation and freezing. Surface runoff water should be drained away from the excavations and not allowed to pond. Foundation excavations should be concreted as soon as practical after they are excavated. If possible, all footing concrete should be poured the same day the excavation is made. If an excavation is left open for an extended period, a thin mat of lean concrete should be placed over the bottom to minimize damage to the bearing surface from weather or construction activities. Foundation concrete should not be placed on frozen or saturated subgrades.

As discussed earlier, extreme care must be exercised when making excavations close to existing facilities including product piping, drains, conduits, tunnels, vaults, utilities, on-grade equipment or other nearby surface/below-grade structures to prevent undermining or damage to the supported facilities. Open excavations for new footings should not extend below the bearing level of any adjacent footings or pipe/tunnel inverts. If excavations must be extended deeper than any existing footings or inverts, provisions should be made either to underpin these below-grade structural elements or to provide lateral support system to prevent movement of existing structures during the time the nearby excavations are open. For conventional footings, the support measures may include temporary support systems or underpinning of any existing footings, floor slabs or pipe/tunnel inverts adjacent to open excavations. Furthermore, if the new building foundations are located within the zone of influence of any existing footings or other below-grade structural elements, it will also be necessary to account for the loading of existing structural elements on the support systems as well as the new foundations. The zone of influence of a footing may be considered to extend from the edge of the footing bottom in a downward direction away from the footing at a slope of 1H: 1V.

Nearby foundation elements bearing at different levels should be designed and constructed so that the least lateral distance between them is equivalent to or greater than the difference in their bearing levels. To achieve a change in the level of a strip footing, we recommend the footing be gradually stepped at a grade no steeper than two units horizontal to one unit vertical (2H: 1V).

Resistance to lateral loads may be provided by frictional resistance between the bottom of concrete footings and the underlying soils and by passive soil pressure against the sides of the footings. The coefficient of friction between poured-in-place concrete footings and underlying soils may be taken as 0.30. Passive pressure available in compacted fill or undisturbed native soils may be taken as equivalent to the pressure exerted by a fluid weighing 200 pounds per cubic foot (pcf). The above recommended values include a factor of safety of 1.5; therefore, frictional and passive resistance may be used in combination without reduction.

Total settlements of spread and strip footings will vary, depending on the size of the footing and the actual load supported. Footing settlements have been estimated based on anticipated loading conditions. If the recommendations outlined in this report are followed, total and differential settlements of the new building supported on shallow foundations are anticipated to be within approximately 1 inch and ½ inch, respectively. As a precaution, structural and utility connections to new construction supported on shallow foundations should be deferred until a majority of the dead load resulting from construction has been applied. Careful field control during construction will substantially reduce the actual settlements that occur. It is imperative that all fill and backfill materials placed beneath, above and against the sides of the foundations be thoroughly compacted at appropriate moisture content and density as described in the Site Preparation section of this report. We recommend that all footings be suitably reinforced to reduce the effects of normal differential settlements associated with local variations in subsoil conditions.

Alternate Drilled Pier Foundations

If the use of conventional shallow foundations is considered to be impractical due to the presence of significant depth of existing fill materials, limited site space or close proximity to nearby structures, below-grade utilities, tunnels and other adjacent structural elements, the proposed building may be supported using an alternate foundation option consisting of a system of shallow straight shaft drilled piers bearing in suitable native soils consisting of very stiff to hard silty clays underlying the existing fill materials at the site. Based on the subsoil conditions encountered at the boring locations, suitable native bearing soils for drilled pier foundations designed to derive support from end bearing are anticipated at a minimum depth of about 15 feet below existing surface. We recommend the pier foundations at the proposed building location be extended through the upper fill soils to bear into the lower suitable native clay soils. Drilled piers supported by very stiff to hard native clay soils anticipated at or near Elevations 701½ to 702 feet may be designed based on a net allowable bearing pressure of 3,000 psf provided the piers are established to a minimum embedment depth of 15 feet and with a diameter of 36 inches or less. The excavations for drilled piers should not be extended deeper than necessary during construction to avoid disturbing the native clay soils or not leaving a sufficient thickness of suitable clay soils between the bearing level of pier foundations and the lower clay soils.

In all cases, the drilled piers must bear at a depth of at least 3½ feet below the surrounding ground surface for protection against frost penetration. The weight of the buried portion of the pier may be ignored when calculating downward axial loads on the pier. A one-third increase in the allowable capacity may be used for consideration of transient loads such as wind or seismic. If deemed necessary by the structural engineer, consideration may be given to use of grade beams to span between the piers and to provide additional lateral resistance and maintain foundation alignment and integrity. If utilized, the grade beams should be founded at a depth of at least 3½ feet for protection against frost heave.

For resisting lateral loads, drilled piers may be designed by using a passive soil resistance of 200 psf/ft of pier embedment depth up to a maximum of 1,800 psf. The allowable lateral pressures recommended herein are based on anticipated subsoil conditions and that the drilled piers will be located no closer together than three pier diameters on-center. If pier foundations are installed at a spacing of less than 3 pier diameters, it will be necessary to revise and reduce the lateral pressure recommendations above. Regardless of the materials encountered, no passive soil resistance should be considered to a depth equal to one pier diameter. The above pressures should be considered to act only on the projected area of the foundation. The magnitude of lateral movement of drilled piers subjected to overturning loads is dependent upon and is a non-linear function of the magnitude of the applied lateral load and load eccentricity with respect to the ground surface. The drilled pier foundations should be designed to maintain both force and moment equilibrium.

Depending on their depth and location, the excavations for drilled piers are anticipated to extend through the upper existing fill materials and terminate in underlying native clay soils. Excavation bottom soil conditions may consist of variable consistency and/or relative density soils depending upon the final excavation depth. The pier foundation excavations should be evaluated at the time of construction for bottom stability based on actual site soil conditions. All pier foundation excavations should be checked and tested to verify that adequate in-situ soil bearing pressures, compatible with the design value, are achieved. The determination of the required depth of excavation at each pier foundation location should be performed by a qualified person. If sufficient bearing pressure is not available at the design bearing level, the pier size will need to be increased in the field until suitable bearing soils are encountered in order to adequately accommodate the structural loads. Because the piers are anticipated to be end bearing, all loose material should be removed from the pier excavation, prior to placement of reinforcing steel and concrete.

