May 26, 2021

MAX-2019012.00

Mr. John Cashell, Town Planner
Attn: Planning Board
Town Hall
1 Library Street
Georgetown, MA 01833

SUBJECT: Response to Peer Review & Planning Board Comments
Proposed Transfer Station
Carleton Drive, Georgetown, MA

Dear Mr. Cashell:

Greenman-Pedersen Inc. (GPI) previously prepared a *Traffic Impact and Access Study*[1] (TIAS) and *Updated Traffic Impact Analysis*[2] submitted to the Planning Board for the proposed transfer station to be located on Carleton Drive in Georgetown, Massachusetts. Following review of these documents, the Town's traffic peer review consultant, Ron Müller & Associates (RMA) requested that additional traffic count data be collected along Carleton Drive to obtain existing daily vehicle trips and truck trips. In addition, RMA requested that GPI collect empirical trip generation data at similar waste & recycling transfer facilities with a capacity of over 500 tons per day to compare this data to the trip generation estimates contained in the *Updated TIAS*. GPI provided a summary of this additional data to the Planning Board in a letter dated April 19, 2021. RMA and the Planning Board provided additional comments relative to this data during a Planning Board meeting on April 28, 2021. GPI has prepared this letter to respond to the comments and questions raised by RMA and the Planning Board during this meeting.

Carleton Drive Traffic Counts

At the request of the Town's peer review consultant, RMA, GPI collected an ATR count on Carleton Drive just east of Route 133 on Friday, March 19, 2021 to Thursday, March 25, 2019 to obtain a full week of count data in order to estimate existing daily weekday and weekend traffic volumes along Carleton Drive. The raw data resulting from this traffic count was summarized in Table 1 of GPI's April 28, 2021 *Response to Comments* letter. However, the volumes had not been adjusted to account for seasonal variation or impacts due to COVID-19. Therefore, GPI has provided an updated summary of the existing traffic volumes that include adjustments for seasonal variation and COVID-19.

Based on MassDOT historic traffic counts, as summarized in the *Updated TIA,* traffic volumes in the surrounding area in March are typically 10.8 percent lower than average-month conditions. Therefore, the raw March 2021 traffic volumes were increased by 10.8 percent to represent average-month conditions. The seasonally adjusted traffic volumes are summarized in Table 1.

To determine whether any adjustment to the traffic volumes was needed to account for COVID-19, GPI compared the seasonally adjusted traffic volumes collected in March 2021 to the seasonally adjusted volumes collected in February 2019 (pre-COVID) during the weekday AM (6:00 AM – 9:00 AM), weekday PM (3:00 PM

[1] *Traffic Impact and Access Study – Proposed Transfer Station, Carleton Drive – Georgetown, Massachusetts*; Greenman-Pedersen, Inc.; March 2019.
[2] *Updated Traffic Impact Analysis, Proposed Transfer Station, Carleton Drive, Georgetown, Massachusetts*; Greenman-Pedersen, Inc.; April 26, 2019.

– 6:00 PM), and Saturday midday (11:00 AM – 1:00 PM) peak periods to identify whether any reduction in traffic volumes occurred due to COVID-19. The traffic volumes during the weekday AM peak period were higher in March 2021 (post COVID) than in February 2019 (pre-COVID); however, the volumes during the weekday PM and Saturday midday peak hours were lower in March 2021 (post COVID) than in February 2019 (pre-COVID). This may be an indication that the time of day when the trips were occurring shifted earlier in the day following COVID-19. On average, the traffic volumes the volumes post-COVID (March 2021) were 28.8 percent lower than the volumes collected pre-COVID (February 2019). Therefore, GPI increased the seasonally adjusted March 2021 traffic volumes by 28.8 percent to account for impacts due to COVID-19.

The detailed calculations and adjustments are provided as an Attachment to this letter and the resulting traffic volumes with all seasonal and COVID adjustments are summarized in Table 1 below. It should be noted that March 2021 adjusted traffic were higher than the February 2019 adjusted traffic volumes during the weekday AM and Saturday midday peak hours; however, the February 2019 volumes were higher than the March 2021 adjusted volumes during the weekday PM peak hour. Therefore, the weekday PM peak hour traffic volumes shown in Table 1 below reflect the more conservative (higher) February 2019 counts.

TABLE 1
Existing Traffic Volume Summary

Location/Time Period	*Daily Volume (vpd)* [a]	*Peak Hour Volume (vph)* [b]	*% Heavy Vehicles* [c]
Carleton Drive east of Route 133:			
Weekday Daily	542	--	9%
Weekday AM Peak Hour	--	49	11%
Weekday PM Peak Hour	--	54	2%
Saturday Daily	206	--	0%
Saturday Midday Peak Hour	--	24	0%

[a] In vehicles per day.
[b] In vehicles per hour.
[c] Percentage of daily traffic representing single-unit or multi-unit heavy vehicles.

Empirical Trip Generation Data

At the request of the Town's peer review consultant, RMA, GPI previously collected empirical trip generation counts at two other transfer facilities with capacities of over 500 tons per day to verify the accuracy of the trip generation estimates contained in GPI's April 2019 *Updated TIA*. These sites included the Casella Waste facility on Hardscrabble Road in Auburn, MA, which has a capacity of 650 tons per day, and the Covanta Semass facility on Cranberry Highway in Wareham, MA, which has a capacity of 1,200 tons per day. The results of these counts were summarized in GPI's April 19, 2021 *Response to Comments* letter, which contained trip rates per tonnage capacity obtained at each of the two facilities studied. RMA recommended that GPI obtain additional information on the actual tonnage processed on each of the days counts at each of the other facilities in order to estimate a trip rate per tonnage processed to verify that GPI's trip estimates for the proposed Georgetown location represent a worst-case scenario. Therefore, the Applicant contacted Covanta Semass and Casella to obtain information on the tonnage of material received on each of the days that traffic counts were collected. Based on this information, the facilities operated at approximately 66 to 84 percent capacity on the days that traffic counts were performed. The detailed information on the tonnage received on each day is provided as an Attachment to this letter.

GPI estimated trip generation rates per tonnage received at the Auburn and Wareham facilities for each of the following time periods:

- Weekday Daily
- Weekday AM Peak Hour
- Weekday PM Peak Hour

These trip rates were then applied to the proposed 500-ton capacity of the G Mello facility in Georgetown to estimate the trips generated by G Mello on a maximum capacity day. Table 2 provides a summary of the resulting trip rates and site-generated trip estimates. The detailed count data is provided as an Attachment to this letter.

TABLE 2
Empirical Trip Generation Summary

Time Period / Direction	Average Empirical Trip Rate per Tonnage Received [a]	Estimated Georgetown Total Trips	
		Based on Empirical Trip Rates [b]	Georgetown (From TIAS) [c]
Weekday Daily			
Total Trips	1.260	630	890
Truck Trips	*0.567*	*284*	*250*
Weekday AM Peak Hour			
Enter	0.026	13	58
Exit	0.027	13	58
Total	0.053	26	116
Weekday PM Peak Hour			
Enter	0.006	3	0
Exit	0.008	4	6
Total	0.014	7	6

[a] Average trip rate per tonnage received estimated from ATR counts collected March 18 – 22, 2021 at Casella Waste in Auburn, MA and ATR counts collected March 18 – 19, 2021 at Covanta Semass in Wareham, MA
[b] Trips estimated using average trip rates per tonnage received from Casella Waste in Auburn, MA and Covanta Semass in Wareham, MA
[c] Site-generated trips estimated using counts from existing G Mello facility in Georgetown, combined with Applicant-provided data on anticipated truck trip increases (Included in GPI's April 2019 Updated TIA).

As shown in Table 2, the trip rates based on total trips per tonnage received obtained from the Auburn and Wareham facilities are significantly lower than those previously estimated in GPI's April 2019 *Updated TIAS* for the proposed G Mello facility. GPI previously estimated the proposed facility to generate 890 total vehicle trips on a typical weekday, while the rates obtained from the Auburn and Wareham facilities suggest that the proposed G Mello facility would generate only 630 daily vehicle trips. One of the main reasons for this difference may be the volume of residential waste received by the Auburn and Wareham facilities in comparison to the Georgetown facility. Approximately 67 percent of the vehicles generated by the Auburn facility were from truck trips due to a higher percentage of commercial waste received at this facility and lower volume of residential waste in comparison to the proposed Georgetown location. Approximately 31 percent of the vehicles generated by the Wareham facility were truck trips, which is equivalent to the percentage trucks anticipated at the proposed Georgetown facility.

It should be noted that *“truck trips”* refers to commercial construction and landscape vehicles, box trucks, roll-off container trucks, packer trucks, and tractor-trailer / transfer trailer trucks. Not all of the *truck trips* generated by the facility will be tractor-trailer trucks. GPI has estimated that the proposed G Mello facility in Georgetown will generate up to 20 transfer trailers (40 trips) on a peak weekday, which represents approximately five percent of the total trips generated by the facility.

Tonnage Per Vehicle

In addition, Planning Board Member Bruce Fried requested that GPI also provide a table summarizing the tons per vehicle generated at both the Auburn and Wareham locations in comparison to that estimated for the proposed Georgetown location. Table 3 provides a summary of the tonnage processed each day at each of the facilities in comparison to the number of vehicle trips entering each facility on the same day. This data was used to estimate the tonnage per vehicle as requested by Mr. Fried. It should be noted that the number of vehicle trips entering each facility includes employee trips, as well as transfer trailers that would not be bringing materials to the site. It was not possible to separate these trips from other entering trips based on the counts that were collected. Therefore, the tonnage per vehicle ratios may appear artificially low. However, to provide an appropriate comparison to the proposed Georgetown facility, GPI also included the employee trips and transfer trailer trips in the “entering vehicle” count when comparing the tonnage to number of entering vehicles. Table 3 provides a summary of the resulting tonnage per vehicle from all three sites.

TABLE 3
Tonnage per Entering Vehicle Summary

	Auburn			Braintree / Wareham			
Location	**3/18/2021**	**3/19/2021**	**3/22/2021**	**3/18/2021**	**3/19/2021**	**Average Empirical**	**Georgetown**
Tonnage Processed	490	516	643	797	1012	--	500
Daily Entering Vehicles	221	195	222	861	893	--	445
Tonnage per Vehicle (In)	**2.22**	**2.65**	**2.90**	**0.93**	**1.13**	**1.96**	**1.12**

As shown in Table 3, the proposed Georgetown facility is anticipated to experience a tonnage per entering vehicle ratio that will be similar to the existing Wareham facility and significantly lower than the existing Auburn facility. This is likely due to the higher volume of residential waste received at the Georgetown facility. As previously noted, approximately 67 percent of the vehicles generated by the Auburn facility represented trucks, which are able to transport a higher tonnage of material per vehicle. The Wareham facility experienced approximately 31 percent truck trips, which is similar to the percentage anticipated for the proposed Georgetown facility.

Pavement Cores

The Applicant previously provided a commitment to provide a pavement mill and overlay of Carleton Drive as necessary to repair the roadway surface and to accommodate the additional traffic generated by the proposed transfer station. RMA requested that the Applicant conduct pavement cores in three locations along Carleton Drive, spaced between the existing driveways, to assess whether the existing pavement and subbase is adequate to accommodate the additional traffic generated by the proposed facility with only a mill and overlay.

The Applicant retained Miller Engineering & Testing, Inc. (Miller) to conduct pavement cores as part of a subsurface exploration program to assess the adequacy of the roadway. Carleton Drive is approximately 1600 feet long. A total of 11 pavement cores were conducted, spaces at approximately 150 feet apart on alternating sides of the roadway. The detailed *Subsurface Exploration Program and Geotechnical Engineering Evaluation*[3] is provided as an Attachment to this letter. The report notes that the existing asphalt pavement is exhibiting symptoms of distress as a result of thermal and age-related shrinkage and subgrade fatigue, and that the existing base course and subbase course soils below the roadway are currently unsuitable to support the anticipated loads and intensities. Based on these findings, Miller recommends full depth reconstruction of the roadway. Table 4 provides a summary of the recommended roadway pavement section based on Miller's report.

TABLE 4
Proposed Roadway Pavement Section

Material	Specification	Thickness (Inches)
Asphalt Wearing Course	12.5 mm Superpave Surface Course MHD Mix SSC - 12.5	2.0
Asphalt Binder Course	19.0 mm Superpave Intermediate Course MHD Mix SIC – 190	3.0
Base Course	Dense Graded Crushed Gravel MHD Item M1.03.01	12.0
Subbase Course	Existing Silty Gravel Subgrade or Reclaimed Pavement Borrow (MI.09.0)	20.0

Should you have any questions, or require additional information, please contact me directly at (603) 766-5223.

Sincerely,

GREENMAN-PEDERSEN, INC.

Rebecca L. Brown, P.E.
Senior Project Manager

ATTACHMENTS – COVID Adjustment Data, Empirical Trip Rate Calculations, Pavement Evaluation Report

cc: Ron Müller, Ron Müller & Associates
Jason Mello, G Mello Disposal Corp.
Nancy McCann, McCann & McCann, P.C.
Planning Board Members

[3] *Subsurface Exploration Program and Geotechnical Engineering Evaluation, Proposed Pavement Improvements, Carleton Drive, Georgetown, MA;* Miller Engineering & Testing, Inc.; May 21, 2021.

	February 2019 Volumes		March 2021 Volumes		
	Raw Data	Seasonally Adjusted	Raw Data	Seasonally Adjusted	COVID Adjustment (Based on Seasonally Adjusted Feb 2019)
Seasonal Adjustment		16.5%		10.8%	
Weekday AM (6 AM - 9 AM)	73	85	93	103	-17.5%
Peak Hour (7:30 - 8:30 AM)	**35**	**41**	**44**	**49**	**-16.3%**
Weekday PM (3 PM - 6 PM)	126	147	76	84	75.0%
Peak Hour (4:45 - 5:45 PM)	**46**	**54**	**45**	**50**	**8.0%**
Saturday Midday (11 AM - 1 PM)	28	33	15	17	94.1%
Peak Hour (12:00 - 1:00 PM)	**6**	**7**	**13**	**14**	**-50.0%**

Average COVID Adjustment	28.8%

Weekday Daily Volume			380	421	**542**
Weekday AM Peak Hour					49
Weekday PM Peak Hour					54
Saturday Daily Volume			144	160	**206**
Saturday Midday Peak Hour					14

Vehicle Type	Weekday Daily Traffic Volumes				
	Raw Data (March 2021)	Seasonally Adjusted	**Existing Volumes (COVID Adjusted)**	Site-Generated Trips	**Build Trips**
Group 1 (Passenger Cars, Panel Trucks, Pick-Ups)	346	383	**493**	610	**1103**
Group 2 (Single-Unit Trucks, Small Contractor, Landscape)	24	27	**35**	100	**135**
Group 3 (Multi-Unit Trucks, Packers, Roll-Offs, Transfer Trailers)	10	11	**14**	180	**194**
TOTAL	380	421	**542**	890	**1432**

Trip Generation Comparison - Empirical Data (Based on Tonnage Received)

Time Period / Direction	Trip Rate per Tonnage Received: Auburn 3/18/2021	Auburn 3/19/2021	Auburn 3/22/2021	Braintree / Wareham 3/18/2021	Braintree / Wareham 3/19/2021	Average	Estimated Total Trips: Georgetown (From TIAS)	Average Empirical Rates
Weekday Daily								
Total Trips	0.914	0.768	0.714	2.149	1.755	1.260	890	630
Truck Trips	0.620	0.529	0.471	0.701	0.514	0.567	250	284
Weekday AM Peak Hour								
Enter	0.047	0.016	0.031	0.029	0.008	0.026	58	13
Exit	0.039	0.025	0.033	0.024	0.013	0.027	58	13
Total	0.086	0.041	0.064	0.053	0.021	0.053	116	27
Weekday PM Peak Hour								
Enter	0.004	0.008	0.011	0.003	0.004	0.006	0	3
Exit	0.004	0.010	0.019	0.003	0.005	0.008	6	4
Total	0.008	0.018	0.030	0.006	0.009	0.014	6	7

