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Geotechnical Report

OWENSVILLE SCENIC REGIONAL LIBRARY
OWENSVILLE, MISSOURI

March 2015

SCENIC REGIONAL LIBRARY
Owner

WASHINGTON ENGINEERING & ARCHTECTURE, P.S.
Civil Engineer/Architect

SCI No. 2015-5027.10

STATE OF MISSOURI
SHAWNNA L. ERTER
NUMBER
PE-2005000979
3/2/15
PROFESSIONAL ENGINEER

SCI ENGINEERING, INC.

CONSULTANTS IN DEVELOPMENT, DESIGN AND CONSTRUCTION
GEOTECHNICAL
ENVIRONMENTAL
NATURAL RESOURCES
CULTURAL RESOURCES
CONSTRUCTION SERVICES

March 2, 2015

Mr. Steven W. Campbell
Scenic Regional Library
304 Hawthorne Drive
Union, Missouri 63084

RE: Geotechnical Report
Owensville Scenic Regional Library
Owensville, Missouri
SCI No. 2015-5027.10

Dear Mr. Campbell:

Attached is our *Geotechnical Report*, dated March 2015. It should be read in its entirety, and our recommendations applied to the design and construction of the project. Selected excerpts from the report are highlighted below:

- Existing fill was encountered to depths of 1 to 3 feet (El. 944.8 to El. 946.1) in four of the seven borings across the site. In order to eliminate all risk associated with existing fill, it would need to be completely removed and replaced or recompacted. Additional recommendations associated with differing amounts of risk are provided in this report.

- Expansive fat clay soils were encountered across the site. Where the bearing and/or subgrade soils consist of expansive clay soils, we recommend that they be remediated to a minimum depth of 3 feet beneath the floor slab; and to a depth of 2 feet below shallow foundations, as further discussed in this report.

- Shallow spread footing foundations may be designed for maximum net allowable soil bearing pressures of 2,000 and 2,400 pounds per square foot (psf) for continuous strip footings and isolated column footings respectively.

- Based on the soils encountered and the anticipated depth to rock, Site Class C should be used for foundation design, with seismic design parameters for the site as follows: $F_a = 1.20$, $F_v = 1.68$, $S_{DS} = 0.27$ and $S_{D1} = 0.14$. The Seismic Design Category (SDC) for the site is C.

- Auger refusal was encountered on apparent bedrock at depths of 13.5 and 12.5 feet (El. 933.0 and El. 932.6) in Borings B-3 and B-7, respectively. We do not anticipate that bedrock will be encountered during the footing excavations, but may be encountered in deep utility excavations.

We appreciate the opportunity to be of service, and look forward to working with you during the construction phase of the project. SCI should participate in a meeting prior to clearing/grading of the site. Such meetings are valuable in reviewing and clarifying project requirements and responsibilities.

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If you have any questions or comments, please call.

Respectfully,

SCI ENGINEERING, INC.

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Enclosure
Geotechnical Report

C: Mr. Tim Sturholdt, AIA; Washington Engineering & Architecture, P.S.

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OWENSVILLE SCENIC REGIONAL LIBRARY
OWENSVILLE, MISSOURI

1.0 INTRODUCTION

At the request of Mr. Timothy Sturholdt of Washington Engineering & Architecture, P.S. (WEA), SCI Engineering conducted a geotechnical exploration for the proposed library. The purpose of our exploration was to characterize and evaluate the subsurface conditions, provide recommendations for foundations, and address other geotechnical aspects. Our services were provided in general accordance with our proposal dated January 27, 2015 and authorized by Mr. Steven Campbell of Scenic Regional Library on February 3, 2015.

2.0 SITE AND PROJECT DESCRIPTION

We understand that a new structure is currently being planned for a site located at the southwest corner of East Madison Avenue and South Olive Street (Highway 19) in Owensville, Missouri. The location of the site is shown on the *Vicinity and Topographic Map*, Figure 1 and the existing site features are shown on the *Aerial Photograph*, Figure 2. Currently, much of the site is grass covered, with some remnants of previous development on the southern and western ends of the site. The previous development on the site consisted of a slab-on-grade, single-story school that was demolished in 2014.

The proposed structure is a slab-on-grade, single-story structure with footprint of approximately 8,000 square feet, with associate parking on the northern and eastern ends of the site. The proposed construction is shown on the *Site Plan*, Figure 3. Detailed grading plans were not available at the time of this report however we anticipate that minimal cuts and fills will be required to achieve the proposed design grades.

Structural loads were not available at the time of this report; however, we anticipate that the building will be lightly loaded, with column loads of less than 200 kips and wall loads of less than 4 kips per lineal foot. If these loads will be exceeded, then SCI should be contacted to review our recommendations.

We have not reviewed, nor are we aware of, any previous studies on this specific site, by SCI or others, that would affect the preparation of this report. However, SCI recently completed a Phase One Environmental Site Assessment for the subject site. These results were presented under a separate cover.

3.0 SUBSURFACE CONDITIONS

A total of seven borings were drilled at the approximate locations shown on the *Aerial Photograph* and the *Site Plan*. Detailed information regarding the nature and thickness of the soils and rock encountered, and the results of the field sampling and laboratory testing are shown on the Boring Logs in Appendix A. The boring locations were selected and staked in the field by WEA and the surface elevations at the boring locations were later provided to SCI.

3.1 Soil Profile

Existing fill consisting of lean clay (CL in accordance with the Unified Soil Classification System and ASTM D 2488-06), and fat clay (CH) containing varying amounts of crushed rock and sand were encountered in four of the seven borings to depths of approximately 1 to 3 feet (El. 944.8 to El. 946.1). It is believed that the fill was placed either during construction of the former school sometime in the 1950's, or as part of the recent demolition work. Based on the results of the field and laboratory testing, the existing fill appears to have been placed with some level of compactive effort. However, documentation in regards to the placement of the fill material was not available at the time of this report.