Groundwater was observed during drilling at approximate depths of 28½ feet and 29½ feet below ground surface (about Elevations 688½ feet and 687 feet) at boring locations B-1 and B-2. Upon completion, groundwater levels rose to as much as 6 to 7 feet depth (about Elevation 710 feet) below ground surface within the borings. In addition, borings encountered existing sandy fills extending to a depth of about 12 feet below surface, which could provide a natural flow path for groundwater to enter drilled pier excavations. If drilled pier foundations are used on the project, the pier excavations are expected to extend to a depth of at least about 15 feet. Therefore, possibility exists that groundwater may be encountered during pier excavations for the project. It will be necessary to control groundwater inflows into pier excavations. Fluctuations in the groundwater levels and/or trapped water within the granular soils should be anticipated especially after a precipitation event and with seasonal variations following periods of prolonged precipitation. Therefore, the actual water levels at the site may vary at the time of construction.

Caving was observed in the soil borings at approximate depths of 26 to 28 feet (Elevations 690½ to 689 feet). Therefore, to prevent the sides of the pier excavations from collapsing and to control groundwater seepage from the granular soils, a temporary steel casing will be required in the construction of the drilled piers. The casing should be extended into the suitable native clay stratum in order to seal the casing to prevent soil or water intrusion into the pier shaft excavation, prior to placing reinforcing steel and concrete. The casing may be full or partial depth depending upon the subsurface conditions encountered at the time of construction. We recommend that the design plans indicate that casing (or sufficient side support) be required for all pier excavations for the installation of pier foundations. The casing could be drilled (i.e., twisted) into position prior to shaft excavations, such that the surrounding soils providing drilled pier lateral resistance to structure loads do not experience significant disturbance. At locations, where drilled piers will be installed in close proximity to any nearby structures, below-grade utilities or other facilities, it is recommended that the piers be installed by twisting the casing into the ground instead of driving or vibrating as the excavation proceeds. Additional discussion regarding pier installation construction methods is provided below.

The selection of the location of final pier foundation elements for the new building will be influenced by factors including prevailing subsurface conditions at the pier locations, presence of utilities and proximity to other above or below-grade facilities. Based on our site observations and the MISS DIG markings noted in the field at the time of our field explorations, as well as the presence of existing facilities anticipated within or near the proposed construction area, extreme care must be exercised by the contractor when performing excavations near these existing facilities that are to remain in order to protect them from potential damage. The contractor should be fully aware of the locations of all existing above or below-grade facilities such as utilities, piping and tunnels before excavating for the pier foundations and be prepared to support or brace the excavations as required so that these existing facilities are not impacted by the construction and they also do not impede the construction operations. As an added precaution, we recommend that the contractor be required to review readily available historical utility maps and as-built drawings for all nearby structures at the local city or county offices in addition to any utility plans and as-built drawings made available by the project owner prior to approval of the final layout of the pier foundation locations. Furthermore, detailed construction procedures should be submitted by the contractor for review and approval by the engineer.

It is anticipated that if drilled pier foundations are used, the project will involve construction-related activities including installation of temporary casing, pier shaft excavations and compaction of soils around the pier foundations. To prevent risk of damage to any nearby facilities, utilities and other improvements from excessive caving or potential ground loss, or vibrations associated with these construction activities, the contractor should take all precautions in selecting appropriate construction means and methods. Furthermore, the contractor should exercise utmost care in selecting the type of method used for installing the temporary casing in order to prevent settlement of or damage to adjacent facilities. Use of vibratory hammers to install casing is not recommended and should not be allowed on the project. This is to prevent potential densification of the granular soils and associated settlements of any nearby utility lines and/or other adjacent facilities that could result from construction methods involving use of vibratory hammers or other types of equipment producing significant vibrations. A qualified representative of the project owner should be onsite during drilled pier excavation and installation operations to continuously observe and verify suitable bearing materials are encountered, bearing surface has been properly cleaned, piers are plumb and appropriate construction methods are employed for installation of drilled piers.

After the pier excavation is complete, the hole has been cleaned and reinforcement set, concrete should be placed using a concrete pump or by tremie method. It is important that the concrete be placed and casing removed in such a manner as to prevent necking of the drilled pier. Segregation of aggregate during concrete placement should be minimized. When withdrawing the casing during concreting operations, special care should be taken to prevent water and/or soil intrusion into the pier excavation. To prevent such intrusion, a sufficient level of concrete should be maintained above the tip of the casing as it is being withdrawn so as to offset any hydrostatic pressure head that may exist in granular soils above the bearing level.

The drilled pier foundations should be installed by an experienced and competent contractor who will also be responsible for properly installing the piers in accordance with applicable industry standards and generally accepted methods without causing deterioration of subgrade conditions or impacting any existing adjacent structures that are to remain. With regard to the drilled pier construction, the duration of construction must be accounted for. We recommend that construction methods make certain that the pier excavation is not left open overnight prior to placing of concrete. To reduce the lateral movement of the pier shaft, all voids or enlargements in the drilled pier shaft due to over-excavation (if any) or caving soil conditions must be filled with concrete at the time the drilled pier concrete is placed. Initial set of concrete should be achieved before an adjacent pier is drilled.

Provided the site and subgrade preparation and foundation recommendations in this report are followed, the total and differential settlements for individual pier foundations are estimated to be less than 1 inch and ½ inch, respectively. We recommend that all drilled piers be suitably reinforced to withstand the effects of vertical compression and lateral overturning loads. The structural design of drilled piers and/or grade beams including reinforcement details should be performed by the project structural engineer.

