BID PROJECT NO. 1206-10

COUNTY OF MONROE
DEPARTMENT OF TRANSPORTATION

ADDENDUM NO. 1

TO THE

CONSTRUCTION SPECIFICATIONS AND RELATED DOCUMENTS

FOR

PECK ROAD OVER SALMON CREEK
BRIDGE REPLACEMENT

Capital Improvement Project No. 1451.01.05

STATE OF NEW YORK
ROBERT WILLIAM HUTTEMAN
072062
LICENSED PROFESSIONAL ENGINEER

12-17-10

TO ALL BIDDERS:

The following constitutes ADDENDUM NO. 1 of the Contract Documents. Each Bidder shall acknowledge receipt of the Addendum on Page P-5 of the Proposal.

Pages ADD: 1-1 through ADD: 1-29 Date: December 17, 2010

TO ALL BIDDERS:

Pages ADD: 1-1 through ADD: 1-29 constitutes Addendum No. 1 to the Contract Documents. Make the following changes to the Contract Specifications and Related Documents:

A. ADDITIONS TO THE PROJECT CONTRACT DOCUMENTS

1. Pages ADD 1-3 through ADD 1-29 constitute the Geotechnical Report for the project and should be utilized in conjunction with the plans and specifications for the project. This addendum transmits information only. No other changes to the plans and specifications are necessary as a result of the addendum.

BID OPENING DATE REMAINS JANUARY 19, 2011 at 2:00 P.M.

EMPIRE GEO
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A SUBSIDIARY OF SJB SERVICES, INC.

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Geotechnical Evaluation Report for Proposed Bridge Replacement Project Peck Road (CR 183) Over Salmon Creek B.I.N 3317820, P.I.N. 4753.98 Town of Parma Monroe County, New York

Prepared For:

Lu Engineers
2230 Penfield Road
Penfield, New York
14526

Prepared By:

Empire Geo-Services, Inc.
535 Summit Point Drive
Henrietta, New York 14467

STATE OF NEW YORK
WENDEL CARL ARMSTRONG
085856
LICENSED PROFESSIONAL ENGINEER
1/9/09

Project No. RE-08-029
January 2009

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January 9, 2009
Project No. RE-08-029

Mr. Jason Messenger
Lu Engineers
2230 Penfield Road
Penfield, New York 14526

Re: Geotechnical Evaluation Report for
Proposed Bridge Replacement Project
Peck Road (CR 183) Over Salmon Creek
B.I.N 3317820, P.I.N. 4753.98
Town of Parma
Monroe County, New York

Dear Mr. Messenger:

Empire Geo-Services, Inc. (Empire) is pleased to submit this letter report summarizing our subsurface exploration and geotechnical evaluation completed for the proposed replacement of the existing bridge carrying Peck Road (CR183) over Salmon Creek in the Town of Parma, Monroe County, New York. The approximate location of the project site is shown on Figure 1. This work was completed in accordance with our proposal, dated October 9, 2008, and the subsequent authorization by Lu Engineers (Lu).

Project Description

The proposed Peck Road bridge replacement project is located on Peck Road, between Clarkson Parma Town Line Road and approximately 1.6 miles west of Spencer Road in the Town of Parma. The existing bridge consists of an approximate 36 feet long single span, steel girder deck bridge, supported on concrete abutments, with associated wing walls. The existing bridge carries Peck Road in a general east - west direction over Salmon Creek. The bottom of the existing creek (mud line) is approximately 10 feet below the existing road surface.

Based on the information provided by Lu, we understand the project will include replacement of the existing bridge along the same general alignment. The proposed bridge structure is expected to consist of a pre-cast concrete arched span bridge system, with a span of about 36 feet and a rise of about 11 feet. It is anticipated that the bottom of bridge foundations will bear approximately 13 feet below the existing road surface. The expected total loads (live + dead) on the footings will be approximately 19 kips per linear feet.

Subsurface Exploration

The subsurface exploration program consisted of two (2) test borings drilled by our affiliated drilling company, SJB Services, Inc. (SJB), on October 30th, 2008. The test borings are designated as borings B-1 and B-2. The approximate locations of the test borings are shown on Figure 2.

The test boring locations were initially selected and plotted on a site plan provided by Lu, entitled "Roadway Plan". The test boring locations were then established in the field by SJB using tape measurements referenced to existing site features. Optical survey level techniques were utilized to determine the existing ground surface elevations at the test boring locations. The ground surface elevations were referenced an existing survey benchmark designated "Benchmark #2" located at the existing southeast corner of the bridge, as shown on Figure 2. The benchmark elevation is designated as 348.82 feet, as shown on Figure 2.

The test borings were made using a Central Mine Equipment (CME) model 550X, rubber floatation tire, all terrain type vehicle mounted drill rig, using hollow stem auger and split spoon sampling techniques. The test borings were advanced by augering and split spoon sampling to depths of 28.2 and 28.7 feet below the existing ground surface (bgs), where auger refusal conditions were encountered. Split spoon samples and Standard Penetration Tests (SPTs) were taken continuously to a depth of about 10 feet and in intervals of five feet or less for the remaining depth of the test borings. Split spoon samples and SPTs were completed in general accordance with *ASTM D1586 – "Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils"*.

After auger refusal conditions were met, the test borings were further advanced with rock core sampling in general accordance with *ASTM D 2113 – "Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation"*. The rock coring was advanced 5 feet at both test boring locations.

A geologist prepared the test boring logs based on visual observations of the recovered soil samples and a review of the driller's field notes. The soil samples were described based on a visual/manual estimation of the grain size distribution, along with characteristics such as color, relative density, consistency, moisture, etc. The recovered rock cores were also described, including characteristics such as color, rock type, hardness, weathering, bedding thickness, core recovery and rock quality designation (RQD). The test boring logs are presented in Appendix A, along with general information and a key of terms and symbols used to prepare the logs.

Subsurface Conditions

The general stratigraphy encountered at the two test boring locations were generally similar. These conditions consisted of an upper surficial zone of fill soils, which generally consisted of silty sand, gravelly sand, and clayey silt, with inclusions of organics and occasional cobble fragments. The fill soils extended to a depth of about 13 to 15 feet bgs. Standard Penetration Test (SPT) "N" values obtained in the fill soils ranged from 4 to 21 indicating the fill soils appear to have been placed in a generally uncontrolled manner. It should be expected that the fill thickness will increase near the existing bridge abutments and will extend to at least the bottom of the existing abutment foundations or the excavations made to construct the foundations.