Beneath the fill soils and any surficial topsoil, the natural soils generally consisted of interbedded layers of lean clay (CL), fat clay (CH) with varying amounts of sand, chert gravel, and sandstone fragments, and shaley clay to boring termination depths of 12.5 to 15.0 feet (El. 930.7 to El. 933.0). A layer of clayey gravel (GC) was encountered in Boring B-3 at a depth of approximately 10.5 feet (El. 935.4) and extended to the auger refusal depth of 13.3 feet (El. 932.6). The natural soils ranged from medium stiff to hard in consistency, with Standard Penetration Test (SPT) N-Values of 6 to 77 blows per foot (bpf), averaging 21 bpf. In general, the SPT N-values increased with depth.

Moisture contents within the natural soils typically ranged from 10 to 38 percent, averaging 24 percent. In general, the moisture contents decreased with depth. To characterize the shrink and swell potential of the subgrade soils, Atterberg limits testing was performed on samples from Boring B-4 and B-7, which resulted in liquid limits of 46 and 49, with corresponding plasticity indices of 28 and 31, respectively. These results indicate that the soil is low to medium plastic in nature. Dry densities and unconfined compressive strengths obtained on the Shelby tube samples from B-4 and B-7, were measured at 103 and 82 pounds per cubic foot (pcf), and at 1.8 and 1.4 kips per square foot (ksf), respectively.

3.2 Bedrock Profile

Auger refusal was encountered on apparent bedrock at depths of 13.5 and 12.5 feet (El. 933.0 and El. 932.6) in Borings B-3 and B-7, respectively. Additionally, split-spoon sampler refusal occurred in Borings B-2, B-5, and B-6 at depths of 9.5 to 14.5 feet (El. 932.2 to El. 937.6). Auger or split-spoon sampler refusal is a designation applied to any material that cannot be further penetrated by the power auger or sampler without extraordinary effort, and is indicative of a very hard or very dense material, usually boulders or bedrock. Documented geology, including the *Missouri Interactive Maps* provided on the *Center for Applied Research and Environmental Systems (CARES)* website, bedrock at the site consists of the Pennsylvanian Undifferentiated, which is composed of shale, limestone sandstone and coal.

3.3 Groundwater

Groundwater was encountered in Boring B-1 at a depth of 1.5 feet. Based on the lack of groundwater in the remaining borings, and the existing fill was observed at this location, we believe the groundwater encountered is likely trapped or perched water within the existing fill soils. The groundwater level depends on seasonal and climatic variations, and may be present at different depths in the future. In addition, without extended periods of observation, accurate groundwater level measurements may not be possible, particularly in low permeability soils. We do not anticipate that groundwater will influence the construction of the foundations, but may be encountered in deeper utility excavations.

4.0 DESIGN RECOMMENDATIONS

4.1 Existing Fill

Existing fill was encountered in four of the seven borings to depths of approximately 1 to 3 feet (El. 944.8 to El. 946.1). Some of the fill soils also contain potentially expansive fat clay soils. It is believed that the fill was placed either during construction of the former school sometime in the 1950’s, or as part of the recent demolition work. Based on the results of limited field and laboratory testing on the existing fill, it appears that the existing fill was placed with some level of compactive effort. However, documentation in regards to the placement and compaction of the fill material is not available. As a result, there is some risk of settlement or other performance problems if foundations, floor slabs, and pavements are supported by the fill. In order to totally eliminate this risk, all of the existing fill would have to be excavated and either recompacted or replaced.

It is recommended where the fill will underlie the foundations, floor slabs, and pavements, that the fill be excavated and either recompacted, or replaced. The overexcavation should extend at least 5 feet beyond the outside edge of the footprints to facilitate uniform compaction of the replacement materials, and may require additional widening at the corners to allow equipment access for proper compaction. Given the size of the site, and the depths of the fill encountered at the boring locations, this could likely be completed during the general site grading activities.

However, the cost of entirely removing and replacing the fill beneath the floor slab and pavement areas may not justify the potential benefit gained; and some risk of settlement of the floor slab or pavements may be acceptable to the Owner. If the owner is willing to accept some risk of future settlements, the fill could be left in place beneath the floor slabs and pavements, with proper proofrolling and treatment as described later in this report. However, the fill beneath the footings will still require remediation. Remediation options for the fill include removal and replacement with structural fill, or recompaction. Some of the fill contains potentially expansive fat clay soils. These soils are not suitable to be used as fill because of their potential for shrinkage and swelling.

As an alternative to removal and replacement, the building foundations could be extended downwards through the fill to bear on natural soils. The design depth of the footings and the thickness of the fill will influence which alternative method of dealing with the fill is most desirable. Where fill extends only a couple feet below the base of a foundation, deepening the footing through the fill may be more expedient and cost effective. In areas where the existing fill extends more than a couple feet below the base of a foundation, deepening the footing becomes more difficult, potentially requiring significant amounts of additional excavation and shoring or bracing.

4.2 Expansive Clay Remediation

Expansive clay soils were encountered across the site. These soils are susceptible to excessive volume changes with variations in moisture contents. We anticipate that some remediation of the expansive soils will be required beneath the building foundations, floor slabs, and pavements. Where the bearing and/or subgrade soils consist of expansive clay soils, we recommend that they be removed to a minimum depth of 3 feet beneath the bottom of the floor slabs, and to a depth of 2 feet below the shallow building foundations. The overexcavation should extend at least 5 feet beyond the outside edge of the footprints to facilitate uniform compaction of the replacement materials, and may require additional widening at the corners to allow equipment access for proper compaction. The overexcavation should be backfilled with properly compacted low plastic soil or 1-inch minus crushed limestone. As an alternative, the overexcavation for shallow foundations may be backfilled with lean concrete, which will not require widening of the overexcavation.

As an alternative to overexcavation and replacement, the fat clay may be remediated by the addition of lime in combination with a recompaction operation. If lime stabilization is performed, we recommend thoroughly mixing in "Code L" (a locally available calcium oxide by-product also known as lime kiln dust) at a rate of 7 percent, or approximately 8 pounds of Code L per cubic foot of soil, to the depths and lateral limits described in the preceding paragraph. Water may need to be added during mixing to allow for proper hydration of the lime. Pulverizing and tilling equipment, such as “gators,” are preferred for mixing the lime into the soil. The treated soil should be placed in compacted lifts as discussed in the “Fill Materials and Compaction” section.

