# GEOTECHNICAL ENGINEERING STUDY LOT C 

for

# Hudson Logistics Center Hudson, New Hampshire 

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## EXECUTIVE SUMMARY

In support of the proposed industrial park development in Hudson, New Hampshire, Langan conducted a geotechnical subsurface exploration and prepared a geotechnical engineering study to provide geotechnical design and construction recommendations. Specifically, this report addresses Lot $C$ within the overall development. The remaining two lots (Lots A and B) are addressed in separate reports.

Existing grades on the 111 acre lot generally slope down from the southwest to the east (about el +175 to +125 ). The design concept includes the construction of a distribution warehouse having a footprint of about 530,000 square feet (sf) and a proposed finished floor elevation (FFE) of about el +149 . Proposed site grades generally range from about el +125 to +175 . The remaining development includes new access roads, parking areas, loading docks, utilities, and stormwater features.

At this time, the site grading has not been finalized. As such, the recommendations provided here are subject to change when the revised site grading is complete. If the grading approach changes, a revised geotechnical engineering report may be required as the grading affects our recommendations.

Our subsurface exploration was performed between June and July, 2020 and consisted of borings (60), test pits (31), observation wells (4), laboratory testing, and infiltration tests (2).

The general subsurface conditions across the entire lot consisted of a surficial layer of topsoil (about 2 to 24 inches thick), underlain by discontinuous layers of fill (about 1 to 3 feet thick), sand/silt (about 1 to up to 24 feet thick), glacial till (about 2 to up to 23 feet thick), and bedrock (top of about el +108 to +143 ). Groundwater was encountered or observed across the site (about el +120 to +151 ). Within the proposed building footprint, bedrock was encountered from about el +108 to +151 and groundwater was encountered or observed from about el +125 to +149 .

The proposed warehouse building can be supported on a conventional shallow foundation system using an allowable bearing pressure of 3,000 pounds per square foot (psf) bearing on the natural sand/silt, glacial till, bedrock, or compacted structural fill. Total and differential settlements are estimated to be 1 inch and $1 / 2$ inch or less, respectively. The proposed slab areas can be constructed as conventional slab-on-grade bearing on the natural sands, glacial till, or proof-rolled existing fill.

Site Class D and Seismic Design Category B may be used in design.

The following design and construction premiums were identified:

- The natural sand is generally poorly graded and both the sand and glacial till materials have a fines contents ranging from $4 \%$ to $23 \%$. Mixing the sand and glacial till with a more granular material may be required such that the materials are well graded to meet the specifications for structural fill and so that the material are not as sensitive to moisture.
- Groundwater was encountered across the site from about 4 to 20 feet below grade (about el +120 to +151).
- Temporary groundwater dewatering will be required throughout construction where excavations extend to below groundwater.
- Groundwater was encountered within 4 feet and above proposed select paved areas. Permanent dewatering (underdrains) will be required at the western side of the lot for up to 250,000 square feet of paved areas.
- Groundwater was encountered one foot below the proposed slab elevation for the building. Permanent dewatering (sub-slab underdrains) will be required for up to 10,000 square feet of the building area.
- Bedrock was encountered across the site from about 12 to 36 feet below grade (about el +108 to +151).
- Bedrock was encountered in one test pit within the proposed building in the asphalt paved areas. If bedrock is encountered within the building, rock removal will be required.
- Rock removal will be required for site areas to the west.
- Select wetlands are proposed for filling as part of the development. All unsuitable materials (i.e. water, organic materials, etc.) must be removed prior to filling. Dewatering activities should be expected in these areas.
- The foundations for the proposed water towers have not been designed yet as they are a delegated design. Ground improvement may be required for the water towers; however this should be determined by the water tower design engineer of record.
- Topsoil will need to be segregated, as it is not suitable for re-use beneath structural areas (pavements, buildings, retaining walls, etc.). Topsoil may be re-used in landscaped areas, pending approval.


## INTRODUCTION

This report presents our geotechnical engineering study for the proposed industrial park development in Hudson, New Hampshire. Specifically, this report addresses Lot C within the overall development. The remaining two lots (Lots A and B) are addressed in separate reports.

The purposes of this study were to explore subsurface conditions, evaluate feasible foundation options, and develop geotechnical engineering recommendations. Services were performed in accordance with our authorized proposal (19 September 2019 and revised 1 July 2020).

Our approach and recommendations were developed considering the following plans, design criteria, preliminary loads, and design bulletin. Any changes to the design scheme must be reviewed by Langan for effects on our recommendations.

- Site development plans prepared by Langan (August 2020 progress print).
- "Design Criteria and Outline Specification for the Development of 2019-2020 NA Traditional Non-Sort Facility, Version 7.0" prepared by Ford \& Associates Architects, Inc. (10 September 2019).
- Column Loading Map prepared by HSA \& Associates, Inc. (received 20 July 2020).
- Design bulletin DB-0088 NACF Pavement Design Criteria and Guidelines (3 March 2020).

At this time, the site grading is still progressing. As such, the recommendations provided here are subject to change with the revised site grading.

Elevations are referenced from a "Topographic Subdivision Plan, Hudson Logistics Center" (21 April 2020) prepared by Hayner/Swanson, Inc. referencing the National Geodetic Vertical Datum of 1929 (NGVD29).

## SITE DESCRIPTION

## Overall

The overall about 320-acre site is occupied by the Green Meadow Golf Club at 59 Steele Road in Hudson, New Hampshire. The site is bounded by Sagamore Bridge Road to the north, commercial properties, streams/wetlands and New Hampshire Route 3A to the east, residential neighborhoods to the south along Fairway and Eagle Drives, and the Merrimack River to the west. Figure 1 shows the site location and surrounding properties.

The golf club consists of a 39-hole golf course including wooded areas, open fairways, water features, and sand traps. Structures include a two-story clubhouse, one-story maintenance
building, and pump houses. Grades generally slope up from the east to the center of the site and slope down from the center to the west towards the Merrimack River.

Multiple utilities run throughout the site to support the existing golf course (irrigation, electric, stormwater, etc.).

## Lot C

Lot $C$ is about 111 acres and is located on the southeast part of the overall site. Site grades generally slope down from the southwest to east (about el +175 to +125 ). High points (between about el +160 and +175 ) exist in the southwestern part of the lot. Elevations typically vary in the north part of the lot between about el +135 and +150 , and at the center and south parts of the lot between about el +145 and +160 . Low areas exist along the eastern part of the lot (about el +125 to +135 ). Wetlands exists along the western part of the site with grades typically below about el +125 .

## PROPOSED DEVELOPMENT

## Overall

The overall proposed development will include demolition of the existing club golf course and ancillary structures, and the construction of three distribution warehouses on separate lots. No basement levels are proposed. Each proposed warehouse will have associated parking stalls, loading docks, access roads, landscaped areas, and stormwater basins. Additionally, one aboveground water tank is proposed for each lot (to be designed by others).

Several fill retaining walls up to about 10 feet high are proposed throughout the overall site.

Two new access roadways are proposed (Walmart Boulevard to the north and Green Meadow Drive to the south) to connect the three lots to Route 3A to the east. Walmart Boulevard will extend towards Route 3A from the northeast corner of Lot $A$ and Green Meadow Drive will extend towards Route 3A from the east between Lots A and C. The roadways will traverse the existing wetlands and streams using a pipe culvert.

A boat ramp is being contemplated at the Merrimack River adjacent to Lot B. Explorations and associated recommendations for this area and the boat ramp are beyond the scope of this study.

## Lot C

Though the building has not been engineered at this time per the structural engineer, the structural engineer had provided the general proposal building information here. Table 1 details the proposed building information. No internal mezzanine areas are proposed.

Proposed grades vary between about el +125 to +175 . The proposed FFE is about el +149 with an about 4 foot drop to adjacent grades at the loading docks to the west, where the pavement grades generally slope away from the building. Pavement areas vary between about el +132 and +147 . Proposed infiltration basins are located at the north, east and west of the lot (about el +128 to +143 ). A proposed soil berm to the south varies between about el +125 and +175 . The proposed roadway runs along the eastern part of the lot and typically varies between about el +132 and +140 .

## Table 1. Proposed Site Development

| Proposed Building |  | Estimated Grades Within the <br> Proposed Building Footprint |  |  | Proposed Structural Loads |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| Stories <br> (\#) | Footprint <br> (SF) | Existing | Proposed <br> FFE | Resulting <br> Cuts/Fills <br> (ft) | Building <br> Column <br> (kips) | Wall Loads <br> (kips/foot) |
| One | 530,000 | $\mathrm{el}+133$ <br> to +169 | el +149 | Cut $=19$ <br> Fill $=16$ | 190 to 220 | 9 to 11 |

## REVIEW OF AVAILABLE INFORMATION

## Regional Geology

The surficial geology map from the United States Department of Agriculture (Figure 2) indicates the overburden is loamy sand. The bedrock geology map from the United State Geologic Survey (Figure 3) indicates the bedrock below the site is granofels.

## Federal Emergency Management Agency Flood Map

We reviewed the Flood Insurance Rate Map (FIRM) for the town of Hudson, New Hampshire, published by the Federal Emergency Management Agency (FEMA), Map No. 33011C0656D and 33011C0658D effective 25 September 2009 (Figure 4). Table 2 gives a summary of the findings.

Table 2. Flood Mapping

|  | Flood Mapping ${ }^{\mathbf{1 , 2}}$ |
| :---: | :---: |
| Building Area | Site and Roadway Areas |
| Zone X (not shaded) | Eastern Edge: Zone X (not shaded) \& Zone A |

## Available Historic Information

We reviewed historic topographic maps (1893 to 2012) and aerial photographs (1938 to 2016) for the overall site. Historic information provided in Appendix A.

Pre-1893 - The site is shown as undeveloped with an unnamed stream running through the southeast part of the site. The surrounding areas also appear to be undeveloped.

Late 1910s to 1920s - The site is shown as mostly undeveloped, with unidentified structures and an access road in the eastern part of the site.

1930s to 1950s - The unknown structures from the late 1910s and 1920s are no longer shown on the topographic maps. Parts of the southeast and northern areas of the site are developed as agricultural fields with associated structures and access roads.

Early 1960s to Present - The site is developed as a golf course with a residential building in the east. Site development features include a clubhouse, maintenance building, access roads, asphalt-paved parking, and water features. Topographic maps show existing gravel pits in the western part of the site from 1965 through 1987. Aerial maps show similar gravel pits to the west and northwest of the maintenance building from 1963 through 1995. The site has remained similar to its current state since about 1965.

## Available Geotechnical Report

We have reviewed a geotechnical engineering report titled "Preliminary Geotechnical Engineering Study" prepared by GZA GeoEnvironmental, Inc. (May 2006). Relevant information is attached in Appendix B. The report includes 21 borings, 22 test pits, and 3 field permeability tests performed around the site. Identified design and construction premiums for the overall site included shallow groundwater reported to the west, shallow refusal on bedrock reported to the north, and potentially liquefiable soils reported to the east.

[^0]
## SUBSURFACE EXPLORATION

Langan performed a subsurface exploration consisting of borings, observation wells, test pits, and infiltration tests throughout the proposed development area. All work was overseen by a Langan field engineer. An exploration location plan is shown in Figure 5.

## Borings

Standard Penetration Test (SPT) N-values ${ }^{3}$ were documented and soil samples were generally obtained continuously to a depth of 12 feet and every 5 feet thereafter. Disturbed soil samples were obtained using a standard 2-inch-outer-diameter split-spoon sampler driven by a 140-pound automatic or safety hammer in accordance with ASTM D1586, Standard Penetration Test. See Tables 3 and 4 for additional information regarding the boring program.

Recovered soil samples were visually examined and classified in the field in general accordance with the Unified Soil Classification System (USCS). Soil classifications, N-values, and other field observations were recorded on our field logs provided in Appendix C.

Bedrock was cored in selected borings using a $2-7 / 8$-inch NO core barrel. The core barrel was equipped with a diamond cutting bit in accordance with ASTM D2113, Rock Core Drilling. Rock type, percent recovery (REC) ${ }^{4}$ and Rock Quality Designation (RQD) ${ }^{5}$ were determined for each the core run.

Table 3. Summary of Boring Subcontractors

| Date Range | Drilling Companies | Drilling Equipment |
| :--- | :--- | :---: |
| 1 <br> 2 June to <br> 2 | Suly, 2020* | SoilTesting, Inc. |
|  | Seaboard Geotechnical \& 550X ATV Rig, CME55 Truck- <br> Environmental Drilling Services |  |
|  | Ctlantic Testing Laboratories <br> Limited | Diedrich D50 Track Rig, Mobile Drill B52 <br> Truck-mounted Rig |

[^1]Table 4. Summary of Borings

| Total <br> (\#) | Subtotal <br> (\#) | Boring Locations | Boring ID's | Depth Range (ft) | Elevation Range (Bottom of Boring) |
| :---: | :---: | :---: | :---: | :---: | :---: |
| 60 | 30 | Proposed Building Areas | $\begin{gathered} \text { C-B-BOR-01, } \\ \text { C-B-BOR-01A, } \\ \text { C-B-BOR-02 to } \\ \text { C-B-BOR-29 } \\ \hline \end{gathered}$ | 14 to 34 | $\begin{gathered} \text { el }+140 \\ \text { to } \\ +108 \end{gathered}$ |
|  | 6 | Proposed Roadway Areas | $\begin{aligned} & \text { C-R-BOR-01 to } \\ & \text { C-R-BOR-04, } \\ & \text { C-R-BOR-06, } \\ & \text { C-R-BOR-06A } \end{aligned}$ | 17 to 22 | $\begin{gathered} \text { el }+117 \\ \text { to } \\ +107 \end{gathered}$ |
|  | 24 | Proposed Site Areas | $\begin{gathered} \text { C-S-BOR-01 to } \\ \text { C-S-BOR-24 } \end{gathered}$ | 13 to 22 | $\begin{gathered} \hline \mathrm{el}+153 \\ \text { to } \\ +110 \end{gathered}$ |

## Test Pits

Test pit were excavated throughout the site to further observe the subsurface soils and to perform infiltration testing. See Tables 5 and 6 for additional information regarding the exploration program. Test Pit logs are provided in Appendix D, and photographs are provided in Appendix E.

Table 5. Summary of Test Pit Subcontractor

| Date Range | Test Pit Company | Test Pit Equipment |
| :---: | :---: | :---: |
| 29 May to 30 June, 2020* | Polster Industries, LLC | CAT 304E, CAT 305E, Takeuchi TB260 |

*Dates reflect duration of the overall exploration program (i.e. Lots A, B, and C)

Table 6. Summary of Test Pits

| Total (\#) | Subtotal <br> (\#) | Test Pit Locations | Test Pit ID's | Depth Range (ft) | Elevation Range (Bottom of Test Pit) |
| :---: | :---: | :---: | :---: | :---: | :---: |
| 31 | 14 | Proposed Building Areas | $\begin{gathered} \hline \text { C-B-TP-01 to } \\ \text { C-B-TP-10, } \\ \text { C-B-TP-10A, } \\ \text { C-B-TP-10B, } \\ \text { C-B-TP-11 to } \\ \text { C-B-TP-12 } \end{gathered}$ | 4 to 9 | $\begin{gathered} \mathrm{el}+128 \\ \text { to } \\ +155 \end{gathered}$ |
|  | 3 | Proposed Roadway Areas | C-R-TP-01 to C-R-TP-03 | 7 to 8 | $\begin{gathered} \hline \mathrm{el}+123 \\ \text { to } \\ +124 \end{gathered}$ |
|  | 14 | Proposed Site Areas | $\begin{gathered} \hline \text { C-S-TP-01 to } \\ \text { C-S-TP-08, } \\ \text { C-S-TP-11, } \\ \text { C-S-TP-13 to } \\ \text { C-S-TP-17 } \end{gathered}$ | 5 to 8 | $\begin{gathered} \mathrm{el}+120 \\ \text { to } \\ +157 \end{gathered}$ |

## Groundwater Observation Wells

Groundwater observation wells were installed throughout the site. See Table 7 for a summary of observation wells installed. Well construction logs are provided in Appendix F.