Seismic Site Class

In general, seismic activity and earthquake potential throughout the State of Michigan is considered to be somewhat low. General industry guidelines were followed in estimating the seismic site classification for the project. The site classification is typically based on the relative stiffness for soil and rock layers within a 100-foot soil/rock column. However, none of the borings completed for this investigation were taken to a 100-foot completion depth. Therefore, the idealized soil/rock column used for estimating seismic site classification is based on our review of the regional geologic conditions and past project experience in the general site vicinity. Based on the review of subsurface conditions encountered in the borings completed for this investigation, as well as our knowledge of regional geologic setting and general subsurface conditions from past project experience in the site vicinity, the subject site may be classified as Site Class D in accordance with the definitions given in Section 1613.5.2 of the 2009 Michigan Building Code, which is based on the 2012 International Building Code®.

8.0 SUPPORT OF FLOOR SLABS

The subgrade resulting from the satisfactory completion of site preparation activities recommended in Section 6.0 above can be used for the support of concrete floor slabs-on-grade anticipated for the proposed building. However, due to existing underlying fill materials and other variations within subsoils at the proposed building location, the possibility of some floor slab settlement cannot be precluded. Accordingly, we recommend that all ground-supported concrete floor slabs be suitably reinforced and separated from the foundation system to allow for independent movement. Further, in order to protect the subgrade soils from construction-related disturbances, reduce differential settlements of the existing fill materials and equalize moisture conditions beneath the slab as well as provide a stable working platform, it is recommended that the floor slab be supported on a minimum 6-inch well-compacted layer of free draining granular base course material such as MDOT 6AA (coarse aggregate). If no floor slab settlement can be tolerated, alternate measures such as complete removal of existing fill materials or structural support of floor slabs using grade beams will be necessary. Provided that the site conditioning recommendations outlined in the Site Preparation section of this report are followed and a minimum of 6-inch granular base is placed beneath the floor slabs, a modulus of subgrade reaction value of 100 pounds per cubic inch (pci) may be used for the design of floor slabs supported on existing fill at the site. This estimated value corresponds to a 1 foot by 1 foot plate load test.

Based on our past experience from other projects involving construction of on-grade concrete slabs, there is sometimes substantial time lag between initial grading and the time when the contractor is ready to construct the slab-on-grade. Even though the subgrade soils may have been prepared and compacted adequately during initial grading, exposure to weather and construction traffic can impact the integrity of subgrade soils. Therefore, prior to the construction of on-grade concrete slabs for the new building, the floor slab subgrade should be closely evaluated by a qualified representative of the testing agency. We suggest that provisions be included in the project specifications for the contractor to restore the subgrade soils to an acceptable condition prior to construction of slabs. Such restoration may include moisture conditioning of the surficial soils and re-compaction to the project requirements.

If the floor slab is to be covered with moisture sensitive flooring or coatings, consideration should be given to the use of a 4-inch thick layer of sand underlain by a no less than 10-mil thick plastic sheet vapor barrier beneath the floor slab. The placement of vapor retarder/barrier should be in accordance with the project specific needs, current version of the American Concrete Institute (ACI) 302.1 guidelines, local building codes and the recommendations of the flooring manufacturer. Special care should be exercised during construction activities to prevent damage to the vapor retarder.

9.0 TEMPORARY EXCAVATIONS

Excavations for the project should comply with the current Michigan Department of Labor and Regulatory Affairs (LARA) requirements, *i.e.*, the Michigan Occupational Safety and Health Act (known as MIOSHA) and related Federal OSHA regulations, as well as any additional local regulations or owner requirements must be strictly followed and adequate protection provided for workers and adjacent structures. We are providing the information below solely as a service to our client. Under no circumstances should the information provided herein be inferred to mean that GeoTran is assuming responsibility for temporary excavations, construction site safety, activities of the contractor, or for design, installation, maintenance and performance of any shoring, bracing, underpinning, or other similar systems. Such responsibility is not implied and should not be inferred.

All cuts deeper than 5 feet should be properly sloped or otherwise structurally retained to provide stable and safe working conditions. Construction site safety generally is the sole responsibility of the contractor. The contractor is also solely responsible for designing and constructing stable, temporary excavations and must shore, slope, or bench the sides of the excavations as required to maintain stability of both excavation sides and bottom. The contractor should be aware that slope height, slope inclination, and excavation depths should in no case exceed those specified in the local, state, or federal safety regulations including OSHA Health and Safety Standards for Excavations, 29CFR Part 1926, or successor regulations. Excavations must be performed and evaluated under the supervision of the contractor's designated competent person. The competent person must verify the soil conditions based on actual materials encountered during excavation activities and field conditions at the time of excavation in order to determine the permissible temporary slope inclinations. In areas, where there is insufficient space to allow for proper side slopes for excavations due to adjacent structures, utilities or other surface or below-grade facilities, vertical walls with properly designed and installed lateral bracing, or a combination of slopes and braced vertical walls may be used. The contractor should thoroughly review the site conditions and available as-built drawings for all existing facilities located within or directly adjacent to the construction area, be aware of existing utility locations, tunnels, piping, vaults, and adjacent buildings before initiating excavation activities; and be prepared to support or brace the existing facilities, as appropriate.

With time and the presence of seepage and/or wet weather, the stability of temporary cuts can be significantly reduced. Therefore, construction should proceed as quickly as possible to limit the time the excavations are left open. In addition, runoff water should be prevented from entering the excavations, by collecting and disposing of outside the construction limits. To prevent runoff from adjacent areas from entering the excavation, a perimeter berm may be constructed at the top of the excavation or slope. Additionally, temporary cut slopes, where utilized, should be covered with plastic or Visqueen sheeting to help minimize erosion during wet weather and closely observed in the field until the foundation installation and backfilling activities are complete.

Construction traffic and excavated material stockpiles should be kept away from excavations a minimum distance equal to the full depth of the excavation, unless the resulting surcharge loads are accounted for in the design of the lateral bracing system. In addition, the effect of the existing building foundations, buried piping, tunnels or any other nearby structures must also be considered in the design of the bracing system. The contractor's proposed excavation plan, support systems and sequence of construction should be reviewed by a qualified engineer prior to allowing the contractor to commence work.