The method of treatment described above is based on generally accepted standards in the local engineering community; however, swell pressures and volume change potential greater than can be mitigated by this method may exist. Consequently, the owner should recognize that there is an inherent, but reduced risk that damage may occur, even after remedial treatment of the subgrade soil.

4.3 Shallow Foundations

Shallow spread footing foundations bearing on natural lean clay, remediated existing fill or remediated expansive clay soils are appropriate for support of the foundations. Based on the soils encountered during our exploration, shallow foundations can be sized for maximum net allowable bearing pressures of 2,000 and 2,400 pounds per square foot (psf) for continuous strip footings and isolated column footings, respectively.

Exterior footings and foundations in unheated areas of the buildings should be provided with at least 30 inches of soil cover for frost protection. Interior footings in heated areas can be located at nominal depths below the finished floor. For footings designed and constructed in accordance with our recommendations, total settlement should be less than 1 inch, and differential settlement between adjacent footings should be less than ¾ inch.

4.4 Seismic Considerations

Ground shaking at the foundation of structures and liquefaction of the soil under the foundation are the principle seismic hazards to be considered in design of earthquake-resistant structures. Liquefaction occurs when a rapid buildup in water pressure, caused by the ground motion, pushes sand particles apart, resulting in a loss of strength and later densification as the water pressure dissipates. This loss of strength can cause bearing capacity failure while the densification can cause excessive settlement. Potential earthquake damage can be mitigated by structural and/or geotechnical measures or procedures common to earthquake resistant design.

4.4.1 Design Earthquake

According to International Building Code (2009 edition) (IBC 2009), structures such as the one proposed for this project are required to be designed to a design earthquake with a 2 percent Probability of Exceedance (PE) over a 50-year exposure period (i.e. a 2,475-year design earthquake). The 2 percent PE in 50-year design earthquake has a Moment Magnitude (Mw) of 7.7 and a Peak Ground Acceleration (PGA) of 0.11g, as determined from data provided by the IBC 2009 and the United States Geological Survey (USGS) National Seismic Hazard Mapping Project.

4.4.2 International Building Code Site Classification

Based on procedures outlined in the IBC 2009 and our geotechnical explorations for the subject site, including the borings through predominantly medium stiff to very stiff near surface cohesive soil and the depth to bedrock, the site can be classified as Site Class C. Using the procedures outlined in Section 1613 of the IBC 2009, the calculated weighted average undrained shear strength (s_u) is in excess of the 2,000 psf required to be classified as Site Class C. Seismic design parameters for the site are as follows: F_A = 1.20, F_V = 1.68, S_{DS} = 0.27 and S_{D1} = 0.14. The Seismic Design Category (SDC) for the site is C.

4.4.3 Liquefaction Potential Analysis

The liquefaction potential analysis for the site was conducted using data from the field exploration and laboratory test results and the techniques outlined in the National Center for Earthquake Engineering (NCEER) Technical Report NCEER-97-0022. Based on our analyses, the soils at the project site have sufficient strength values to resist liquefaction and/or a fines content that make the threat of liquefaction minimal during the design earthquake. While the amount of the seismically induced settlement is dependent on the magnitude and distance from the seismic event, we estimate that the settlements from the design earthquake will be negligible and relatively uniform in nature so liquefaction mitigation techniques are not required.

4.5 Floor Slabs

We recommend that all concrete slabs be designed using a modulus of subgrade reaction (k) of 150 pounds per square inch per inch of deflection (pci) if bearing on newly placed, low plastic structural fill, natural low plastic lean clay soils, remediated existing fill or remediated expansive clay soils. If the existing fill is to remain in place, a reduced modulus of subgrade reaction (k) of 100 pci should be used. All slabs should be supported on a minimum 4-inch-thick layer of crushed stone. This will help to distribute concentrated loads and equalize moisture conditions beneath the slabs.

It is generally preferable to maintain structural separation between the floor slabs and the foundation walls and column pads using isolation joints. We also suggest that joints be placed in the floor and pool deck slabs on no more than 15-foot intervals in any direction. Such joints permit slight movements of the independent elements and help reduce random cracking that might otherwise be caused by restraint of shrinkage, slight rotations, heave, or settlement.

We recommend that 6-mil-thick polyethylene sheeting be placed immediately beneath the building floor slab and above the crushed rock or gravel, to reduce the transfer of capillary moisture to the interior slab. However, without careful attention to curing of the floor slabs, the polyethylene sheeting can cause excessive shrinkage cracking and "curling."

4.6 Corrosivity

Laboratory testing, including percent moisture, resistivity, and pH, was performed on a sample from B-4. The laboratory results are included below in Table 4.1. Based on the test results and criteria established in the “10-Point System” specified by American Water Works Association (AWWA) C-105, the site currently scores less than 10. With the 10-point system, scores exceeding 10 indicate increased risk of corrosion to subsurface steel and ductile iron piping.

Based on our knowledge from previous tests for projects on base, we consider Type I cement to be adequate for construction. We are not aware of any concrete damage in this area arising from exposure to soils with high sulfate contents. We advise that ACI-recommended reinforcement cover should also be considered.

Table 4.1 – Corrosivity Test Results

Analyses	Results
Percent Moisture (%)	23
Resistivity (Ohms-cm)	7,500
pH	6.8

4.7 Below-Grade Walls

Below-grade walls at this site may include minor retaining walls designed to accommodate surface grade changes around the proposed building. The maximum toe pressure for below-grade walls should not exceed the bearing pressure previously given for continuous strip footings. Retaining walls may be designed with an allowable coefficient of friction between the base of the concrete footing and the soil subgrade of 0.3.