Beneath the fill soils, a stratum of indigenous highly weathered Shale with inclusions of silt was encountered, which extended to a depth of about 28.2 feet to 28.7 feet bgs, where auger refusal was encountered. Following auger refusal, 5 feet of rock core was completed at each test boring location. The bedrock recovered from the test borings are generally described as red-brown, slightly weathered to sound, soft to medium hard, thinly bedded to bedded Shale. The core recovery at test borings B-1 and B-2 were 99% and 98%, respectively. The RQD ranged from 77% to 86% indicating the recovered rock core generally has a "good" rock mass quality.

Water level measurements were made in the borings at the completion of drilling. Freestanding water was present during drilling and sampling at test boring B-2 at a depth of about 8 feet bgs. No freestanding water was apparent at test boring B-1. We note, however, the groundwater may not have had sufficient time to accumulate in the borings within the time period that had elapsed from the completion of drilling operations and the time of measurement.

Based on these measurements, it appears a general groundwater condition may be present at approximately 8 feet bgs and below, which generally coincides with the depth to the water level within Salmon Creek. Although not observed during our investigation, it is possible that perched or trapped groundwater conditions could exist within the more permeable fill soils, which overly the less permeable highly weathered shale and silt stratum and the shale bedrock. Perched groundwater conditions can be particularly more prevalent following heavy or extended periods of precipitation and during seasonally wet periods. Both perched and permanent groundwater conditions can be expected to vary with location and with changes in soil conditions, precipitation, seasonal conditions, and with fluctuations in the adjacent stream levels.

The individual test boring logs in Appendix A should be referenced for more detailed information regarding the subsurface and groundwater conditions encountered at the boring locations.

Geotechnical Considerations and Recommendations

General:

The following general considerations and recommendations are provided to assist with planning the design and construction of the proposed replacement bridge foundations and the associated site work. More detailed recommendations are presented in the subsequent sections of this report.

Based on our analysis of the conditions encountered in the test borings, it is Empire's opinion that the proposed replacement bridge could be supported with spread foundations; however it does not appear to be a practical option for this project, due to the groundwater conditions which may require substantial dewatering methods and a significant amount of excavation that may be required to embed the foundations to provide adequate scour protection.

As an alternative to spread foundations, a deep foundation consisting of drilled piers or micro-piles bearing within the Shale bedrock could be considered to support the replacement bridge foundations given the relatively shallow depth to competent bedrock, from the proposed bottom of the foundations.

The use of a deep foundation will limit the amount of excavation required and the construction dewatering requirements. In addition, the scour protection as required with a spread foundation system would be reduced or eliminated. We point out that there can be some potential for boulders and cobbles within the fill soils, which should be anticipated with drilled pier installation. In addition, the drilled pier excavations may require the use of temporary casing and/or drilling slurry to stabilize the pier excavations.

The use of driven piles, do not appear to be a practical option for this project. It is expected that pre-drilling through the highly weathered Shale and silt would be necessary to bear the driven piles on competent shale bedrock. Accordingly, drilled piers or micro-piles as discussed above would appear to be a more practical foundation option.

Drilled Pier Foundation Design:

Drilled pier foundations (i.e. drilled, cast in-place caisson foundations) bearing on the Shale bedrock could be used to support the proposed bridge structure. Drilled pier foundations should be seated a nominal 2 feet into the competent bedrock to insure that the piers are bearing on competent bedrock.

It is anticipated that the drilled piers would need to be initially advanced to a depth of about 28 feet below the existing road surface (i.e. about El. 320.0 feet), prior to seating the drilled piers an additional 2 feet into the bedrock. It should be expected that the depth of the competent bedrock, between and away from the test boring locations, may vary and may require adjustments in the bearing elevation based on actual conditions encountered at the time of construction. Accordingly, close inspection of the deep foundation bearing grades, by qualified geotechnical personnel, is recommended at the time of construction.

The allowable axial (compressive) load capacity of drilled piers bearing within the Shale bedrock can be computed based on a maximum net allowable end bearing pressure of 12 tons per square foot (tsf). A minimum pier diameter of 2.5 feet is recommended. Drilled piers should be spaced no closer than 3 pier diameters, center to center. Pile caps and or grade beams should be embedded a minimum of 4 feet for frost protection.

It is estimated that drilled pier foundations bearing in sound bedrock will undergo insignificant settlement when designed and constructed in accordance with our recommendations.

Micro-Pile Foundation Design:

Drilled micro-piles are another foundation option, which could be considered to support the proposed bridge foundations. Micro-piles (steel cased/concrete piles) would be typically 6 to 10 inches in diameter and socketted into the competent Shale bedrock to develop their axial capacity. Similar to the drilled piers, it is anticipated that the micro-piles would need to be initially advanced to a depth of about 28 feet below the existing road surface (i.e. about El. 320.0 feet), prior to socketting them into the competent Shale bedrock.

The micro-pile foundations are generally designed and installed by a Specialty Contractor qualified and experienced in such construction methods. Therefore, it is general practice for the Engineer to develop a performance specification for the micro-pile and then have the installation contractor provide a suitable pile design, considering the logistics of the installation and the subsurface conditions. The diameter of the micro-pile (i.e. steel casing size), depth of effective bedrock embedment, steel reinforcing, and cement grout strength can be varied by the Specialty Contractor based on the structural design requirements, as well as considering the sizes/costs of casing pipe available on the market. The micro-piles should be spaced no closer than 3 feet, center to center.

An allowable side shear resistance (bond strength) of 25 pounds per square inch (psi), developed between the concrete micro-pile and the Shale bedrock socket can be used for design. A concrete/grout with a minimum compressive strength of 4,000 psi should be used. Drilled micro-piles should be embedded at least five (5) feet into the competent Shale bedrock. The effective compression bond length can be the entire length of the rock socket into the competent bedrock.

Based on the above criteria, the following table summarizes the allowable axial compressive capacity for a 7-inch and 10-inch diameter micro pile, with 5 and 10 feet of effective bond length in the competent bedrock.

Estimated Allowable Axial Compressive Capacity		
Micro-Pile Diameter (inch)	Embedment Length into Competent Shale (feet)	Allowable Axial Compressive Capacity per Pile (tons)
7	5	16
7	10	33
10	5	24
10	10	47

Accordingly, other capacities can be developed by varying the diameter and /or embedment length within the effective bond zone.

It is estimated that the drilled micro-pile foundations will undergo insignificant total settlement when designed and constructed in accordance with our recommendations.

Abutment / Wing Wall Design:

The design of abutment and wing walls should be based on lateral earth pressures caused by the load of backfill against the wall and the surcharge effects from permanent or temporary loads. Abutment and wing walls, which are designed for restrained or non-yielding conditions, should be designed using "at rest" lateral earth pressures. Wing walls, which are allowed to yield, can be designed on the basis of "active" lateral earth pressures.