Table 7. Summary of Observation Wells

| Total <br> (\#) | ID | Depth <br> (ft) | Bottom of Observation Well <br> Elevation |
| :---: | :--- | :---: | :---: |
|  | C-B-BOR-02(OW) | 10 | $\mathrm{el}+122$ |
| 4 | C-B-BOR-16(OW) | 25 | $\mathrm{el}+133$ |
|  | C-B-BOR-20(OW) | 20 | $\mathrm{el}+137$ |
|  | C-S-BOR-04(OW) | 10 | $\mathrm{el}+121$ |

## Lab Testing

Selected samples were sent to a testing laboratory to confirm visual classifications and to determine index properties (physical and mechanical). Testing for chlorides and sulfates was performed at the structural engineer's request. See Table 8 for a summary of the completed laboratory tests. Laboratory results are provided in Appendix G.

Table 8. Laboratory Testing Summary

| Test Description | ASTM Standard | Quantity |
| :--- | :---: | :---: |
| Grain Size | ASTM D-6913 | 9 |
| Moisture | ASTM D-2216 | 12 |
| Percent Passing No. 200 | ASTM D-1140 | 1 |
| Organic Matter | ASTM D2974 | 1 |
| Chlorides | ASTM D-512 | 2 |
| Sulfates | ASTM D-516 | 2 |

## SUBSURFACE CONDITIONS

## Subsurface Materials

The subsurface conditions generally consist of a surficial layer of topsoil underlain by layers of discontinuous fill, sand/silt, glacial till, and finally bedrock. A summary of subsurface materials is provided in Table 9. A description of subsurface materials encountered is provided below in order of increasing depth.

Table 9. Subsurface Conditions

| Layer | Thickness <br> (feet) | Top <br> Elevation <br> Range | N-Value <br> Range | Average <br> Density | Fines <br> Content <br> (\%) | Moisture <br> Content <br> (\%) |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| Topsoil | 2-inches to <br> 24 -inches | el +170 <br> to <br> +127 | 2 to 28 | Loose | N/A | N/A |
| Fill | 1 to 3 | el +127 <br> to <br> +125 | 7 | Loose | N/A | N/A |
| Sand/Silt | 1 up to 24 | el +169 <br> to <br> +107 | 3 to <br> Refusal 6 | M. Dense | Sand: 4 to 23 <br> Silt: 60 to 91 | Sand: 2 to 11 <br> Silt: 18 to 48 |
| Glacial | 2 up to 23 | el +165 <br> to <br> +108 | 20 to <br> Refusal | V. Dense | 23 to 31 | 9 to 13 |

Topsoil - A layer of topsoil was encountered in 54 borings and 31 test pits. The topsoil generally consists of brown to dark brown fine to medium sand with varying proportions of gravel, roots,

[^2]and silt. In the remaining 6 borings, the surficial material was consistent with the fill or natural sand material.

Fill - Below the topsoil, a layer of fill was encountered in one boring and five test pits. The fill is generally composed of light brown to dark brown fine to medium sand with varying amounts of gravel, roots, debris, and silt. Note an abandoned rubble well was encountered in one test pits (C-B-TP-10B) to a depth of about 9 feet. The fill layer is generally classified as poorly graded sand (SP) in accordance with the USCS.

Sand/Silt - Below the fill or topsoil, a layer of sand, with some silty sand and silt pockets, was encountered in all borings. The sand is generally composed of light brown to brown fine to coarse sand with varying amounts of gravel and silt. The silt, which was limited to discrete and discontinuous areas, is generally composed of light brown to brown silt with varying amounts of fine sand and gravel. Note that higher SPT N-values (Table 9) within the sand/silt layer are likely the result of obstructions (boulders, cobbles, or gravel) blocking the sampler. The sand layer, and silty layers within, are generally classified as poorly graded sand (SP), silty sand (SM), and silt (ML) in accordance with the USCS.

Glacial Till - Below the sand/silt, a layer of glacial till was encountered. The glacial till is generally composed of brown to grayish brown fine to coarse sand with varying amounts of gravel, silt, and weathered rock fragments. Note that higher SPT N -values (Table 9) within the glacial till layer are likely the result of obstructions (boulders, cobbles, or gravel) blocking the sampler. The glacial till layer is generally classified as silty sand (SM) in accordance with the USCS.

Bedrock - Below the glacial till, a layer of bedrock was inferred or cored in 21 borings. A summary of encountered bedrock is provided in Table 10. The bedrock consists of gray schist, fine to medium grained, moderately weathered, close to very close fractures, and moderate dipping and horizontal fractures. Up to five-foot-long rock cores were taken in nine borings during our exploration. The REC and RQD of the rock core samples ranged from about $35 \%$ to $100 \%$ and $0 \%$ to $91 \%$, respectively. Though not encountered in the borings or test pits, the lower REC and RQD values may be indicative of weathered bedrock.

Table 10. Summary Bedrock Information

| Location | Bedrock Depth |  |  |  |
| :---: | :---: | :---: | :---: | :---: |
|  | Cored |  | Inferred |  |
|  | Depth (ft) | Elevation | Depth (ft) | Elevation |
| Proposed <br> Building <br> Areas | 12 to 34 | el +112 <br> to <br> +130 | 15 to 36 | el +108 to +140 <br> (Bedrock was <br> foundation up to <br> el +151 in one test pit: <br> C-B-TP-10) |
| Proposed <br> Roadway <br> Areas | Not Performed | Not Performed | 20 | el +116 |
| Proposed <br> Site <br> Areas | Not Performed | Not Performed | 16 to 22 | $\mathrm{el}+122$ <br> to <br> +143 |

Groundwater - A summary of groundwater is provided in Table 11. Groundwater, if encountered, should be expected to fluctuate with seasons, precipitation, construction activities, irrigation activities, etc.

Table 11. Summary Groundwater Information

| Location | Groundwater Depth |  |  |  |
| :---: | :---: | :---: | :---: | :---: |
|  | Observation Wells/Test Pits |  | Inferred in Borings |  |
|  | Depth (ft) | Elevation | Depth (ft) | Elevation |
| $\begin{array}{c}\text { Proposed } \\ \text { Building } \\ \text { Areas }\end{array}$ | 6 to 14 | $\mathrm{el}+126$ |  |  |
| to |  |  |  |  |
| +144 |  |  |  |  |$)$

## Infiltration Testing

Infiltration rates were measured in the proposed stormwater systems as specified by the civil engineer. Infiltration tests were performed in accordance with the New Hampshire Code of Administrative Rules (Env-Wq 1500). A summary of average infiltration rates at each location is presented in Table 12. A detailed summary of infiltration tests is provided in Appendix H . Generally, the measured infiltration rates are higher than the rates in the available geotechnical
report. Final design infiltration rates should be selected by the civil engineer based on the stormwater system design and allowable infiltration rates.

Table 12. Infiltration Test Results Summary

| Location | Surface <br> Elev. | Test <br> Depth <br> $(\mathrm{ft})$ | Test <br> Elev. | Measured <br> Infiltration Rate <br> $(\mathrm{in} / \mathrm{hr})$ | Material Type |
| :---: | :---: | :---: | :---: | :---: | :---: |
| C-S-TP-01 | el +128 | 4 | el +124 | 11 | Light brown silty fine SAND, <br> about 2-inch-thick fine to <br> medium lenses |
| C-S-TP-17 | $\mathrm{el}+133$ | 3 | $\mathrm{el}+130$ | 79 | Light brown silty fine SAND |

## Sulfate \& Chloride Testing

Chemical analyses were performed on select samples generally obtained from soils within 5 feet of both proposed grades and the finished floor elevation. The soluble sulfate and chloride concentrations were both less than 10 parts-per-million. A summary of laboratory testing is provided in Appendix G. Based on the laboratory testing, the sulfate exposure class ${ }^{7}$ is SO and the chloride exposure class ${ }^{7}$ is C 1 given the presence of groundwater. Consideration could be given to using chloride exposure class CO for building slabs as a vapor barrier is proposed below.

## GEOTECHNICAL DESIGN RECOMMENDATIONS

## Additional Explorations \& Analysis

As the design progresses, we recommend the following additional exploration and analysis work be performed to advance the geotechnical design and construction recommendations:

- Test pits should be completed along the northern part of Green Meadow Drive as access was not provided during our exploration program.
- Groundwater levels should be obtained throughout design for additional measurements and potential refinements to recommendations for permanent water controls. Additionally, groundwater readings should be collected when watering of the course has stopped and after the site irrigation system is decommissioned as leaks in the system or surface-level infiltration from the system may affect groundwater levels.
- Additional design and coordination work should be performed with respect to site and sub-slab underdrain systems.

[^3]- The retaining walls will need to be designed by a design engineer registered in New Hampshire. Design should include all internal and external stability checks.
- The water tower foundations will need to be designed by others as this is a delegated design.
- Temporary works for pre-cast/tilt-up wall panels will need to be designed by others as this is a delegated design.


## Liquefaction

We evaluated the liquefaction potential of non-cohesive soil below the groundwater table and up to 50 feet below the ground surface (as required by the New Hampshire Building Code) using the procedure outlined by Youd et. al (2001). The Youd et. al method is considered to be the state-of-practice procedure as recommended by the National Earthquake Hazard Reduction Program. The method presents an empirical relationship between the earthquake demand represented by the Cyclic Stress Ratio (CSR), and the soil resistance to dynamic loading represented by the Cyclic Resistance Ratio (CRR). Field $N$-values are converted to $N_{1,60, \text { cs }}$ by applying corrections for hammer energy efficiency, soil overburden pressure, borehole diameter, rod length, sampler lining, and fines content.

The available geotechnical engineering report indicated a potentially liquefiable area to the east (in the vicinity of GZA boring B-18). As part of our subsurface exploration and evaluation, we performed borings in the vicinity of boring B-18 and analyzed the results.

Our analysis was performed on a sample set of borings that were potentially liquefiable across the lot.

Input parameters included a peak ground acceleration of 0.200 g (from USGS). Our analysis indicates an adequate factor of safety for liquefaction for explorations advanced within the building and roadway/site areas. We concluded that liquefaction need not be considered in the design. Plots showing factors of safety versus depth are provided as Figures 6 and 7 for the building and roadway/site areas, respectively.

## Seismic Design

This section presents seismic design recommendation, in accordance with the 2019 New Hampshire State Building Code (International Building Code 2015). We have considered the soil conditions encountered in the borings to be consistent and representative of the soil conditions in the top 100 feet of soil at this lot.

Table 13. Seismic Design Values

| Description | Parameter | Recommended Value |
| :--- | :--- | :--- |
| Mapped Spectral Acceleration for short periods ${ }^{10}:$ | $\mathrm{S}_{\mathrm{s}}$ | 0.238 g |
| Mapped Spectral Acceleration for 1-sec period ${ }^{11}:$ | $\mathrm{S}_{1}$ | 0.075 g |
| Site Class: | -- | $\mathbf{D}$ - Stiff Soil Profile |
| Site Coefficient: | $\mathrm{F}_{\mathrm{a}}$ | 1.6 |
| Site Coefficient: | $\mathrm{F}_{\mathrm{v}}$ | 2.4 |
| 5\% damped design spectral response acceleration at <br> short periods: | $\mathbf{S}_{\mathrm{Ds}}$ | $\mathbf{0 . 2 5 4} \mathbf{~ g}$ |
| 5\% damped design spectral response acceleration at <br> 1-sec period: | $\mathbf{S}_{\mathbf{D} 1}$ | $\mathbf{0 . 1 2 0} \mathbf{~ g}$ |
| Anticipated Risk Category | -- | $\mathbf{I I}$ |
| Seismic Design Category | -- | $\mathbf{B}$ |

Based on the above spectral accelerations and the anticipated risk category, we have estimated the Seismic Design Category (SDC). The structural engineer is responsible for confirming the appropriate use group, occupancy category, and final SDC for the proposed structure.

## Building Foundations

The materials encountered at the anticipated footing elevation consist of fill, sand/silt, and glacial. Bedrock was encountered in one test pit within the building footprint and to the west of the proposed building in the truck court/parking area; as such, bedrock may be encountered at the bottom of footing elevation as well. The existing fill and silt are not suitable for foundation support. The proposed structure and guard house can be supported on shallow foundations bearing on structural fill, sand/silt, glacial till, weathered rock, or bedrock using an allowable bearing pressure of $3,000 \mathrm{psf}$. If desired, a higher bearing pressure for footings bearing on weathered rock or bedrock could be provided. Footing subgrades should be prepared in accordance with the Subgrade Preparation section of this report.

All exterior footings should be constructed 48 inches or deeper below the lowest adjacent grade for frost protection. Interior footings in heated spaces may be constructed at a convenient depth below the slab; however, all bottoms of footings should be at least 1.5 feet below the finishedfloor elevation. Interior footings in non-heated spaces, or where frost protection is not provided

[^4]throughout construction, should be protected from frost (e.g., lowering footings, backfilling, heaters/blankets, etc.).

Isolated column footings should have a minimum dimension of 3 feet, and strip footings should have a minimum width of 2 feet even if smaller dimensions can be justified using the recommended allowable bearing pressure.

Foundations should not be located so that one foundation is within the zone of influence of an adjacent foundation. The zone of influence is taken as a $1 \mathrm{H}: 1 \mathrm{~V}$ projection extending outward and downward from the edge of the foundation.

## Building Settlement

Total settlement of the structure is estimated to be on the order of 1 inch or less, provided the bearing pressure recommended here is used and the subgrade preparation work described here is performed. Differential settlements of adjacent new structure columns are expected to be about $1 / 2$ inch. The majority of the settlement is expected to take place during construction.

## Water Tower

The design engineer of record should confirm that the bearing capacity and calculated settlements (based on the water tower loads) are acceptable for use with a shallow foundation design. If not, the water tower design engineer of record should determine if supplemental foundation recommendations are required. Ground improvement to achieve higher bearing capacities may be required.

Given the design of the water tower is not finalized, we recommend that an allowance for ground improvement (stone columns up to 25 feet long) be provided for initial cost estimating until a final design can be prepared by others.

## Building Floor Slabs

We recommend that ground-floor slabs be constructed as a slab-on-grade bearing on natural soils, structural fill, or compacted existing fill prepared in accordance with the recommendations here. Additional recommendations for sub-slab underdrains are provided below. If bedrock or weathered rock is encountered, it should be over-excavated a minimum of 2 feet below the proposed bottom of slab elevation and replaced with structural fill or gravel; additional rock removal may be required for sub-slab utilities and should be coordinated as the design progresses. The slab-on-grade supporting short-term loads over smaller areas (e.g., vehicle wheel
loads) ${ }^{12}$ should be designed for a modulus of subgrade reaction of 125 pounds per cubic inch (pci). The slab-on-grade supporting long-term loads over larger areas (e.g., uniform or rack loading) should be designed for a reduced modulus of subgrade reaction of 80 pci .