A pre-construction survey for the project should be considered owing to the close proximity of the site or construction area to existing nearby facilities. The survey should record the elevation and horizontal position of all existing installations directly bordering the construction area and may consist of photographs, videotaping, etc. Vibration monitoring may be considered if heavy construction equipment capable of producing substantial vibrations is utilized on the project. Furthermore, if shallow conventional footings that will require significant amounts of undercutting and replacement with engineered fill are selected for the new building, a settlement survey should be performed on a weekly basis during excavation and on a monthly basis, approximately one month after the excavations have been completed, at a minimum.

10.0 GROUNDWATER CONTROL

Groundwater was encountered within borings at approximate Elevations of 688½ to 687 feet during drilling and at about Elevation 710 feet upon completion of borings. Based on the water level data obtained in the borings and depending upon the location of groundwater or water-bearing granular strata and the amount of precipitation prior to and during construction, appreciable amounts of groundwater infiltration may be encountered within shallow footing construction excavations for the project. Groundwater levels are subject to seasonal, climatic and other variations and may be different at other times and locations than those stated in this report. We anticipate that accumulations of surface water runoff or groundwater seepage in foundation excavations can be controlled by conventional dewatering methods such as standard pumping from small dug sumps formed at the base of the excavations and located outside of the zone of influence of footings, and provided that inflows from any overlying saturated granular seams and layers are controlled. In addition, a layer of crushed concrete, coarse aggregate or mud mat may be required to stabilize wet soils at the bottom of excavations.

Excavations that terminate more than one to two feet below groundwater level are expected to encounter moderate to heavy volumes of groundwater. In addition, a "quick" condition may develop as groundwater migrates towards excavations, resulting in the disturbance of soils, and a reduction in their supporting capabilities. Based on these considerations and because standard sump and pump methods may not be adequate, positive or special groundwater control measures, such as dewatering wells or well points will be required before making excavations below the groundwater level. It should be noted that granular soils that contain more than about 15 percent silt and clay will be difficult to effectively dewater and may require closely spaced dewatering systems. Dewatering systems, where used, should be properly designed to prevent soil fines from being pumped out of the subsurface soil layers. The discharge water from the dewatering system should be monitored to verify that this condition does not develop. In addition, consideration should be given in the design of the dewatering system such that the groundwater levels are not drawn down too deep so as to affect the existing structures. The contractor should be prepared to provide a dewatering system during construction that is capable of maintaining dry excavations.

In general, where groundwater is encountered within foundation excavations for drilled piers (if used), temporary casing is recommended to control groundwater seepage and to keep the existing granular fills from collapsing into the pier excavation. It is important that the contractor be prepared for installing drilled piers below the groundwater level in saturated granular soils. Dewatering of wet shafts should not be allowed and is not recommended. If the use of temporary casing is found to be ineffective in controlling groundwater inflows, it may be necessary to use bentonite slurry to stabilize the excavation sides and/or bottom. The use of slurry will require that concrete be placed within the pier by tremie method. Additional information regarding groundwater control measures in drilled pier excavations is discussed in Section 7.0 above.

11.0 DATA REVIEW AND FIELD VERIFICATION

We should be provided the opportunity to review geotechnical portions of the final plans and specifications. The purpose of the review will be to verify that the intent of our recommendations set forth in this report have been correctly interpreted and included in the design of the project.

We recommend that a qualified firm be retained to provide observation and testing services during the earthwork and foundation construction phases of the proposed project. This is to verify the anticipated subsurface conditions are present and observe compliance with the design concepts, specifications and recommendations. Also, field verification allows appropriate design or construction changes to be made in a timely manner if conditions differ from those anticipated prior to the start of construction.

12.0 LIMITATIONS

We have prepared this report exclusively for Wade Trim, Inc. for the project specifically described in this report. Our professional services have been performed, our findings obtained and our recommendations prepared in accordance with the generally accepted geotechnical engineering practice, as it exists in the project area at the time of our study. No other warranty or representation, expressed or implied, is included or intended in this report.