Below-grade walls should also be designed to withstand lateral earth pressures caused by the weight of the backfill, including slopes behind the walls; and any surcharge, such as adjacent floor loads. We recommend the equivalent fluid unit weights tabulated in Table 4.2 below for lateral earth pressures, in pounds per cubic foot (pcf), be used in the design of below-grade walls. The indicated values assume that positive drainage is provided to prevent buildup of hydrostatic pressure. Expansive clay soils should not be used to backfill the wall excavations. Values for granular material should only be used if the granular backfill extends upwards and outwards the full height of the wall at a slope of 45 degrees or flatter from its base. In this case, exterior granular backfill should be capped with approximately 2 feet of cohesive soil to reduce the potential for surface water infiltration into the granular backfill. With clean granular backfill, filter fabric, such as Mirafi 140N, should be placed along the interface between the soil and granular backfill to reduce the potential for infiltration of the soil into the granular material.

Table 4.2 - Recommended Lateral Earth Pressures

Backfill Type	Equivalent Fluid Unit Weights	
	At-Rest Earth Pressures (pcf)	Active Earth Pressures (pcf)
Cohesive Soil	70	50
Granular Material (1-inch minus)	60	40
Free-Draining Granular Material (1-inch clean)	50	30

At-rest earth pressures should be used for restrained or fixed-headed walls that are restricted from rotation, such as loading dock or basement walls connected to floor joists or beams, or a wing wall attached to a basement wall. Active earth pressures should be used for free-headed walls where the base remains fixed and deflection at the top of the wall of approximately 1 inch for each 10 feet of wall height is allowed, such as a retaining wall.

The above values are applicable when the surface of the backfill behind the wall is horizontal. Upward sloped or loaded backfill will result in increased values. In addition to lateral earth pressures, below-grade walls should be designed to resist any surcharge loads, including shallow building foundations and traffic. These surface loads can be modeled as uniform lateral loads, equivalent to one-half of the surface load, acting at the halfway point on the wall.

A passive soil resistance modeled by an equivalent fluid unit weight of 250 pcf may be used for natural soil against the face of the exterior base or a key below the base of the wall. The upper 2 feet of soil backfilled against the exterior face of the walls and uncontrolled backfill soils should be ignored when calculating the lateral resistance. Lower passive pressure should be used if the ground surface slopes downward away from the face of the wall.

We recommend that all below-grade walls be provided with a drainage system. A minimum 4-inch diameter, perforated drainpipe should be used, and placed at foundation level. Granular drainage material, consisting of 1-inch clean crushed rock, classified as GP by ASTM D 2487, with less than 5 percent of the rock passing the No. 200 sieve, should be placed a minimum of 6 inches in all directions around the drainage pipe. Synthetic filter fabric, such as Mirafi 140N or equivalent, should encapsulate the drainpipe and granular drainage material. The pipe should be sloped to drain by gravity or through weepholes located on approximately 10-foot centers for above-grade retaining walls, or to a sump with a pump for below-grade walls where positive drainage by gravity cannot be achieved. Alternately, drainage can be provided directly through the weepholes without a drain pipe, provided that filter fabric is used or other measures are taken to prevent the granular backfill from migrating out through the weepholes. Any interior sumps must be isolated “watertight” from the interior subgrade to prevent the movement of moisture from the sump into the underlying soils.

4.8 Site Grading and Drainage

Surface contouring and site drainage should be provided to reduce surface water infiltration around the perimeter of the club house, beneath the floor slab and the pool deck slabs. All grades should be sloped away from the structures and pool deck. Roof and surface drainage should be collected and discharged such that water is not permitted to infiltrate the backfill of the building.

Large trees and shrubs should be planted away from exterior footings as they may cause drying and shrinkage of the foundation soils and, with the passage of time, potentially detrimental settlement of the building floor slabs and foundations. A minimum distance of 20 feet or the mature tree's dripline, whichever is greater, is suggested.

We recommend that all final slopes have a maximum inclination of 3 horizontal to 1 vertical (3H:1V), and that a crest of at least 10 feet in width or a distance equivalent to the total height of the slope, whichever is less, be provided around the structures before the surface slopes down and away. We do not anticipate that slopes steeper or taller than 15 feet in total height slopes will be required. However, if they are proposed, the slopes should be brought to our attention and individually addressed and evaluated by SCI on a case-by-case basis.

4.9 Underground Utilities

Underground utilities can provide a pathway for water to migrate below the floor, pool deck and pool slabs. Drain and utility pipes beneath the slabs should have tight joints to prevent leakage. If utility excavations are backfilled with free-draining granular materials, then cutoffs should be provided at the exterior walls to reduce the potential for water to migrate beneath the structures. Impermeable cutoffs may consist of a 3-foot-long "plug" of cohesive soil or bentonite soil mix, or a 1-foot-long plug of lean concrete. Soil may be used for the balance of the backfill.

With the exception of individual service lines to the building that intersect foundations perpendicularly, below-grade utilities should not be located within the stress influence zone of the building foundations. Accordingly, below-grade utilities should be located outside a zone extending 45 degrees downward and outward from the edge of the footings.

5.0 SITE DEVELOPMENT AND CONSTRUCTION CONSIDERATIONS

5.1 Site Preparation

Within the construction area, existing pavements, structures, and below-grade components to be abandoned must be properly demolished and the debris removed from the site. Existing foundations, slabs and utilities, as well as their associated backfill, should be removed from below and at least 10 feet beyond the proposed building footprints. As an exception, deep utilities may be grouted in place rather than being removed. However, the existing backfill associated with deep utilities should be removed and replaced or recompacted. Outside this area, existing foundation walls and footings deeper than 3 feet below the proposed subgrade may be left in place. Excavations resulting from the removal of existing site improvements should be backfilled with properly compacted fill.

Upon removal and backfilling of the existing site improvements, areas to be cut or to receive fill should be stripped of any surface vegetation or organic topsoil. The strippings should be removed and stockpiled for later placement in landscaped or common ground areas, as appropriate. After stripping, the site should be proofrolled by systematically passing over the subgrade to achieve complete coverage with proper compaction or loaded construction equipment, and observing the subgrade for pockets of excessively soft, wet, or disturbed soil, or otherwise unacceptable materials.