The lateral earth pressures can be computed using the following soil parameters where the wall backfill is a suitable granular material and contains a proper foundation drain(s) as discussed below. Water must not be allowed to collect against the backside of the exposed wall section unless the wall is designed for the additional hydrostatic pressure.

Recommended Soil Parameters for Abutment and Wing Wall Design:

Coefficient of At-Rest Lateral Earth Pressure – 0.50

Coefficient of Active Lateral Earth Pressure – 0.33

Coefficient of Passive Lateral Earth Pressure – 3.00

Angle of Internal Friction – 30 Degrees

Total Unit Weight of Soil – 125 pcf

Submerged Unit Weight of Soil – 65 pcf

Surcharge Load Coefficient – 0.50

Abutment and Wing Wall Drainage:

The abutment and wing walls should be constructed with foundation drains to intercept any groundwater that may tend to collect against the walls. The drainage system must be properly designed, installed and maintained for long-term performance. The design should include such features as clean-outs to properly maintain the system. The abutment wall drain system should extend to the bottom of the exposed section of the wall.

The foundation drainage system should include a geotextile, selected considering drainage and filtration, installed around drainage stone surrounding a slotted under-drain pipe. The drainage stone should be sized in accordance with the pipe slotting or perforations. A crushed aggregate conforming to NYSDOT Standard Specifications Section 703-02, Size Designation No. 2 (1 inch washed gravel or stone) is generally acceptable. The foundation drainage stone and surrounding geotextile should extend above the drainpipe a minimum of 2 feet.

A pervious granular backfill or a suitable geosynthetic drainage composite (i.e. Miradrain 5000) should be placed against the abutment wall to intercept groundwater and allow drainage to the under-drain system. If a soil material is used it should have a minimum width of 2 feet. Concrete Sand, which meets the minimum requirements of NYSDOT Standard Specifications Section 703-07 (100 percent passing 3/8 inch sieve to maximum of 3 percent passing a No. 200 sieve), is generally acceptable.

It is recommended that the backfill placed behind the abutment walls beyond the drainage system be a Suitable Granular Fill or a Structural Fill, as described in Appendix B.

Site Preparation and Construction Recommendations

Construction Dewatering:

Existing stream flow and groundwater conditions will need to be addressed and controlled during excavation and construction of the new replacement bridge. The creek as well as possible groundwater seepage from the existing granular fill soils should be expected to fluctuate with seasonal and precipitation events.

The amount of groundwater that could be encountered will depend on the excavation location, depth, the permeability of the soils encountered and the actual creek and groundwater conditions at the time of construction. The sand, silty sand and silt soils are expected to yield more substantial quantities of groundwater and could undergo rapid excavation bottom and sidewall instability, if not properly dewatered in advanced of performing the excavation work.

Dewatering procedures and surface water control should be implemented prior to excavation and maintained below the proposed excavation bottom. It is anticipated that sump and pump methods of dewatering, along with drainage stone layers, underdrains, cofferdams and stream diversion will generally be sufficient to control surface water and groundwater conditions for the bridge structure construction. More substantial methods of dewatering, such as deep sumps, deep wells and/or vacuum well points, may be necessary where excavations must extend further below the groundwater and/or where more permeable fill and indigenous soils are encountered in the presence of groundwater.

It is recommended that the contractor excavate some test pits in advance of the excavation work, particularly where deeper excavations are required, to ascertain potential groundwater conditions and plan the dewatering that will be necessary. A construction dewatering plan should be developed to properly dewater the site based on the soil and groundwater conditions. Groundwater dewatering plans should include implementation of measures to control erosion, sedimentation and the migration of soil fines, and discharge requirements.

Excavation and Subgrade Preparation for Grade Beams and Drilled Pier/Micro-Pile Cap Construction:

Existing foundations and structures, which are present at the locations of the new bridge structure elements, should be removed in their entirety. Excavation to the proposed subgrades for grade beams/pile cap construction should be performed using a method, which minimizes disturbance to the bearing grades. If any soils containing organics or otherwise deleterious soil material are encountered, they should be removed in their entirety. In addition, any ridges or loose soil left by machine excavation should be manually trimmed and removed prior to constructing the grade beams and drilled pier caps.

The proposed subgrades should be observed and evaluated by a representative of Empire, prior to placement of engineered fill and/or grade beams / pile caps. Any placement and compaction of Structural Fill should be observed and tested by a representative of Empire.

All subgrades for grade beam/pile cap construction should be protected from precipitation and surface water. Water should not be allowed to accumulate on the soil subgrades and the subgrades should not be allowed to freeze, either prior to or after construction of foundations. If subgrades are not protected and degrade, they must be undercut/removed accordingly.

After completion of the foundation construction, the excavations should be backfilled as soon as possible and prior to construction of the superstructure. The backfill behind the abutment walls and wing walls, beyond the drainage system components, should consist of Structural Fill or Suitable Granular Fill, as recommended in Appendix B.

Drilled Pier Foundation Construction:

It should be anticipated that the drilled pier foundation construction, will encounter groundwater, as previously discussed. Accordingly, proper dewatering of the pier excavation should be required to lower the groundwater levels in order to observe the pier excavation and bearing surface, and to allow construction of the pier in-the-dry.

Alternatively, special procedures for placing the concrete below groundwater, could also be considered. However, there is less control during construction in-the-wet, and therefore, there may be greater risk associated with this construction procedure. If this method is used, the concrete must be placed in a manner that displaces the water from the hole, such as using a tremie. It is recommended that drilled piers constructed in-the-wet be constructed only by contractors qualified and experienced in such construction methods. In both cases installation of temporary casing during drilling may also be necessary to prevent the potential sloughing of wet or loose soils into the excavation.

Initially, the excavation should extend to the top of competent bedrock. A nominal 2 feet deep rock socket should then be drilled. All soil and loose bedrock should be removed. The contractor should also be prepared to encounter and handle potential large cobble and boulders in the fill soils.

The final bearing surface should be level or near level. Where drilled piers are constructed in-the-dry, the holes should be dry for observation by qualified and experienced geotechnical personnel. Plumbness of the caisson or pier should be maintained within 1% of the total length. Where piers are constructed in the wet, the concrete must be placed in a manner that displaces water or slurry from the hole. Casing removal during concrete placement should proceed in a manner that prevents or reduces to the extent possible, surrounding soil and water from protruding into the space that will be occupied by concrete. Qualified and experienced geotechnical personnel should monitor the drilled pier construction.