We recommend a minimum 6-inch-thick layer of $3 / 4$-inch clean crushed stone be included beneath the slabs to protect the prepared subgrade and to serve as a capillary break. Additional assessment is on-going regarding recommendations for a permanent drainage design.

A vapor barrier should be used below the ground-floor slab to limit transmission of water vapor through the slab. We recommend a vapor barrier with a minimum thickness of 20 mils. Omission of a vapor barrier can lead to floor-covering problems including delamination and mold. Additional waterproofing measures may be required pending the on-going recommendations for permanent drainage design. The contractor may elect to place up to 4 -inches of a fine to medium sand (i.e., stone dust) above the vapor barrier for slab constructability considerations. The sand layer should have a maximum particle diameter of $3 / 16$-inch and should consist of hard durable sand free from ice and snow, roots, sod and other deleterious matter. The vapor barrier should be coordinated with the environmental requirements for the development.

## Permanent Groundwater Control

## Building Areas

Perimeter wall and footing drains should be installed to divert groundwater flow away from the structure during prolonged precipitation, snowmelt, or utility breaks. Manufactured geocomposite drainage panels or a 12-inch-wide layer of $3 / 4$-inch washed crushed stone should be installed against the outside of all perimeter walls and should extend to within 1 foot of adjacent surface grade. In the truck-court areas, gravel should be used. The drainage panels (or crushed stone) should connect to a perforated footing drain at the base of the footing having a minimum diameter of 6 inches. The footing drains should be connected to the site stormwater system and where possible drain by gravity. Where used, drainage panels should be secured in place and the filter-fabric side must face the soil. If washed crushed stone is used, it should be wrapped with a geotextile filter fabric.

As noted, the grading plans are currently being finalized. We recommend modeling anticipated post construction groundwater elevations to determine if permanent dewatering measures for site features (sub-slab underdrain, pavement underdrains, etc.) are required.

[^5]Groundwater levels (el +125 to +149 ) are up to within 1 foot of the proposed top of slab elevation (el +150 ) within about 10,000 square feet of the proposed building (generally on the western side of the building). We propose modeling these areas further, but as the grading plans are still being finalized, we recommend that allowances and unit rates be carried for permanent dewatering measures at this point in the design (i.e. sub-slab underdrains).

A preliminary design groundwater elevation of el +153 should be used (i.e. 4 feet above the highest recorded groundwater levels to date). Underdrains should consist of a minimum of a 12-inch-thick gravel layer (3/4-inch washed, crush stone) beneath the slab. Geotextile filter fabric should be placed between the soil subgrade and the stone. Within the stone, an inter-connected grid network of 6-inch diameter SCH-80 PVC pipes should be placed. The pipes should be spaced at 15 feet on-center. The pipes should be routed to internal sump pits and connected to the site stormwater system to discharge via gravity. A minimum of one sump pit per 5,000 square-foot (or tributary area) of underdrain area should be assumed at this time.

Additionally, we recommend a perforated pipe, having a minimum diameter of 6 inches, be located on the in-board side of the truck-court foundation wall (western side of the building) at the bottom of footing elevation. The pipe should be routed to the site stormwater system. A 12inch thick gravel ( $3 / 4$-inch washed, crushed stone) trench wrapped in filter fabric should encapsulate the perforated pipe and extend from the bottom of footing to bottom of slab elevation.

## Site Areas

Groundwater was encountered to the west of the building above and within 4 feet of the proposed pavement and truck court grades for about 250,000 square feet of the overall pavement footprint. We recommend that allowances and unit rates be carried for permanent dewatering measures at this point in the design (i.e. pavement underdrains). The pavement underdrain design will be included on the civil plans.

Underdrains should consist of a minimum of a 12-inch-thick gravel layer (3/4-inch washed, crush stone) beneath the pavement. Filter fabric should be placed between the soil subgrade and the stone. Within the stone, an inter-connected grid network of 6-inch diameter $\mathrm{SCH}-80 \mathrm{PVC}$ pipes should be placed. The pipes should be spaced at 20 feet on-center. The pipes should be routed to the site stormwater system to discharge via gravity.

## Pavement Design

We have provided recommendations for minimum asphalt-pavement sections using 115\% of the daily traffic loading provided by the traffic engineer (Langan) detailed in Table 14. The pavement
sections were designed using a California Bearing Ratio (CBR) of 10 for proofrolled site soils or properly placed compacted fill. CBR testing must be performed in pavement areas at the start of construction to confirm the design assumptions. A life expectancy of 20 years was used for flexible pavements and 30 years for rigid pavements. Pavement design calculations are provided in Appendix I. Refer to subsequent sections for subgrade preparation procedures.

We have prepared the following site-wide (i.e. all three lots) pavement design recommendations for the overall site.

Table 14: Proposed Daily Traffic Loading

| Area | Passenger Cars (\#) |  | Light Trucks (\#) |  | Tractor Trailers (\#) |  |
| :--- | :---: | :---: | :---: | :---: | :---: | :---: |
|  | Proposed | $\mathbf{1 1 5 \%}$ | Proposed | $\mathbf{1 1 5 \%}$ | Proposed | $\mathbf{1 1 5 \%}$ |
| Lot A: | 651 | $\mathbf{7 4 9}$ | $\mathrm{n} / \mathrm{a}$ | $\mathbf{n} / \mathbf{a}$ | 131 | $\mathbf{1 5 1}$ |
| Lot B: | 326 | $\mathbf{3 7 5}$ | 25 | $\mathbf{2 9}$ | 40 | $\mathbf{4 6}$ |
| Lot C: | 354 | $\mathbf{4 0 7}$ | $\mathrm{n} / \mathrm{a}$ | $\mathbf{n} / \mathbf{a}$ | 60 | $\mathbf{6 9}$ |
| Northern Access Roadway <br> (Walmart Blvd.): | 390 | $\mathbf{4 4 9}$ | $\mathrm{n} / \mathrm{a}$ | $\mathbf{n} / \mathbf{a}$ | 131 | $\mathbf{1 5 1}$ |
| Southern Access Roadway <br> (Green Meadow Drive): | 941 | $\mathbf{1 , 0 8 2}$ | 25 | $\mathbf{2 9}$ | 100 | $\mathbf{1 1 5}$ |

Table 15: Standard \& Heavy Duty Flexible Pavement Sections (Site Areas)

| Material | Thickness (in) |  |
| :--- | :---: | :---: |
|  | Standard Duty | Heavy Duty |
| Area: | Passenger car drive aisles <br> \& parking stalls | Access drives \& truck <br> courts |
| Top (Finish) Course: | 2.0 inches | 2.0 inches |
| Asphalt Pavement Binder Course: | 2.0 inches | 3.0 inches |
| Processed Aggregate and Gravel <br> (NH DOT Item No. 304.3): | 8.0 inches | 12.0 inches |
| One pavement design provided for all three lots. Lots A and C control the pavement design. Traffic loading for Lot A used in the <br> pavement calculations. |  |  |
| Processed aggregate and gravel course has been increased by 2 inches from the minimum calculated pavement sections given <br> the anticipated underlying loose fine sands. |  |  |

Table 16: Standard, Heavy, Extra Heavy Duty Rigid Pavement Sections (Site Areas)

| Material | Thickness (in) / Materials |  |  |
| :---: | :---: | :---: | :---: |
|  | Standard Duty | Heavy Duty | Extra Heavy Duty |
| Area: | Passenger car drive aisles \& parking stalls | Access drives \& truck courts | Dolly pads \& loading/unloading aprons |
| Concrete (4,500 psi 28-day strength, 6\% air-entrained, chloride resistant): | 5.0 | 8.0 | 8.0* |
| Processed Aggregate and Gravel (NH DOT Item No. 304.3): | 6.0 | 8.0 | 8.0 |
| Continuous Reinforcing Each Way: | \#3 bar at 22-inch on-center | \#3 bar at 16-inch on-center | \#3 bar at 16-inch on-center |
| Per the design criteria, dowels are to be used at construction joints. <br> Minimum calculated design heavy and extra heavy duty rigid pavement sections increased to match the design criteria minimum cross-section ( 8.0 inches of concrete and 6.0 inches of processed aggregate and gravel). |  |  |  |
|  |  |  |  |
| Processed aggregate and gravel course has been increased by 2 inches from the minimum calculated/design criteria pavement sections given the anticipated underlying loose fine sands. |  |  |  |
| *Extra heavy duty rigid pavement shall be enhanced with a minimum of 7.5 pounds of synthetic macrofibers per cubic yard of concrete. |  |  |  |

Table 17. Heavy Duty Flexible Pavement Section (Roadways)

| Material | Thickness (in) |  |
| :--- | :---: | :---: |
|  | Northern Access <br> Roadway <br> (Walmart Blvd.) | Southern Access <br> Roadway <br> (Green Meadow Drive) |
| Top (Finish) Course: | 1.5 | 1.5 |
| Asphalt Pavement Binder Course: | 2.5 | 2.5 |
| Crushed Gravel (NH DOT Item No. 304.2): | 6.0 | 6.0 |
| Gravel (NH DOT Item No. 304.3): | 12.0 | 12.0 |
| Minimum calculated design pavement section increased to match the Town of Hudson minimum typical cross-section for <br> subdivision streets (commercial/industrial) Town of Hudson Engineering Department, Engineering Technical Guidelines \& Typical <br> Details, Detail R-1 (revised February 2020) (4 inches of hot bituminous pavement, 6 inches of crushed gravel, and 12 inches of <br> gravel). |  |  |

## Retaining Walls

Site fill-retaining walls may be designed as geogrid reinforced modular block walls (such as Mesa, Keystone, Versa-lok, or Redi-Rock type walls) or gravity-type retaining walls, depending on the location and size of the proposed wall.

Retaining walls can be designed using a moist unit weight of 130 pounds per cubic foot and a drained angle of internal friction of $30^{\circ}$. Site retaining walls, where movement is acceptable, can be designed using active earth pressures. Walls where movement cannot be tolerated should be designed for at-rest earth pressures. The parameters described above presume (1) the wall backfill materials (i.e., within the reinforced zones) are select imported granular soils, (2) full drainage is provided behind the reinforced zone and wall facing to prevent the buildup of hydrostatic pressure, (3) that surface loads at the top of the retaining walls will consist of parking and driving areas and vehicles, and (4) the slope at the top of the retaining wall is level. Presuming the aforementioned fill, fill placement, and compaction requirements are adhered to, a coefficient of active earth pressure ( $K a=0.33$ ) or a coefficient of at-rest earth pressure ( $K o=0.50$ ) can be used as appropriate. The fill used may consist of imported materials that satisfy the minimum strength parameters specified here and gradation requirements specified by the wall designer. Design parameters should be confirmed during construction via laboratory testing on the actual proposed backfill materials, and adjustment of the pressures should be made by the designer where appropriate to consider these factors.

Retaining-wall foundations should bear on natural soils (if fill or silt is encountered it should be fully removed and replaced) or well-compacted structural/engineered fill compacted with at least six coverages of a minimum 5-ton static-drum-weight vibratory roller. Soft or otherwise unsuitable natural or fill identified by the geotechnical engineer in the field during proofrolling and compaction should be removed and replaced with approved compacted structural/engineered fill. Backfill behind the walls should be placed as discussed in the Fill Materials, Placement and Compaction Criteria section of this report. Over-compaction should be avoided behind the walls.

The proposed retaining wall design (including calculations and global stability and groundwater mounding analyses) and construction means and methods should be provided and signed and sealed by a Professional Engineer licensed in the State of New Hampshire.

## GEOTECHNICAL CONSTRUCTION RECOMMENDATIONS

## Site Preparation

All existing foundations, floor slabs, and utilities should be completely removed within 10 feet of the proposed footprint. Given the current use of the site, we expect below-grade irrigation infrastructure to be encountered throughout the lot. Below grade structures outside the building footprint can be abandoned in place provided they are removed to at least 3 feet below finished subgrade levels, 2 feet below proposed utilities, and to eliminate conflicts with new utilities or structures. Slabs left in place should be sufficiently broken up to allow water to drain and so that a geotechnical engineer can observe whether voids exist beneath the slab. Existing asphalt pavement and concrete walkways should be completely removed.

Existing utilities within the building footprint should be completely removed. Existing utilities outside of the proposed building footprint should be removed or abandoned in place by completely filling with grout.

Excavations made to remove below grade elements should be backfilled with approved, compacted fill in accordance with the Excavation, Fill, Placement, and Compaction Criteria section of this report and any environmental requirements.

Clearing and grubbing of trees and vegetation designated for removal (including root systems) should be performed. Buried debris should be completely removed beneath proposed building slab, footing, and pavement locations. Given the former and current uses of the site, bury holes with topsoil, tree stumps, or similar unknown objects should be expected throughout. Topsoil should be stripped from the proposed building and pavement areas and should be stockpiled and protected from erosion. Topsoil will be evaluated by the landscape architect (Langan) for reuse in landscape areas and coordinated with the environmental engineer (Langan). All clearing and stripping activities should be performed in strict accordance with the approved soil-erosion and sediment-control plan and the environmental reports prepared for the project.

Existing wetlands slated for removal should be completely dewatered at the on-set and maintained dry during backfilling activities. Once dewatered, all organic and silty materials should be completely removed to the top of natural granular soils, weathered rock, or bedrock. A choker 2-foot-thick layer of 3 - to 6 -inch diameter stone should be placed at the subgrade. A layer of filter fabric should be placed above the stone. The resulting excavation should be backfilled with structural fill as described here.

All demolition and site-clearing work should be performed in accordance with any environmental requirements established for the site, and all local, state, and federal regulations. All debris and trees and other vegetation should be properly disposed of off-site in accordance with applicable regulations. All construction work should be performed so as not to adversely impact the neighboring buildings, off site structures or utilities, including the existing utilities and trees that are to remain. Protection of these elements should be provided as necessary. Before beginning grading or placing fill, any miscellaneous trash, debris, or other unsuitable materials should be removed from the site.

## Subgrade Preparation

All soil footing and utility-trench subgrades, except bedrock, should be proofrolled with six overlapping coverages of a double-drum 1-ton walk-behind vibratory roller (such as a Bomag BW75 or equivalent).

Along the western edge of the building, footings for the truck-court may bear on bedrock. If a footing or adjacent footings will bear on rock and soil, a transition zone should be created. For
adjacent footings, the rock should be over-excavated a minimum of 12 inches and replaced with $3 / 4$ inch crushed gravel. For strip footings, rock should be over-excavated a minimum of 12 inches for 5 horizontal feet in either direction (total of 10 feet) from the point of bearing material transition and replaced with $3 / 4$-inch gravel. The specific requirements will be based on the field conditions observed at the subject location and the geotechnical engineer's subsequent recommendations.

All slab subgrade areas should be proofrolled with six overlapping coverages of a vibratory drum roller having a minimum static drum weight of 10 tons. Once the slab is fully compacted, a proofroll with a fully loaded dump truck should be performed. The maximum acceptable depression under the fully loaded dump truck is $1 / 2$ inch. If depressions greater than a $1 / 2$ inch are observed, corrective action must be taken by the contractor.

Soft areas identified during proofrolling should be excavated and replaced with approved structural fill. The actual extent of necessary removal and replacement should be determined by a qualified Langan geotechnical engineer. Care should be taken when proofrolling near any existing underground utilities that are to remain.