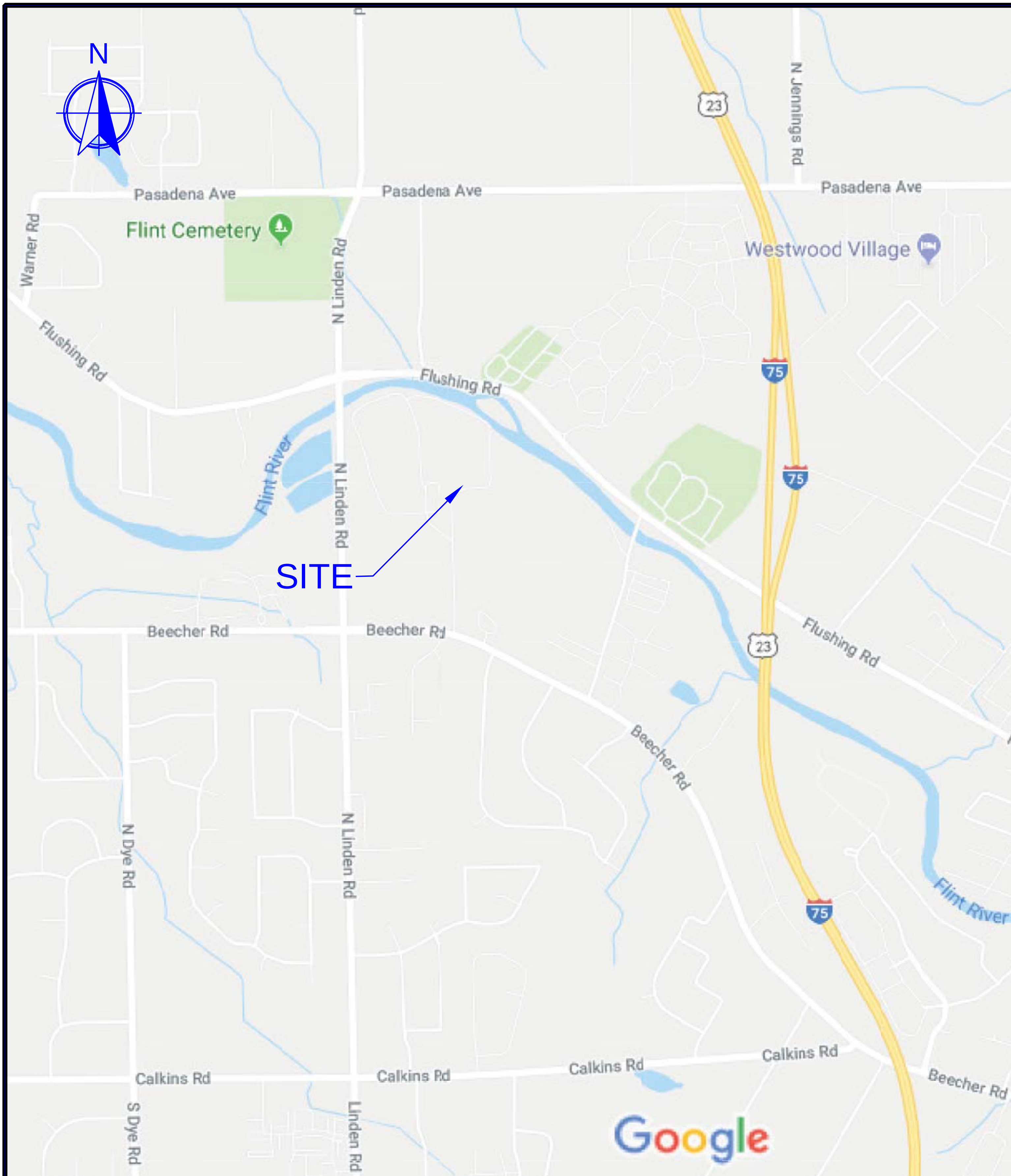
Our recommendations for this project were developed utilizing subsurface information from the soil borings performed at the site. At this time, we would like to note that borings only depict the subsurface conditions at the specific locations and time at which they were made. The subsurface conditions at other locations on the site may vary from those occurring at the boring locations that we have explored to date. If significant variations are exposed during construction, they should be brought to our attention as it may be necessary for us to reevaluate the recommendations of this report.

The conclusions and recommendations presented in this report have been developed based upon the data obtained from the borings and our current understanding of the proposed construction. Any revision in the plans for the proposed construction from those anticipated in this report should be brought to our attention to determine whether any changes in the foundation or site preparation recommendations are necessary. This report reflects our opinion as of this date, based on the results of the study described herein and on the information provided during the course of the study. The results of this study may not be relied upon by entities other than those identified above without the prior knowledge and written consent of GeoTran.

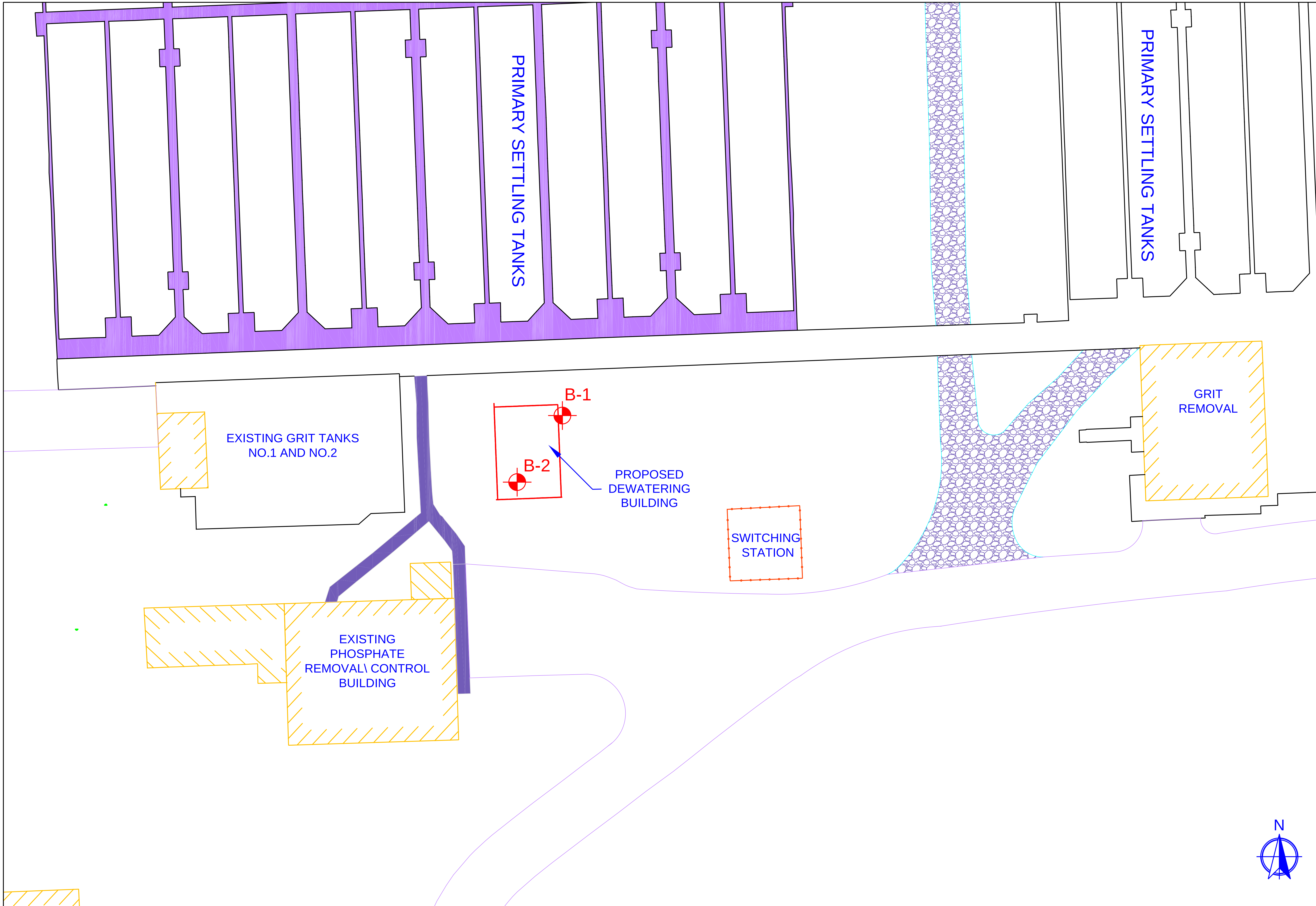
The scope of the current study was limited to geotechnical exploration of the subsurface conditions for the support of the proposed dewatering building and other related aspects of construction. No chemical, environmental, or hydrologic testing or analyses were performed as part of this study.