Micro-Pile Foundation Construction:

The micro-pile boring should initially extend to the top of competent bedrock and casing should be advanced to the competent bedrock surface with the boring. After encountering the competent bedrock, a down-hole hammer, core barrel or tri-cone bit should be used to drill into the bedrock, beyond the casing, to the design embedment depth. All loose soil and rock should be removed from the drill hole. The contractor should also be prepared to encounter and handle potential large cobble and boulders in the fill soils.

Plumbness of the micro-pile should be maintained within 1% of the total length. A qualified individual should observe all micro-pile installations and prepare a report summarizing the installation process. In addition, at least one of the micro-piles should be load tested by the contractor to twice the allowable or working load, to confirm that adequate capacity has been developed.

Excavation Slope Stability and Shoring:

Excavations must be adequately sloped back and/or properly supported (i.e. shored, braced, shielded etc.) in accordance with OSHA requirements as a minimum. Based on the test boring information, it would appear that the overall soil conditions encountered would be generally classified as Type C soil in accordance with OSHA criteria.

Based on the OSHA Type C soil criteria, unsupported excavations less than 20 feet would need to be sloped backed to at least a 1.5 H (min) to 1 V slope. It is noted, however, that slopes encountering groundwater conditions may be unstable using this criteria, and therefore may require flatter slopes in conjunction with proper dewatering in order to maintain stable and safe conditions. The contractor should confirm the OSHA soil classification and excavation requirements at the time of construction based on actual location and soil and groundwater conditions present. The contractor shall be solely responsible for all excavation safety.

Concluding Remarks

This report was prepared to assist in planning and design for the proposed replacement of the existing bridge on Peck Road (CR183) over Salmon Creek in the Town of Parma, Monroe County, New York. The report has been prepared for the exclusive use of Lu Engineers and other members of the design team, for specific application to this site and this project only.

The recommendations were prepared based on Empire Geo-Services, Inc.'s understanding of the proposed project, as described herein, and through the application of generally accepted soil and foundation engineering practices. No warranties, expressed or inferred, are made by the conclusions, opinions, recommendations or services provided.

Empire Geo-Services, Inc. should be informed of any changes to the planned construction so that it may be determined if any changes to the recommendations presented in this report are necessary. Important information regarding the use and interpretation of this report is presented in Appendix C.

If you have any questions or wish to discuss this information, please do not hesitate to contact our office at any time. Thank you for considering Empire Geo-Services, Inc. for this work.

Sincerely,

EMPIRE GEO-SERVICES, INC.

John Danzer by GY

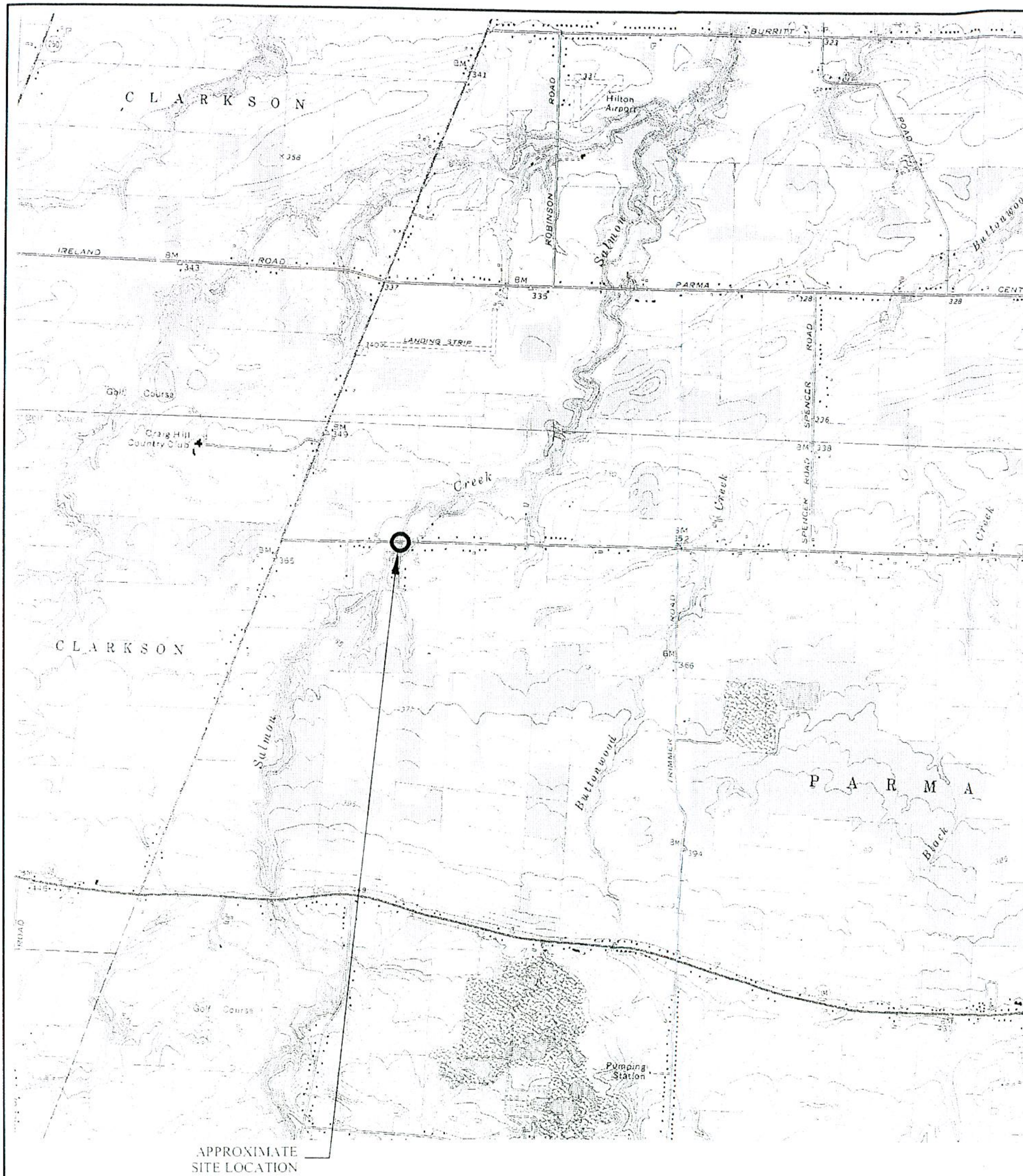
Wendel C. Armstrong, P.E.
Geotechnical Engineer

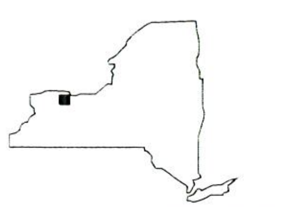
John J. Danzer, P.E.
Senior Geotechnical Engineer
and Project Reviewer