Percentage Truck Trips

	Georgetown (From TIAS)	Average Empirical Rates
Weekday	28%	45%

Casella Waste - Hardscrabble Road, Auburn, MA

Capacity = 650 tons / day

Time Period	Thursday 3/18/21			Friday 3/19/21			Monday 3/22/21		
	Enter (NB)	Exit (SB)	TOTAL	Enter (NB)	Exit (SB)	TOTAL	Enter (NB)	Exit (SB)	TOTAL
12:00 AM	1	1	2	0	0	0	0	0	0
1:00 AM	7	11	18	2	0	2	0	0	0
2:00 AM	2	1	3	0	0	0	7	0	7
3:00 AM	3	1	4	4	1	5	1	1	2
4:00 AM	6	1	7	12	5	17	2	1	3
5:00 AM	8	9	17	8	9	17	4	2	6
6:00 AM	13	14	27	5	6	11	8	5	13
7:00 AM	18	11	29	8	8	16	17	17	34
8:00 AM	18	21	39	10	19	29	18	19	37
9:00 AM	12	11	23	17	14	31	19	27	46
10:00 AM	20	18	38	18	17	35	13	16	29
11:00 AM	26	26	52	22	20	42	24	22	46
12:00 PM	20	20	40	26	17	43	22	21	43
1:00 PM	33	29	62	26	34	60	18	24	42
2:00 PM	11	19	30	12	13	25	25	30	55
3:00 PM	14	15	29	13	21	34	18	22	40
4:00 PM	3	5	8	7	10	17	15	13	28
5:00 PM	1	2	3	4	6	10	4	8	12
6:00 PM	3	2	5	0	0	0	1	3	4
7:00 PM	0	0	0	0	0	0	4	3	7
8:00 PM	2	0	2	0	0	0	1	2	3
9:00 PM	0	1	1	0	0	0	0	0	0
10:00 PM	0	7	7	1	1	2	0	0	0
11:00 PM	0	2	2	0	0	0	1	1	2
TOTAL	**221**	**227**	**448**	**195**	**201**	**396**	**222**	**237**	**459**
TRUCK TRIPS	**152**	**152**	**304**	**132**	**141**	**273**	**141**	**162**	**303**

AM Peak Hour (7:30 - 8:30 AM)	23	19	42	8	13	21	20	21	41
PM Peak Hour (4:45 - 5:45 PM)	2	2	4	4	5	9	7	12	19
Saturday Midday (11:00 AM - 12:00 PM)	--	--	--	--	--	--	--	--	--

Trip Rates per Tonnage Capacity

Weekday Daily	0.34	0.349	0.689	0.3	0.309	0.609	**0.342**	**0.365**	**0.707**
Weekday AM Peak Hr	**0.035**	**0.029**	**0.064**	0.012	0.02	0.032	0.031	0.032	0.063
Weekday PM Peak Hr	0.003	0.003	0.006	0.006	0.008	0.014	**0.011**	**0.018**	**0.029**
Saturday Daily	--	--	--	--	--	--	--	--	--
Saturday Midday Peak Hr	--	--	--	--	--	--	--	--	--

Truck Trips per Tonnage Capacity	**0.468**	0.42	0.466

Daily Tonnage Received	490	516	643
Percentage of Capacity	75%	79%	99%

Trip Rates per Tonnage Received

Weekday Daily	**0.451**	**0.463**	**0.914**	0.378	0.39	0.768	0.345	0.369	0.714
Weekday AM Peak Hr	**0.047**	**0.039**	**0.086**	0.016	0.025	0.041	0.031	0.033	0.064
Weekday PM Peak Hr	0.004	0.004	0.008	0.008	0.01	0.018	**0.011**	**0.019**	**0.03**

Truck Trips per Tonnage Received	**0.62**	0.529	0.471

Covanta SEMASS - 141 Cranberry Highway, Braintree / Wareham, MA

Capacity = 1200 tons / day

Time Period	Thursday 3/18/21			Friday 3/19/21		
	Enter (SB)	Exit (NB)	TOTAL	Enter (SB)	Exit (NB)	TOTAL
12:00 AM	1	6	7	4	4	8
1:00 AM	8	7	15	7	12	19
2:00 AM	3	5	8	4	4	8
3:00 AM	12	3	15	7	1	8
4:00 AM	30	8	38	31	12	43
5:00 AM	139	42	181	149	26	175
6:00 AM	95	89	184	97	104	201
7:00 AM	52	41	93	40	37	77
8:00 AM	33	36	69	29	33	62
9:00 AM	57	43	100	60	50	110
10:00 AM	28	28	56	36	36	72
11:00 AM	37	41	78	38	48	86
12:00 PM	66	80	146	73	74	147
1:00 PM	49	43	92	39	35	74
2:00 PM	37	43	80	37	48	85
3:00 PM	49	60	109	40	55	95
4:00 PM	32	50	82	34	44	78
5:00 PM	85	82	167	103	84	187
6:00 PM	26	107	133	33	108	141
7:00 PM	0	12	12	5	36	41
8:00 PM	3	8	11	4	10	14
9:00 PM	13	10	23	17	15	32
10:00 PM	4	3	7	4	5	9
11:00 PM	2	5	7	2	2	4
TOTAL	**861**	**852**	**1713**	**893**	**883**	**1776**
TRUCK TRIPS	**280**	**279**	**559**	**262**	**258**	**520**

AM Peak Hour (7:30 - 8:30 AM)	**23**	**19**	**42**	**8**	**13**	**21**
PM Peak Hour (4:45 - 5:45 PM)	**2**	**2**	**4**	**4**	**5**	**9**

Trip Rates per Tonnage Capacity

Weekday Daily	0.718	0.71	1.428	**0.744**	**0.736**	**1.48**
Weekday AM Peak Hr	**0.019**	**0.016**	**0.035**	0.007	0.011	0.018
Weekday PM Peak Hr	0.002	0.002	0.004	**0.003**	**0.004**	**0.007**

Truck Trips per Tonnage Capacity	**0.466**	0.433

Daily Tonnage Received	797	1012
Percentage of Capacity	66%	84%

Trip Rates per Tonnage Received

Weekday Daily	**1.08**	**1.069**	**2.14931**	0.882	0.873	1.754941
Weekday AM Peak Hr	**0.029**	**0.024**	**0.053**	0.008	0.013	0.021
Weekday PM Peak Hr	0.003	0.003	0.006	**0.004**	**0.005**	**0.009**

Truck Trips per Tonnage Received	**0.701**	0.514

MILLER ENGINEERING & TESTING INC.

GEOTECHNICAL / SOIL BORINGS / ENVIRONMENTAL / SOILS / CONCRETE / MASONRY / STEEL / ROOFING / ASPHALT INSPECTION

Mail all correspondence to: 100 SHEFFIELD ROAD · PO BOX 4776 · MANCHESTER, NH 03108-4776 · TELEPHONE (603)668-6016 · Fax (603)668-8641

May 21, 2021

Rebecca Brown, P.E.
GPI
116 South River Road, Building B, Suite 1
Bedford, NH 03110

RE: Subsurface Exploration Program and
Geotechnical Engineering Evaluation
Proposed Pavement Improvements
Carleton Drive
Georgetown, MA

Project No. 21.067.NH

Dear Ms. Brown:

Miller Engineering & Testing, Inc. has recently completed a subsurface exploration program and subsequent geotechnical engineering evaluation of the existing asphalt roadway pavements for Carleton Drive located in Georgetown, Massachusetts. The exploration program and this engineering evaluation were performed in accordance with our proposal, Ref. File 162-21(R1), dated April 21, 2021.

This report summarizes the results of the recent subsurface investigation and includes a description of the subsurface exploration program and the conditions encountered, along with a geotechnical evaluation of the existing bituminous pavements. In addition, recommendations are provided to address roadway pavement improvements.

The geotechnical evaluation and recommendations presented herein are based, in part, on widely spaced test borings. The nature and extent of variations between exploration locations may not become evident until construction. Should significant variations become evident during construction, it may be necessary to re-evaluate the conclusions and recommendations contained herein. The contents of this letter report are subject to the limitations in Appendix A.

SITE AND PROJECT DESCRIPTION

The proposed roadway improvements will occur over the entire, approximately 1600-foot, length of Carleton Drive, located in Georgetown, Massachusetts. The existing cul-de-sac roadway provides access from Massachusetts Route 133 (East main Street) to several small commercial and industrial buildings.

Resent traffic data has indicated that the current traffic volume on the existing roadway consist of primarily light passenger vehicle traffic (+/-500 vpd) and limited medium to heavy vehicle traffic (+/-50 vpd).

Carleton Roadway is bordered by low, rolling, pine woodlands along the southwest perimeter. Several vernal pools were noted near the edges of the roadway, in this area, indicating a potential seasonal ground water issue in proximity to the roadway surface. These woodlands transition to low lying wetlands along the southeastern half of the Drive. In this area, the roadway was only slightly elevated above the wetland indicating a potential water table in close proximity to the pavement subgrade. Relatively flat, slightly elevated, woodlands exist along the northern perimeter of the Drive in and around two large multi-purpose commercial and industrial buildings with surrounding asphalt paved parking and roadways. Topography along Carleton Drive is relatively flat and slopes down gently from the west to the east. Inspection of the down-sloping roadway shoulders indicate that the original roadway was constructed on an elevated berm through the existing topography.

A visual inspection of the existing roadway indicated signs of moderate to severe pavement distress along the entire length of the roadway. Moderate to severe rutting was observed in both traffic lanes throughout the roadway length. Several edge cracking and secondary cracking was noted along the southern edge of roadway pavements within the western half od the drive. Potholes were observed to be scattered, irregularly, throughout the western half of the roadway. A few larger patch repairs indicate the need for significant, past, roadway base and pavement repairs.

It appears that the existing roadway pavements were resurfaced, with a 1- to 1.5-inch-thick asphalt wearing course overlay, at sometime within the previous 10 to 12-years; however, the current condition of the wearing course is considered to be in poor condition. Severe raveling of the pavement surface was noted along the length of the roadway as well as severe longitudinal joint cracking at the roadway centerline. At numerous locations, the existing wearing course had been mechanically stripped, by plows, to expose the underlying pavement surface. Inspection of this strata showed no indicating of the underlying surface being properly milled or tack coated, prior to the installation of the pavement surface overlay.

SUBSURFACE EXPLORATION PROGRAM

A subsurface exploration program was completed at the project site on April 29, 2021. The exploration program was performed by a drill crew from Miller Engineering & Testing, Inc. The exploration program consisted of the advancement of eleven (11) test borings, B-1 through B-11, which were advance along the length of the roadway, at approximate 150-foot spacings, and alternating drive lanes. Test borings were advanced from the existing roadway surface to depths ranging from 2.5 to 8-feet. The approximate locations of the test borings are shown on the attached Subsurface Exploration Location Plan, Figure 1.

The purpose of the subsurface exploration program was to:

- Assess the nature, consistency and relative density of the soils encountered at the site, provide soil samples for visual classification, and perform standard penetration testing.
- Assess the thickness and conditions of the existing pavement structure and subgrade soils encountered at the test boring locations.
- Assess the thickness and condition of the existing asphalt pavements.
- Asses the depth to groundwater, if encountered.

The test borings at the site were advanced using a truck mounted CME-45 drill rig, turning hollow-stem augers. Subgrade soil samples were typically collected immediately below the bituminous pavements and then continuously to the termination depths of the test borings. Sampling was performed using a 1-3/8 inch, inside diameter, split barrel spoon sampler. Standard Penetration Tests (SPT's), which were used to assess the soils relative density, were performed at sampling depths by driving the split spoon sampler 18-24 inches with a 140-pound hammer falling 30 inches. Standard penetration testing was performed in general accordance with ASTM Method D-1586. The standard penetration resistance, or N-Value, is defined as the number of hammer blows required to drive the sampler between the 6 and 18-inch increments.

Details of the specific conditions observed at the test boring locations are provided on the Test Boring Logs in Appendix B.

SUBSURFACE CONDITIONS ENCOUNTERED

The results of the test borings indicate that subsurface conditions at the test boring locations generally consist of bituminous asphalt pavements overlaying thin gravel fill, over silty sand fill, in-turn overlaying the naturally-occurring organic silt and/or the silty, granular subgrade. Groundwater was observed in several of the test borings at depths of approximately five (5) to six (6) feet below the existing roadway surface.

The subsurface conditions observed at the test boring locations are summarized below:

BITUMINOUS ASPHALT PAVEMENTS

Bituminous asphalt pavements were encountered at each of the test boring locations, immediately at the ground surface. The asphalt pavements were observed to range in thickness from two-and-one-half (2.5) inches to four-and-one-half (4.5) inches, but typically averaged approximately three (3) inches in thickness.

The following table summarizes the results of the pavement thicknesses at the test boring locations:

Table #1
Asphalt Coring Results

Boring Location	Approx. Pavement Thickness (in)	Boring Location	Approx. Pavement Thickness (in)
B-1	3	B-7	2.5
B-2	3.5	B-8	2.5
B-3	4	B-9	4.5
B-4	3	B-10	4
B-5	2.5	B-11	3
B-6	4		

BASE COURSE FILL MATERIAL

Medium dense to dense, granular, crushed gravel fill materials were encountered immediately underlying the asphalt pavements at the test boring locations. This material generally consisted of brown to dark-brown, fine to coarse sand with little to some gravel and little to trace amounts of silt. The crushed gravel fill materials were observed to range in thickness from approximately four (4) to ten (10) inches, but was generally estimated to be approximately six (6) inches in average thickness along the length of the roadway.

GRANULAR FILL MATERIAL

Granular fill materials were encountered immediately underlying the crushed gravel fills at most of the test boring locations. The granular fill materials generally consisted of medium-dense to dense, brown, fine to medium sand, with little to some coarse sand, little to some gravel and little to some silt.

The granular fills encountered at the test boring locations appear to have been installed to elevate the roadway to the design grades were observed to extend to depths ranging from one (1) to greater than six (6) feet below the existing roadway surface, but typically extended to four (4) feet below the pavement surface. The granular fills typically extended to the underlying naturally occurring subgrade, with the exception of Test Borings B-1 and B-2, where the granular fill materials extended beyond the termination depths of the test borings.

NATURALLY OCCURRING ORGANIC SILT DEPOSITS

Naturally occurring organic silt deposits were observed immediately underlying the granular fill deposits at test borings B-3, B-5, B-6 and B-8. At these locations, the subgrade below the fills consisted of loose, dark-brown to brown, organic silts with little to trace amounts of fine sand and trace amounts of root matter. This material was observed to range in thickness from twelve (12) to twenty-four (24) inches.

The organic silt deposits observed in the test borings appear to be consistent with the relatively incompressible basal soils typically located below organic peat deposits. Based on the lack of fibrous peat, it is likely that the naturally occurring organic, fibrous peats were removed during the original roadway construction, leaving the underlying, relatively uncompressible, organic silts to remain below the roadway fills.

<u>NATURALLY OCCURRING GRANULAR DEPOSITS</u>

Naturally occurring granular deposits were encountered immediately underlying the granular fill and organic silt deposits at each of the test boring locations, with the exceptions of test borings B-1, B-2, B-4 and B-8, and extended to the termination depths of the test borings.

The naturally occurring granular deposits were noted to vary between loose, brown to orange, fine sand and silt with trace amounts of gravel to medium-dense, brown, fine to medium sand and gravel with little to some silt.

<u>AUGER PENETRATION REFUSAL</u>

Auger penetration refusal was encountered at test boring B-4 at a depth of 2.5 feet below the ground surface.

Test boring B-4 was advanced in the vicinity of an existing roadway surface repair and patch. Approximately twelve (12) to fourteen (14) inches of crushed stone was encountered immediately underlying the asphalt patch, and extended to the termination depth of the test boring. The presence of the surface pavement patch and the crushed stone fill indicated a previous roadway and/or utility repair. In order to prevent damage to what may be an underlying utility, auger penetration was terminated upon encountering significant grinding of the drilling equipment. Based on surrounding test borings within a 150-foot radius, it is unlikely that the penetration refusal is due to the presence of bedrock or very large boulders.

<u>Groundwater Observations</u>

Groundwater observations were made at each of the test boring locations, upon their completion, and after a brief stabilization period. Also, the moisture content of retrieved soil samples was assessed to aid in the determination of the groundwater level.

The following table summarizes the results of the groundwater measurements:

Table #2
Groundwater Measurement Results

Boring Location	**Approx. Groundwater Depth (ft)**	**Boring Location**	**Approx. Groundwater Depth (ft)**
B-1	Not Encountered	B-7	Not Encountered
B-2	Not Encountered	B-8	Not Encountered
B-3	6	B-9	6
B-4	Not Encountered	B-10	5
B-5	6	B-11	5
B-6	6		

It should be noted that groundwater fluctuates from time to time as a result of season, temperature, precipitation, adjacent structures and other environmental conditions. Groundwater conditions at other times, therefore, may be different from those observed and recorded herein.

The results of the groundwater observations at the test boring locations are indicated on the Test Boring Logs in Appendix B.