GeoTran
Consultants, LLC

APPENDIX



GeoTran Consultants, LLC GEOTECHCNIAL ENGINEERING AND CONSULTING SERVICES 10315 E. GRAND RIVER SUITE 201 BRIGHTON, MI 48116 PHONE: 810-229-6805 FAX: 810-229-6561 WWW.GEOTRANCONSULTANTS.COM	SITE LOCATION MAP PROPOSED DEWATERING BUILDING FLINT WASTEWATER TREATMENT PLANT FLINT, MICHIGAN	PROJECT NO.: 17-09002G-10	FILE NAME: 17-09002G-10_001.DWG	
		DRAWN BY: M. LUCKHAM	DATE: 1-22-2018	FIGURE NO.: 1
		REVIEWED BY: D. YIP	DATE 1-22-2018	
		SCALE: NOT TO SCALE	SHEET NO.: 1 OF 1	



GeoTran
Consultants, LLC

GEOTECHNICAL
ENGINEERING AND
CONSULTING SERVICES

10315 E. GRAND RIVER,
SUITE 201
BRIGHTON, MICHIGAN 48116

PHONE: (810)-229-6805
FAX: (810)-229-6561

WWW.GEOTRANCONSULTANTS.COM

BORING LOCATION PLAN
PROPOSED DEWATERING BUILDING
FLINT WASTEWATER TREATMENT PLANT
FLINT, MICHIGAN

LEGEND:

B-1 SOIL BORING LOCATION

SOIL BORINGS WERE PERFORMED BY DLZ/AMERICAN DRILLING ON JANUARY 18, 2018 UNDER THE OBSERVATION OF GEOTRAN CONSULTANTS, LLC.

DRAWING REFERENCE: BASE PLAN PROVIDED BY WADE TRIM.

PROJECT NO.: 17-09002G-10
DRAWN BY: M. LUCKHAM
DATE: 02-07-2018
REVIEWED BY: D. YIP
DATE: 02-08-2018
FILE: 17-09002G-10_002.dwg
SCALE: 1 IN = 30 FT
FIGURE NO.: 2

SHEET 1 OF 1

GeoTran Consultants, LLC

GENERAL NOTES

TERMINOLOGY

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

SAMPLE DESIGNATIONS

AS - Auger Sample - directly from auger flight
BS - Miscellaneous Sample - bottle or bag
SS - 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
VS - Vane Shear
HA - Hand Auger Sample
PID - Photo Ionizataion Detector

PARTICLE SIZES

Boulders -	Greater than 12 inches (305mm)
Cobbles -	3 inches (76.2mm) to 12 inches (305mm)
Gravel - Coarse -	3/4 inches (19.05 mm) to 3 inches (76.2mm)
Fine -	No. 4 - 3/16 inches (4.75mm) to 3/4 inches (19.05 mm)
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)

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

COHESIVE SOILS

Consistency	Unconfined Compressive Strength (psf)	Approximate Range of (N)
Very Soft	Below 500	0 - 2
Soft	500 - 1000	3 - 4
Medium Stiff	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 Dense	36 - 65	11 - 30
Dense	66 - 85	31 - 50
Very Dense	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.

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.

DEPOSITIONAL FEATURES

Parting -	as much as 1/16 inch thick
Seam -	1/16 inch to 1/2 inch thick
Layer -	1/2 inch to 12 inches thick
Stratum -	greater than 12 inches thick
Pocket -	small, erratic deposit of limited lateral extent
Lens -	lenticular deposit
Varved -	alternating seams or layers of silt and/or clay and sometimes fine sand
Occasional -	one or less per foot of thickness
Frequent -	more than one per foot of thickness
Interbedded -	applied to strata of soil or beds of rock lying between or alternating with other strata of different nature

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).

LOG OF SOIL BORING NO.: B-1

GeoTran Consultants, LLC

Project Name: Proposed Dewatering Building

Project Location: Flint Wastewater Treatment Plant, Michigan

Project Number: 17-09002G-10

Client: Wade Trim

Date: 1/18/2017

GROUND SURFACE ELEVATION: 717.0 ft ±

ELEV. (ft)	SAMPLE TYPE/ NUMBER	REC. (in.)	BLOWS/ 6 INCHES	STD. PEN. RESIST. N-VALUE	POCKET PEN. (psf)	PROFILE DESCRIPTION	DEPTH (ft)	MOIST. CONT. (%)	DRY DENSITY (pcf)	UNCONF. COMP. ST. (psf)	LIQUID LIMIT	PLASTICITY INDEX	Loss on Ignition (%)
716.5						TOPSOIL: Dark Brown CLAYEY SAND with Organic Matter	0.5						
715	SS-1	18	14 16 9	25		FILL: Dark Brown SANDY CLAY with Trace of Gravel							
	SS-2	18	4 5 3	8			5						
710	SS-3	14	6 6 8	14									
	SS-4	18	6 5 5	10			10						
705 / 705.0						Very Stiff to Hard Gray SILTY CLAY with Trace of Sand, Gravel and Occasional Silty Sand Layers	12.0						
	SS-5	18	5 7 9	16	6500		15						
700													
	SS-6	0	11 10 15	25			20						
695													
	SS-7	18	9 16 19	35	>9000		25						

Stratification lines represent approximate boundaries; In-situ, transition may be gradual.