Soil footing subgrades should be excavated level and if any cobbles or boulders are encountered at the footing subgrade level such that a relatively level subgrade is not achieved, the cobbles or boulders should be removed and replaced with compacted structural fill, compacted $3 / 4$-inch crushed stone, or lean concrete. All soil subgrades for footings or slabs should be compacted to the project specified compaction criteria.

If foundations are not poured in a timely manner, the subgrade should be protected with a leanconcrete mud mat to protect the footing subgrades.

Steps should be taken by the contractor to control and remove surface-water runoff and precipitation. When soil is wet and subjected to construction traffic, previously acceptable subgrades can soften and become unacceptable. A smooth-drum roller should be used to seal the surface and provide for better drainage. We also recommend crowning or sloping the subgrade to provide positive drainage off the subgrades.

## Removal/Replacement

If encountered beneath foundations, a minimum of 3-feet of the miscellaneous fill, or otherwise deleterious material, should be removed within the foundation zone of influence (i.e. 1 H to 1 V downward projection from the edge). The resulting material should be proofrolled in accordance with the Subgrade Preparation section outlined herein. The resulting excavation should be backfilled with structural fill in compacted lifts.

An abandoned rubble well was encountered in one test pits (C-B-TP-10B) to a depth of about 9 feet. The well and any surrounding unsuitable fill should be completely removed and the resulting excavation backfilled with structural fill as outlined here.

Placement of additional fill materials in foundation areas, if required, should be performed in accordance with the Excavation, Fill, Placement, and Compaction Criteria recommendations outlined herein.

## Excavation, Fill, Placement, and Compaction Criteria

Excavation through the fill and the underlying sand/silt and glacial till can likely be performed using conventional earthmoving equipment (e.g., backhoes, excavators, dozers, etc.). Excavations made for footings and utilities should be conducted to minimize disturbance to the subgrade (i.e., backhoe with a smooth-edge bucket). Larger equipment may be required for removal of obstructions such as boulders, etc.

Within the proposed building footprint, the top of competent rock (either refusal of the drilling equipment or rock coring) was encountered from about el +108 to el +140 . Though at one test pit location (C-B-TP-10), bedrock was inferred at el +151 ). Given a proposed finished floor elevation of el +150 , rock removal within the proposed building is anticipated.

Within the proposed roadway and site areas, the top of competent rock (either refusal of the drilling equipment or rock coring) was encountered from about el +116 to el +143 . Based on the current site grading, rock removal may be required to the west of the proposed building in the truck court and parking areas.

- Bedrock should be removed to a minimum of 6 inches below the proposed pavement section a minimum of 10 feet horizontal feet beyond. The resulting excavation should be backfilled with compacted $3 / 4$-inch stone. A layer of filter fabric should be placed between the $3 / 4$-inch stone and the pavement section.
- In truck court and parking areas where utilities that are sensitive to settlement transition from bearing on rock to bearing on soil, rock should be over-excavated a minimum of 12 inches for 5 horizontal feet in either direction (total of 10 feet) from the point of bearing material transition and replaced with $3 / 4$-inch gravel to reduce the potential for differential settlements. The specific requirements will be based on the field conditions observed at the subject location, the geotechnical engineer's subsequent recommendations, and the sensitivity of the utility to differential settlement.

Rock excavation techniques will be required to excavate to the required elevations. Blasting may be required. The actual means and methods required for rock excavation should be selected by the contractor based upon experience and capabilities. All blasting should be performed in accordance with the applicable state and local regulations and in a manner such than no on-site or off-site structures or features are adversely impacted.

All excavations should be properly sloped or braced and conform with applicable OSHA regulations including, but not limited to, temporary shoring, trench boxes, temporary rock stabilization, or proper benching or both.

All excavation and backfilling must be performed in accordance with the project environmental engineer's recommendations.

The following types of fill can be used.

Structural Fill - Structural fill should be well-graded sand and gravel having a maximum particle size of 3 inches and no more than $10 \%$ passing the No. 200 sieve. Additionally, the structural fill should be free of organics, clay, roots, concrete, other non-soil constituents, and other deleterious or compressible materials. Any approved imported structural fill should be "certified clean fill" free of hazardous substances and meeting all local, state, federal and the New Hampshire Department Environmental Services regulations.

Material Reuse - The contractor may reuse the on-site granular fill, sand, or glacial till as structural fill provided the soils meet the requirements for structural fill outlined above and is approved by the environmental engineer. The silt may not be used as structural fill. Note that samples obtained within the fill, sand, and glacial till layers have a fines content (material passing the No. 200 sieve) ranging from about 6\% to 48\%; therefore, select soils will be sensitive to moisture. The overall amount of soil that can be reused will be dependent on the amount of fines present within the soil, the contractor's ability to add stone, the time of year the earthwork is carried out (e.g., potentially inclement weather), and the ability of the earthwork contractor to stage, aerate and process the material to facilitate placement and compaction. The existing shallow sand generally has a uniform gradation and low silt content (poorly graded) which may be difficult to compact to specifications without systematic application of water to each layer or blending the material to create a well-graded fill. In addition, the contractor may need to place the material in thinner lifts to achieve the compaction requirements specified herein.

General Fill - On-site soils not meeting the requirements for structural fill can be used as general fill for site landscape and other nonstructural areas (e.g., landscaped areas) if environmentally suitable for reuse. The fill and silt layers may be used as general fill, if required.

Compaction Criteria - All fill should be placed in uniform 12-inch-thick loose lifts and compacted. Fill in landscaped areas should be compacted to $90 \%$ of its maximum dry unit weight as determined by ASTM D1557; all other fill should be compacted to at least $95 \%$. In restricted areas where only hand-operated compactors can be used, the maximum lift


#### Abstract

thickness should be limited to 8 inches. The appropriate water content at the time of compaction should be plus or minus $2 \%$ points of optimum as determined by the laboratory compaction tests of proposed fill. No backfill should be placed on areas where free water is standing or on frozen subsoil areas.


## Groundwater Control

Across the lot, groundwater was encountered from about el +120 to +151 . Based on the proposed grades, we expect that groundwater will be encountered along the western part of the building. Temporary groundwater control in this area, and potentially throughout the site, will be required.

We anticipate that dewatering will be required during construction. Water infiltration to the foundation excavation can likely be controlled using gravity-fed sump pumps via gravel trenches or sumps assisted with collector trenches. Deeper systems such as well points may be required. The final dewatering measures required should be evaluated and designed by the contractor. The dewatering measures implemented should adequately dewater all foundation-related excavations such that compaction of footing subgrades is feasible.

Collection of rainwater runoff will also be needed during the excavation of the removal and replacement program and during the subgrade preparation work. Water runoff is expected to be controlled with the use of gravel-lined collection trenches, pits and submersible pumps. Care should be taken to ensure that drainage is provided during all phases of excavation work. Environmental pretreatment of groundwater, if necessary, is beyond the scope of this study. Collected water should be discharged in accordance with applicable regulations and any environmental requirements.

## SERVICES DURING DESIGN, CONSTRUCTION DOCUMENTS AND CONSTRUCTION QUALITY ASSURANCE

During final design, Langan should be retained to consult with the design team as geotechnical questions arise. Technical specifications and design drawings should incorporate our recommendations. When authorized, we will assist the design team in preparing specification sections related to geotechnical issues such as earthwork, shallow foundations, backfill, retaining walls, and excavation support. Langan should also, when authorized, review the project plans and contractor submittals relating to materials and construction procedures for geotechnical work to confirm the designs incorporate the intent of our recommendations.

Langan has explored and interpreted the site subsurface conditions and developed the foundation design recommendations contained here, and is therefore best suited to perform qualityassurance observation and testing of geotechnical-related work during construction. The work
requiring quality-assurance confirmation or special inspections per the Building Code includes, but is not limited to, earthwork, shallow foundations, backfill, retaining walls, and excavation support.

Recognizing that construction observation is the final stage of geotechnical design, qualityassurance observation during construction by Langan is necessary to confirm the design assumptions and design elements, to maintain our continuity of responsibility on this project, and allow us to make changes to our recommendations, as necessary. The foundation system and general geotechnical construction methods recommended herein are predicated upon Langan's assisting with the final design and providing construction observation services for the owner. If Langan is not retained for these services, we cannot assume the role of geotechnical engineer of record, and the entity providing the final design and construction observation services must serve as the engineer of record.

## LIMITATIONS

The conclusions and recommendations provided in this report result from our interpretation of the geotechnical conditions existing at the site inferred from a limited number of borings and test pits, and information provided by Hillwood. Actual subsurface conditions may vary. Recommendations provided are dependent upon one another and no recommendation should be followed independent of the others.

Any proposed changes in structures or their locations should be brought to Langan's attention as soon as possible so we can determine whether such changes affect our recommendations. Information on subsurface strata and groundwater levels shown on the logs represent conditions encountered only at the locations indicated and at the time of our exploration. If different conditions are encountered during construction, they should immediately be brought to Langan's attention for evaluation because they might affect our recommendations.

This report has been prepared to assist the owner, architect, and structural engineer in the design process and is only applicable to the design of the specific project identified. The information in this report cannot be used or depended on by engineers or contractors involved in evaluations or designs of facilities (including underpinning, grouting, stabilization, etc.) on adjacent properties beyond the limits of that which is the specific subject of this report.

Environmental issues (such as permitting or potentially contaminated soil and groundwater) are outside the scope of this study and are addressed in a separate Langan evaluation.

FIGURES








## APPENDIX A HISTORIC INFORMATION



This report includes information from the following map sheet(s).

NW N NE

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SITE NAME: 59 Steele Road ADDRESS: 59 Steele Road Hudson, NH 03051
CLIENT: Langan Environmental Services

This report includes information from the following map sheet(s).

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| 0 Miles | 0.25 | 0.5 | 1 | 1.5 |

NW N NE

SITE NAME: 59 Steele Road ADDRESS: 59 Steele Road Hudson, NH 03051 CLIENT: Langan Environmental Services


SITE NAME: 59 Steele Road ADDRESS: 59 Steele Road Hudson, NH 03051 Langan Environmental Services
W

SITE NAME: 59 Steele Road ADDRESS: 59 Steele Road Hudson, NH 03051 Langan Environmental Services





SITE NAME: 59 Steele Road ADDRESS: 59 Steele Road CLIENT: $\quad$ Langan Environmental Services



SITE NAME: 59 Steele Road ADDRESS: 59 Steele Road Hudson, NH 03051 CLIENT: Langan Environmental Services



SITE NAME: 59 Steele Road ADDRESS: 59 Steele Road Hudson, NH 03051 CLIENT: Langan Environmental Services





TP, Nashua South, 1965, 7.5-minute

SITE NAME: 59 Steele Road ADDRESS: 59 Steele Road Hudson, NH 03051 Langan Environmental Services



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SITE NAME: 59 Steele Road ADDRESS: 59 Steele Road Hudson, NH 03051 CLIENT: Langan Environmental Services




SITE NAME: 59 Steele Road ADDRESS: 59 Steele Road Hudson, NH 03051 CLIENT: Langan Environmental Services







SITE NAME: 59 Steele Road ADDRESS: 59 Steele Road Hudson, NH 03051
CLIENT: Langan Environmental Services



## APPENDIX B AVAILABLE GEOTECHNICAL REPORT

TAble 1
SUMMARY OF TEST BORINGS AND TEST PITS
River Place
Hudson, New Hampshire

| Test Boring Designation ${ }^{1}$ | Notes | Ground Surface Elev.$+/-(\text { feet })^{2}$ | Exploration Depth (feet) | Groundwater ${ }^{3}$ |  | Thickness of Deposit (feet) |  |  |  |  |  |  | Refusal |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  |  | $\begin{array}{\|c\|} \hline \text { Depth to } \\ \text { (feet) } \\ \hline \end{array}$ | $\begin{gathered} \text { Elev. of } \\ \text { (feet) } \end{gathered}$ | Topsoil | Subsoil | Sult | Sand | Silty Sand | Gravelly Sand | Peat | Depth to (feet) | Elev. of (feet) |
| B-1 | 6 | 136.0 | 30.2 | NA |  |  |  |  | 29.7 |  |  |  | 30.2 | 105.8 |
| B-2 |  | 150.6 | 22.0 | NA |  | 0.2 |  |  | $>21.5$ |  |  |  | NE |  |
| B-3 |  | 138.7 | 22.0 | NA |  | 1.0 | 1.0 | $>13.5$ | 6.5 |  |  |  | NE |  |
| B-4 |  | 132.8 | 22.0 | NA |  | 1.0 | 1.5 | 3.5 | $>16$ |  |  |  | NE |  |
| B-5 | 6 | 153.9 | 13.2 | NA |  | 1.0 |  |  | 11.2 |  |  |  | 13.2 | 140.7 |
| B-6 |  | 119.8 | 22.0 | 15.0 | 104.8 | 0.5 | 1.5 |  |  | $>20$ |  |  | NE |  |
| B-7 |  | 111.2 | 22.0 | 6.0 | 105.2 | 0.5 | 2.0 | $>13.5$ | 6.5 |  |  |  | NE |  |
| B-8 |  | 116.6 | 27.0 | 21.0 | 95.6 | 0.3 | 2.2 |  |  | $>24.5$ |  |  | NE |  |
| B-9 |  | 147.5 | 37.0 | 25.0 | 122.5 |  |  |  | 8.5 | $>28.5$ |  |  | NE |  |
| B-10 (OW) | 4 | 112.9 | 25.0 | 19.6 | 93.3 | 2.0 |  |  | >23 |  |  |  | NE |  |
| B-11 | 6 | 169.6 | 10.5 | NA |  | 1.0 | 1.0 |  |  | 8.0 | 0.5 |  | 10.5 | 159.1 |
| B-12 | 6 | 132.1 | 20.8 | 3.0 | 129.1 | 2.0 |  |  |  | 18.8 |  |  | 20.8 | 111.3 |
| B-13 | 6 | 127.8 | 15.1 | NA |  | 0.5 |  |  |  | 14.6 |  |  | 15.1 | 112.7 |
| B-13A | 6 | 128.1 | 19.7 | 5.6 | 122.5 | 0.5 |  |  |  | 15.0 | 3.6 |  | 19.1 | 109.0 |
| B-14 |  | 133.3 | 11.0 | 3.6 | 129.7 | 1.2 | 1.3 |  | $>8.5$ |  |  |  | NE |  |
| B-I5 |  | 133.7 | 12.0 | 3.7 | 130.0 | 0.5 |  |  | $>11.5$ |  |  |  | NE |  |
| B-16 | 5 | 129.7 | 12.0 | 6.0 | 123.7 | 1.0 | 1.0 |  | $>6$ | 4.0 |  |  | NE |  |
| B-17 (OW) | 5 | 132.6 | 19.0 | 10.3 | 122.3 | 0.5 |  | 7.0 | $>11.5$ |  |  |  | NE |  |
| B-18 | 5 | 132.4 | 12.0 | 5.5 | 126.9 | 1.0 | 1.0 |  | $>10$ |  |  |  | NE |  |
| B-19 | 6 | 149.2 | 16.5 | 15.0 | 134.2 | 1.0 | 1.0 |  | 9.9 | 2.1 | 2.5 |  | 16.5 | 132.7 |
| B-20 (OW) |  | 133.1 | 11.0 | 3.8 | 129.3 | 0.7 | 1.3 |  | $>3.5$ | 5.5 |  |  | NE |  |
| TP-1 |  | 146.6 | 7.0 | NE |  | 0.5 |  |  | 3.5 | $>3$ |  |  | NE |  |
| TP-2 |  | 135.1 | 7.0 | NE |  | 0.3 |  |  |  | $>6.7$ |  |  | NE |  |
| TP-3 |  | 138.5 | 7.0 | NE |  | 0.5 |  |  |  | $>6.5$ |  |  | NE |  |
| TP-4 |  | 157.7 | 6.5 | NE |  | 0.5 |  |  |  |  | $>6$ |  | NE |  |
| TP-5 | 6 | 136.7 | 2.5 | NE |  |  |  |  |  |  | $>2.5$ |  | 2.5 | 134.2 |
| TP-5A | 6 | 136.7 | 2.5 | NE |  |  |  |  | 2.5 |  |  |  | 2.5 | 134.2 |
| TP-6 |  | 131.3 | 7.0 | 7.0 | 124.3 | 1.5 |  |  |  | $>5.5$ |  | , | NE |  |
| TP-7 |  | 138.5 | 7.0 | NE |  | 0.5 |  |  |  | $>6.5$ |  |  | NE |  |
| TP-8 |  | 119.1 | 7.0 | NE |  | 0.5 | 0.8 |  |  | $>5.7$ |  |  | NE |  |
| TP-9 |  | 137.2 | 7.0 | NE |  | 0.7 |  |  |  | $>6.3$ |  |  | NE |  |
| TP-10 |  | 119.0 | 7.0 | NE |  | 0.5 |  |  |  | $>6.5$ |  |  | NE |  |
| TP-11 |  | 109.6 | 7.0 | NE |  | 1.5 |  | $>5.5$ |  |  |  |  | NE |  |
| TP-12 |  | 134.1 | 7.0 | NE |  | 0.5 |  | 4.0 | $>2.5$ |  |  |  | NE |  |
| TP-13 |  | 139.9 | 6.5 | NE |  | 0.4 |  |  | $>5$ | 1.1 |  |  | NE |  |
| TP-14 |  | 138.1 | 6.0 | NE |  | 0.3 |  |  | $>4.5$ | 1.2 |  |  | NE |  |
| TP-15 | 7 | 150.0 | 6.5 | NE |  | 0.5 |  |  | $>2.2$ | 3.8 |  |  | NE |  |
| TP-16 |  | 142.5 | 7.0 | NE |  | 0.8 |  |  | $>4.8$ | 1.4 |  |  | NE |  |
| TP-17 |  | 135.8 | 7.0 | NE |  | 0.5 |  |  | $>5$ | 1.5 |  |  | NE |  |
| TP-18 |  | 126.5 | 6.5 | 5.4 | 121.1 | 0.2 |  |  |  | 4,0 |  | $>2.5$ | NE |  |
| TP-19 |  | 127.7 | 7.0 | NE |  | 0.8 |  |  |  | $>6.2$ |  |  | NE |  |
| TP-20 |  | 133.2 | 7.0 | 4.8 | 128.4 | 0.7 |  |  |  | $>6.3$ |  |  | NE |  |
| TP-21 |  | 127.7 | 6.8 | 6.7 | 121.0 | 0.5 |  |  |  | $>6.3$ |  |  | NE |  |
| TP-22 |  | 146.3 | 7.0 | NE |  | 0.4 |  | $>0.8$ | 5.8 |  |  |  | NE |  |