LABORATORY TESTING

A total of ten (10) samples, identified as MET Laboratory Numbers L21096-A through J, were collected from the split-spoon samples during the test boring program. The samples were submitted to the laboratory for grain size analysis testing, to aid in the evaluation of the engineering characteristics of the fill material located immediately below the asphalt pavements at the site to assess their suitability for re-use and to adequately support the pavement structure. The results of the laboratory testing are provided in the Grain Size Distribution Reports in Appendix C.

EXISTING AND PROPOSED TRAFFIC LOADS AND INTENSITIES

Existing and proposed traffic loads and volumes were developed by representatives of GPI for use in the proposed pavement design evaluation. Current composite traffic data was classified into three vehicle loading scenarios and then adjusted for seasonal volume to model the current traffic loads and intensities. Subsequently, the predicted volume and traffic intensities were determined and added to the existing traffic determinations to prepare an estimate of anticipated daily traffic volumes after the construction of a proposed Transfer Station. A summary of the traffic calculations used to develop the proposed design pavement section are provided below:

Table #3
Traffic Load and Intensity Results

Vehicle Type	Weekday Daily Traffic Volumes				
	Raw Data (March 2021)	Seasonally Adjusted	**Existing Volumes (COVID Adjusted)**	Site-Generated Trips	**Build Trips**
Group 1 (Passenger Cars, Panel Trucks, Pick-Ups)	346	383	**493**	610	**1103**
Group 2 (Single-Unit Trucks, Small Contractor, Landscape)	24	27	**35**	100	**135**
Group 3 (Multi-Unit Trucks, Packers, Roll-Offs, Transfer Trailers)	10	11	**14**	180	**194**
TOTAL =	380	421	**542**	890	**1432**

Based on the data provided herein, and assuming a 20-year design period for the roadway, it was determined that the roadway would be subjected to traffic loading of 3.3×10^6 ESAL's (Equivalent 18-kip Single Axel Load Applications) over its design life.

ENGINEERING EVALUATION

Based on the result of a combination of visual inspections and test borings performed at the site, the existing asphalt pavements within Carleton Drive are exhibiting symptoms of moderate to severe distress as a result of a combination of thermal and age-related shrinkage and subgrade fatigue. It is recommended that the existing bituminous pavements at the site be rehabilitated to provide improved safety and serviceability to adequately support the anticipated traffic loads and intensities.

The existing roadway pavement section was observed to consist of approximately three (3) inches of bituminous asphalt pavement overlying approximately six (6) inches of medium-dense crushed gravel, overlying thirty (30) inches of medium-dense, silty, gravely sands, in turn overlying the naturally occurring silty subgrade.

A structural analysis of the existing pavement section was performed to assess the suitability of the existing pavement structure to support the design traffic loadings. The result of the analysis indicating that the existing pavement section is inadequate to support the design traffic over a 20-year design period.

An alternatively pavement section, consisting of a 2-inch asphalt overlay of the existing pavements (5-inches of pavement total) was evaluated. The results of the evaluation also indicated that the proposed section would be inadequate to support design traffic intensities.

A third pavement section was considered, in which the existing asphalt pavements would be reclaimed into the underlying base course gravels, to produced a reclaimed stabilized base which would support a new 4-inch asphalt pavement section. Analysis of the capacity of a reclaimed pavement section also failed to provide adequate support of the design traffic loading.

The results of the pavement section analyses indicate that the existing base course and subbase course soils below the roadway are currently unsuitable to support the design traffic loads and intensities. Furthermore, it does not appear likely that the existing base course gravels can be adequately improved, in-place via reclamation processes, to support the design traffic. As a result, it will b necessary to over-excavate and replace the existing base course gravels and a portion of the subbase course soils to facilitate the installation of a new, more substantial, crushed gravel base course and heavier asphalt pavement section to support design traffic loading.

PAVEMENT DESIGN AND CONSTRUCTION RECOMMENDATIONS

Bituminous asphalt pavements for the proposed project were analyzed and designed in accordance with procedures developed in the "1993 AASHTO Guide for the Design of Pavement Structures". This analysis method was used in combination with the results of the test borings to determine the recommended pavement section thicknesses, which will adequately support the anticipated low traffic loading intensity. In addition to traffic loading and intensities, the AASHTO analysis method also considers subgrade strength, environmental effects and serviceability requirements.

The results of the analysis indicate that the existing base course soils below paved roadway areas of the site are unsuitable for support of the proposed roadway should be removed and replaced to support the design traffic loading and intensities. The existing asphalt pavements, the underlying crushed gravel base course and portions of the underlying subbase course fills should be over-excavated and replaced with compacted Dense Graded Crushed Gravel. Upon completion of the over-excavation and replacement of the base and subbase soils, new asphalt pavements should then be installed to a minimum thickness of five (5) inches.

The full depth pavement reconstruction should be performed as follows:

1. The first item of earthwork should consist of mechanically excavating and removing the existing asphalt pavements, crushed gravel base course soils and portions of the underlying subbase course silty, gravely fills, to a minimum depth of seventeen (17) inches below the proposed finished asphalt roadway grades.

   The excavated asphalt, base and subbase soils are considered unsuitable for reuse below the proposed roadway section and should be removed from the site.

   If the existing asphalt pavements were pulverized and blended with the underlying base course crushed gravels, prior to excavation, the resulting material may meet the minimum requirements for Reclaimed Stabilized Base and could be used in raise in-grade areas of the site below the proposed Dense Graded Crushed Gravel fills. This material may also be suitable for reuse below proposed asphalt paved parking or low-volume roadway area of the proposed transfer station and could be stockpiled for future use.

2. Following the over-excavation and removal process of the existing pavements, and gravels, the exposed subgrade should be graded and compacted to allow for the installation of a new Base Course gravel fills.

   The exposed subgrade soils are anticipated to consist of the existing brown, medium-dense, silty, gravely, fine to medium sands with trace amounts of coarse sand. Prior to subsequent backfilling, this material should be compacted to a minimum of 95% of the material's maximum dry density, as determined by ASTM Method D1557.

   Subgrade areas where distress in the existing pavements has occurred, as a result of settlement (e.g., manholes, catch basins, etc..), should be over-excavated and replaced with compacted engineered backfill materials prior to installation of the new Base Course gravel fills. At these areas, the top 24-inches of existing backfill material, installed around the buried structures, should be over-excavated and replaced with Dense Graded Crushed Stone, Item M2.01.7 meeting the minimum requirements of the current Massachusetts Highway Department Standard Specifications for Highways and Bridges (MHDSSHB). This material should be installed in 12-inch maximum loose lifts and be compacted to a minimum of 95% of the material's maximum dry density, as determined by ASTM Method D1557.

   The over-excavation and replacement zone around the structures should be extended a minimum of five (5) feet laterally beyond the structures.

   In the event localized wet areas are encountered in the subgrade, which may create unstable conditions, the unstable areas should be over-excavated by a minimum of 12-inches and be replaced with ¾-inch crushed stone, completely wrapped in filter fabric (Mirafi 140N, or equal). The crushed stone and fabric should be keyed into the subgrade using the effort of four (4) passes of a 700-pound vibratory plate compactor.

3. Upon completion of the subgrade preparation and compaction, Base Course gravel fills, consisting of a minimum of 12-inches of Dense Graded Crushed Stone, should be installed to the bottom of the proposed asphalt pavement elevations. The Dense Graded Crushed Stone materials should meet the minimum requirements of Item M2.01.7, of the current Massachusetts Highway Department Standard Specifications for Highways and Bridges (MHDSSHB). This material should be installed in 12-inch maximum loose lifts and be compacted to a minimum of 95% of the material's maximum dry density, as determined by ASTM Method D1557.

4. All existing covers, boxes and structures located within the reconstructed pavement sections shall be raised or lowered, as necessary, to construct rim elevations level and flush with the finished asphalt surface.

5. Upon completion of the subgrade and base course preparation procedures, asphalt pavements should be installed in the minimum thickness provided in the table below:

Table #4
Proposed Roadway Pavement Section

Material	Spec	Thickness (Inches)
Asphalt Wearing Course	12.5mm Superpave Surface Course MHD Mix SSC - 12.5	2.0
Asphalt Binder Course	19.0mm Superpave Intermediate Course MHD Mix SIC - 190	3.0
Base Course	Dense Graded Crushed Gravel MHD Item M1.03.1	12.0
Subbase Course	Existing Silty Gravel Subgrade or Reclaimed Pavement Borrow (M1.09.0)	20.0

6. The bituminous pavements placed in full depth pavement reconstruction areas should be designed and installed in accordance with Section 460, of the current Massachusetts Highway Department Standard Specifications for Highways and Bridges, except where noted herein, or as directed and approved by the Engineer. The Hot Mix Asphalt courses shall meet the minimum material mix specifications for 12.5mm and 19.0mm dense graded SUPERPAVE mix, with a PG 64-22 Binder.

   The Surface Course and Binder Course pavements should be installed and compacted to between 92 and 97 percent of the materials Theoretical Maximum Density as determined by AASHTO Method T 209.

FINAL DESIGN AND CONSTRUCTION MONITORING

It is recommended that a qualified geotechnical engineer or his representative be retained to provide engineering services during the site preparation pavement construction phases of the project. This will become particularly important relative to the excavation of unsuitable materials and placement and compaction of engineered fill at the project site. This allows for design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction. The adequacy of fill compaction should be determined by field density testing as fill is placed and compacted.

Representative samples of all backfill soil materials and asphalt materials should be submitted to Miller Engineering & Testing, Inc. for testing to establish their optimum water/asphalt contents and maximum dry densities, and to compare their gradation characteristics with the requirements of the current Massachusetts State Department of Transportation Standard Specifications. In this manner, compaction criteria can be developed which will provide the materials with adequate strength and minimal distortion.

Lastly, it is recommended that this firm be retained to prepare final design plans and to prepare earthwork and bituminous concrete related project specifications. In the event that any changes in the nature and/or scope of the proposed pavement improvements are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report modified or verified in writing by Miller Engineering & Testing, Inc.

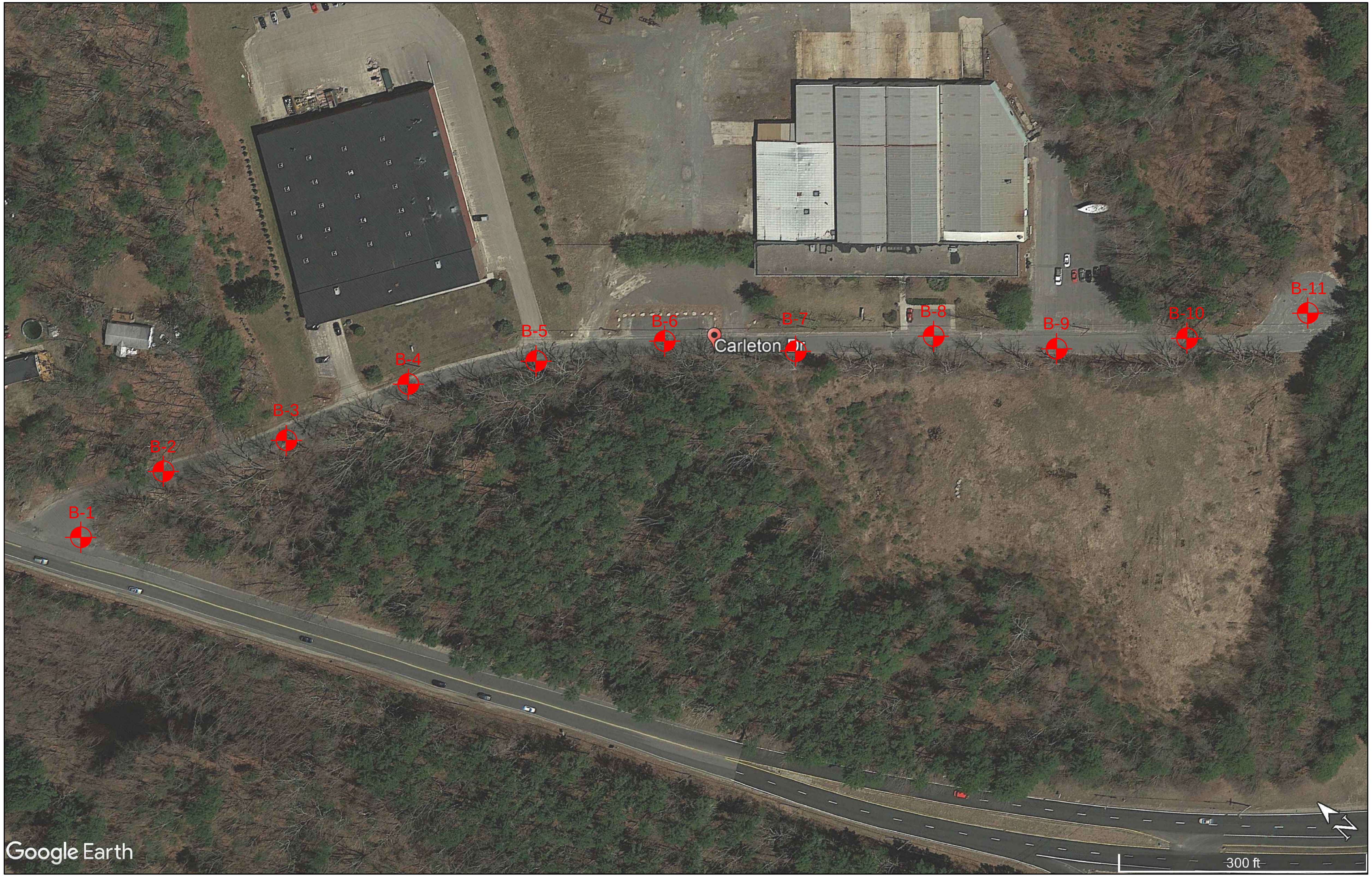
We trust the report meets with your current needs. Should you have any questions, please do not hesitate to contact us.

Respectfully,

MILLER ENGINEERING & TESTING, INC.

Gerald N. Gendron
Senior Staff Engineer

Figures



MILLER ENGINEERING & TESTING, INC.
100 SHEFFIELD ROAD - PO BOX 4776
MANCHESTER, NEW HAMPSHIRE 03108
TEL (603) 668-6016 // FAX (603) 668-8641
EMAIL ENGRS@MILLERENGANDTESTING.COM

PROJECT: CARLETON ROAD
GEORGETOWN, MA

PROJECT NO. 21.067.NH

DATE: MAY 21, 2021

SUBSURFACE EXPLORATION LOCATION PLAN

FIGURE NO. 1

Appendix A

LIMITATIONS

Explorations

1. The analyses, recommendations and designs submitted in this report are based in part upon the data obtained from subsurface explorations. The nature and extent of variations between these explorations may not become evident until construction. If variations then appear evident, it will be necessary to re-evaluate the recommendations of this report.

2. The generalized soil profile described in the text is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretation of widely spaced explorations and samples; actual soil transitions are probably more gradual. For specific information, refer to the boring logs.

3. Water level readings have been made in the drill holes at times and under conditions stated on the boring logs. These data have been reviewed and interpretations have been made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, temperature, and other factors differing from the time measurements were made.

Review

4. It is recommended that this firm be retained to review final design plans and specifications. In the event that any changes in the nature, design, or location of the structures are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of the report modified or verified in writing by Miller Engineering & Testing, Inc.

Construction

5. It is recommended that this firm be retained to provide soils engineering services during the excavations and foundation construction phases of the work. This is to observe compliance with the design concepts, specifications, or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

Use of Report

6. This report has been prepared for the exclusive use of **GPI** for the **Proposed Pavement Improvements to Carleton Drive** located in **Georgetown, Massachusetts** in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

7. This soil and pavement engineering report has been prepared for this project by Miller Engineering & Testing, Inc. This report was completed for design purposes and may be limited in its scope to prepare an accurate bid. Contractors wishing a copy of the report may secure it with the understanding that its scope is limited to design considerations only.