Soft areas or otherwise unacceptable materials, if encountered, should be removed and replaced with structural fill or stabilized prior to placing additional fill. If removal of soft soils is impractical due to their excessive depth, they should be stabilized or “bridged over” in a manner approved by SCI. “Bridging” of the soft soils can often be accomplished by working 2- to 4-inch clean crushed rock into the softer soils and then placing a geofabric, such as Mirafi 600X or equivalent, prior to placing additional fill.

5.2 Fill Materials and Compaction

Prior to fill placement and compaction, the upper 8 inches of the exposed subgrade should be scarified, moisture conditioned, and recompacted. Structural fill, including aggregate base course, should be placed in maximum 8-inch-thick loose lifts and mechanically compacted to at least 90 percent of its Modified Proctor maximum dry density (ASTM D 1557). We recommend that any fill placed in building areas have a liquid limit less than 45 and a plasticity index less than 25. If higher plasticity soils are placed within 2 feet of shallow building foundations or within 3 feet of the floor slab subgrades, then remediation will be required. Acceptable non-organic fill soils include materials designated CL, ML, CL-ML, GP, and GW by ASTM D 2487.

Prior to compaction, the soil may require moisture adjustment. During warm weather, moisture reduction can generally be accomplished by disking or otherwise aerating the soil. When air drying is not feasible, a moisture reducing chemical additive, such as hydrated lime, could be incorporated into the soil. During dry weather, some addition of moisture may be required to facilitate compaction. This should also be done in a controlled manner using a tank truck with a spray bar. The moistened soil should be thoroughly blended with a disk or pulverizer to produce a uniform moisture content. If construction is performed during the winter season, fill materials should be carefully observed to see that no frozen soil is placed as fill or remains in the base materials upon which fill is placed.

Backfill for foundation walls and retaining walls may consist of low plastic lean clay or 1-inch minus crushed limestone. We advise performing field density tests on at least every other lift to monitor compaction. As an alternate, we suggest using 1-inch clean crushed limestone to provide improved drainage and to reduce lateral pressures on the walls. Due to a slight risk of migration of soil fines into the clean rock, a synthetic filter fabric, such as Mirafi 140N or equivalent, should be placed between the soil face of the excavation and the crushed limestone. If clean rock is used, it may be placed in 2-foot-thick lifts and tamped or tracked to achieve adequate densification. Exterior clean rock backfill should be capped with cohesive soil to reduce the potential for surface water infiltration.

Backfill placed next to walls should be compacted with hand operated equipment and not large self-propelled or machine operated equipment, which could result in potential overcompaction and higher lateral pressures. Compaction should be reduced within approximately 1 foot of the walls, and the walls should be observed periodically for signs of movement. If movement is detected, it may be necessary to provide bracing and/or change backfill procedures.

In addition to the minimum density requirements listed above, the soil must be stable, i.e., not “pumping” or rutting excessively under construction traffic, prior to placing additional fill or constructing foundations, floor slabs, and pavements. Field density tests should be performed on each lift of fill to document that proper compaction is achieved.

5.3 Bedrock Considerations

Bedrock was encountered at depths of 12.5 and 13.3 feet (El. 933.0 and El. 932.6) in Borings B-7 and B-3, respectively. Although proposed grades are not currently available, we do not anticipate that rock excavation will be required for the building foundations. However, rock excavation may be required for deeper site utilities, such as storm and sanitary sewers. Although not characterized by rock coring, we anticipate that most, if not all, of the bedrock encountered below our auger refusal depths will require chipping or other rock removal methods such as blasting. If blasting is required at the site, it should be controlled to keep peak velocities at the existing structures and property lines to less than 2 inches per second, unless local ordinances require more stringent criteria as velocities greater than this could cause damage. A pre-blast survey of adjacent structures is recommended; and vibration monitoring during blasting operations is advisable, particularly until the amount of explosives to be used is determined. Blasts using small amounts of explosives or a number of delays should be considered to reduce blasting damage.

5.4 Shallow Excavations

SCI should observe all footing excavations, and floor slab and pavement subgrades for problem areas, such as soft zones, areas of unsuitable existing fill or areas of untreated expansive clay soils, prior to placing concrete. Excessive disturbance of siltier soils in footing excavations should be avoided and could complicate construction. The potential for such disturbance will increase during wetter times of the year. Excavations that have been excessively disturbed should be overdeepened to approved undisturbed soils. Overexcavation and replacement with structural fill should be performed where inadequate bearing materials are present in footing excavations.

The base of all excavations should be clean, free of loose soil or uncompacted fill, relatively dry and maintained near their optimum moisture content. Excavations should be protected from extreme temperatures, precipitation, and construction disturbances. To reduce the possibility of desiccation or saturation of the foundation soils, we recommend that the concrete be placed as soon as possible after excavations are made.

Groundwater is not anticipated to be encountered in the excavations. However, in most situations, small amounts of groundwater seepage into the excavations can be handled by means of gravity ditching and a sump pump. If greater flows are experienced, SCI should be retained to provide additional consultation.

5.5 Subgrade Considerations

Floor pavement subgrades may be subjected to construction traffic and exposure to weather for an extended period and significant problems may be incurred. It may be necessary to proofroll the subgrade, in both cut and fill areas, and recompact the subgrade immediately prior to placing base rock for the floor slabs, pool deck slabs, or pool slabs. In addition, subgrades covered with base rock may be very slow to dry if precipitation occurs after placing the base rock. Therefore, we recommend that proofrolling and placement of the base rock be done as close to the time of pouring the slabs or decks as is practical. Proofroll passes should be limited, particularly on silty subgrades, to reduce the potential for pumping of moisture from deeper within the soil profile.

Special measures may be required to facilitate construction during wet or cold weather, or where excessive areas of soft soils are identified. These measures may include, but are not limited to, the addition of lime to the subgrade soils for drying purposes, or the removal of soft spongy soils and their replacement with crushed limestone. Soft areas should be selectively undercut and backfilled with properly compacted cohesive soil. A geotextile, such as Mirafi 600X, or geogrid, such as Tensar TX140, or equivalent, may be used to help stabilize particularly soft areas. Where possible, the subgrade should be sloped to provide drainage.