. Refer to Appendix B for test boring logs and Appendix C for test pit logs.
2. Approximate ground surface elcvation information was interpolated from survey information presented on a plan entitled "Boring/Test Pit/Observation Well Location Plan, 59 Steele Road, Hudson, New Hampshire," prepared by Hayner/Swanson, Inc. of Nashua, New Hampshire, dated April 2006.
3. Groundwater readings shown for test borings with observation wells installed were measured in groundwater observation wells on April 14, 2006. Italicized groundwater readings represent groundwater readings taken during drilling or test pit excavation and do not represent stabilized levels.
4. Cobble layer encountered from 15 to $\mathbf{I} .5$ feet below ground surface. Sand deposit thickness shown does not include cobble layer thickness.
5. Boring terminated due to running sands.
6. Refusal encountered due to boulders or bedrock.
7. Approximate ground surface elevation was interpolated from topography site plan provided by Hayner Swanson.

TABLE 2

## SUMMARY OF LABORATORY TESTING

River Place
Hudson, New Hampshire

| Boring / Test Pit <br> No. | Sample <br> No. | Depth (feet) | Soil Description | Grain Size Distribution |  |  | $\begin{gathered} \text { Natural } \\ \text { Water } \\ \text { Content (\%) } \end{gathered}$ |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  |  | Gravel | Sand | Silt |  |
| B-1 | S-3 | 10-12 | Fine to coarse SAND, some Gravel, trace Silt | 21.0 | 73.7 | 5.3 | 4.4 |
| B-2 | S-2 | 5-7 | Medium to coarse SAND, little Gravel, trace Silt | 15.0 | 80.0 | 5.0 | 3.1 |
| B-3 | S-2 | 5-7 | Medium to coarse SAND and Gravel, trace Silt | 36.9 | 60.4 | 2.7 | 3.3 |
| B-4 | S-2A | 5-6.8 | SILT and fine Sand | 0.1 | 48.8 | 51.1 | 20.4 |
| B-5 | S-3 | 10-12 | Fine to medium SAND, some Gravel, little Silt | 33.9 | 46.3 | 19.8 | 5.0 |
| B-8 | S-2 | 5-7 | Fine to medium SAND, some Silt | 0.0 | 73.4 | 26.6 | 7.5 |
| B-9 | S-2 | 5-7 | Fine to medium SAND, trace Silt | 0.2 | 95.9 | 3.9 | 5.8 |
| B-11 | S-2 | 4-6 | Fine to medium SAND, some Silt | 0.1 | 79.4 | 20.5 | 7.0 |
| B-15 | S-2 | 5-7 | Fine to coarse SAND, little Silt, trace Gravel | 7.5 | 75.9 | 16.6 | 24.3 |
| B-16 | S-1B | 0-2 | SILT, trace fine Sand | 0.0 | 4.8 | 95.2 | 33.7 |
| B-17(0W) | S-2 | 4-6 | SILT, some fine Sand | 0.2 | 30.0 | 69.8 | 25.4 |
| B-18 | S-3 | 10-12 | Fine to medium SAND, trace Silt | 0.0 | 93.2 | 6.8 | 26.4 |
| TP-1 | S-3 | 3.5 | Medium to coarse SAND, little Gravel, trace Silt | 10.8 | 85.6 | 3.6 | 4.0 |
| TP-2 | S-2 | 1.5 | SILT and fine Sand | 0.0 | 44.1 | 55.9 | 13.3 |
| TP-4 | S-1 | 2 | GRAVEL and medium to coarse Sand, trace Silt | 51.3 | 44.0 | 4.7 | 4.4 |
| TP-5A | S-1 | 1 | Fine to coarse SAND, some Silt, little Gravel | 19.6 | 55.5 | 24.9 | 7.6 |
| TP-6 | S-2 | 2-3 | Fine to medium SAND, some Silt | 0.0 | 68.0 | 32.0 | 14.1 |
| TP-9 | S-2 | 2 | Fine to medium SAND and Silt, trace Gravel | 5.3 | 59.3 | 35.4 | 10.2 |
| TP-13 | S-3 | 3 | Medium to coarse SAND, trace Silt | 0.5 | 97.5 | 2.0 | 4.4 |

Notes:

1. Refer to Appendix D for laboratory results.























## APPENDIX C

TEST PIT LOGS



Notes:

1. Soil samples were scre ened for total volatile organic compounds (VOCS) using a TEI Model 580 b organic vapor meter referenced to an isobutylene-in-air standard. Total VOCS detected are reported in parts per million (ppm) in the "Field Test Data" column. "ND" indicates no VOCS detected.

| Test Pit Plan |  | Boulder Class | Prop | ons Used | Abbreviations | GROUNDWATER |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| 8 | Letter Desigralion | Size Range Classification |  |  | $\mathrm{F}=$ Fine |  |  |
|  | A | $6^{\prime \prime}$ - $17^{\prime \prime}$ | TRACE (TR.) | 0. 10\% | $\mathrm{M}=$ Medium | ( ) Encountered |  |
|  | B | $18^{\prime \prime}=36$ |  |  | $C=$ Coarse | (X) Not Encountered |  |
|  | c | $36^{\prime \prime}$ and Larger | LITTLE (LI.) | 10-20\% | $\mathrm{V}=$ Very |  |  |
|  |  | Excavation Efort | SOME (SO) | 20-35\% | FM $=$ Finte to mediunt | Elapsed Time to | Depth to |
| NORTH |  | Excavalion Efort | Some (SO) | 20-35\% | $\mathrm{GR}=\mathrm{Gray}$ | Reading (Hours) | Groundwater |
| Volume $=$ 3,1 cu. yd. |  | M ----Moderate | AND | 35-50\% | $\mathrm{BN}=$ Browm |  |  |
|  |  | D....-Difficult |  |  | YEL $=$ Yellow |  |  |

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Notes:

1. Soil samples were screened for total volatile organic compounds (VOCS) using a TEI Model 580 b organic vapor meter referenced to an isobutylene-in-air standard. Total VOCS detected are reported in parts per million (ppm) in the "Field Test Data" column. "ND" indicates no VOCS detected.

| Test Pit Filan |  | Boulder Class | Prop | ons Used | Abbreviations | GROUNDWATER |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| 8 | Lelter Desigration | Size Range Classification |  |  | $\mathrm{F}=$ Fine |  |  |
|  | A | $6^{\prime \prime}-17{ }^{\prime \prime}$ | TRACE (TR.) | 0-10\% | $\mathrm{M}=$ Medium | ( ) Encountered |  |
|  | B | $18^{\prime \prime}-36^{\prime \prime}$ |  |  | $\mathrm{C}=$ Coarse | (X) Not Encountered |  |
| 4 | C | 36" and Larger | LITTLE (LI.) | 10-20\% | $\mathrm{V}=\mathrm{V}$ ery |  |  |
|  |  |  |  |  | F/M $=$ Fine to medium | Elapsed Time lo | Deput to |
| NORTH |  | Excavation Effort E-W-Esy | SOME (SO.) | 20-35\% | F/C = Fine to coarse $G R=G r a y$ | Reading (Hours) | Groundwater |
| Volume $=8.3 \mathrm{cu} . \mathrm{yd}$. |  | M ---Moderate | AND | 35-50\% | $\mathrm{BN}=$ Brown |  |  |
|  |  | D - - Dimicult |  |  | YEL, - Yellow |  |  |





Notes:

1. Soil samples were screened for total volatile organic compounds (VOCS) using a TEI Model 580b organic vapor meter referenced to an isobutylene-in-air standard. Total VOCS detected are reported in parts per million (ppm) in the "Field Test Data" column. "ND" indicates no VOCS detected.



Notes:

1. Soil samples were screened for total volatile organic compounds (VOCS) using a TEI Model 580 b organic vapor meter referenced to an isobutylene-in-air standard. Total VOCS detected are reported in parts per million (ppm) in the "Field Test Data" column. "ND" indicates no VOCS detected.






## Notes:

1. Soil samples were screened for total volatile organic compounds (VOCS) using a TEI Model 580 b organic vapor meter referenced to an isobutylene-in-air standard. Total VOCS detected are reported in parts per million (ppm) in the "Field Test Data" column. "ND" indicates no VOCS detected.

|  | Letter Designation A B C | Boulder Class |  |  | Abbreviations$\begin{aligned} & \mathrm{F}=\text { Fine } \\ & \mathrm{M}=\text { Medlum } \\ & \mathrm{C}=\text { Coarse } \\ & \mathrm{V}=\mathrm{Very} \end{aligned}$ | GROUNDWATER |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  | Sizc Range Classification$\begin{gathered} 6^{\prime \prime}-37^{\prime \prime} \\ 18^{\prime \prime} \cdot 36^{\prime \prime} \\ 36^{\prime \prime} \text { and Larger } \end{gathered}$ | TRACE (TR) 0-10\% |  |  |  |  |
|  |  |  |  |  | $\begin{array}{ll}\text { ( } \mathrm{X} \text { ) } & \text { Encountered } \\ \text { Not Encountered }\end{array}$ |  |
|  |  |  |  |  |  |  |  |  |
|  |  |  | LITTLE (LI.) | 10-20\% |  |  |
|  |  |  | SOME (SO.) | 20-35\% | F/M = Finc to medium <br> F/C $=$ Fine to coarse <br> $G R=G r a y$ | Elapsed Time to Reading (Hours) | Depth to Groundwater |
|  |  | Excavation Effort |  |  |  |  |  |
|  |  | M --.-Moderate | AND | 35-50\% | $\mathrm{BN}=$ Brown |  |  |
|  |  | D .....Difficult |  |  | YEL, $\times$ Yellow |  |  |





Notes:

1. Soil samples were screened for total volatile organic compounds (VOCS) using a TEI Model 580 b organic vapor meter referenced to an isobutylene-in-air standard. Total VOCS detected are reported in parts per million (ppm) in the "Field Test Data" column. "ND" indicates no VOCS detected.





Notes:

1. Frost encountered.
2. Soil samples were screened for total volatile organic compounds (VOCS) using a TEI Model 580 b organic vapor meter referenced to an isobutylene-in-air standard. Total VOCS detected are reported in parts per million (ppm) in the "Field Test Data" column. "ND" indicates no VOCS detected.