Appendix B

TEST BORING LOG

MILLER ENGINEERING & TESTING, INC.
100 Sheffield Road - Manchester, NH 03103
Ph. (603) 668-6016 - Fax: (603) 668-8641

Project: Carleton Drive, Georgetown, MA
Project No: 21.067.NH
Date Start: 04-29-21
Date End: 04-29-21

Sheet 1 **of** 1
Boring No: B-1
Location: See Plan
Approx. Surface Elev:

	CASING	SAMPLER
Type	HSA	SS
Size	2-1/4" ID	1-3/8" ID
Hammer		140 lbs.
Fall		30"

GROUNDWATER OBSERVATIONS

Date	Depth	Casing At	Stabilization Period
04-29-21	None	6'	Upon Completion

Depth/ Elev.	Cas bl/ft	SAMPLE Sample No.	Depth Range	Pen.	Rec.	BLOWS 0-6"	6-12"	12-18"	18-24"	Strata Change	Sample Description	Notes
0		-	0.0-0.3	3							-: 3" Asphalt	
		S-1	0.5-2.0	18	9		30	27	20		S-1: Brown, fine to coarse sand, some gravel, little silt (FILL)	
2		S-2	2.0-4.0	24	7	17	15	10	8		S-2: Brown, fine to coarse sand, some silt, little gravel (cemented oiled soil in sample) (FILL)	
4		S-3	4.0-6.0	24	8	8	17	41	47		S-3: Brown, fine to coarse sand, some silt and gravel (FILL)	
6											BORING TERMINATED AT 6 ft	
8												
10												
12												

Driller: R. Marcoux
Helper: J. Donahue
Inspector: T. Young

COHESIVE CONSISTENCY (Blows/Foot)
0-2 VERY SOFT
2-4 SOFT
4-8 MEDIUM STIFF
8-15 STIFF
15-30 HARD

COHESIONLESS (Blows/Foot)
0-4 VERY LOOSE
4-10 LOOSE
10-30 MEDIUM DENSE
30-50 DENSE
50+ VERY DENSE

PROPORTIONS USED
TRACE: 0-10%
LITTLE: 10-20%
SOME: 20-35%
AND: 35-50%

NOTES:

REMARKS: THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITION MAY BE GRADUAL.
WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS.
FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

TEST BORING LOG

MILLER ENGINEERING & TESTING, INC.
100 Sheffield Road - Manchester, NH 03103
Ph. (603) 668-6016 - Fax: (603) 668-8641

Project: Carleton Drive, Georgetown, MA
Project No: 21.067.NH
Date Start: 04-29-21
Date End: 04-29-21

Sheet 1 **of** 1
Boring No: B-2
Location: See Plan
Approx. Surface Elev:

	CASING	SAMPLER	Date	Depth	Casing At	Stabilization Period
			GROUNDWATER OBSERVATIONS			
Type	HSA	SS	04-29-21	None	6'	Upon Completion
Size	2-1/4" ID	1-3/8" ID				
Hammer		140 lbs.				
Fall		30"				

Depth/ Elev.	Cas bl/ft	Sample No.	Depth Range	Pen.	Rec.	Blows 0-6"	Blows 6-12"	Blows 12-18"	Blows 18-24"	Strata Change	Sample Description	Notes
0		-	0.0-0.3	3.5							-: 3.5" Asphalt	
		S-1	0.5-2.0	18	6		25	35	39		S-1: Brown, fine to coarse sand, some gravel, little silt (FILL)	
2		S-2	2.0-4.0	24	12	19	17	20	20		S-2: Brown, fine to coarse sand, some gravel and silt (cemented oiled soil in sample) (FILL)	
4		S-3	4.0-6.0	24	9	11	29	25	10		S-3: Brown, fine to coarse sand, some gravel and silt (FILL)	
6											BORING TERMINATED AT 6 ft	
8												
10												
12												

Driller: R. Marcoux
Helper: J. Donahue
Inspector: T. Young

COHESIVE CONSISTENCY (Blows/Foot)
0-2 VERY SOFT
2-4 SOFT
4-8 MEDIUM STIFF
8-15 STIFF
15-30 HARD

COHESIONLESS (Blows/Foot)
0-4 VERY LOOSE
4-10 LOOSE
10-30 MEDIUM DENSE
30-50 DENSE
50+ VERY DENSE

PROPORTIONS USED
TRACE: 0-10%
LITTLE: 10-20%
SOME: 20-35%
AND: 35-50%

NOTES:

REMARKS: THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITION MAY BE GRADUAL. WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

TEST BORING LOG

MILLER ENGINEERING & TESTING, INC.
100 Sheffield Road - Manchester, NH 03103
Ph. (603) 668-6016 - Fax: (603) 668-8641

Project: Carleton Drive, Georgetown, MA
Project No: 21.067.NH
Date Start: 04-29-21
Date End: 04-29-21

Sheet 1 **of** 1
Boring No: B-3
Location: See Plan
Approx. Surface Elev:

	CASING	SAMPLER
Type	HSA	SS
Size	2-1/4" ID	1-3/8" ID
Hammer		140 lbs.
Fall		30"

GROUNDWATER OBSERVATIONS

Date	Depth	Casing At	Stabilization Period
04-29-21	6'	8'	Upon Completion

Depth/ Elev.	Cas bl/ft	Sample No.	Depth Range	Pen.	Rec.	Blows 0-6"	Blows 6-12"	Blows 12-18"	Blows 18-24"	Strata Change	Sample Description	Notes
0		-	0.0-0.3	4							-: 4" Asphalt	(1)
		S-1	0.5-2.0	18	6		15	38	42		S-1: Dark brown, crushed gravel (FILL)	(2)
2		S-2	2.0-3.5	18	8	16	25	15			S-2: Brown, fine to coarse sand, some gravel, little silt (FILL)	
		S-2A	3.5-4.0	6	5				9		S-2A: Olive (mottled), fine to medium sand, some silt, trace gravel (FILL)	
4		S-3	4.0-6.0	24	14	3	2	4	2		S-3: Dark brown/brown, organic silt, trace fine sand (peat)	
6		S-4	6.0-8.0	24	18	3	2	3	6		S-4: Brown, silt, trace fine sand, wet	(3)
8											BORING TERMINATED AT 8 ft	
10												
12												

Driller: R. Marcoux
Helper: J. Donahue
Inspector: T. Young

COHESIVE CONSISTENCY (Blows/Foot)
0-2 VERY SOFT
2-4 SOFT
4-8 MEDIUM STIFF
8-15 STIFF
15-30 HARD

COHESIONLESS (Blows/Foot)
0-4 VERY LOOSE
4-10 LOOSE
10-30 MEDIUM DENSE
30-50 DENSE
50+ VERY DENSE

PROPORTIONS USED
TRACE: 0-10%
LITTLE: 10-20%
SOME: 20-35%
AND: 35-50%

NOTES:
(1) Pavement patch in road.
(2)Rock fragments in tip of split-spoon.
(3) Brown, fine sand, trace silt in tip of splt-spoon.

REMARKS: THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITION MAY BE GRADUAL.
WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS.
FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

TEST BORING LOG

MILLER ENGINEERING & TESTING, INC.
100 Sheffield Road - Manchester, NH 03103
Ph. (603) 668-6016 - Fax: (603) 668-8641

Project: Carleton Drive, Georgetown, MA
Project No: 21.067.NH
Date Start: 04-29-21
Date End: 04-29-21

Sheet 1 **of** 1
Boring No: B-4
Location: See Plan
Approx. Surface Elev:

	CASING	SAMPLER
Type	HSA	SS
Size	2-1/4" ID	1-3/8" ID
Hammer		140 lbs.
Fall		30"

GROUNDWATER OBSERVATIONS

Date	Depth	Casing At	Stabilization Period
04-29-21	None	2.5'	Upon Completion

Depth/ Elev.	Cas bl/ft	Sample No.	Depth Range	Pen.	Rec.	0-6"	6-12"	12-18"	18-24"	Strata Change	Sample Description	Notes
0		-	0.0-0.3	3							-: 3" Asphalt	
		S-1	0.5-1.2	8	4		17	65/2:	35/0"		S-1: Dark brown, fine to coarse sand, some crushed gravel, little silt (FILL) Auger Refusal at 2.5'	
2												
											BORING TERMINATED AT 2.5 ft	
4												
6												
8												
10												
12												

Driller: R. Marcoux
Helper: J. Donahue
Inspector: T. Young

COHESIVE CONSISTENCY (Blows/Foot)	COHESIONLESS (Blows/Foot)	PROPORTIONS USED
0-2 VERY SOFT	0-4 VERY LOOSE	TRACE: 0-10%
2-4 SOFT	4-10 LOOSE	LITTLE: 10-20%
4-8 MEDIUM STIFF	10-30 MEDIUM DENSE	SOME: 20-35%
8-15 STIFF	30-50 DENSE	AND: 35-50%
15-30 HARD	50+ VERY DENSE	

NOTES:

REMARKS: THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITION MAY BE GRADUAL. WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

TEST BORING LOG

MILLER ENGINEERING & TESTING, INC.
100 Sheffield Road - Manchester, NH 03103
Ph. (603) 668-6016 - Fax: (603) 668-8641

Project:	Carleton Drive, Georgetown, MA
Project No:	21.067.NH
Date Start:	04-29-21
Date End:	04-29-21
Sheet	1 of 1
Boring No:	B-5
Location:	See Plan
Approx. Surface Elev:	

	CASING	SAMPLER
Type	HSA	SS
Size	2-1/4" ID	1-3/8" ID
Hammer		140 lbs.
Fall		30"

GROUNDWATER OBSERVATIONS

Date	Depth	Casing At	Stabilization Period
04-29-21	6'	8'	Upon Completion

Depth/ Elev.	Cas bl/ft	Sample No.	Depth Range	Pen.	Rec.	Blows 0-6"	Blows 6-12"	Blows 12-18"	Blows 18-24"	Strata Change	Sample Description	Notes
0		-	0.0-0.2	2.5							-: 2.5" Asphalt	
		S-1	0.4-2.0	19	7	4/1"	28	25	21		S-1: Dark brown to brown, fine to coarse sand, some crushed gravel, little silt (FILL)	
2		S-2	2.0-3.2	14	9	20	37	50/2"			S-2: Brown, fine to coarse sand, some gravel and silt (FILL)	
4		S-3	4.0-6.0	24	9	8	4	1	1		S-3: Dark brown, organic silt, trace fine sand, trace fine organic roots (peat)	
6		S-4	6.0-7.5	18	14	2	3	8			S-4: Brown, fine sand, some silt, trace fine roots, wet	
		S-4A	7.5-8.0	6	6				14		S-4A: Brown, fine sand, trace silt, wet	
8											BORING TERMINATED AT 8 ft	
10												
12												

Driller: R. Marcoux
Helper: J. Donahue
Inspector: T. Young

COHESIVE CONSISTENCY (Blows/Foot)	COHESIONLESS (Blows/Foot)	PROPORTIONS USED
0-2 VERY SOFT	0-4 VERY LOOSE	TRACE: 0-10%
2-4 SOFT	4-10 LOOSE	LITTLE: 10-20%
4-8 MEDIUM STIFF	10-30 MEDIUM DENSE	SOME: 20-35%
8-15 STIFF	30-50 DENSE	AND: 35-50%
15-30 HARD	50+ VERY DENSE	

NOTES:

REMARKS: THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITION MAY BE GRADUAL. WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

TEST BORING LOG

MILLER ENGINEERING & TESTING, INC.
100 Sheffield Road - Manchester, NH 03103
Ph. (603) 668-6016 - Fax: (603) 668-8641

Project:	Carleton Drive Georgetown, MA
Project No:	21.067.NH
Date Start:	04-29-21
Date End:	04-29-21
Sheet	1 **of** 1
Boring No:	B-6
Location:	See Plan
Approx. Surface Elev:	

	CASING	SAMPLER	GROUNDWATER OBSERVATIONS			
			Date	**Depth**	**Casing At**	**Stabilization Period**
Type	HSA	SS	04-29-21	6'	8'	Upon Completion
Size	2-1/4" ID	1-3/8" ID				
Hammer		140 lbs.				
Fall		30"				

Depth/ Elev.	Cas bl/ft	Sample No.	Depth Range	Pen.	Rec.	0-6"	6-12"	12-18"	18-24"	Strata Change	Sample Description	Notes
0		-	0.0-0.3	4							-: 4" Asphalt	
		S-1	0.5-2.0	18	10		25	23	21		S-1: Brown, fine to coarse sand, some gravel, trace silt (FILL)	
2		S-2	2.0-4.0	24	12	17	17	22	21		S-2: Dark brown/brown, fine to coarse sand, some gravel and silt (FILL)	
4		S-3	4.0-5.0	12	6	8	3				S-3: Dark brown/brown, fine to coarse sand, some gravel and silt (FILL)	
		S-3A	5.0-6.0	12	6			3	2		S-3A: Dark brown, organic silt, little fine sand (peat)	
6		S-4	6.0-8.0	24	8	3	3	11	19		S-4: Brown, fine to medium sand, some gravel and silt	
8											BORING TERMINATED AT 8 ft	
10												
12												

Driller: R. Marcoux
Helper: J. Donahue
Inspector: T. Young

COHESIVE CONSISTENCY (Blows/Foot)
0-2 VERY SOFT
2-4 SOFT
4-8 MEDIUM STIFF
8-15 STIFF
15-30 HARD

COHESIONLESS (Blows/Foot)
0-4 VERY LOOSE
4-10 LOOSE
10-30 MEDIUM DENSE
30-50 DENSE
50+ VERY DENSE

PROPORTIONS USED
TRACE: 0-10%
LITTLE: 10-20%
SOME: 20-35%
AND: 35-50%

NOTES:

REMARKS: THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITION MAY BE GRADUAL. WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

TEST BORING LOG

MILLER ENGINEERING & TESTING, INC.
100 Sheffield Road - Manchester, NH 03103
Ph. (603) 668-6016 - Fax: (603) 668-8641

Project:	Carleton Drive Georgetown, MA	**Sheet**	1 **of** 1
		Boring No:	B-7
Project No:	21.067.NH	**Location:**	See Plan
Date Start:	04-29-21		
Date End:	04-29-21	**Approx. Surface Elev:**	

			GROUNDWATER OBSERVATIONS			
	CASING	**SAMPLER**	**Date**	**Depth**	**Casing At**	**Stabilization Period**
Type	HSA	SS	04-29-21	None	6'	Upon Completion
Size	2-1/4" ID	1-3/8" ID				
Hammer		140 lbs.				
Fall		30"				

Depth/ Elev.	Cas bl/ft	Sample No.	Depth Range	Pen.	Rec.	0-6"	6-12"	12-18"	18-24"	Strata Change	Sample Description	Notes
0		-	0.0-0.2	2.5							-: 2.5" Asphalt	
		S-1	0.3-2.0	20	9	4/2"	3/2"	35	34		S-1: Brown, fine to coarse sand, some gravel, little silt (FILL)	
2		S-2	2.0-3.5	18	9	28	25	23			S-2: Brown, fine to coarse sand, some gravel, little silt (1.5" layer of cemented oiled soil in middle of sample) (FILL)	
		S-2A	3.5-4.0	6	6				19		S-2A: Olive/brown (mottled), fine to medium sand, some silt, little gravel	
4		S-3	4.0-6.0	24	8	6	7	9	13		S-3: Brown, fine to coarse sand, some silt and gravel	
6											BORING TERMINATED AT 6 ft	
8												
10												
12												

Driller: R. Marcoux
Helper: J. Donahue
Inspector: T. Young

COHESIVE CONSISTENCY (Blows/Foot)
0-2 VERY SOFT
2-4 SOFT
4-8 MEDIUM STIFF
8-15 STIFF
15-30 HARD

COHESIONLESS (Blows/Foot)
0-4 VERY LOOSE
4-10 LOOSE
10-30 MEDIUM DENSE
30-50 DENSE
50+ VERY DENSE

PROPORTIONS USED
TRACE: 0-10%
LITTLE: 10-20%
SOME: 20-35%
AND: 35-50%

NOTES:

REMARKS: THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITION MAY BE GRADUAL.
WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS.
FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

TEST BORING LOG

MILLER ENGINEERING & TESTING, INC.
100 Sheffield Road - Manchester, NH 03103
Ph. (603) 668-6016 - Fax: (603) 668-8641

Project:	Carleton Drive, Georgetown, MA	**Sheet**	1 of 1
		Boring No:	B-8
Project No:	21.067.NH	**Location:**	See Plan
Date Start:	04-29-21		
Date End:	04-29-21	**Approx. Surface Elev:**	

	CASING	SAMPLER	GROUNDWATER OBSERVATIONS Date	Depth	Casing At	Stabilization Period
Type	HSA	SS	04-29-21	None	6'	Upon Completion
Size	2-1/4" ID	1-3/8" ID				
Hammer		140 lbs.				
Fall		30"				

Depth/ Elev.	Cas bl/ft	SAMPLE Sample No.	Depth Range	Pen.	Rec.	BLOWS 0-6"	6-12"	12-18"	18-24"	Strata Change	Sample Description	Notes
0		-	0.0-0.2	2.5							-: 2.5" Asphalt	
		S-1	0.5-1.0	6	2		25				S-1: Brown, fine to coarse sand, some crushed gravel, little silt (FILL)	
		S-1A	1.0-2.0	12	7			26	29		S-1A: Brown, fine to coarse sand, some gravel, trace silt	(1)
2		S-2	2.0-3.5	18	10	31	28	26			S-2: Brown, fine to coarse sand, some gravel, little silt (2" layer of cemented oiled soil in middle of sample) (FILL)	(2)
		S-2A	3.5-4.0	6	3				15		S-2A: Brown, fine to medium sand, some silt, little gravel (FILL)	
4		S-3	4.0-6.0	24	16	2	2	1	3		S-3: Dark brown, organic silt, some fine sand, trace fine organic roots (peat)	
6											BORING TERMINATED AT 6 ft	
8												
10												
12												

Driller: R. Marcoux
Helper: J. Donahue
Inspector: T. Young

COHESIVE CONSISTENCY (Blows/Foot)
0-2 VERY SOFT
2-4 SOFT
4-8 MEDIUM STIFF
8-15 STIFF
15-30 HARD

COHESIONLESS (Blows/Foot)
0-4 VERY LOOSE
4-10 LOOSE
10-30 MEDIUM DENSE
30-50 DENSE
50+ VERY DENSE

PROPORTIONS USED
TRACE: 0-10%
LITTLE: 10-20%
SOME: 20-35%
AND: 35-50%

NOTES: (1) Rock in tip of split-spoon.
(2) 2" layer of asphalt in middle of sample.