Attachments:

Figure 1 – Site Location Plan
Figure 2 – Subsurface Exploration Plan
Appendix A - Subsurface Exploration Logs
Appendix B – Fill Material and Earthwork Recommendations
Appendix C – Geotechnical Report Limitations

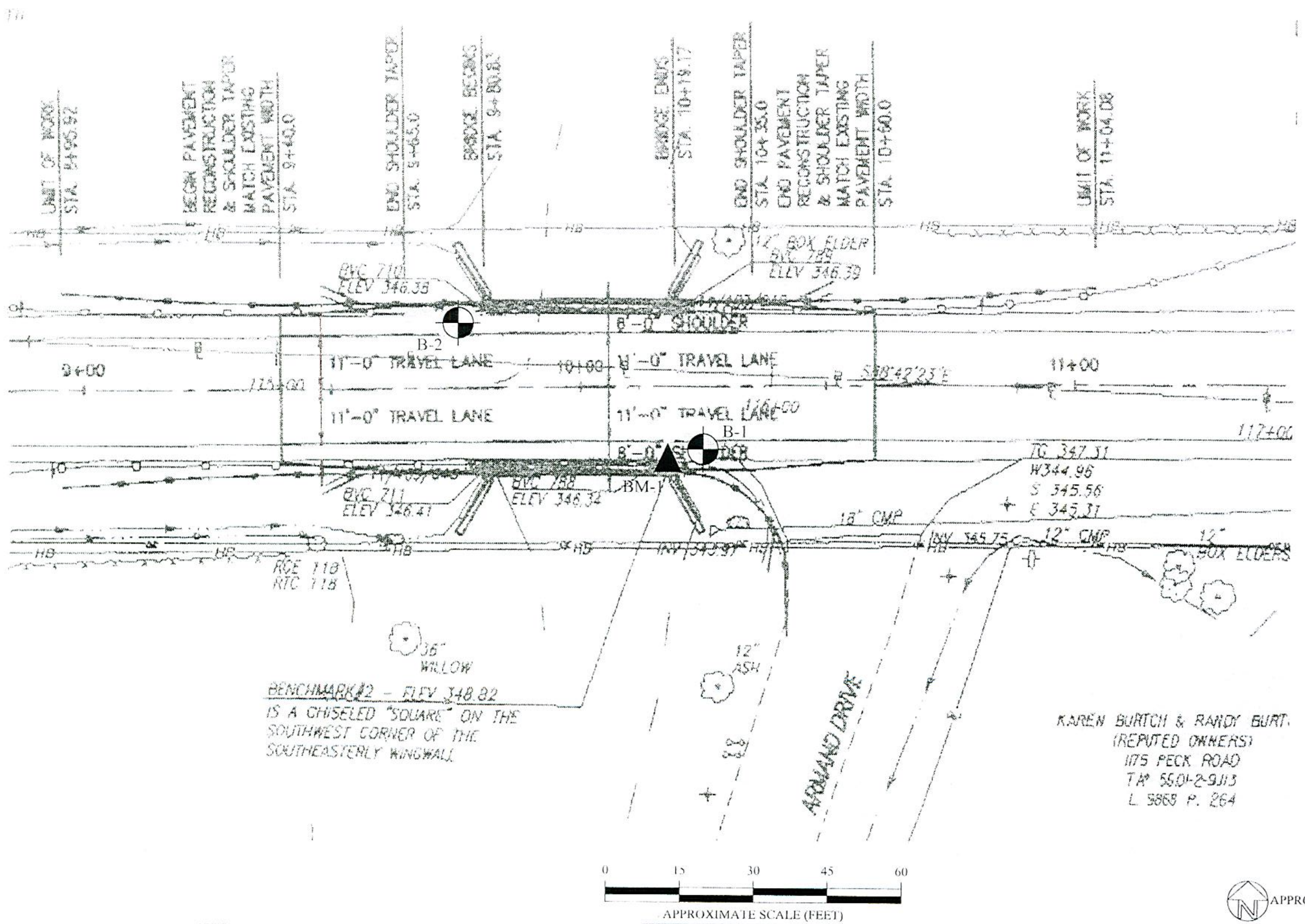
FIGURES





U.S.G.S. QUADRANGLE SPENCERPORT

EMPIRE GEO SERVICES INC a subsidiary of SJB Services, Inc.	PROPOSED BRIDGE REPLACMENT PROJECT PECK ROAD (CR 183) OVER SALMON CREEK B.I.N. 33172820, P.I.N. 4753.98 TOWNS OF PARMA MONROE COUNTY, NEW YORK		
SITE LOCATION PLAN	DR BY WCA	SCALE: NONE	PROJ NO RE-08-029
	CHKD BY: JJD	DATE: JANUARY 2009	FIGURE NO 1



LEGEND

B-1 — APPROXIMATE LOCATION AND DESIGNATION OF TEST BORINGS DRILLED ON OCTOBER 30, 2008 BY SJB

BM-1 — APPROXIMATE LOCATION OF BENCHMARKS USED BY SJB/EMPIRE. SEE REPORT FOR DESCRIPTION AND ELEVATION.

NOTES:

1. BASEPLAN SHOWING EXISTING SITE FEATURES PROVIDED BY LU ENGINEERS, ENTITLED "ROADWAY PLAN".

2. THE BORING LOCATIONS WERE APPROXIMATELY DETERMINED IN THE FIELD BY SJB PERSONNEL WITH SURVEY TAPE MEASUREMENTS FROM PHYSICAL MAPPING FEATURES. THE LOCATION OF THE BORINGS SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.

EMPIREGEO SERVICES INC
a subsidiary of SJB Services, Inc.