Total Drilling Depth: 38.9 ft

Drilling Contractor: DLZ/American Drilling

Driller: V. Dearing

Drilling Method:
CME 75 Truck Mounted Drilling Rig, Using 2-1/4-inch I.D. Hollow Stem Auger to End of Boring.

Backfill Procedure:
Borehole backfilled with excavated materials.

Groundwater Levels:

At Time of Drilling: 28.5 ft

End of Drilling: 7 ft ; Caved at 28 ft

Notes:

Logged By: D. Yip **Reviewed By:** M. Luckham

Figure No.: 4

LOG OF SOIL BORING NO.: B-1

GeoTran Consultants, LLC

Project Name: Proposed Dewatering Building

Project Location: Flint Wastewater Treatment Plant, Michigan

Project Number: 17-09002G-10

Client: Wade Trim

Date: 1/18/2017

SAMPLE DATA						PROFILE DESCRIPTION			LABORATORY DATA					
ELEV. (ft)	SAMPLE TYPE/ NUMBER	REC. (in.)	BLOWS/ 6 INCHES	STD. PEN. RESIST. N-VALUE	POCKET PEN. (psf)	GRAPHIC LOG	GROUND SURFACE ELEVATION: 717.0 ft ±	DEPTH (ft)	MOIST. CONT. (%)	DRY DENSITY (pcf)	UNCONF. COMP. ST. (psf)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	Loss on Ignition (%)
690														
	SS-8	18	10 14 16	30	>9000		Very Stiff to Hard Gray SILTY CLAY with Trace of Sand, Gravel and Occasional Silty Sand Layers	30						
685							685.0 / 32.0							
	SS-9	10	49 50/4"	50/4"			Very Dense Gray SILTY SAND with Little Clay and Gravel	35						
680														
	SS-10	5	100/5"	100/5"			678.1 / 38.9							
							End of Boring at 38.9 ft.							
675														
670														
665														
660														

LOG OF SOIL BORING NO.: B-2

SHEET 1 OF 2

GeoTran Consultants, LLC

Project Name: Proposed Dewatering Building

Project Location: Flint Wastewater Treatment Plant, Michigan

Project Number: 17-09002G-10

Client: Wade Trim

Date: 1/18/2018

SAMPLE DATA						PROFILE DESCRIPTION			LABORATORY DATA					
ELEV. (ft)	SAMPLE TYPE/ NUMBER	REC. (in.)	BLOWS/ 6 INCHES	STD. PEN. RESIST. N-VALUE	POCKET PEN. (psf)	GRAPHIC LOG	GROUND SURFACE ELEVATION: 716.5 ft ±	DEPTH (ft) 0	MOIST. CONT. (%)	DRY DENSITY (pcf)	UNCONF. COMP. ST. (psf)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	Loss on Ignition (%)
						715.5	TOPSOIL: Dark Brown CLAYEY SAND with Organic Matter	1.0						
715	SS-1	18	4 5 9	14										
	SS-2	18	8 16 20	36				5						
710	SS-3	14	9 6 6	12			FILL: Brown SAND with Trace of Clay and Gravel							
	SS-4	18	3 3 6	9				10						
705						704.5		12.0						
	SS-5	18	4 6 7	13	4500			15						
700														
	SS-6	18	5 11 19	30	>9000		Very Stiff to Hard Gray SILTY CLAY with Trace of Sand, Gravel and Occasional Silty Sand Layers	20						
695														
	SS-7	18	5 12 16	28	>9000			25						

Stratification lines represent approximate boundaries; In-situ, transition may be gradual.

Total Drilling Depth: 38.8 ft

Drilling Contractor: DLZ/American Drilling

Driller: V. Dearing

Drilling Method:
CME 75 Truck Mounted Drilling Rig, Using 2-1/4-inch I.D. Hollow Stem Auger to End of Boring.

Backfill Procedure:
Borehole backfilled with excavated materials.

Groundwater Levels:

At Time of Drilling: 29.5 ft

End of Drilling: 6 ft ; Caved at 26 ft

Notes:
Boring was offset from the initital location due to concrete obstruction encountered at about 5.5 ft.

Logged By: D. Yip

Reviewed By: M. Luckham

Figure No.: 5

LOG OF SOIL BORING NO.: B-2

GeoTran Consultants, LLC

Project Name: Proposed Dewatering Building

Project Location: Flint Wastewater Treatment Plant, Michigan

Project Number: 17-09002G-10

Client: Wade Trim

Date: 1/18/2018

ELEV. (ft)	SAMPLE TYPE/ NUMBER	REC. (in.)	BLOWS/ 6 INCHES	STD. PEN. RESIST. N-VALUE	POCKET PEN. (psf)	GRAPHIC LOG	GROUND SURFACE ELEVATION: 716.5 ft ±	DEPTH (ft)	MOIST. CONT. (%)	DRY DENSITY (pcf)	UNCONF. COMP. ST. (psf)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	Loss on Ignition (%)
690	SS-8	18	4, 6, 11	17	5000		Very Stiff to Hard Gray SILTY CLAY with Trace of Sand, Gravel and Occasional Silty Sand Layers	30						
685							684.5	32.0						
	SS-9	5	50/5"	50/5"			Very Dense Gray SILTY SAND with Little Clay and Gravel	35						
680	SS-10	3	50/3"	50/3"			677.7	38.8						
							End of Boring at 38.8 ft.							
675														
670														
665														
660														