REMARKS: THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITION MAY BE GRADUAL. WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

TEST BORING LOG

MILLER ENGINEERING & TESTING, INC.
100 Sheffield Road - Manchester, NH 03103
Ph. (603) 668-6016 - Fax: (603) 668-8641

Project: Carleton Drive, Georgetown, MA
Project No: 21.067.NH
Date Start: 04-29-21
Date End: 04-29-21

Sheet 1 **of** 1
Boring No: B-9
Location: See Plan
Approx. Surface Elev:

	CASING	SAMPLER
Type	HSA	SS
Size	2-1/4" ID	1-3/8" ID
Hammer		140 lbs.
Fall		30"

GROUNDWATER OBSERVATIONS			
Date	**Depth**	**Casing At**	**Stabilization Period**
04-29-21	6'	6'	Upon Completion

Depth/ Elev.	Cas bl/ft	Sample No.	Depth Range	Pen.	Rec.	0-6"	6-12"	12-18"	18-24"	Strata Change	Sample Description	Notes
0		-	0.0-0.4	4.5							-: 4.5" Asphalt	
		S-1	0.4-2.0	19	6	2/1"	37	23	17		S-1: Brown, fine to coarse sand, some gravel, little silt (FILL)	(1)
2		S-2	2.0-2.5	6	4	10					S-2: Brown, fine to coarse sand, some gravel, little silt (FILL)	
		S-2A	2.5-4.0	18	10		4	3	3		S-2A: Orange brown, silt	
4		S-3	4.0-6.0	24	6	2	1	3	10		S-3: Brown, fine sand, some silt, trace gravel	(2)
6											BORING TERMINATED AT 6 ft	
8												
10												
12												

Driller: R. Marcoux
Helper: J. Donahue
Inspector: T. Young

COHESIVE CONSISTENCY (Blows/Foot)
0-2 VERY SOFT
2-4 SOFT
4-8 MEDIUM STIFF
8-15 STIFF
15-30 HARD

COHESIONLESS (Blows/Foot)
0-4 VERY LOOSE
4-10 LOOSE
10-30 MEDIUM DENSE
30-50 DENSE
50+ VERY DENSE

PROPORTIONS USED
TRACE: 0-10%
LITTLE: 10-20%
SOME: 20-35%
AND: 35-50%

NOTES:
(1) Rock fragments at top of sample and in tip of split-spoon.
(2) Piece of gravel at bottom of sample.

REMARKS: THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITION MAY BE GRADUAL.
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FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

TEST BORING LOG

MILLER ENGINEERING & TESTING, INC.
100 Sheffield Road - Manchester, NH 03103
Ph. (603) 668-6016 - Fax: (603) 668-8641

Project: Carleton Drive, Georgetown, MA
Project No: 21.067.NH
Date Start: 04-29-21
Date End: 04-29-21

Sheet 1 **of** 1
Boring No: B-10
Location: See Plan
Approx. Surface Elev:

	CASING	SAMPLER
Type	HSA	SS
Size	2-1/4" ID	1-3/8" ID
Hammer		140 lbs.
Fall		30"

GROUNDWATER OBSERVATIONS

Date	Depth	Casing At	Stabilization Period
04-29-21	5'	6'	Upon Completion

Depth/ Elev.	Cas bl/ft	Sample No.	Depth Range	Pen.	Rec.	Blows 0-6"	Blows 6-12"	Blows 12-18"	Blows 18-24"	Strata Change	Sample Description	Notes
0		-	0.0-0.3	4							-: 4" Asphalt	
		S-1	0.3-2.0	20	9	10/2"	34	23	20		S-1: Brown, fine to coarse sand, some gravel, little silt (FILL)	
2		S-2	2.0-4.0	24	4	17	7	5	6		S-2: Brown, fine to medium sand, some silt, little gravel (FILL)	
4		S-3	4.0-4.5	6	4	5					S-3: Dark brown, fine to medium sand, some silt, little gravel	
		S-3A	4.5-6.0	18	7		6	7	11		S-3A: Brown, fine to coarse sand, some gravel, trace silt, wet	(1)
6											BORING TERMINATED AT 6 ft	
8												
10												
12												

Driller: R. Marcoux
Helper: J. Donahue
Inspector: T. Young

COHESIVE CONSISTENCY (Blows/Foot)
0-2 VERY SOFT
2-4 SOFT
4-8 MEDIUM STIFF
8-15 STIFF
15-30 HARD

COHESIONLESS (Blows/Foot)
0-4 VERY LOOSE
4-10 LOOSE
10-30 MEDIUM DENSE
30-50 DENSE
50+ VERY DENSE

PROPORTIONS USED
TRACE: 0-10%
LITTLE: 10-20%
SOME: 20-35%
AND: 35-50%

NOTES: (1) Rock fragments in tip of split spoon.

REMARKS: THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITION MAY BE GRADUAL. WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

TEST BORING LOG

MILLER ENGINEERING & TESTING, INC.
100 Sheffield Road - Manchester, NH 03103
Ph. (603) 668-6016 - Fax: (603) 668-8641

Project: Carleton Drive, Georgetown, MA
Project No: 21.067.NH
Date Start: 04-29-21
Date End: 04-29-21

Sheet 1 **of** 1
Boring No: B-11
Location: See Plan
Approx. Surface Elev:

	CASING	SAMPLER
Type	HSA	SS
Size	2-1/4" ID	1-3/8" ID
Hammer		140 lbs.
Fall		30"

GROUNDWATER OBSERVATIONS

Date	Depth	Casing At	Stabilization Period
04-29-21	5'	6'	Upon Completion

Depth/ Elev.	Cas bl/ft	Sample No.	Depth Range	Pen.	Rec.	Blows 0-6"	Blows 6-12"	Blows 12-18"	Blows 18-24"	Strata Change	Sample Description	Notes
0		-	0.0-0.3	3							-: 3" Asphalt	
		S-1	0.4-1.0	7	7	3/1"	15				S-1: Brown, fine to coarse sand, some gravel, little silt (FILL)	
		S-1A	1.0-2.0	12	3			15	13		S-1A: Brown, fine to medium sand, some silt, little gravel	
2		S-2	2.0-4.0	24	12	8	13	9	17		S-2: Brown, fine to medium sand, some silt, little gravel	(1)
4		S-3	4.0-6.0	24	13	13	12	12	14		S-3: Brown, fine to coarse sand, little gravel, trace silt, wet	
6											BORING TERMINATED AT 6 ft	
8												
10												
12												

Driller: R. Marcoux
Helper: J. Donahue
Inspector: T. Young

COHESIVE CONSISTENCY (Blows/Foot)
0-2 VERY SOFT
2-4 SOFT
4-8 MEDIUM STIFF
8-15 STIFF
15-30 HARD

COHESIONLESS (Blows/Foot)
0-4 VERY LOOSE
4-10 LOOSE
10-30 MEDIUM DENSE
30-50 DENSE
50+ VERY DENSE

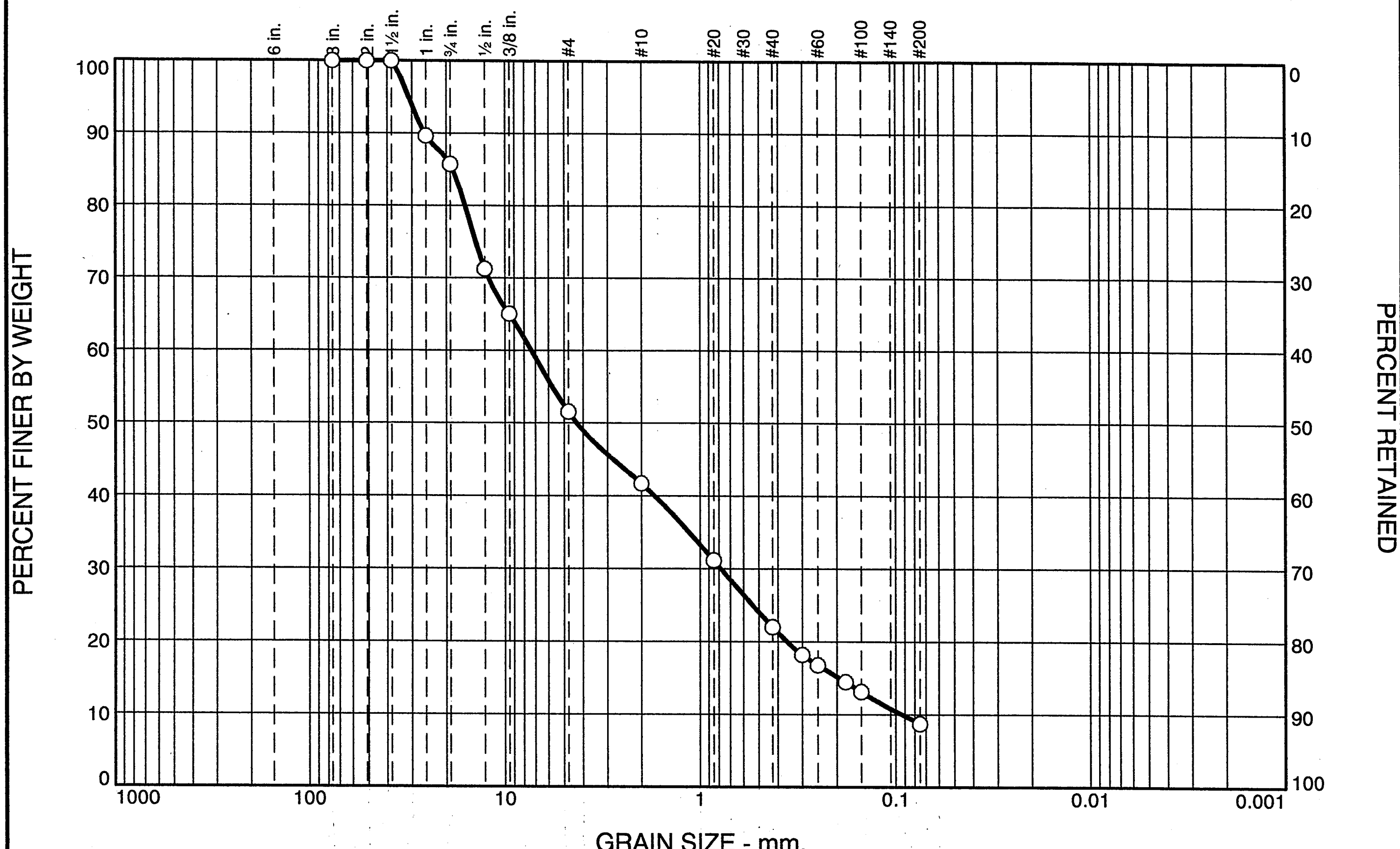
PROPORTIONS USED
TRACE: 0-10%
LITTLE: 10-20%
SOME: 20-35%
AND: 35-50%

NOTES: (1) Rock in tip of split-spoon.

REMARKS: THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITION MAY BE GRADUAL. WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

Appendix C

GRAINSIZE DISTRIBUTION REPORT - AGGREGATE GRADING



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	14	34	10	20	13	9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100		
2"	100		
1.5"	100		
1"	90		
.75"	86		
.5"	71		
.375"	65		
#4	52		
#10	42		
#20	31		
#40	22		
#50	18		
#60	17		
#80	14		
#100	13		
#200	8.8		

* (no specification provided)

Material Description

Brown, fine to coarse sand, some gravel, little silt (FILL)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 25.8393 D_{85}= 18.4965 D_{60}= 7.3399
D_{50}= 4.2896 D_{30}= 0.7793 D_{15}= 0.1944
D_{10}= 0.0926 C_u= 79.29 C_c= 0.89

Classification

USCS= AASHTO=

Remarks

BORING JAR SAMPLE.

Source of Sample: B-1 **Depth:** .5
Sample Number: S-1

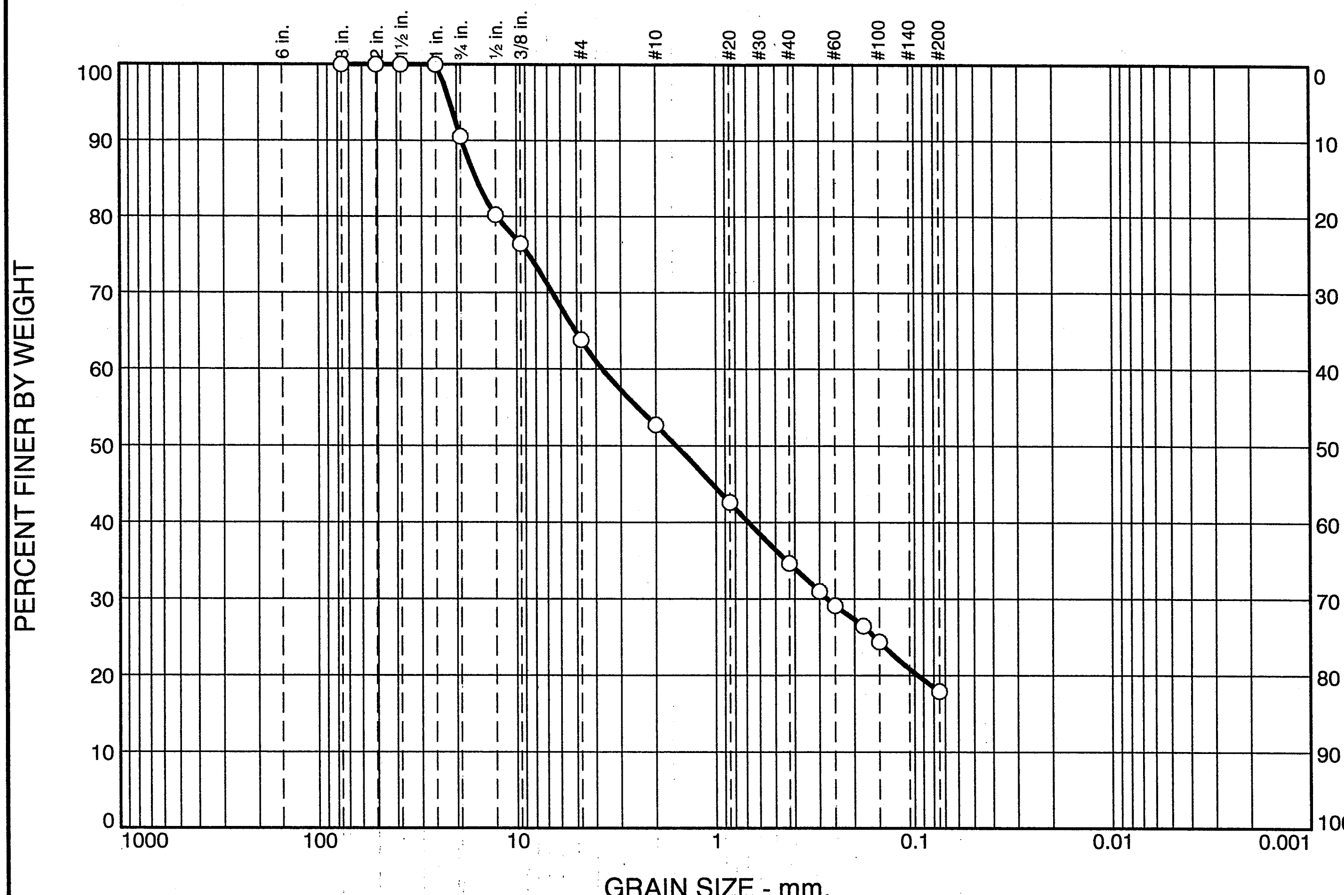
Date: 5-5-21

MILLER ENGINEERING & TESTING, INC.