SUBSURFACE EXPLORATION PLAN

PROPOSED BRIDGE REPLACMENT PROJECT
PECK ROAD (CR 183) OVER SALMON CREEK
B.I.N. 33172820, P.I.N. 4753.98
TOWNS OF PARMA
MONROE COUNTY, NEW YORK

DR BY: WCA	APPROXIMATE SCALE: AS SHOWN	PROJ NO.: RE-08-029
CHKD BY: JJD	DATE: JANUARY 2009	FIGURE NO: 2

APPENDIX A

SUBSURFACE EXPLORATION LOGS

DATE			
STARTED ______	**SJB SERVICES, INC.**		PROJ. No. ______
FINISHED ______	**SUBSURFACE LOG**		HOLE No. ______
			SURF. ELEV. ______
SHEET ____ OF ____	SERVICES, INC.		G.W. DEPTH ______

PROJECT ______________________ LOCATION ______________________

DEPTH (ft)	SAMPLES	SAMPLE No.	BLOWS ON SAMPLER 0/6	6/12	12/18	18/24	N	BLOWS ON CASING C	SOIL OR ROCK CLASSIFICATION	NOTES
0		1	3	3	4	8	7	10	3" TOPSOIL	Groundwater at 10' upon completion, and 5' 24 hrs. after completion
								15	Brown SILT, some Sand, trace clay, ML (Moist-Loose)	
								50/.5		
5									Gray SHALE, medium hard, weathered, thin bedded, some fractures	Run#1, 2.5'-5.0' 95% Recovery 50% RQD

1 2 3 4 5 6 7 (numbered features explained on reverse) 8 9 10

TABLE I

Symbol	Sample Type
	Split Spoon Sample
	Shelby Tube Sample
	Geoprobe Macro-Core
	Auger or Test Pit Sample
	Rock Core

TABLE II

Identification of soil type is made on basis of an estimate of particle sizes, and in the case of fine grained soils also on basis of plasticity.

Soil Type	Soil Particle Size	
Boulder	>12"	Coarse Grained (Granular)
Cobble	3" - 12"	
Gravel - Coarse	3" - 3/4"	
- Fine	3/4" - #4	
Sand - Coarse	#4 - #10	
- Medium	#10 - #40	
- Fine	#40 - #200	
Silt - Non Plastic (Granular) Clay - Plastic (Cohesive)	<#200	Fine Grained

TABLE III

The following terms are used in classifying soils consisting of mixtures of two or more soil types. The estimate is based on weight of total sample.

Term	Percent of Total Sample
"and"	35 - 50
"some"	20 - 35
"little"	10 - 20
"trace"	less than 10

(When sampling gravelly soils with a standard split spoon, the true percentage of gravel is often not recovered due to the relatively small sampler diameter.)

TABLE IV

The relative compactness or consistency is described in accordance with the following terms:

Granular Soils Term	Blows per Foot, N	Cohesive Soils Term	Blows per Foot, N
Very Loose	0 - 4	Very Soft	0 - 2
Loose	4 - 10	Soft	2 - 4
Firm	10 - 30	Medium	4 - 8
Compact	30 - 50	Stiff	8 - 15
Very Compact	>50	Very Stiff	15 - 30
		Hard	>30

(Large particles in the soils will often significantly influence the blows per foot recorded during the penetration test)

TABLE V

Term	Meaning
Varved	Horizontal uniform layers or seams of soil(s).
Layer	Soil deposit more than 6" thick.
Seam	Soil deposit less than 6" thick.
Parting	Soil deposit less than 1/8" thick.
Laminated	Irregular, horizontal and angled seams and partings of soil(s).

TABLE VI

Rock Classification Term		Meaning
Hardness	- Soft	Scratched by fingernail
	- Medium Hard	Scratched easily by penknife
	- Hard	Scratched with difficulty by penknife
	- Very Hard	Cannot be scratched by penknife
Weathering	- Very Weathered	Judged from the relative amounts of disintegration, iron staining, core recovery, clay seams, etc.
	- Weathered	
	- Sound	

Rock Classification Term		Meaning	
Bedding	- Laminated	(<1")	Natural breaks in Rock Layers
	- Thin Bedded	(1" - 4")	
	- Bedded	(4" - 12")	
	- Thick Bedded	(12" - 36")	
	- Massive	(>36")	

(Fracturing refers to natural breaks in the rock oriented at some angle to the rock layers)

ADD 1-20

GENERAL INFORMATION & KEY TO SUBSURFACE LOGS

The Subsurface Logs attached to this report present the observations and mechanical data collected by the driller at the site, supplemented by classification of the material removed from the borings as determined through visual identification by technicians in the laboratory. It is cautioned that the materials removed from the borings represent only a fraction of the total volume of the deposits at the site and may not necessarily be representative of the subsurface conditions between adjacent borings or between the sampled intervals. The data presented on the Subsurface Logs together with the recovered samples provide a basis for evaluating the character of the subsurface conditions relative to the project. The evaluation must consider all the recorded details and their significance relative to each other. Often analyses of standard boring data indicate the need for additional testing or sampling procedures to more accurately evaluate the subsurface conditions. Any evaluation of the contents of this report and recovered samples must be performed by qualified professionals. The following information defines some of the procedures and terms used on the Subsurface Logs to describe the conditions encountered, consistent with the numbered identifiers shown on the Key opposite this page.

1. The figures in the Depth column define the scale of the Subsurface Log.

2. The Samples column shows, graphically, the depth range from which a sample was recovered. See Table I for descriptions of the symbols used to represent the various types of samples.

3. The Sample No. is used for identification on sample containers and/or Laboratory Test Reports.

4. Blows on Sampler - shows the results of the "Penetration Test", recording the number of blows required to drive a split spoon sampler into the soil. The number of blows required for each six inches is recorded. The first 6 inches of penetration is considered a seating drive. The number of blows required for the second and third 6 inches of penetration is termed the penetration resistance, N.

5. Blows on Casing - Shows the number of blows required to advance the casing a distance of 12 inches. The casing size, hammer weight, and length of drop are noted at the bottom of the Subsurface Log. If the casing is advanced by means other than driving, the method of advancement will be indicated in the Notes column or under the Method of Investigation at the bottom of the Subsurface Log. Alternatively, sample recovery may be shown in this column, or other data consistent with the column heading.

6. All recovered soil samples are reviewed in the laboratory by an engineering technician, geologist or geotechnical engineer, unless noted otherwise. Visual descriptions are made on the basis of a combination of the driller's field descriptions and noted observations together with the sample as received in the laboratory. The method of visual classification is based primarily on the Unified Soil Classification System (ASTM D 2487) with regard to the particle size and plasticity (See Table No. II), and the Unified Soil Classification System group symbols for the soil types are sometimes included with the soil classification. Additionally, the relative portion, by weight, of two or more soil types is described for granular soils in accordance with "Suggested Methods of Test for Identification of Soils" by D.M. Burmister, ASTM Special Technical Publication 479, June 1970. (See Table No. III). Description of the relative soil density or consistency is based upon the penetration records as defined in Table No. IV. The description of the soil moisture is based upon the relative wetness of the soil as recovered and is described as dry, moist, wet and saturated. Water introduced into the boring either naturally or during drilling may have affected the moisture condition of the recovered sample. Special terms are used as required to describe soil deposition in greater detail; several such terms are listed in Table V. When sampling gravelly soils with a standard two inch diameter split spoon, the true percentage of gravel is often not recovered due to the relatively small sampler diameter. The presence of boulders and large gravel is sometimes, but not necessarily, detected by an evaluation of the casing and sampler blows or through the "action" of the drill rig as reported by the driller.