5.6 Excavation Bracing Requirements

In the *Federal Register*, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, Part 1926, Subpart P." This document was issued to provide for the safety of workers entering excavations, including utility trenches, basements, footings, and others. All operations should be performed under the supervision of qualified site personnel in accordance with OSHA regulations.

5.7 Erosion Control and Land Disturbance Monitoring Program

Appropriate erosion and sediment control measures, such as proper contouring during site grading activities, the installation of siltation fences, and/or inlet protection, should be used during construction to keep eroded materials from being carried onto adjacent properties or waterbodies. Depending on the length of time the subgrade is exposed and the amount of siltation that occurs, it may be necessary to periodically remove materials collected by the sediment control systems. Timely sodding and/or seeding of sloped surfaces will help reduce this potential problem.

SCI recommends following the procedures detailed in the Stormwater Pollution Prevention Plan (SWPPP). Any site disturbing more than one acre of ground must obtain a Land Disturbance Permit from the Missouri Department of Natural Resources (MDNR). As part of the permit compliance procedures, weekly and rain-event site observations must be performed to document the changing site conditions and maintenance of control measures.

6.0 CONSTRUCTION MONITORING PROGRAM

The following list summarizes SCI's recommendations for a construction monitoring program. These services are recommended to provide quality assurance in assessing design assumptions and to document earth-related construction procedures for compliance with plans, specifications, and good engineering practice. SCI should be retained to:

- Participate in a formal preconstruction meeting with the Owner's Representative, Civil Engineer, and Contractor, prior to construction at the site.

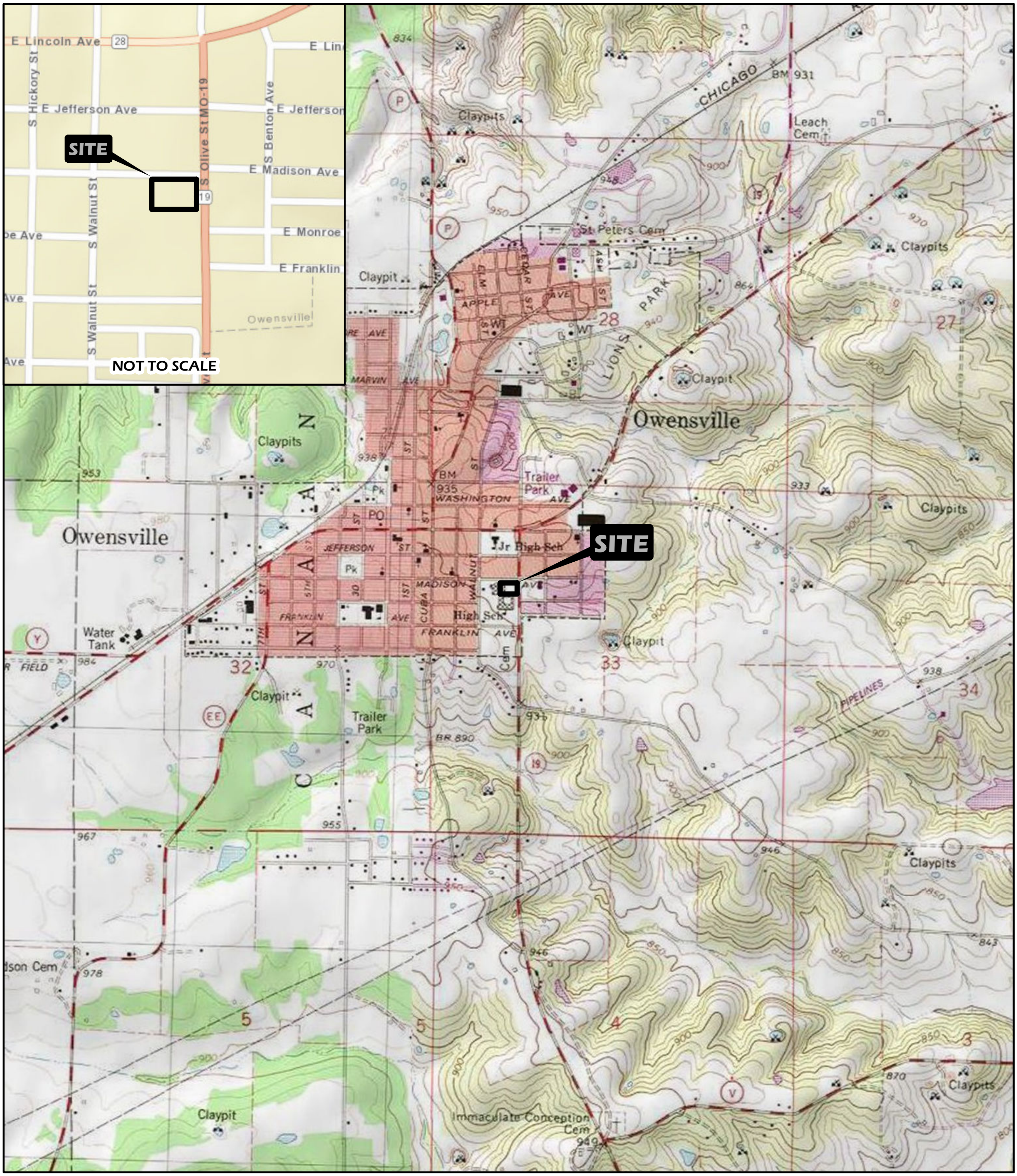
- Observe site preparation activities prior to construction, including removal and backfilling of existing site improvements, stripping and proofrolling.
- Conduct and document weekly and rain-event observations at the site, maintain and update on-site paperwork, and provide submittals required by the SWPPP and Land Disturbance Permit.
- Assess the suitability of potential fill materials, including both on-site and off-site sources.
- Monitor placement and compaction of structural fill and backfill.
- Observe foundation excavations, floor slab and pavement subgades to assess the impact of existing fill, expansive clay soils, and to recommend the extent of remedial measures.
- Observe footing excavations for adequacy of bearing materials.
- Observe the floor slab and pool deck subgrades prior to placing base rock.
- Observe backfilling of below-grade utility excavations.
- Provide quality assurance testing of structural concrete materials.