Client: GPI
Project: Carleton Drive
Project No: 21.067.NH **Figure** L21131A

Tested By: DM/BM

GRAINSIZE DISTRIBUTION REPORT - AGGREGATE



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	9	27	11	18	17	18	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100		
2"	100		
1.5"	100		
1"	100		
.75"	91		
.5"	80		
.375"	76		
#4	64		
#10	53		
#20	43		
#40	35		
#50	31		
#60	29		
#80	26		
#100	24		
#200	18		

* (no specification provided)

Material Description

Brown, fine to coarse sand, some silt, little gravel (cemented oiled soil in sample) (FILL)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 18.7592 D_{85}= 15.9245 D_{60}= 3.7109
D_{50}= 1.5785 D_{30}= 0.2728 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

BORING JAR SAMPLE.

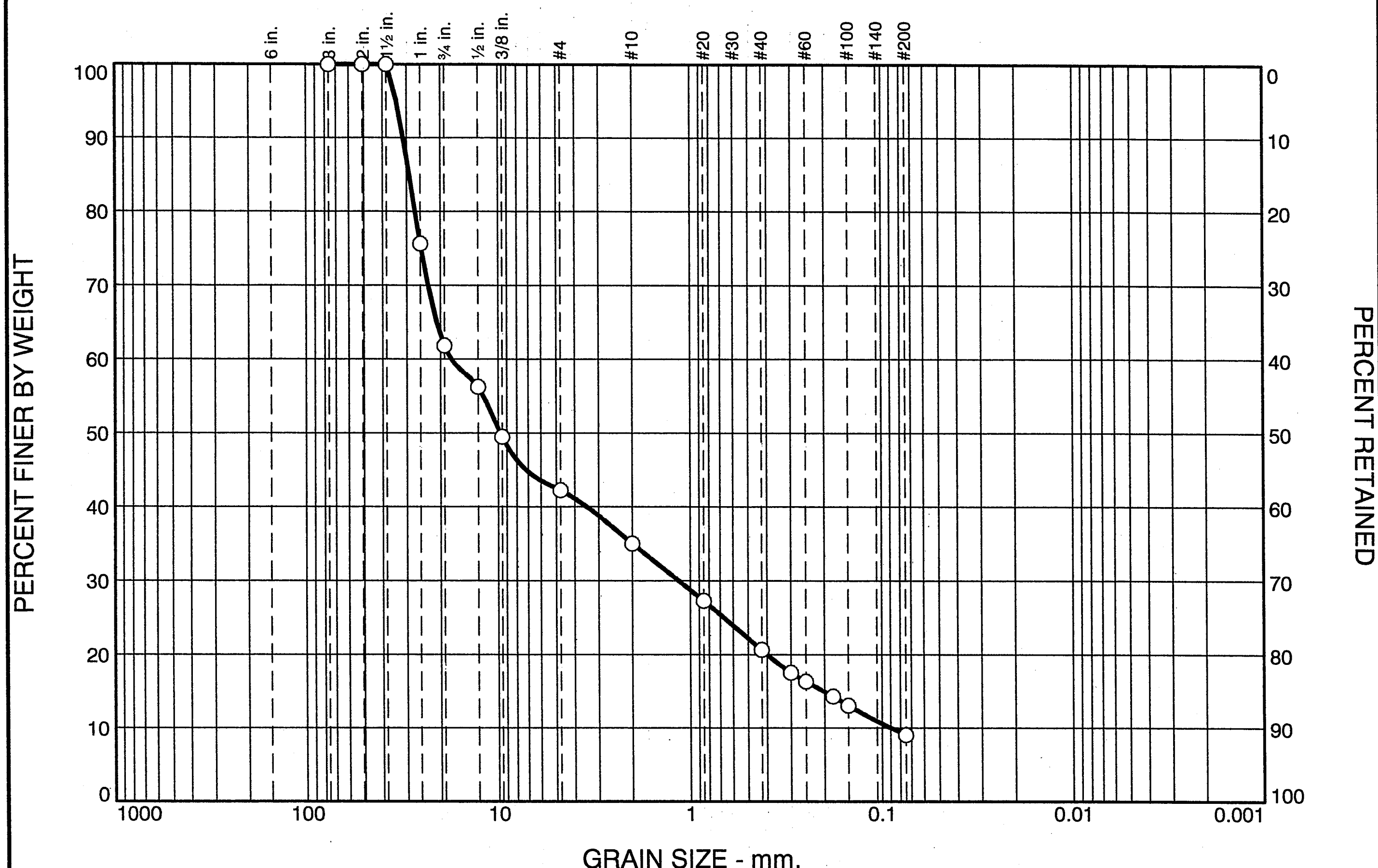
Source of Sample: B-1 **Depth:** 2
Sample Number: S-2

Date: 5-5-21

MILLER ENGINEERING & TESTING, INC.

Client: GPI
Project: Carleton Drive
Project No: 21.067.NH **Figure** L21131B

GRAINSIZE DISTRIBUTION REPORT - AGGREGATE



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	38	20	7	14	12	9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100		
2"	100		
1.5"	100		
1"	76		
.75"	62		
.5"	56		
.375"	49		
#4	42		
#10	35		
#20	27		
#40	21		
#50	18		
#60	16		
#80	14		
#100	13		
#200	9.1		

* (no specification provided)

Material Description

Brown, fine to coarse sand, some gravel, little silt (FILL)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 31.1882 D_{85}= 29.0255 D_{60}= 17.5544
D_{50}= 9.7393 D_{30}= 1.1428 D_{15}= 0.2016
D_{10}= 0.0893 C_u= 196.50 C_c= 0.83

Classification

USCS= AASHTO=

Remarks

BORING JAR SAMPLE.

Source of Sample: B-2 **Depth:** .5
Sample Number: S-1

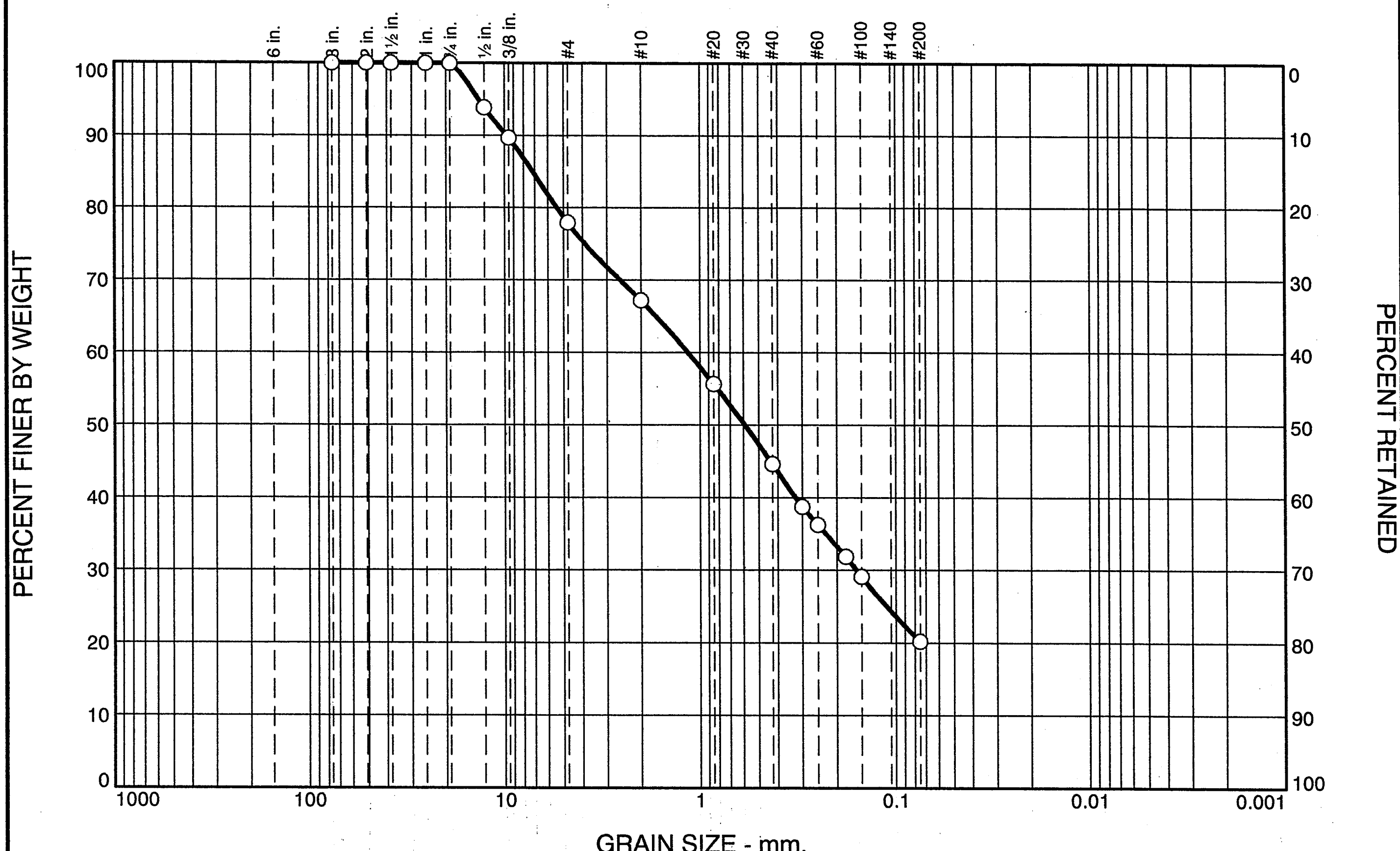
Date: 5-5-21

MILLER ENGINEERING & TESTING, INC.

Client: GPI
Project: Carleton Drive

Project No: 21.067.NH **Figure** L21131C

GRAINSIZE DISTRIBUTION REPORT - AGGREGATE GRADING



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	22	11	22	25	20	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100		
2"	100		
1.5"	100		
1"	100		
.75"	100		
.5"	94		
.375"	90		
#4	78		
#10	67		
#20	56		
#40	45		
#50	39		
#60	36		
#80	32		
#100	29		
#200	20		

* (no specification provided)

Material Description

Brown, fine to coarse sand, some gravel and silt (cemented oiled soil in sample) (FILL)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 9.7216 D_{85}= 7.1474 D_{60}= 1.1489
D_{50}= 0.5869 D_{30}= 0.1592 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

BORING JAR SAMPLE.

Source of Sample: B-2 **Depth:** 2
Sample Number: S-2

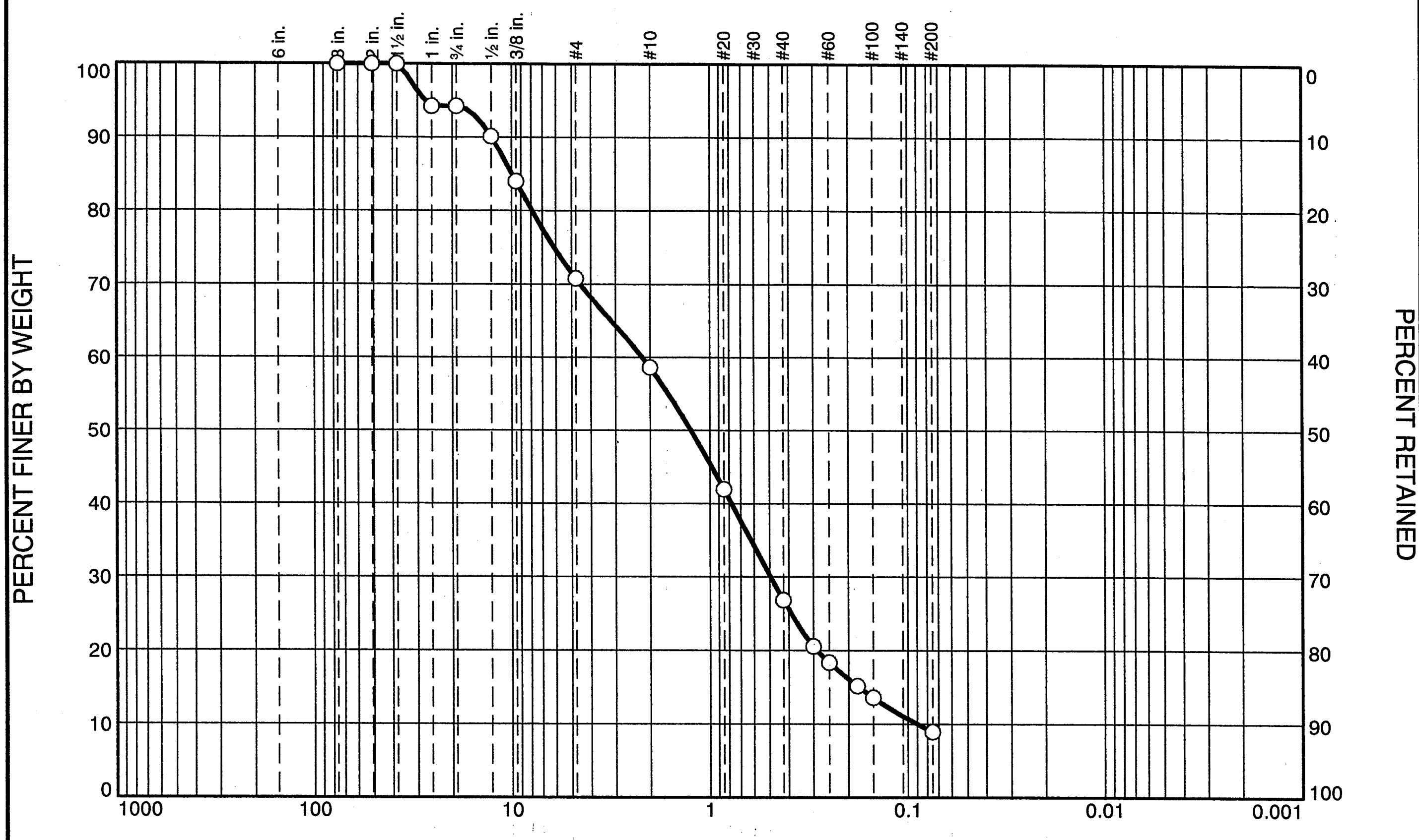
Date: 5-5-21

MILLER ENGINEERING & TESTING, INC.

Client: GPI
Project: Carleton Drive
Project No: 21.067.NH **Figure** L21131D

Tested By: DM/BM

GRAINSIZE DISTRIBUTION REPORT - AGGREGATE GRADING



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	6	23	12	32	18	9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100		
2"	100		
1.5"	100		
1"	94		
.75"	94		
.5"	90		
.375"	84		
#4	71		
#10	59		
#20	42		
#40	27		
#50	21		
#60	18		
#80	15		
#100	14		
#200	9.0		

* (no specification provided)

Material Description

Brown, fine to coarse sand, some gravel, trace silt (FILL)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 12.6009 D_{85}= 9.9531 D_{60}= 2.1902
D_{50}= 1.2429 D_{30}= 0.4940 D_{15}= 0.1762
D_{10}= 0.0894 C_u= 24.50 C_c= 1.25

Classification

USCS= AASHTO=

Remarks

BORING JAR SAMPLE.