7. Rock description is based on review of the recovered rock core and the driller's notes. Frequently used rock classification terms are included in Table VI.

8. The stratification lines represent the approximate boundary between soil types and the transition may be gradual. Solid stratification lines delineate apparent changes in soil type, based upon review of recovered soil samples and the driller's notes. Dashed lines convey a lesser degree of certainty with respect to either a change in soil type or where such change may occur.

9. Miscellaneous observations and procedures noted by the driller are shown in this column, including water level observations. It is important to realize the reliability of the water level observations depends upon the soil type (water does not readily stabilize in a hole through fine grained soils), and that any drill water used to advance the boring may have influenced the observations. The ground water level will fluctuate seasonally, typically. One or more perched or trapped water levels may exist in the ground seasonally. All the available readings should be evaluated. If definite conclusions cannot be made, it is often prudent to examine the conditions more thoroughly through test pit excavations or groundwater observation wells.

10. The length of core run is defined as the length of penetration of the core barrel. Core recovery is the length of core recovered divided by the core run. The RQD (Rock Quality Designation) is the total length of pieces of NX core exceeding 4 inches divided by the core run. The size core barrel used is also noted in the Method of Investigation at the bottom of the Subsurface Log.

DATE	
START	10/30/2008
FINISH	10/30/2008
SHEET	1 OF 1

SJB SERVICES, INC.
SUBSURFACE LOG

SJB SERVICES, INC.

HOLE NO.	B-1
SURF. ELEV	348.4
G.W. DEPTH	See Notes

PROJECT: Proposed Bridge Replacement Project
LOCATION: Peck Road (CR 183) over Salmon Creek
Town of Parma, Monroe County, New York
PROJ. NO.: RE-08-029

DEPTH FT.	SMPL NO.	BLOWS ON SAMPLER 0/6	6/12	12/18	N	SOIL OR ROCK CLASSIFICATION	NOTES
	1	4	3			Brown-Gray fine SAND and Silt,	
		5	4		8	numerous crushed stone fragments, tr. gravel (moist, FILL)	
	2	4	4			Brown f-c SAND, some f-c Gravel, tr. silt (moist, FILL)	
		4	3		8		
5	3	3	2				
		2	2		4		
	4	2	2			Brown fine SAND, some Clayey Silt, little f-c Gravel,	
		3	7		5	occasional cobble fragments (moist, FILL)	
	5	2	2				
10		2	4		4	Contains "and" Clayey Silt	
15						Red-Brown Highly Weathered SHALE Rock and Silt (moist)	REF = Sample Spoon Refusal
	6	45	50/0.2		REF		
20							
	7	50/0.2			REF		
25							
	8	50/0.4			REF		Auger refusal encountered at 28.2' NQ '2' SIZE CORE
30						Red-Brown SHALE Rock, slightly weathered to sound, soft to medium hard, bedded	RUN = 28.2' - 33.2' REC = 99 % RQD = 86 %
35						Boring Complete at 33.2'	No freestanding water encountered during overburden sampling
40							

N = NO. BLOWS TO DRIVE 2-INCH SPOON 12-INCHES WITH A 140 LB. PIN WT. FALLING 30-INCHES PER BLOW

CLASSIFIED BY: Geologist

DRILLER: K. Fuller

DRILL RIG TYPE: CME-550X

METHOD OF INVESTIGATION: ASTM D-1586 USING HOLLOW STEM AUGERS

DATE	
START	10/30/2008
FINISH	10/30/2008
SHEET	1 OF 1

SJB SERVICES, INC.
SUBSURFACE LOG

HOLE NO.	B-2
SURF. ELEV	348.3
G.W. DEPTH	See Notes

PROJECT: Proposed Bridge Replacement Project
PROJ. NO.: RE-08-029
LOCATION: Peck Road (CR 183) over Salmon Creek, Town of Parma, Monroe County, New York

DEPTH FT.	SMPL NO.	BLOWS ON SAMPLER 0/6	6/12	12/18	N	SOIL OR ROCK CLASSIFICATION	NOTES
	1	4	8			Gray Crushed STONE Fragments and f-c Sand, little silt (moist, FILL)	
		13	12		21		
	2	7	7			Brown f-c SAND, little-some Silt, little f-c Gravel (moist, FILL)	
		3	3		10		
5	3	4	12			Contains tr. clay	
		5	4		17		
	4	4	4			Brown-Gray Clayey SILT, some fine Sand, tr. gravel, tr. organics (moist, FILL)	
		3	3		7		
	5	3	4				
10		3	5		7		
15	6	50/0.4			REF	Red-Brown Highly Weathered SHALE Rock, tr. silt (moist)	REF = Sample Spoon Refusal
20	7	50/0.4			REF		
25	8	50/0.4			REF		NQ '2' SIZE CORE
	9	50/0.3			REF		
30						Red-Brown SHALE Rock, slightly weathered to sound, soft, thinly bedded to bedded	RUN = 28.7' - 33.7' REC = 98 % RQD = 77 %
35						Boring Complete at 33.7'	Freestanding water first encountered at 8.0' during overburden sampling
40							

N = NO. BLOWS TO DRIVE 2-INCH SPOON 12-INCHES WITH A 140 LB. PIN WT. FALLING 30-INCHES PER BLOW

CLASSIFIED BY: Geologist

DRILLER: K. Fuller

DRILL RIG TYPE: CME-550X

METHOD OF INVESTIGATION: ASTM D-1586 USING HOLLOW STEM AUGERS

APPENDIX B

FILL MATERIAL AND EARTHWORK RECOMMENDATIONS

APPENDIX B

FILL MATERIAL AND EARTHWORK RECOMMENDATIONS

I. Material Recommendations

A. Structural Fill

Structural Fill should consist of a crusher run stone, free of clay, organics and friable or deleterious particles. As a minimum, the crusher stone should meet the requirements of New York State Department of Transportation, Standard Specifications, Item 304.12 M – Type 2 Subbase, with the following gradation requirements.

Sieve Size Distribution	Percent Finer by Weight
2 inch	100
¼ inch	25-60
No. 40	5-40
No. 200	0-10

B. Subbase Stone

The subbase stone course placed as the aggregate course beneath slab on grade and pavement construction should conform to the same material requirements as Structural Fill as stated above.