GeoTran Consultants, LLC

SUMMARY OF LABORATORY TEST DATA

PROJECT NAME: Proposed Dewatering Building

PROJECT NUMBER: 17-09002G-10

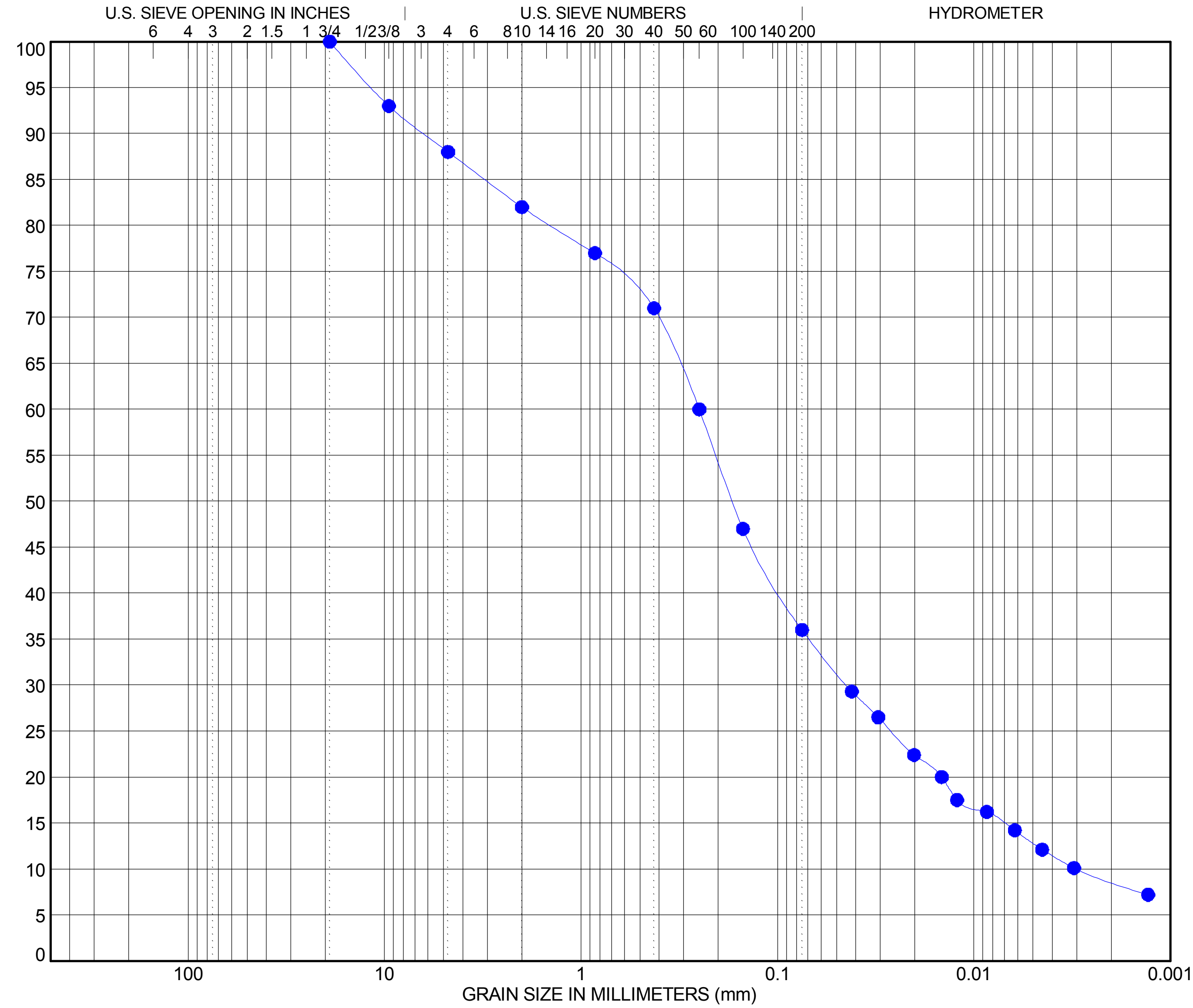
PROJECT LOCATION: Flint Wastewater Treatment Plant, Michigan

Boring Number	SAMPLE			LABORATORY ANALYSIS															
	Number	Depth (ft)	Elevation (ft)	Moisture Content (%)	Dry Density (pcf)	Grain Size Distribution (%)							Atterberg Limits			Loss on Ignition (%)	Unconfined Compressive Strength (psf)	Failure Strain (%)	Specific Gravity
						Gravel	Sand			Silt	Clay	%<#200 Sieve	Liquid Limit	Plastic Limit	Plasticity Index				
							Coarse	Medium	Fine										
B-1	SS-10	38.9	678.1	--	--	12	6	11	35	23	13	36	--	--	--	--	--	--	--

Figure No.: 6

GRAIN SIZE ANALYSIS

Soil Boring No.	Sample No.	Sample Depth	Sample Elev.	Description
B-1	**SS-10**	**38.9 ft**	**678.1 ft**	Gray SILTY SAND with Little Clay and Gravel



% COBBLES	% GRAVEL		% SAND			% SILT	% CLAY
	Coarse	Fine	Coarse	Medium	Fine		
0.0	0.0	12.0	6.0	11.0	35.0	23.2	12.8

D_{85}	D_{60}	D_{50}	D_{30}	D_{10}	COEFFICIENTS	
					Cc	Cu
3.0822	0.25	0.1688	0.045	0.003	2.64	83.10

ATTERBERG LIMITS		
LL	PL	PI
--	--	--

REMARKS:

Sampled By: D. Yip **Checked By:** M. Luckham

GeoTran Consultants, LLC

Project Name: Proposed Dewatering Building
Project Location: Flint Wastewater Treatment Plant, Michigan
Client: Wade Trim
Project Number: 17-09002G-10
Date: 1/18/2017

Figure No.: 7

GEOTRAN GRAIN SIZE CURVES - GEOTRAN STD-2012.GDT - 2/8/18 11:27 - C:\USERS\GEOTRAN CONSULTANTS\GOOGLE DRIVE\GEOTRAN -MAIN\PROPOSALS_PROJECTS\2017\17-09002G_FLINTWWTP\REPORT\17-09002G-10.GPJ