Source of Sample: B-6 **Depth:** .5
Sample Number: S-1

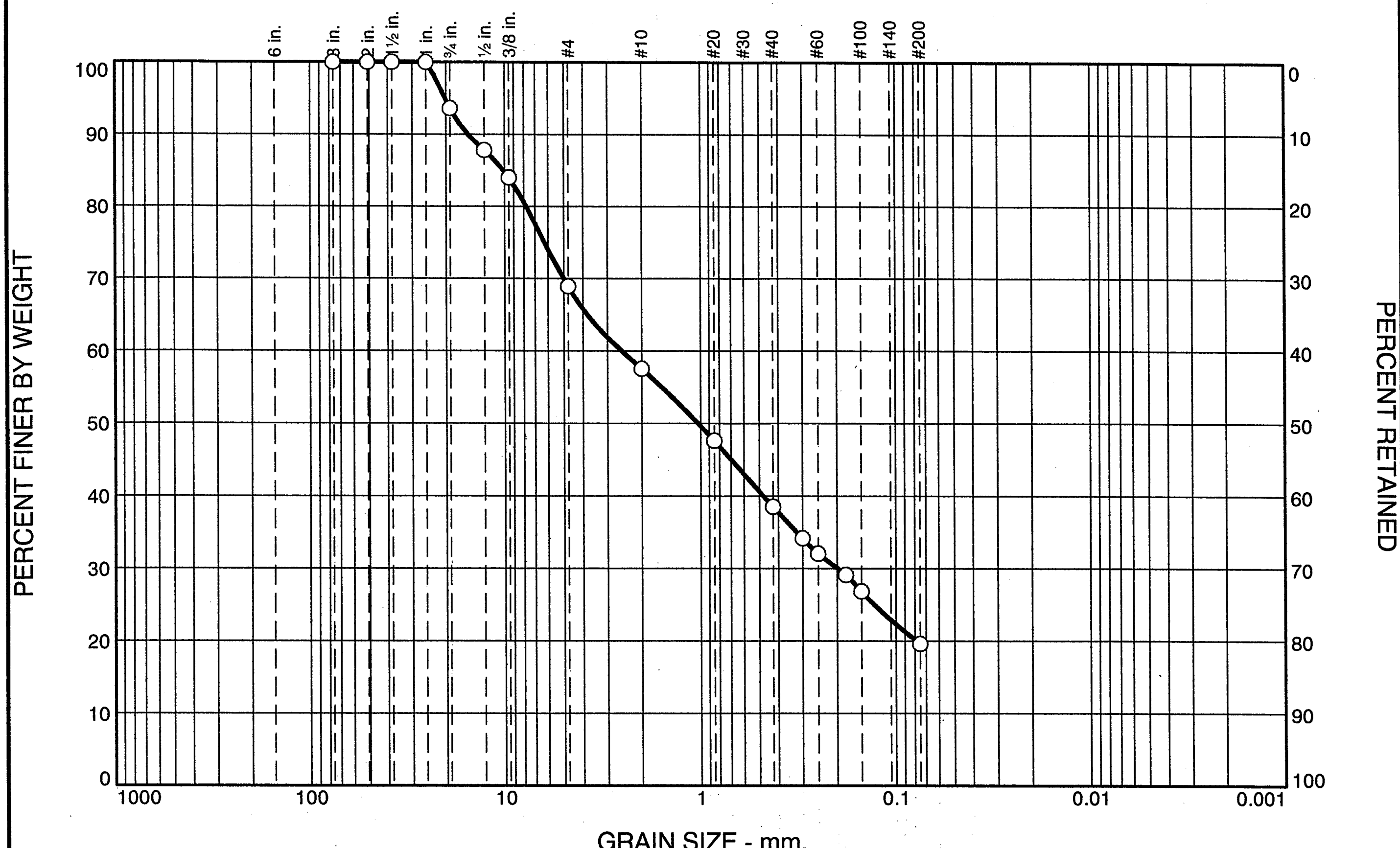
Date: 5-5-21

MILLER ENGINEERING & TESTING, INC.

Client: GPI
Project: Carleton Drive
Project No: 21.067.NH **Figure** L21131E

Tested By: DM/BM

GRAINSIZE DISTRIBUTION REPORT - AGGREGATE GRADING



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	6	25	11	19	19	20	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100		
2"	100		
1.5"	100		
1"	100		
.75"	94		
.5"	88		
.375"	84		
#4	69		
#10	58		
#20	48		
#40	39		
#50	34		
#60	32		
#80	29		
#100	27		
#200	20		

* (no specification provided)

Material Description

Dark brown/brown, fine to coarse sand, some gravel and silt (FILL)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 15.3604 D_{85}= 10.1482 D_{60}= 2.5275
D_{50}= 1.0248 D_{30}= 0.1965 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

BORING JAR SAMPLE.

Source of Sample: B-6 **Depth:** 2
Sample Number: S-2

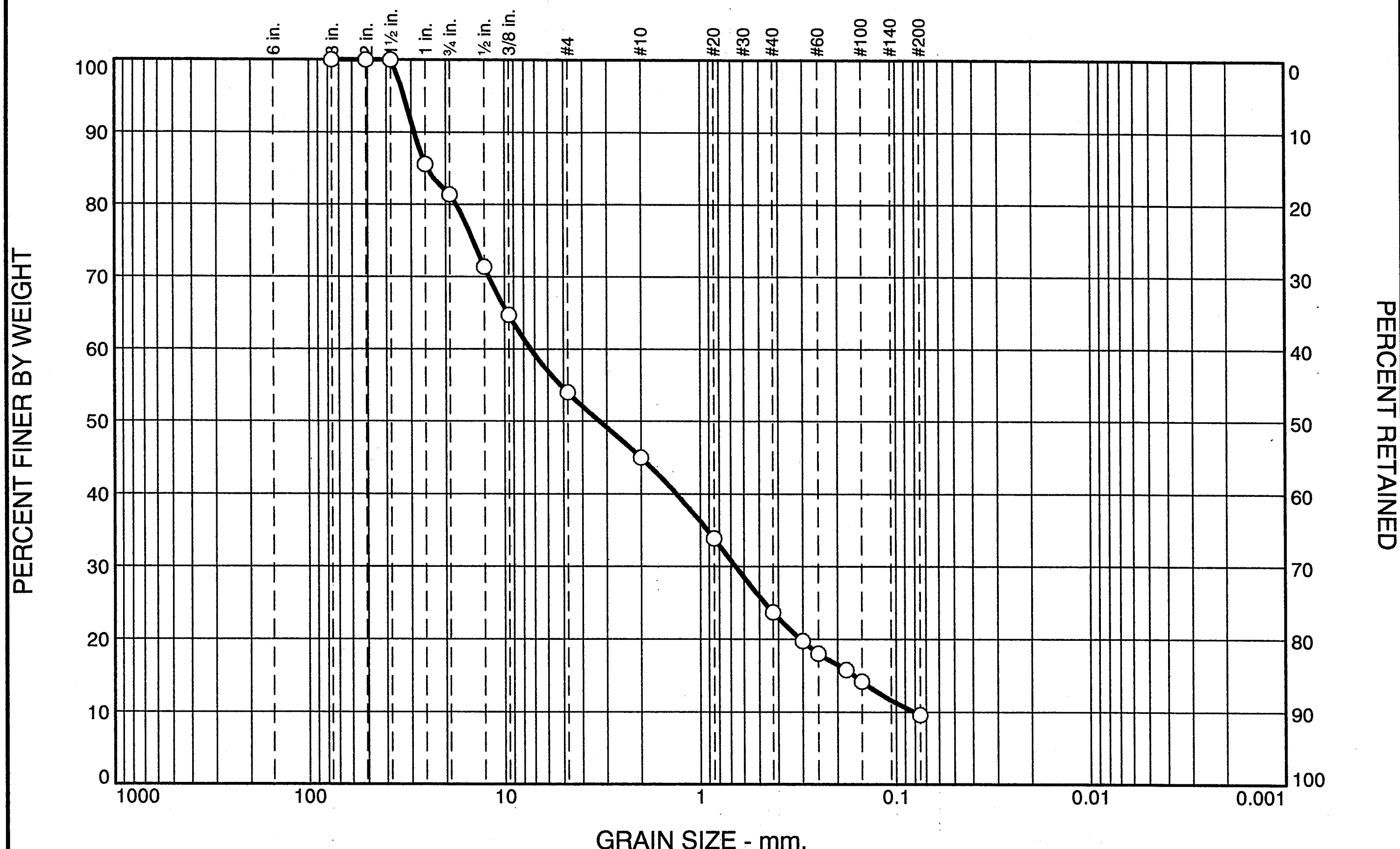
Date: 5-5-21

MILLER ENGINEERING & TESTING, INC.

Client: GPI
Project: Carleton Drive
Project No: 21.067.NH **Figure** L21131F

Tested By: DM/BM

GRAINSIZE DISTRIBUTION REPORT - AGGREGATE GRADING



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	19	27	9	21	14	10	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100		
2"	100		
1.5"	100		
1"	86		
.75"	81		
.5"	71		
.375"	65		
#4	54		
#10	45		
#20	34		
#40	24		
#50	20		
#60	18		
#80	16		
#100	14		
#200	9.6		

* (no specification provided)

Material Description

Brown, fine to coarse sand, some gravel, trace silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 28.8242 D_{85}= 24.8329 D_{60}= 7.3391
D_{50}= 3.2636 D_{30}= 0.6588 D_{15}= 0.1640
D_{10}= 0.0805 C_u= 91.17 C_c= 0.73

Classification

USCS= AASHTO=

Remarks

BORING JAR SAMPLE.

Source of Sample: B-8 **Depth:** 1
Sample Number: S-1A

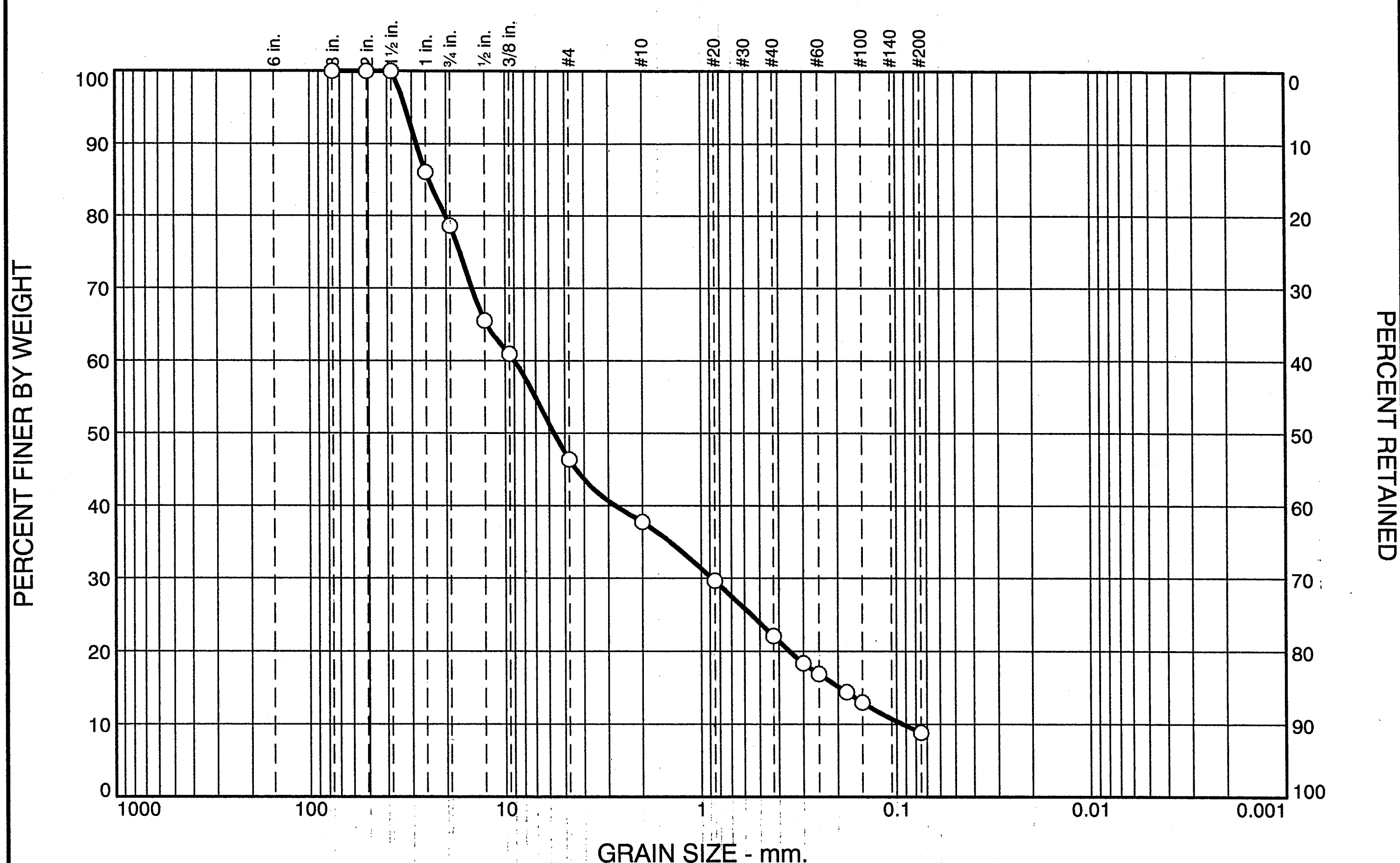
Date: 5-5-21

MILLER ENGINEERING & TESTING, INC.

Client: GPI
Project: Carleton Drive
Project No: 21.067.NH **Figure** L21131G

Tested By: DM/BM

GRAINSIZE DISTRIBUTION REPORT - AGGREGATE GRADING



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	21	33	8	16	13	9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100		
2"	100		
1.5"	100		
1"	86		
.75"	79		
.5"	66		
.375"	61		
#4	46		
#10	38		
#20	30		
#40	22		
#50	18		
#60	17		
#80	14		
#100	13		
#200	8.8		

* (no specification provided)

Material Description

Brown, fine to coarse sand, some gravel, little silt (2" layer of cemented oiled soil in middle of sample) (FILL)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 28.2689 D_{85}= 24.5309 D_{60}= 8.9791
D_{50}= 5.6547 D_{30}= 0.8783 D_{15}= 0.1947
D_{10}= 0.0934 C_u= 96.14 C_c= 0.92

Classification

USCS= AASHTO=

Remarks

BORING JAR SAMPLE.

Source of Sample: B-8 **Depth:** 2
Sample Number: S-2

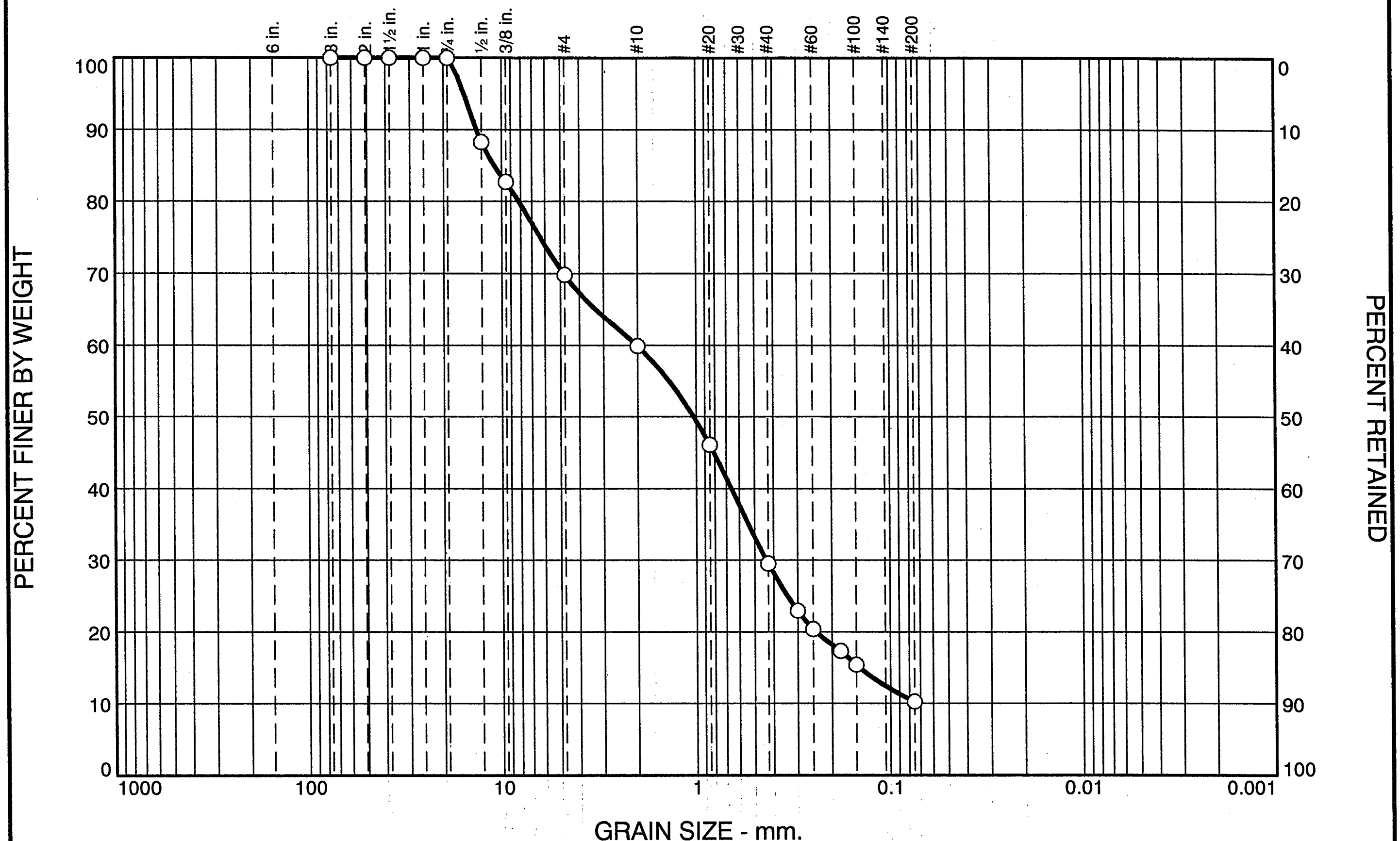
Date: 5-5-21

MILLER ENGINEERING & TESTING, INC.