C. Suitable Granular Fill

Suitable soil material, classified as GW, GP, GM, SW, SP and SM soils using the Unified Soil Classification System (ASTM D-2487) and having no more than 85-percent by weight material passing the No. 4 sieve, no more than 20- percent by weight material passing the No. 200 sieve and which is generally free of particles greater than 6 inches, will be acceptable as Suitable Granular Fill. It should also be free of topsoil, asphalt, concrete rubble, wood, debris, clay and other deleterious materials. Suitable Granular Fill can be used as foundation backfill and as subgrade fill to raise site grades beneath slab-on-grade and pavement construction.

Material meeting the requirements of New York State Department of Transportation, Standard Specifications, Item 203.07M – Select Granular Fill, or Item 203.20 Select Granular Subgrade is acceptable for use as Suitable Granular Fill.

II. <u>Placement and Compaction Requirements</u>

Structural Fill placed beneath foundations should be compacted to dense stable matrix, where its total thickness over the indigenous soil subgrades is 1 foot or, and to a minimum of 95 percent of the maximum dry density as measured by the modified Proctor test (ASTM D1557), where its total thickness will exceed 1 foot. All controlled fill placed beneath pavement construction, beneath utilities and behind abutment walls and wing walls should be compacted to a minimum of 95 percent of the maximum dry density as measured by the modified Proctor test (ASTM D1557). Fill placed in non-loaded grass areas can be compacted to a minimum of 90 percent of the maximum dry density (ASTM D1557).

Placement of fill should not exceed a maximum loose lift thickness of 6 to 9 inches with the exception of subbase courses beneath pavement construction, which can be placed in one lift, provided it does not exceed 15 inches. The loose lift thickness should be reduced in conjunction with the compaction equipment used so that the required density is attained.

Fill should have a moisture content within two percent of the optimum moisture content prior to compaction. Subgrades should be properly drained and protected from moisture and frost. Placement of fill on frozen subgrades is not acceptable. It is recommended that all fill placement and compaction be monitored and tested by a representative of Empire Geo-Services, Inc.

III. <u>Quality Assurance Testing</u>

The following minimum laboratory and field quality assurance testing frequencies are recommended to confirm fill material quality and post placement and compaction conditions. These minimum frequencies are based on generally uniform material properties and placement conditions. Should material properties vary or conditions at the time of placement vary (i.e. moisture content, placement and compaction, procedures or equipment, etc.) Then additional testing is recommended. Additional testing, which may be necessary, should be determined by qualified geotechnical personnel, based on evaluation of the actual fill material and construction conditions.

A. <u>Laboratory Testing of Material Properties</u>

- Moisture content (ASTM D-2216) - 1 test per 4000 cubic yards or no less than 2 tests per each material type.
- Grain Size Analysis (ASTM D-422) - 1 test per 4000 cubic yards or no less than 2 tests per each material type.

- Liquid and Plastic Limits (ASTM D-4318) 1 test per 4000 cubic yards or no less than 2 tests per each material type. Liquid and Plastic Limit testing is necessary only if appropriate, based on material composition (i.e. clayey or silty soils).
- Modified Proctor Moisture Density Relationship (ASTM D-1557) 1 test per 4000 cubic yards or no less than 1 test per each material type. A maximum/minimum density relationship (ASTM D-4253 and ASTM D-4254) may be an appropriate substitute for ASTM D-1557 depending on material gradation.

B. Field In-Place Moisture/Density Testing (ASTM D-3017 and ASTM D-2922)

- Backfilling along trenches and foundation walls - 1 test per 50 lineal feet per lift.
- Backfilling isolated excavations (i.e. column foundations, manholes, etc.) 1 test per lift.
- Filling in open areas for slab-on-grade construction - 1 test per 2500 square feet per lift.

GEOTECHNICAL REPORT LIMITATIONS

Empire Geo-Services, Inc. (Empire) has endeavored to meet the generally accepted standard of care for the services completed, and in doing so is obliged to advise the geotechnical report user of our report limitations. Empire believes that providing information about the report preparation and limitations is essential to help the user reduce geotechnical-related delays, cost over-runs, and other problems that can develop during the design and construction process. Empire would be pleased to answer any questions regarding the following limitations and use of our report to assist the user in assessing risks and planning for site development and construction.

PROJECT SPECIFIC FACTORS: The conclusions and recommendations provided in our geotechnical report were prepared based on project specific factors described in the report, such as size, loading, and intended use of structures; general configuration of structures, roadways, and parking lots; existing and proposed site grading; and any other pertinent project information. Changes to the project details may alter the factors considered in development of the report conclusions and recommendations. *Accordingly, Empire cannot accept responsibility for problems which may develop if we are not consulted regarding any changes to the project specific factors that were assumed during the report preparation.*

SUBSURFACE CONDITIONS: The site exploration investigated subsurface conditions only at discrete test locations. Empire has used judgement to infer subsurface conditions between the discrete test locations, and on this basis the conclusions and recommendations in our geotechnical report were developed. It should be understood that the overall subsurface conditions inferred by Empire may vary from those revealed during construction, and these variations may impact on the assumptions made in developing the report conclusions and recommendations. *For this reason, Empire should be retained during construction to confirm that conditions are as expected, and to refine our conclusions and recommendations in the event that conditions are encountered that were not disclosed during the site exploration program.*

USE OF GEOTECHNICAL REPORT: Unless indicated otherwise, our geotechnical report has been prepared for the use of our client for specific application to the site and project conditions described in the report. *Without consulting with Empire, our geotechnical report should not be applied by any party to other sites or for any uses other than those originally intended.*

CHANGES IN SITE CONDITIONS: Surface and subsurface conditions are subject to change at a project site subsequent to preparation of the geotechnical report. Changes may include, but are not limited to, floods, earthquakes, groundwater fluctuations, and construction activities at the site and/or adjoining properties. *Empire should be informed of any such changes to determine if additional investigative and/or evaluation work is warranted.*

MISINTERPRETATION OF REPORT: The conclusions and recommendations contained in our geotechnical report are subject to misinterpretation. *To limit this possibility, Empire should review project plans and specifications relative to geotechnical issues to confirm that the recommendations contained in our report have been properly interpreted and applied.*

Subsurface exploration logs and other report data are also subject to misinterpretation by others if they are separated from the geotechnical report. This often occurs when copies of logs are given to contractors during the bid preparation process. *To minimize the potential for misinterpretation, the subsurface logs should not be separated from our geotechnical report and the use of excerpted or incomplete portions of the report should be avoided.*

OTHER LIMITATIONS: Geotechnical engineering is less exact than other design disciplines, as it is based partly on judgement and opinion. For this reason, our geotechnical report may include clauses that identify the limits of Empire's responsibility, or that may describe other limitations specific to a project. These clauses are intended to help all parties recognize their responsibilities and to assist them in assessing risks and decision making. Empire would be pleased to discuss these clauses and to answer any questions that may arise.