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Notes:

1. Soil samples were screened for total volatile organic compounds (VOCS) using a TEI Model 580b organic wapor meter referenced to an isobutylene-in-air standard, Total VOCS detected are reported in parts per million (ppm) in the "Field Test Data" column. "ND" indicates no VOCS detected.



|  |  |  | cavation E | ipment |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| GZA Rep. | C. Melby | Contractor | New Han | shire Bo | g, Inc. | Date | 3/28/2006 |
|  |  | Operator |  | att Stone |  | Ground Elev. | 133.2 feet |
| Weather | Sunny, 50s | Make | Komatsu | Model | PC 27 | Time Started | 1115 |
|  |  | Capacity | 1.5 feet $^{3}$ | Reach | 10 feet | Time Completed | 1140 |



Notes:
I. Sail samples were screened for total volatile organic compounds (VOCS) using a TEI Model 580 b organic vapor meter referenced to an isobutylene-in-air standard. Total VOCS detected are reported in parts per million (ppm) in the "Field Test Data" column. "ND" indicates no VOCS detected.

|  | Letter Designation$\begin{aligned} & \text { A } \\ & \text { B } \end{aligned}$$\mathrm{c}$ | $\begin{aligned} & \hline \text { Boulder Class } \\ & \text { Size Range Clasgification } \\ & 6 "=17^{\prime \prime} \\ & 18^{\prime \prime}-36^{\prime \prime} \\ & 36^{\prime \prime} \text { and Larger } \end{aligned}$ | Proportions Used |  |  | OROUNDWATER$\left(\begin{array}{ll}x & \text { Enceuntered } \\ \left(\begin{array}{ll}\text { ( }\end{array}\right. & \text { Not Encountered }\end{array}\right.$ |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  |  |  |  |  |  |
|  |  |  | TRACE (TR.) | 0-10\% |  |  |  |
|  |  |  |  |  |  |  |  |
|  |  |  | LITILE (LI.) | 10-20\% |  |  |  |
|  |  | Excavation Effort E--Easy M -...Moderate D .....-Difficult | SOME (SO.) | 20-35\% |  | Elapsed Time ta Reading (Hours) | Depth to Groundwaler |
|  |  |  | AND | 35-50\% |  | 5 minutes | 4.8 feet |
|  |  |  |  |  |  |  |  |
| GZA GeoEnvironmental, Inc. $\quad$ p:04jobs $104.0024050 .00104 .0024050 .011 /$ tplog. $\times$ is |  |  |  |  |  |  |  |



## Notes:

1. Soil samples were screened for total volatile organic compounds (VOCS) using a TEI Model 580b organic vapor meter referenced to ant isobutylene-in-air standard. Total VOCS detected are reported in parts per million (ppm) in the "Field Test Data" column. "ND" indicates no VOCS detected.

| Test Pit Plan | ```Lemer Designation A B C``` | Boulder Class | Proportions Used |  | $\mathrm{F}=\text { Fine }^{\text {Abbreviations }}$ | GROUNDWATER |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  | Size Range Classification |  |  |  |  |  |
|  |  | $6^{4 \prime}-17{ }^{\prime \prime}$ | TRACE (TR.) | 0.10\% | $\mathrm{M}=$ Medium | (X) Encountered |  |
|  |  | $18^{n}-36^{\prime \prime}$ |  |  | $C=$ Coarsc | ( ) Nol Encountered |  |
|  |  | $36^{4 \prime}$ and Larger | LITTLE (LI.) | 10-20\% | $\mathrm{V}=\mathrm{V}$ ¢ry |  |  |
|  |  |  |  |  | F/M = Fine to medium | Elapsed time to | Depth to |
|  |  | Excavation Effort | SOME (SO.) | 20-35\% | F/C = Fine to coarse | Reading (Hours) | Groundwater |
| Volume $=$. 3.0 cu. yd. |  | E -m- Easy | AND | 35-50\% | BN = Brawn | 5 minutes | 6.7 fect |
|  |  | D ....-Dificult | AND | J. 5 | YEL, $=$ Yellow |  |  |



## APPENDIX D

LABORATORY TESTING

## Particle Size Distribution Report



GRAIN SIZE - mm.

| $\%+3 "$ | \% Gravel |  |  | \% Sand |  |  | \% Fines |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | Coarse | Medium | Fine | Coarse | Medium | Fine |  |
| 0.0 | 0.0 | 12.8 | 8.2 | 15.8 | 39.9 | 18.0 |  |


| SIEVE <br> SIZE | PERCENT <br> FINER | SPEC.* <br> PERCENT | PASS? <br> (X=NO) |
| :---: | :---: | :---: | :---: |
| 1 | 100.0 |  |  |
| $3 / 4$ | 96.7 |  |  |
| $1 / 2$ | 91.0 |  |  |
| $3 / 8$ | 87.2 |  |  |
| $\# 4$ | 84.3 |  |  |
| $\# 10$ | 79.0 |  |  |
| $\# 20$ | 69.5 |  |  |
| $\# 40$ | 51.4 |  |  |
| $\# 60$ | 23.3 |  |  |
| $\# 100$ | 9.8 |  |  |
| $\# 200$ | 5.3 |  |  |
|  |  |  |  |
|  |  |  |  |
|  |  |  |  |
|  |  |  |  |
|  |  |  |  |

Material Description
Brown, fine to coarse SAND, some Gravel, trace Silt.
(no specification provided)

## Sample Number: S-3

Source of Sample: B-1
Depth: 10-12 ft.

## Date:



## Particle Size Distribution Report



| SIEVE <br> SIZE | PERCENT <br> FINER | SPEC.* <br> PERCENT | PASS? <br> (X=NO) |
| :---: | :---: | :---: | :---: |
| $3 / 4$ | 100.0 |  |  |
| $1 / 2$ | 96.3 |  |  |
| $3 / 8$ | 93.3 |  |  |
| $\# 4$ | 81.3 |  |  |
| $\# 10$ | 63.1 |  |  |
| $\# 20$ | 36.7 |  |  |
| $\# 40$ | 13.5 |  |  |
| $\# 60$ | 6.3 |  |  |
| $\# 100$ | 3.9 |  |  |
| $\# 200$ | 2.7 |  |  |
|  |  |  |  |
|  |  |  |  |
|  |  |  |  |

(no specification provided)
Sample Number: S-2
Source of Sample: B-3
Depth: 5-7 ft.
Date:
GZA GeoEnvironmental, Inc.
CIfent: W/S Development Associates, LLC
Project: River Place Hudson, NH

(no specification provided)

## Sample Number: S-2A <br> Source of Sample: B-4

Depth: 5-6.8 ft.

## Date:




(no specification provided)
Sample Number: S-2
Source of Sample: B-9
Depth: 5-7 ft.

## Date:

Project: River Place Hudson, NH

(no specification provided)

## Sample Number: S-2

Source of Sample: B-11 Depth: 4-6 ft.

## Date:

## Particle Size Distribution Report



Sample Number: S-2
Source of Sample: B-15
Depth: 5-7 ft.

## Date:


(no specification provided)
Sample Number: S-1B
Source of Sample: B-16
Depth: 0-2 ft.
Date:
GZA GeoEnvironmental, Inc. Client: W/S Development Associates, LLC
Project: River Place Hudson, NH

## Particle Size Distribution Report



Sample Number: S-2B
Source of Sample: B-17
Depth: 4-6 ft.

## Date:

## Particle Size Distribution Report



GRAIN SIZE - mm.

| \% +3" | \% Gravel |  | \% Sand |  |  | \% Fines |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 0.0 | 0.5 | 32.1 | 60.6 | 6.8 |  |


(no specification provided)

Material Description
Brown, fine to medium SAND, trace Silt.

| PL= | Atterberg Limits LL= | $\mathrm{Pl}=$ |
| :---: | :---: | :---: |
| $\begin{aligned} & \mathrm{D}_{85}=0.6726 \\ & \mathrm{D}_{30}=0.1717 \\ & \mathrm{C}_{\mathrm{u}}=3.65 \end{aligned}$ | $\begin{aligned} & \text { Coefficients } \\ & \mathrm{D}_{60}=0.3367 \\ & \mathrm{D}_{15}=0.1167 \\ & \mathrm{C}_{\mathrm{C}}=0.95 \end{aligned}$ | $\begin{aligned} & \mathrm{D}_{50}=0.2538 \\ & \mathrm{D}_{10}=0.0922 \end{aligned}$ |
| USCS= | Classification AASHTO $=$ <br> Remarks |  |

Sample Number: S-3
Source of Sample: B-18
Depth: $10-12 \mathrm{ft}$

## Date:

## Particle Size Distribution Report



| GRAIN SIZE - mm. |  |  |  |  |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| $\%+\mathbf{3 "}^{\prime \prime}$ | \% Gravel |  |  | \% Sand |  | \% Fines |  |
|  | Coarse | Medium | Fine | Coarse | Medlum |  | \% |
| 0.0 | 0.0 | 1.9 | 8.9 | 33.2 | 47.1 | 5.3 | 3.6 |


| SIEVE <br> SIZE | PERCENT FINER | SPEC.* <br> PERCENT | PASS? ( $\mathrm{X}=\mathrm{NO}$ ) |
| :---: | :---: | :---: | :---: |
| 1/2 | 100.0 |  |  |
| 3/8 | 98.1 |  |  |
| \#4 | 94.1 |  |  |
| \#10 | 89.2 |  |  |
| \#20 | 78.2 |  |  |
| \#40 | 31.2 |  |  |
| \#60 | 8.9 |  |  |
| \#100 | 5.0 |  |  |
| \#200 | 3.6 |  |  |
|  |  |  |  |
|  |  |  |  |
|  |  |  |  |
|  |  |  |  |
|  |  |  |  |

(no specification provided)

## Sample Number: S-3

Source of Sample: TP-1
Depth: 3.5 ft .

## Date:

## Particle Size Distribution Report



## Particle Size Distribution Report







## APPENDIX C BORING LOGS























































































## APPENDIX D TEST PIT LOGS

































## APPENDIX E TEST PIT PHOTOGRAPHS

## C-B-TP-01



151010101
Hudson Logistics Center
Hudson, NH


## C-B-TP-02



## C-B-TP-03



151010101
Hudson Logistics Center
Hudson, NH


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## C-B-TP-04



151010101
Hudson Logistics Center Hudson, NH


## C-B-TP-05



151010101
Hudson Logistics Center

## C-B-TP-06



151010101
Hudson Logistics Center Hudson, NH


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## C-B-TP-07



151010101
Hudson Logistics Center

## C-B-TP-08



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Hudson Logistics Center
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Hudson, NH

## C-B-TP-09



151010101

## C-B-TP-10




151010101 Hudson Logistics Center Hudson, NH

## C-B-TP-10A



151010101
Hudson Logistics Center Hudson, NH


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## C-B-TP-10B



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## C-B-TP-11



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## C-B-TP-12



151010101
Hudson Logistics Center

## C-R-TP-01




151010101
Hudson Logistics Center Hudson, NH

## C-R-TP-02




151010101 Hudson Logistics Center Hudson, NH

## C-R-TP-03



151010101
Hudson Logistics Center
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## C-S-TP-01



151010101
Hudson Logistics Center Hudson, NH


LANGAN

## C-S-TP-02



151010101
Hudson Logistics Center Hudson, NH


LANGAN

## C-S-TP-03




## C-S-TP-04



151010101
Hudson Logistics Center Hudson, NH


## C-S-TP-05



151010101
Hudson Logistics Center
Hudson, NH


LANGAN

## C-S-TP-06



151010101
Hudson Logistics Center Hudson, NH


LANGAN

## C-S-TP-07



## C-S-TP-08




## C-S-TP-11



151010101
Hudson Logistics Center Hudson, NH


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## C-S-TP-13



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Hudson Logistics Center
LANGAN

## C-S-TP-14



151010101
Hudson Logistics Center
Hudson, NH


LANGAN

## C-S-TP-15



151010101
Hudson Logistics Center Hudson, NH


## C-S-TP-16



151010101
Hudson Logistics Center Hudson, NH


LANGAN

## C-S-TP-17



151010101
Hudson Logistics Center

## APPENDIX F WELL CONSTRUCTION LOGS \& READINGS

Lot C
Summary of Groundwater Elevations
Hudson, New Hampshire
Langan Project No.: 151010101

| Monitoring Well Lot ID | C |  |  |  |
| :---: | :---: | :---: | :---: | :---: |
| Monitoring Well ID | C-B-BOR-02(OW) | C-S-BOR-04(OW) | C-B-BOR-16(OW) | C-B-BOR-20(OW) |
| Ground Surface Elevation (feet) | 132.0 | 130.5 | 158.0 | 156.5 |
| Installation Date | 6/13/2020 | 6/18/2020 | 6/17/2020 | 6/8/2020 |
| Reference Point | Ground Surface | Ground Surface | Ground Surface | Ground Surface |
| June 20, 2020 |  |  |  |  |
| Depth to Groundwater (feet) | 5.0 | NM | 13.9 | 12.8 |
| Groundwater Elevation (feet) | 127.0 | NA | 144.1 | 143.7 |
| June 30, 2020 |  |  |  |  |
| Depth to Groundwater (feet) | 5.0 | 6.9 | 14.3 | NM |
| Groundwater Elevation (feet) | 127.0 | 123.6 | 143.7 | NA |
| July 1, 2020 |  |  |  |  |
| Depth to Groundwater (feet) | NM | NM | NM | NM |
| Groundwater Elevation (feet) | NA | NA | NA | NA |
| July 19, 2020 |  |  |  |  |
| Depth to Groundwater (feet) | 5.7 | NM | NM | NM |
| Groundwater Elevation (feet) | 126.3 | NA | NA | NA |
| July 20, 2020 |  |  |  |  |
| Depth to Groundwater (feet) | 6.5 | 7.5 | 14.9 | 13.7 |
| Groundwater Elevation (feet) | 125.5 | 123.0 | 143.1 | 142.8 |
| July 29, 2020 |  |  |  |  |
| Depth to Groundwater (feet) | 6.6 | 7.6 | 15.0 | 13.9 |
| Groundwater Elevation (feet) | 125.4 | 122.9 | 143.0 | 142.6 |

Notes:

1. "Depth to Groundwater" results are shown in feet below ground surface. "Groundwater Elevation" is given in feet and references the National Geodetic Vertical Datum of 1929 (NGVD 1929).
2. Ground surface elevations were estimated by Langan by interpolating between the ground surface contours shown on the existing conditions plan provided by Hayner/Swanson, Inc. (HSI) of Nashua, New Hampshire. As such, the elevations should be considered approximate.
3. Abbreviations
$\mathrm{NI}=$ Not Installed
NA = Not Applicable
NM = Not Measured





## APPENDIX G LABORATORY TESTING RESULTS

## Moisture Content of Soil and Rock - ASTM D2216

| Boring ID | Sample ID | Depth | Description | Moisture Content,\% |
| :---: | :---: | :---: | :---: | :---: |
| C-B-BOR-04 | S- 2 | 2-4 ft | Moist, light olive brown silty sand with gravel | 8.5 |
| C-B-BOR-08 | S-4 | 6-8 ft | Moist, light yellowish brown sand with silt | 9.1 |
| C-B-BOR-11 | S- 5 | 8-10 ft | Moist, light olive brown silty sand with gravel | 9.2 |
| C-B-TP-02 | G-1 | 3-4 ft | Moist, dark olive brown silt with organics | 48.3 |
| C-B-TP-06 | S-1 | 0-5 ft | Moist, olive yellow sand | 4.7 |
| C-B-TP-07 | S-1 | 0-5 ft | Moist, light olive brown silty sand | 3.6 |
| C-S-BOR-09 | S- 6 | 10-12 ft | Moist, dark grayish brown silty sand | 10.3 |
| C-S-BOR-12 | S- 5 | $8-10 \mathrm{ft}$ | Moist, light brownish gray silty sand | 11.4 |
| C-S-BOR-20 | S-3 | $4-6 \mathrm{ft}$ | Moist, olive yellow sand with gravel | 2.0 |
| C-S-BOR-22 | S- 7 | 15-17 ft | Moist, olive brown silty sand | 13.2 |



## Moisture Content of Soil and Rock - ASTM D2216

| Boring ID | Sample ID | Depth | Description | Moisture <br> Content, $\%$ |
| :---: | :---: | :---: | :---: | :---: |
| C-S-BOR-23 | S-5 | $8-10 \mathrm{ft}$ | Moist, olive brown sand with gravel | 11.1 |
| C-S-TP-01 | G-1 | 4 ft | Moist, light olive brown sandy silt | 31.7 |
| C-S-TP-17 | G-1 | 2.5 ft | Moist, light yellowish brown sandy silt | 18.4 |


|  | Client: <br> Project: <br> Location: | Lang <br> Proj <br> Hud |  |  | Project No: | GTX-311848 |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| 1 | Boring ID: | --- | Sample Type: | --- | Tested By: | ckg |
| E X P R E S S | Sample ID Depth : |  | Test Date: <br> Test Id: | $\begin{aligned} & 08 / 05 / 20 \\ & 567302 \end{aligned}$ | Checked By: | bfs |

## Amount of Material Passing \#200 Sieve - ASTM D1140

| Boring ID | Sample ID | Depth | Visual Description | Fines, \% |
| :---: | :---: | :---: | :---: | :---: |
| C-B-BOR-05 | $\mathrm{S}-5$ | $8-10 \mathrm{ft}$ | Moist, olive brown silt | 90.6 |