Client: GPI
Project: Carleton Drive
Project No: 21.067.NH **Figure** L21131H

Tested By: DM/BM

GRAINSIZE DISTRIBUTION REPORT - AGGREGATE GRADING



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	30	10	30	20	10	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100		
2"	100		
1.5"	100		
1"	100		
.75"	100		
.5"	88		
.375"	83		
#4	70		
#10	60		
#20	46		
#40	30		
#50	23		
#60	20		
#80	17		
#100	15		
#200	10		

* (no specification provided)

Material Description

Brown, fine to coarse sand, some gravel, little silt (FILL)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 13.4676 D_{85}= 10.9302 D_{60}= 2.0183
D_{50}= 1.0274 D_{30}= 0.4337 D_{15}= 0.1441
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

BORING JAR SAMPLE.

Source of Sample: B-10 **Depth:** .333
Sample Number: S-1

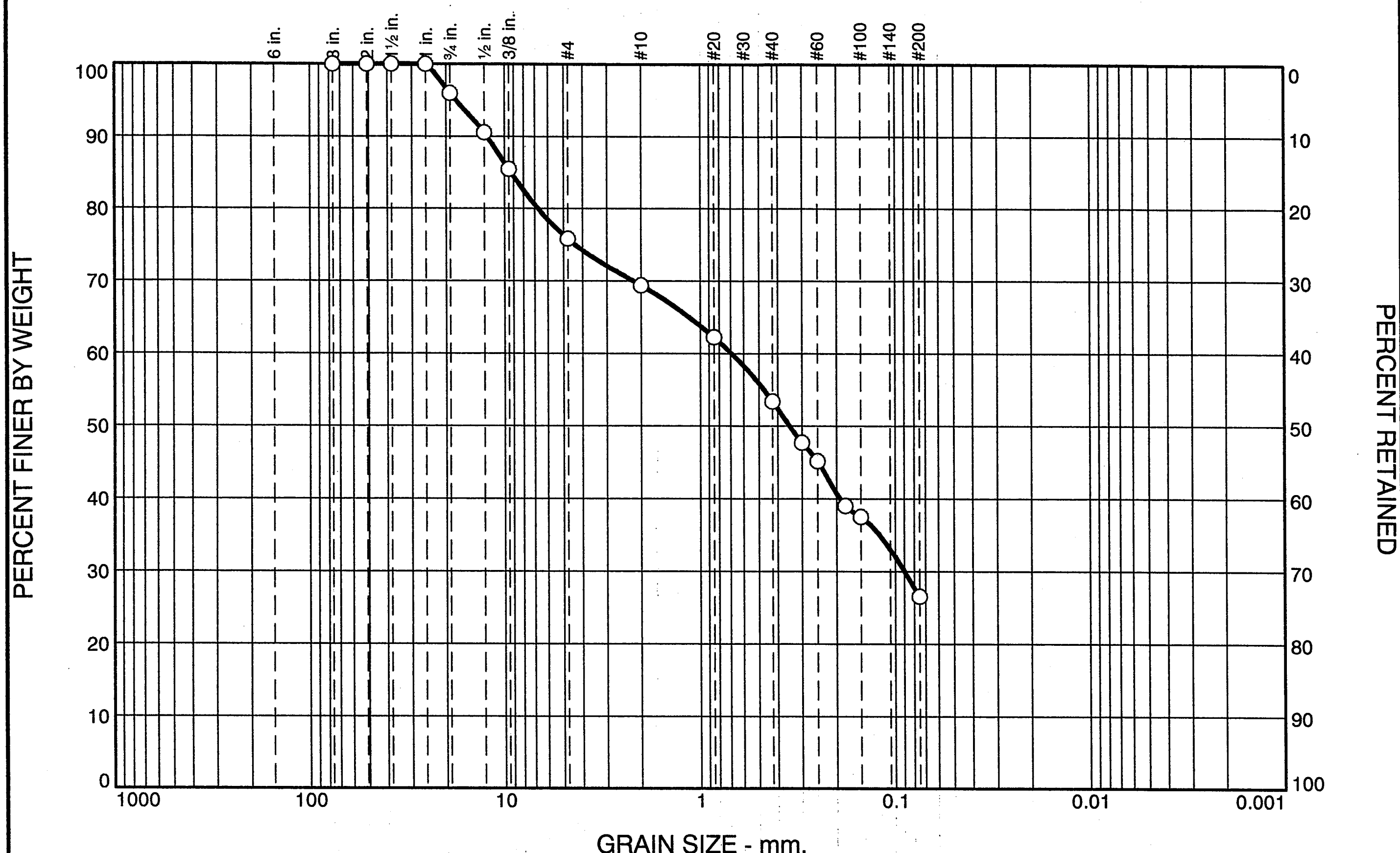
Date: 5-5-21

MILLER ENGINEERING & TESTING, INC.

Client: GPI
Project: Carleton Drive
Project No: 21.067.NH **Figure** L21131I

Tested By: DM/BM

GRAINSIZE DISTRIBUTION REPORT - AGGREGATE GRADING



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	4	20	7	16	26	27	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100		
2"	100		
1.5"	100		
1"	100		
.75"	96		
.5"	91		
.375"	85		
#4	76		
#10	69		
#20	62		
#40	53		
#50	48		
#60	45		
#80	39		
#100	38		
#200	27		

* (no specification provided)

Material Description

Brown, fine to medium sand, some silt, little gravel (FILL)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 12.2801 D_{85}= 9.2605 D_{60}= 0.6883
D_{50}= 0.3473 D_{30}= 0.0893 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

BORING JAR SAMPLE.

Source of Sample: B-10 **Depth:** 2
Sample Number: S-2

Date: 5-6-21

MILLER ENGINEERING & TESTING, INC.

Client: GPI
Project: Carleton Drive
Project No: 21.067.NH **Figure** L21131J

Tested By: DM/BM

Appendix D

Project Information:

Project Name:	Carleton Road	Project No.:	21.081.NH
Location:	Georgetown, MA	Calculated by:	GNG
Client:	GPI	Date:	5/12/2021

Design Information:

Performance Period (years):	20	Reliability % (R):	90
Initital Serviciability (P_o):	4.5	Overall Standard Dev. (So):	0.5
Terminal Servicibility (P_t):	2.0		
Servicibility Loss (ΔPSI = (P_o-P_t):	2.5		

Subgrade Soil Description: Brown, fine to med sand, little to some silt, little to some gravel (FILL)

Resilient Modulus (M_R):	4500 psi	M_R Estimated:	YES
California Bearing Ratio (CBR):		CBR Estimated:	Y / N

Summary of Results:

Standard Duty Pavement Section

Design ESAL:	
Asphalt Wearing Course:	**1.5 in**
Asphalt Binder Course:	**2.0 in**
Base Course:	**6.0 in**
Subbase Course or Existing Subgrade:	**12.0 in**

Heavy Duty Pavement Section

Design ESAL:	
Asphalt Wearing Course:	**2.0 in**
Asphalt Binder Course:	**3.0 in**
Base Course:	**12.0 in**
Subbase Course or Existing Subgrade:	**20.0 in**

Calculate Equivalent 18-Kip Single Aexl Loading (ESALs)

Equivalent Single Axel Loads per Vehicle: SN = 3 P_t = 2

Typical Passenger Vehicle:

Axel Type	Description	Axel Load (kip)	Load Equivalency Factor		Calculated ESALs	
(S)	Front Single Axel	2	LEF =	0.0002	(1 Axel)(0.0002)+(1 Axel)(0.0002) =	**0.0004**
(S)	Rear Single Axel	2	LEF =	0.0002		per car

Typical Light Duty Truck (H20):

Axel Type	Description	Axel Load (kip)	Load Equivalency Factor		Calculated ESALs	
(S)	Front Single Axel	8	LEF =	0.036	(1 Axel)(0.036)+(1 Axel)(0.843) =	**0.879**
(T)	Rear Single Axel	32	LEF =	0.843		per truck

Typical Trailer Truck (HS20):

Axel Type	Description	Axel Load (kip)	Load Equivalency Factor		Calculated ESALs	
(S)	Front Single Axel	8	LEF =	0.036	(1 Axel)(0.036)+2(1 Axel)(0.843)=	**1.722**
(T)	Rear Single Axel	32	LEF =	0.843		per truck
(T)	Trailer Axel	32	LEF =	0.843		

(S) = Single Avel, (T) = Tandem Axel, (3)=Tripple Axel

Traffic Loading Calculations: Performance Period = 20 years

Standard Duty Pavement Section ESALs

Vehicle Type	Current Traffic	Growth Factor	Design Traffic	ESAL Factor	Design ESAL
Passenger Vehicle	1103	No Growth 20.00	8,051,900	0.0004	3,221
Light Trucks	135	No Growth 20.00	985,500	0.879	866,255
				Standard Duty ESALs =	**869,475**

Heavy Duty Pavement Section ESALs

Vehicle Type	Current Traffic	Growth Factor	Design Traffic	ESAL Factor	Design ESAL
Passenger Vehicle	1103	No Growth 20.00	8,051,900	0.0004	3,221
Light Trucks	135	No Growth 20.00	985,500	0.879	866,255
Heavy Trucks	194	No Growth 20.00	1,416,200	1.722	2,438,696
				Heavy Duty ESALs =	**3,308,172**

Estimate Required Structural Number from AASHTO Nomograph

Design Information (From P1):

Reliability % (R):	90
Overall Standard Dev. (So):	0.5
Resilient Modulus psi (M_R):	4500
Servicibility Loss ($\Delta PSI = (P_o - P_t)$):	2.5

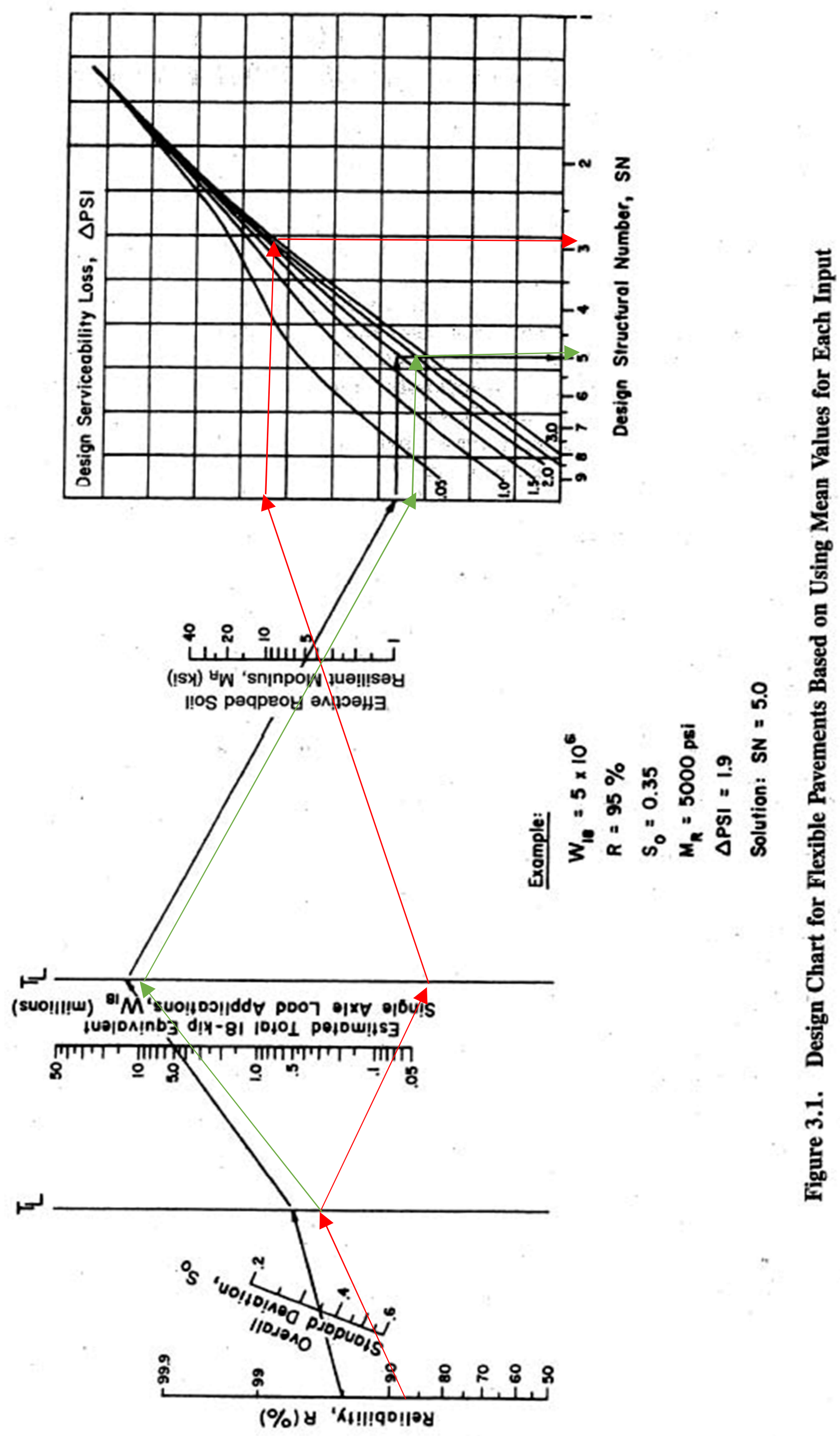
Traffic Information (From P2):

Standard Duty ESALs (W18):	869,475
(millions)	0.87
Heavy Duty ESALs (W18):	3,308,172
(millions)	3.31

Solutions From Nomograph:

Design Structural Number (SN)

Standard Duty =	3.0
Heavy Duty =	5.0



MILLER ENGINEERING & TESTING, INC.	**Carleton Road** **Georgetown, MA** **21.081.NH** **5/12/2021**	AASHTO Flexible Pavement Nomograph	Figure No.: P3 Sheet 3 of 4

Flexible Pavement Section Design

Structural Number (SN) = $D_1(a_1)+D_2(a_2)(m_2)+D_3(a_3)(m_3)$

Material Properties:

Material	Layer (ID) (D)	Layer Properties: Strength Coef. (a)	Layer Properties: Drainage Coef. (m)
Asphalt Wearing Course	1	0.44	-
Asphalt Binder Course	1	0.44	-
Base Course	2	0.12	1.00
Subbase Course	3	0.08	0.90

Standard Duty Pavement Section Design:

Material	Spec	Thickness (Inches)	Layer Strength	Structural Number (SN)
Asphalt Wearing Course	12.5mm Superpave Surface Course MHD Mix SSC - 12.5	1.5	0.44	0.66
Asphalt Binder Course	19.0mm Superpave Intermediate Course MHD Mix SIC - 190	2.0	0.44	0.88
Base Course	Dense Graded Crushed Gravel MHD Item M1.03.1	6.0	0.12	0.72
Subbase Course	Existing Silty Gravel Subgrade or Reclaimed Pavement Borrow (M1.09.0)	12.0	0.08	0.864

Design Structural Number for Section = 3.12
Required Structural Number for Traffic = 3.00
Check Design SN ≥ Required SN = OK

Heavy Duty Pavement Section Design:

Material	Spec	Thickness (Inches)	Layer Strength	Structural Number (SN)
Asphalt Wearing Course	12.5mm Superpave Surface Course MHD Mix SSC - 12.5	2.0	0.44	0.88
Asphalt Binder Course	19.0mm Superpave Intermediate Course MHD Mix SIC - 190	3.0	0.44	1.32
Base Course	Dense Graded Crushed Gravel MHD Item M1.03.1	12.0	0.12	1.44
Subbase Course	Existing Silty Gravel Subgrade or Reclaimed Pavement Borrow (M1.09.0)	20.0	0.08	1.44

Design Structural Number for Section = 5.08
Required Structural Number for Traffic = 5.00
Check Design SN ≥ Required SN = OK

MILLER ENGINEERING & TESTING, INC.	**Carleton Road** **Georgetown, MA** **21.081.NH** **5/12/2021**	Flexible Pavement Section Calculation	Figure No.: P4 Sheet 4 of 4