EXPRESS

| Client: | Langan Engineering |  |  |  |
| :--- | :--- | :--- | :--- | :--- |
| Project: | Project Hudson |  |  |  |
| Location: | Hudson, NH |  | Project No: | GTX-311848 |
| Boring ID: | C-B-TP-02 | Sample Type: | jar | Tested By: |
| Sample ID: | G-1 |  | Test Date: | $06 / 26 / 20$ |
| Cam | Checked By: | jsc |  |  |
| Depth: | $3-4 \mathrm{ft}$ |  | Test Id: | 561437 |
| Test Comment: | --- |  |  |  |
| Visual Description: | Moist, dark olive brown silt with organics |  |  |  |
| Sample Comment: | --- |  |  |  |

## Moisture, Ash, and Organic Matter - ASTM D2974

| Boring ID | Sample ID | Depth | Description | Moisture <br> Content, $\%$ | Ash <br> Content,\% | Organic <br> Matter,\% |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| C-B-TP-02 | G-1 | $3-4 \mathrm{ft}$ | Moist, dark olive brown silt <br> with organics | 48 | 93.3 | 6.7 |


|  | Client: Langan Engineering <br> Project: Project Hudson <br> Location: Hudson, NH |  |  |  | Project No: | GTX-311848 |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | Boring ID: C-B-BOR-04 |  | Sample Typ | jar | Tested By: | ckg |
| EXPRESS | Sample ID: S-2 |  | Test Date: | 06/30/20 | Checked By: | bfs |
|  | Depth: $2-4 \mathrm{ft}$ |  | Test Id: | 561420 |  |  |
|  | Visual Description: Sample Comment: | Moist, light olive brown silty sand with gravel --- |  |  |  |  |

Particle Size Analysis - ASTM D6913


| ASTM | N/A Classification |
| :--- | :--- |
| AASHTO | Silty Gravel and Sand (A-2-4 (0)) |

Sample/Test Description
Sand/Gravel Particle Shape: ANGULAR
Sand/Gravel Hardness : HARD


## Particle Size Analysis - ASTM D6913





| ASTM | N/A Classification |
| :--- | :--- |
| AASHTO | Silty Gravel and Sand (A-2-4 (0)) |

Sample/Test Description
Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD

Particle Size Analysis - ASTM D6913


| ASTM | N/A Classification |
| :--- | :--- |
| AASHTO | Silty Gravel and Sand (A-2-4 (0)) |

Sample/Test Description
Sand/Gravel Particle Shape: ANGULAR
Sand/Gravel Hardness : HARD

Particle Size Analysis - ASTM D6913


| ASTM | N/A Classification |
| :--- | :--- |
| AASHTO | Silty Gravel and Sand (A-2-4 (0)) |

Sample/Test Description
Sand/Gravel Particle Shape: ANGULAR
Sand/Gravel Hardness : HARD


## Particle Size Analysis - ASTM D6913



|  | Client: Langan Engineering <br> Project: Project Hudson <br> Location: Hudson, NH |  |  |  |  | Project No: |  | GTX-311848 |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | Boring ID: C-S-BOR-22 |  |  | Sample Typ | jar | Tested By: | ckg |  |
| EXPRESS | Sample ID: S-7 |  |  | Test Date: | 06/30/20 | Checked By: | bfs |  |
|  | Depth : 15-17 ft |  |  | Test Id: | 561424 |  |  |  |
|  | Test Comm Visual Des Sample Co | ent: <br> ription: <br> mment |  | rown silty sa |  |  |  |  |

## Particle Size Analysis - ASTM D6913



| ASTM | N/A Classification |
| :--- | :--- |
| AASHTO | Silty Gravel and Sand (A-2-4 (0)) |

Sample/Test Description
Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD


## Particle Size Analysis - ASTM D6913



Particle Size Analysis - ASTM D422


Sample/Test Description
Sand/Gravel Particle Shape : ---
Sand/Gravel Hardness : ---

Particle Size Analysis - ASTM D422


Sample/Test Description
Sand/Gravel Particle Shape : ---
Sand/Gravel Hardness : ---



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 GEOTESTING EXPRESS INCORPORATED 125 NAGOG PARK
ACTON MA 01720-3451
USA

| Analysis No. | TS-A2008783 |
| :--- | :--- |
| Report Date | 01 July 2020 |
| Date Sampled | 25 June 2020 |
| Date Received | 29 June 2020 |
| Where Sampled | Acton, MA USA |
| Sampled By | Client |

This is to attest that we have examined: Soil for Project Name: Project Hudson; Site Location Hudson, NH; Job Number: GTX-311848

When examined to the applicable requirements of:

| ASTM D 512-12 | "Standard Test Methods for Chloride Ion in Water" Method B |
| :--- | :--- |
| ASTM D 516-16 | "Standard Test Method for Sulfate Ion in Water" |

Results:
ASTM D 512 - Chloride Method B

| Sample |  | Results |  | Detection Limit |
| :---: | :---: | :---: | :---: | :---: |
|  |  | ppm (mg/kg) | \% ${ }^{1}$ |  |
| C-B-TP-06 |  | 22. | 0.0022 | 10. |
| S-1 | 0-5' |  |  |  |
| C-B-TP-07 |  | 14. | 0.0014 |  |
| S-1 | 0-5' |  |  |  |

NOTE: ${ }^{1}$ Percent by weight as received.
ASTM D 516 - Sulfates (Soluble)

| Sample |  | Results |  | Detection Limit |
| :---: | :---: | :---: | :---: | :---: |
|  |  | ppm (mg/kg) | \% ${ }^{1}$ |  |
| C-B-TP-06 |  | <10 | $<0.0010$ | 10. |
| S-1 | 0-5' | <10. | <0.0010 |  |
| C-B-TP-07 |  | <10. | <0.0010 |  |
| S-1 | 0-5' |  |  |  |

NOTE: ${ }^{1}$ Percent by weight as received
USEPA Laboratory ID UT00930

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## APPENDIX H INFILTRATION TEST LOGS

## LANGAN

INFILTRATION TESTS
C-IT-01 performed in C-S-TP-01

| PROJECT Project Hudson |  |  | PROJECT NO. 151010101 |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| LOCATION | 59 Steele Road, Hudson, NH |  | 6/17/2020 to 6/18/2020 |  |  |  |
| INSPECTOR | Taylor Sisti |  | Sunny, 80s ${ }^{\circ} \mathrm{F}$ |  |  |  |
| PRESOAK $\quad$ Start ${ }^{\text {End }}$ | TIME | DEPTH OF WATER IN HOLE (INCH) | ELEVATION AND DATUM |  |  |  |
|  | 11:44 | 24 | Surface Elevation | Approx. | 127.5 | (NGVD29) |
|  | 13:00 | 1.5 | Top of Hole Elevation | Approx. | 125.5 | (NGVD29) |
| *presoak allowed to continue overnight |  |  | Bottom of Hole Elevation | Approx. | 123.5 | (NGVD29) |

## METHOD OF INFILTRATION TEST

C-S-TP-01 was advanced to a depth of about 2 feet below existing grade. An about 6 -inch diameter, 24 -inch deep hole was dug by hand with a post hole digger. The circumfrence of the hole was then lined with a 6 -inch diameter, 30 -inch long PVC pipe. Before running infiltration tests, the hole was presoaked with 24 inches of water and allowed to drain overnight. The infiltration testing hole was free of water the following morning prior to starting infiltration testing. For each infiltration test, the hole was filled with water to a predetermined depth of 24 inches. Then, the time was recorded after one hour or the time for the water to drain 24 inches was recorded. The tables below outline the calculations for determining the average rate in which the water dissipated. Test pit C-S-TP-01 was advanced to to termination depth following completion of the infiltration test. Groundwater was encountered at about 7.1 ft below grade.

|  | TIME (SEC) | DEPTH OF WATER (IN) | TIVE INTERVAL (SEC) | RATE (IN/MIN) | RATE (IN/HOUR) |  | SOIL CONDITIONS |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| TEST 1 | 0 | 24 | - | - | - | Light brown sandy SILT |  |
|  | 3600 | 12.75 | 3600 | 0.19 | 11.25 |  |  |
|  | Average Rate: |  |  |  | 11.3 | inches/hour |  |
|  | TIME (SEC) | DEPTH OF WATER (IN) | TIME INTERVAL (SEC) | RATE (IN/MIN) | RATE (IN/HOUR) |  | SOIL CONDITIONS |
| TEST 2 | 0 | 24 | - | - | - | Light brown sandy SILT |  |
|  | 3600 | 12 | 3600 | 0.20 | 12.00 |  |  |
|  | Average Rate: |  |  |  | 12.0 | inches/hour |  |


|  | TIME (SEC) | DEPTH OF <br> WATER (IN) | TIME INTERVAL (SEC) | RATE (IN/MIN) | RATE <br> (IN/HOUR) |  | SOIL CONDITIONS |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| TEST 3 | 0 | 24 | - | - | - | Light brown sandy SILT |  |
|  | 3600 | 12.5 | 3600 | 0.19 | 11.50 |  |  |
|  | Average Rate: |  |  |  | 11.5 | inches/hour |  |


|  | TIME (SEC) | DEPTH OF <br> WATER (IN) | TIME <br> INTERVAL <br> (SEC) | RATE <br> (IN/MIN) | RATE <br> (IN/HOUR) | SOIL CONDITIONS |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| TEST 4 | 0 | 24 | - | - | - |  |
|  | 3600 | 12 | 3600 | 0.20 | 12.00 | inches/hour |


| Lowest Average Rate: | 11.3 | inches/hour |
| :--- | :--- | :--- |

## LANEAN

INFILTRATION TESTS
C-IT-17 performed in C-S-TP-17

| PROJECT Project Hudson |  |  | PROJECT NO. 151010101 |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| LOCATION | 59 Steele Road, Hudson, NH |  | DATE 6/17/2020 |  |  |  |
| INSPECTOR | Taylor Sisti |  | WEATHER Sunny, $80 \mathrm{~s}^{\circ} \mathrm{F}$ |  |  |  |
| PRESOAK ${ }^{\text {Start }}$ ( ${ }^{\text {End }}$ | TIME | DEPTH OF WATER IN HOLE (INCH) | ELEVATION AND DATUM |  |  |  |
|  | 14:59 | 24 | Surface Elevation | Approx. | 133 | (NGVD29) |
|  | 15:13 | 0 | Top of Hole Elevation | Approx. | 132.5 | (NGVD29) |
|  |  |  | Bottom of Hole Elevation | Approx. | 130.5 | (NGVD29) |

METHOD OF INFILTRATION TEST
C-S-TP-01 was advanced to a depth of about 0.5 feet below existing grade. An about 6 -inch diameter, 24 -inch deep hole was dug by hand with a post hole digger. The circumfrence of the hole was then lined with a 6 -inch diameter, 30 -inch long PVC pipe. Before running infiltration tests, the hole was presoaked with 24 inches of water and allowed to drain. For each infiltration test, the hole was filled with water to a predetermined depth of 24 inches. Then, the time was recorded after one hour or the time for the water to drain 24 inches was recorded. The tables below outline the calculations for determining the average rate in which the water dissipated. Test pit C-S-TP-17 was advanced to to termination depth following completion of the infiltration test.

|  | TIME (SEC) | DEPTH OF WATER (IN) | TIVE INTERVAL (SEC) | RATE (IN/MIN) | RATE (IN/HOUR) |  | SOIL CONDITIONS |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| TEST 1 | 0 | 24 | - | - | - | Light brown sandy SILT |  |
|  | 711 | 0 | 711 | 2.03 | 121.52 |  |  |
|  | Average Rate: |  |  |  | 121.5 | inches/hour |  |
|  | TIME (SEC) | DEPTH OF WATER (IN) | TIME INTERVAL (SEC) | RATE (IN/MIN) | RATE (IN/HOUR) | SOIL CONDITIONS |  |
| TEST 2 | 0 | 24 | - | - | - | Light brown sandy SILT |  |
|  | 957 | 0 | 957 | 1.50 | 90.28 |  |  |
|  | Average Rate: |  |  |  | 90.3 | inches/hour |  |
|  | TIME (SEC) | DEPTH OF WATER (IN) | $\qquad$ | RATE (IN/MIN) | RATE (IN/HOUR) | SOIL CONDITIONS |  |
| TEST 3 | 0 | 24 | - | - | - | Light brown sandy SILT |  |
|  | 1082 | 0 | 1082 | 1.33 | 79.85 |  |  |
|  | Average Rate: |  |  |  | 79.9 | inches/hour |  |
|  | TIME (SEC) | DEPTH OF WATER (IN) | $\qquad$ | RATE (IN/MIN) | RATE <br> (IN/HOUR) |  | SOIL CONDITIONS |
| TEST 4 | 0 | 24 | ( | - | - | Light brown sandy SILT |  |
|  | 1091 | 0 | 1091 | 1.32 | 79.19 |  |  |
|  | Average Rate: |  |  |  | 79.2 | inches/hour |  |
|  |  | Lowest Average Rate: |  |  | 79.2 | inches/hour |  |

## APPENDIX I PAVEMENT DESIGN

## APPENDIX 1.1 <br> FLEXIBLE PAVEMENT DESIGN SITE AREAS (LOTS A, B, C)

## Project Information:

Project Title: Hudson Logistic Center
Project Town: Hudson
Project State: New Hampshire
Client: Hudson Logistic Center

Project No.: 151010101
Performed By: NA
Date: 6/16/2020
Location: Site Areas (All Lots)

## Design Information:



## Summary of Results

## Standard Section

Heavy Duty Section
Design ESAL: 2,177,920

Design ESAL: 11,422


| LANGAN <br> 555 Long Wharf Drive, New Haven, CT 06511 NEW JERSEY NEW YORK CONNECTICUT PENNSYLVANIA OHIO WASHINGTON, DC FLORIDA TEXAS NORTH DAKOTA CALIFORNIA ABU DHABI ATHENS DOHA DUBAI ISTANBUL PANAMA | Project ${ }^{\text {Hudson Logistic Center }}$ |  | Drawing Title <br> Pavement Design Summary Sheet | $\begin{array}{\|l} \hline \text { Project No. } \\ 151010101 \end{array}$ | Drawing No. |
| :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  | $\begin{array}{\|l\|} \hline \text { Date } \\ 6 / 16 / 2020 \end{array}$ | P. 01 |
|  |  |  |  |  |
|  |  |  | $\begin{array}{\|r} \hline \text { Drawn By } \\ \mathrm{NA} \end{array}$ | Sheet 1 of 4 |

## Calculate Equivalent 18-kip Single Axle Loading (ESALs)

## Equivalent Single Axle Loads per Vehicle

- Typical Car:
(S) Front Single Axle:
(S) Rear Single Axle. 2 kips
- Typical Delivery Van:
(S) Front Single Axle:
(S) Truck Rear Axle: 8 kips
- Typical Truck and Trailer (HS20):
(S) Front Single Axle: 12 kips
(T) Truck Rear Axle: 32 kips
(T) Trailer Axle: 32 kips
(S) = single axle, $(T)=$ Tandem, $(3)=$ Triple Axles

Traffic Loading

- Design Life:

LEF = 0.189
LEF $=0.8905$
LEF $=0.8905$

Calculated ESALs
(1 axle)(0.001045)+(1 axle)(0.00104 0.00209 /car

Calculated ESALs
$(1$ axle $)(0.0343)+(1$ axle $)(0.0343)=\quad \mathbf{0 . 0 6 8 6} /$ truck

Calculated ESALs
((Front axle)(0.189)+(Rear axle)(0.8905)
$+($ Trailer Tandem $)(0.8905))=$
1.97 /truck

Standard Pavement Section

| Vehicle Types | Current Traffic | \% Increase | Design Traffic | ESAL Factor | Design ESAL |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Passenger Cars | 651 | $115 \%$ | $5,465,145$ | 0.00209 | 11,422 |
| Light Trucks | 0 | $115 \%$ | 0 | 0.0686 | 0 |

Standard Design ESAL: $\square$ 11,422

Heavy Duty Pavement Section

| Vehicle Types |  | Growth Factors | Design Traffic | ESAL Factor | Design ESAL |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Passenger Cars | 651 |  |  |  |  |
| Light Trucks | 0 | $115 \%$ | $5,465,145$ | 0.00209 | 11,422 |
| Heavy Trucks | 131 | $115 \%$ | 0 | 0.0686 | 0 |

Heavy Duty Design ESAL: 2,177,920

| LANEAN <br> 555 Long Wharf Drive, New Haven, CT 06511 $\begin{array}{ll}203.562 .5771 & \text { F. } 203.789 .6142\end{array}$ NEW JERSEY NEW YORK CONNEGTICUT PENNSYLVANIA OHIO WASHINGTON, DC FLORIDA TEXAS NORTH DAKOTA CALIFORNIA ABUDHABI ATHENS DOHA DUBAI ISTANBUL PANAMA <br>  | $\int^{\text {Project }}$ Hudson Logistic Center |  | Drawing Title <br> ESAL Calculation | $\begin{array}{r} \hline \text { Project No. } \\ 151010101 \end{array}$ | Drawing No.$\text { P. } 02$ |
| :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  | - ${ }^{\text {Date }}$ 6/16/2020 |  |
|  |  |  |  |  |
|  |  |  | $\begin{array}{\|r\|} \hline \text { Drawn By } \\ N A \end{array}$ | Sheet 2 of 4 |  |



## Flexible Pavement Section Calculation:

| Standard Section: |  | Thickness(inch) |  | TDS |  | Structural Number:$\mathrm{SN}=\mathrm{D} 1(\mathrm{a} 1)+\mathrm{D} 2(\mathrm{a} 2)+\mathrm{D} 3(\mathrm{a} 3)$ |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  |  |  |  |  |  |
| Material | Spec |  |  | $\begin{gathered} \hline \text { SN } \\ 0.88 \end{gathered}$ |  |  |  |
| Bituminuous Concrete Surface Course | Class 2 | D1 | 2.0 |  | a1 | 0.44 |  |
| Bituminuous Concrete Binder Course | Class 1 | D2 | 2.0 | a2 | 0.44 | 0.88 |  |
| Dense Graded Aggregate | Subbase | D3 | 6.0 | a3 | 0.11 | 0.66 |  |
|  | Calculated | ructu | al Num | ber | Sectio | 2.42 |  |
|  | Check | alcula | ed SN | is $>$ | ign S | OK |  |
|  | Design Lig |  | Structu |  | ber S | 1.8 | (from P.03) |

Heavy Duty Section:

|  |  | $\begin{array}{\|c} \text { Thickness } \\ \text { (inch) } \end{array}$ |  | LayerStrength |  | SN |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| Material | Spec |  |  |  |  |  |
| Bituminuous Concrete Surface Course | Class 2 | D1 | 2.0 | a1 | 0.44 |  |
| Bituminuous Concrete Binder Course | Class 1 | D2 | 3.0 | a2 | 0.44 | 1.32 |
| Dense Graded Aggregate | Subbase | D3 | 10.0 | a3 | 0.11 | 1.10 |

> | Calculated Structural Number for Section: | $\mathbf{3 . 3 0}$ |
| ---: | ---: |
| Check Calculated SN is > Design SN: | OK |

Design Heavy Duty Structural Number SN: 3.0 (from P.03)

Minimum Pavement Section

| Material |  | Spec |
| :--- | :---: | :---: |
| Thickness |  |  |
| (inch) |  |  |$|$| Bituminuous Concrete (Total) |  |
| :--- | :--- |
| Dense Graded Aggregate | Subbase |



# APPENDIX 1.2 RIGID PAVEMENT DESIGN SITE AREAS (LOTS A, B, C) 

JOINTED-PLAIN CONCRETE PAVEMENT (JPCP)
DATE CREATED:

Wed Sep 022020 17:41:11 GMT-0400 (Eastern Daylight Time)

## Project Description

| Project Name: | Lot C SD | Owner: |
| :--- | :--- | :--- |
| Designer's Name: | Route: | Zip Code: |

Project Description:

| Design Summary |  |  | Doweled | Undoweled |  |
| :--- | :--- | :--- | :--- | :--- | :--- |
| Recommended Design Thickness: | 5.00 in. | 5.00 in. | Maximum Joint Spacing: | 8 ft. | 8 ft. |
| Calculated Minimum Thickness: | 4.84 in. | 4.84 in. |  |  |  |

## Pavement Structure



## CONCRETE

Compressive Strength: 4500 psi
Modulus of Elasticity: 4000000 psi
Calculated Flexural Strength: 627 psi
Edge Support: $\quad$ Yes
Macrofibers in Concrete.

Macrofibers in Concrete: No

## SUBGRADE



Calculated MRSG Value 9,389 ps

## Project Level

## TRAFFIC

Spectrum Type:
Design Life:

ACI 330 Traffic Spectrum A
30 years
USER DEFINED TRAFFIC
Trucks Per Day: 69
Traffic Growth Rate \%: 0 \% per year
Directional Distribution:
Design Lane Distribution:

100 \%
100 \%

## GLOBAL

| Reliability: | $95 \%$ |
| :--- | :--- |
| \% Slabs Cracked at End of Design Life: | $5 \%$ |

Avg Trucks/Day in Design Lane Over the Design Life: 69 Total Trucks in Design Lane Over the Design Life: 756,068

## Design Method

## Project Description

| Project Name: | Lot C - HD |
| :--- | :--- |
| Designer's Name: |  |
|  | Route: |

## Zip Code:

Project Description:

| Design Summary | Doweled | Undoweled |  | Doweled | Undoweled |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Recommended Design Thickness: | 5.75 in. | 5.75 in. | Maximum Joint Spacing: | 9 ft . | 9 ft . |
| Calculated Minimum Thickness: | 5.63 in. | 5.63 in . |  |  |  |

## Pavement Structure

SUBBASE
Calculated Composite K-Value of Substructure:

Minimum Pavement Section: 8-inches of concrete over 6-inches of aggregate base

Layer
Layer Type
JOINIED PLAN CONCRETE SURFACE


## SUBGRADE

## CONCRETE

Compressive Strength: 4000 psi
Modulus of Elasticity: 4000000 psi
Calculated Flexural Strength: 580 psi

## SUBGRADE



Calculated MRSG Value 9,389 psi

## Project Level

TRAFFIC
Spectrum Type:
Design Life:

ACI 330 Traffic Spectrum D
30 years
USER DEFINED TRAFFIC
Trucks Per Day: 69
Traffic Growth Rate \%: 0 \% per year
Directional Distribution:
Design Lane Distribution:

100 \%
100 \%

GLOBAL

| Reliability: | $95 \%$ |
| :--- | :--- |
| \% Slabs Cracked at End of Design Life: | $5 \%$ |

Avg Trucks/Day in Design Lane Over the Design Life: 69 Total Trucks in Design Lane Over the Design Life: 756,068

## Design Method

## APPENDIX I. 3 <br> FLEXIBLE PAVEMENT DESIGN ROADWAYS

## Project Information:

Project Title: Hudson Logistic Center
Project Town: Hudson
Project State: New Hampshire
Client: Hudson Logistic Center

Project No.: 151010101
Performed By: NA
Date: 6/16/2020
Location: Roadways (Walmart Blvd. \& Green Meadow Drive)

## Design Information:



- Soil Description: FILL \& SP/SM
- USCS Symbol: SP/SM
- California Bearing Ratio (CBR): 10
- Resilient Modulus (MR): $\quad \underline{15000} \mathrm{PSI}$



## Summary of Results

## Northern Access Roadway (Walmart Blvd.)

Design ESAL: 2,173,340

Southern Access Roadway (Green Meadow Drive)
Design ESAL: 1,684,723


| LANGAN <br> ST Long Wharf Drive, New Haven, CT 06511 $\qquad$ ABUDHABI ATHENS DOHA DUBAI ISTANBUL PANAMA $\qquad$ $\qquad$ | Project |  | Drawing Title <br> Pavement Design Summary Sheet Roadways | $\begin{array}{\|l} \hline \text { Project No. } \\ 151010101 \end{array}$ | Drawing No. |
| :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  | $\begin{array}{\|l\|} \hline \text { Date } \\ 6 / 16 / 2020 \\ \hline \end{array}$ | P. 01 |
|  |  |  | Scale Not to Scale |  |
|  |  |  | ${ }^{\text {Drawn By }}$ NA | Sheet 1 of 4 |

## Calculate Equivalent 18-kip Single Axle Loading (ESALs)

## Equivalent Single Axle Loads per Vehicle

- Typical Car:
(S) Front Single Axle: 2 kips
(S) Rear Single Axle: 2 kips

Load Equivalency
Factors:
LEF = 0.001045
LEF $=0.001045$

Calculated ESALs
(1 axle)(0.001045)+(1 axle)(0.00104: $\mathbf{0 . 0 0 2 0 9 ~ / c a r ~}$

Calculated ESALs
$(1$ axle $)(0.0343)+(1$ axle $)(0.0343)=\quad \mathbf{0 . 0 6 8 6} /$ truck

Calculated ESALs
((Front axle)(0.189)+(Rear axle)(0.8905)
+(Trailer Tandem)(0.8905)) =
1.97 /truck
(S) Front Single Axle: 12 kips

LEF = 0.189
$\begin{array}{lrrl}(\mathrm{T}) & \text { Truck Rear Axle: } 32 \mathrm{kips} & \text { LEF }= & 0.8905 \\ (\mathrm{~T}) & \text { Trailer Axle: } 32 \mathrm{kips} & \text { LEF }= & 0.8905\end{array}$
$\begin{array}{lrrl}(\mathrm{T}) & \text { Truck Rear Axle: } 32 \mathrm{kips} & \text { LEF }= & 0.8905 \\ (\mathrm{~T}) & \text { Trailer Axle: } 32 \mathrm{kips} & \text { LEF }= & 0.8905\end{array}$

| (S) Front Single Axle: | 8 kips | LEF $=$ | 0.0343 |
| :--- | :--- | :--- | :--- |
| (S) Truck Rear Axle: | 8 kips | LEF $=$ | 0.0343 |

- Typical Truck and Trailer (HS20):

Traffic Loading $\quad$ ( Design Life: 20 years (From Sheet P.01)

## Northern Access Roadway (Walmart Blvd.)

| Vehicle Types | Current Traffic | \% Increase | Design Traffic | ESAL Factor | Design ESAL |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Passenger Cars | 390 | $115 \%$ | $3,274,050$ | 0.00209 | 6,843 |
| Light Trucks | 0 | $115 \%$ | 0 | 0.0686 | 0 |
| Heavy Trucks | 131 | $115 \%$ | $1,099,745$ | 1.97 | $2,166,498$ |

Heavy Duty Design ESAL: $\qquad$

Southern Access Roadway (Green Meadown Drive)

| Vehicle Types | Current Traffic | \% Increase | Design Traffic | ESAL Factor | Design ESAL |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Passenger Cars | 941 | $115 \%$ | $7,899,695$ | 0.00209 | 16,510 |
| Light Trucks | 25 | $115 \%$ | 209,875 | 0.0686 | 14,397 |
| Heavy Trucks | 100 | $115 \%$ | 839,500 | 1.97 | $1,653,815$ |

Heavy Duty Design ESAL: $\square$
$\square$ Project
LANGAN
555 Long Wharf Drive, New Haven, CT 06511
T: 203.562 .5771
F: 203.789 .6142 www langan $\begin{array}{cc}\text { T: } 203.562 .5771 & \mathrm{~F}: 203.789 .6142 \\ \text { WWww.langan.com } \\ \text { NEW JERSEY NEW YORK CONNECICUT PENNSYLVANIA OHIO }\end{array}$ NEW JERSEY NEW YORK CONNECTICUT PENNSYLVANIA OHIO
WASHINGTON, DC FLORIDA TEXAS NORTH DAKOTA CALFORNIA ABUDHABI ATHENS DOHA DUBAI ISTANBUL PANAMA



| Project | Drawing Title | Project No. |
| :--- | :--- | :--- | :--- |
| Hudson Logistic Center |  | 151010101 |



## Flexible Pavement Section Calculation:



Southern Access Roadway (Green Meadow Drive) Section:

|  |  | Thickness <br> (inch) |  | Layer Strength |  | SN |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| Material | Spec |  |  |  |  |  |
| Bituminuous Concrete Surface Course |  | D1 | 1.5 | a1 | 0.44 | 0.66 |
| Bituminuous Concrete Binder Course |  | D2 | 2.5 | a2 | 0.44 | 1.10 |
| Gravel |  | D3 | 6.0 | a3 | 0.11 | 0.66 |
| Dense Graded Aggregate | Subbase | D4 | 12.0 | a4 | 0.11 | 1.32 |

Calculated Structural Number for Section: 3.74
Check Calculated SN is > Design SN: OK
Design Structural Number SN: 2.9 (from P.03)

Minimum Pavement Section

|  |  | Thickness <br> (inch) |
| :--- | :---: | :---: |
| Mituminuous Concrete (Total) | Spec |  |
| Gravel |  | 4.0 |
| Dense Graded Aggregate |  | 6.0 |


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| :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  | $\begin{array}{\|l} \hline \text { Date } \\ 6 / 16 / 2020 \end{array}$ |  |
|  |  |  | Scale <br> As Shown |  |
|  |  |  | Drawn By NA | Sheet 4 of 4 |  |


[^0]:    ${ }^{1}$ Zone X (not shaded), "areas of minimal flood hazard" (i.e. outside the 500-year flood)
    ${ }^{2}$ Zone A, " $1 \%$ annual chance flood, base flood elevations determined," (i.e. 100-year flood)

[^1]:    ${ }^{3}$ The Standard Penetration Test (SPT) is an in situ testing technique used to infer soil density and consistency. The SPT N-value is defined as the number of blows required to drive a 2-inch-diameter split-barrel sampler 12 inches after an initial penetration of 6 inches using a 140-pound hammer falling freely from 30 inches.
    ${ }^{4}$ Rock Core Recovery (REC) is defined as the ratio of the total length of rock recovered to the total core run length, expressed as a percent.
    5 The RQD is defined as the ratio of the summation of each rock piece greater than 4 inches long (for NX cores) to total core run length, expressed as a percent.

[^2]:    ${ }^{6}$ Refusal defined as 50 blows per 6-inches or greater.

[^3]:    ${ }^{7}$ Exposure class from ACl 318-14.

[^4]:    10 Value obtained from AT Council Hazards by Location as provided by the USGS.
    ${ }^{11}$ Value obtained from AT Council Hazards by Location as provided by the USGS.

[^5]:    12 "Engineering Bulletin, Modulus of Subgrade Reaction - Which One Should be Used?" by Structural Services, Inc. (8 April 2016).