7.0 LIMITATIONS

The recommendations provided herein are for the exclusive use of our client. It is imperative that SCI be contacted by any third-party interests to evaluate the applicability of this report relative to use by anyone other than our client. Our recommendations are specific only to the project described, and are not meant to supersede more stringent requirements of local ordinances. They are based on subsurface information obtained at seven specific boring locations within the project area; our understanding of the project as presented in Section 2.0, “Site and Project Description”; and geotechnical engineering practice consistent with the standard of care. No other warranty is expressed or implied. SCI should be contacted if conditions encountered are not consistent with those described.

We should also be provided with a set of final development plans, once they are available, to review whether our recommendations have been understood and applied correctly, and to assess the need for additional exploration or analysis. Failure to provide these documents to SCI may nullify some or all of the recommendations provided herein. In addition, any changes in the planned project or changed site conditions may require revised or additional recommendations on our part.

The final part of our geotechnical service should consist of direct observation during construction, to observe that conditions actually encountered are consistent with those described in this report, and to assess the appropriateness of the analyses and recommendations contained herein. SCI cannot assume responsibility or liability for the adequacy of its recommendations without being retained to observe construction.



PROJECT NAME	GENERAL NOTES/LEGEND
OWENSVILLE SCENIC REGIONAL LIBRARY OWENSVILLE, MISSOURI	USGS TOPOGRAPHIC MAP OWENSVILLE WEST, MISSOURI QUADRANGLE DATED 1981 OWENSVILLE EAST, MISSOURI QUADRANGLE DATED 1980 10' CONTOURS
VICINITY AND TOPOGRAPHIC MAP	

DRAWN BY	RCV	DATE	JOB NUMBER
CHECKED BY	HHF	03/2015	2015-5027.10

SCALE 1" = 2000'

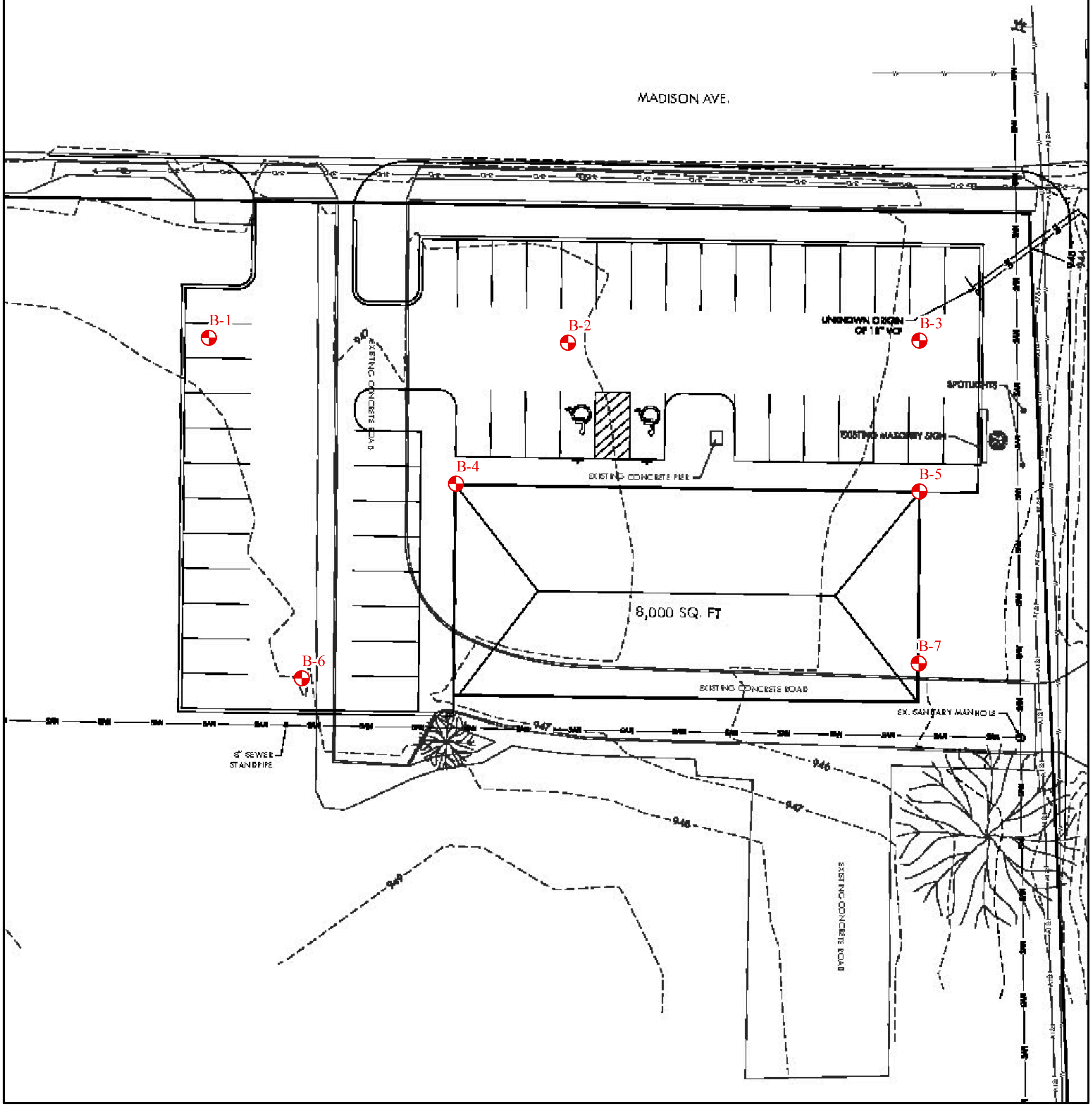
FIGURE 1

PROJECT NAME	OWENSVILLE SCENIC REGIONAL LIBRARY OWENSVILLE, MISSOURI
	AERIAL PHOTOGRAPH
DRAWN BY	RCV
CHECKED BY	HHF
DATE	03/2015
JOB NUMBER	2015-5027.10
SCALE	1" = 100'
FIGURE	2

GENERAL NOTES/LEGEND

INDICATES APPROXIMATE SOIL BORING LOCATIONS.

AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH, DATED 10/2014.



PROJECT NAME				GENERAL NOTES/LEGEND	
OWENSVILLE SCENIC REGIONAL LIBRARY OWENSVILLE, MISSOURI				INDICATES APPROXIMATE SOIL BORING LOCATIONS.	
SITE PLAN				PLAN DATED 1/20/2015 BY WASHINGTON ENGINEERING& ARCHITECTURE. DIMENSIONS AND LOCATIONS ARE APPROXIMATE; ACTUAL MAY VARY. DRAWING SHALL NOT BE USED OUTSIDE THE CONTEXT OF THE REPORT FOR WHICH IT WAS GENERATED.	
DRAWN BY	RCV	DATE	JOB NUMBER		SCALE 1" = 40'
CHECKED BY	HHF	03/2015	2015-5027.10		FIGURE 3

Appendix A

SCI ENGINEERING, INC.
47 St. Andrews Drive
Union, Missouri 63084
636-584-7991 Fax 636-584-7966
www.sciengineering.com

BORING LOG LEGEND AND NOMENCLATURE

Depth is in feet below ground surface. **Elevation** is in feet mean sea level, site datum, or as otherwise noted.

Sample Type

- **SS** Split-spoon sample, disturbed, obtained by driving a 2-inch-O.D. split-spoon sampler (ASTM D 1586).
- **NX** Diamond core bit, nominal 2-inch-diameter rock sample (ASTM D 2113).
- **ST** Thin-walled (Shelby) tube sample, relatively undisturbed, obtained by pushing a 3-inch-diameter, tube (ASTM D 1587).
- **CS** Continuous sample tube system, relatively undisturbed, obtained by split-barrel sampler in conjunction with auger advancement.
- **SV** Shear vane, field test to determine strength of cohesive soil by pushing or driving a 2-inch-diameter vane, and then shearing by torquing soil in existing and remolded states (ASTM D 2573).
- **BS** Bag sample, disturbed, obtained from cuttings.

Recovery is expressed as a ratio of the length recovered to the total length pushed, driven, cored.

Blows Numbers indicate blows per 6 inches of split-spoon sampler penetration when driven with a 140-pound hammer falling freely 30 inches. The number of total blows obtained for the second and third 6-inch increments is the N value (Standard Penetration Test or SPT) in blows per foot (ASTM D 1586). Practical refusal is considered to be 50 or more blows without achieving 6 inches of penetration, and is expressed as a ratio of 50 to actual penetration, e.g., 50/2 (50 blows for 2 inches).

For analysis, the N value is used when obtained by a cathead and rope system. When obtained by an automatic hammer, the N value may be increased by a factor of 1.3.

Vane Shear Strength is expressed as the peak strength (existing state) / the residual strength (remolded state).

Description indicates soil constituents and other classification characteristics (ASTM D 2488) and the Unified Soil Classification (ASTM D 2487). Secondary soil constituents (expressed as a percentage) are described as follows:

Trace	<5
Few	5-15
With	>15-30

Stratigraphic Breaks may be observed or interpreted, and are indicated by a dashed line. Transition between described materials may be gradual.

Laboratory Test Results

- Natural moisture content (ASTM D 2216) in percent.
- Dry density in pounds per cubic foot (pcf).
- Hand penetrometer value of apparently intact cohesive sample in kips per square foot (ksf).
- Unconfined compressive strength (ASTM D 2166) in kips per square foot (ksf).
- Liquid and Plastic Limits (ASTM D 4318) in percent.

RQD (Rock Quality Designation) is the ratio between the total length of core segments 4 inches or more in length and the total length of core drilled. RQD (expressed as a percentage) indicates insitu rock quality as follows:

Excellent	90 to 100
Good	75 to 90
Fair	50 to 75
Poor	25 to 50
Very Poor	0 to 25

BORING LOG

PROJECT Owensville Scenic Regional Library
LOCATION Owensville, Missouri
DRILLER Midwest Drilling, Inc. **HAMMER** Automatic
EQUIPMENT CME 750 w/CFA **ELEVATION** 947.8
BORING NUMBER B-1
SHEET 1 **of** 1
PROJECT NO. 2015-5027.10
DATE DRILLED 02/12/15

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (in/in)	BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					FILL: Brown, lean clay, with coarse sand and gravel									
	1	SS	8/18	3 5 12				14		--				
3					LEAN CLAY (CL): Gray and dark gray, trace fine sand									945
	2	SS	18/18	3 3 5				27		5.0				
6					FAT CLAY (CH): Brown and gray									942
	3	SS	18/18	4 5 6				36		1.5				
9	4	SS	12/18	6 10 12	Becomes brownish-orange with gray, with chert fragments, trace fine sand			26		4.5				939
12														936
	5	SS	11/18	28 37 40	Becomes reddish-orange, with fine to medium sand and sandstone fragments			13		4.5				
15					Boring terminated at 15.0 feet.									933
18														930

WATER LEVEL:
___ NONE OBSERVED WHILE DRILLING
1.5 ft WHILE DRILLING
___ ft ___ HRS AFTER DRILLING
___ ft ___ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Owensville Scenic Regional Library
BORING NUMBER B-2
LOCATION Owensville, Missouri
SHEET 1 **of** 1
DRILLER Midwest Drilling, Inc.
HAMMER Automatic
PROJECT NO. 2015-5027.10
EQUIPMENT CME 750 w/CFA
ELEVATION 947.1
DATE DRILLED 02/12/15

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE RECOVERY (in/in)	SAMPLE BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					FILL: Gray, fat clay, with crushed rock									
	1	SS	15/18	3 3 3	FAT CLAY (CH): Brown			31		4.0				945
3					LEAN CLAY (CL): Brown with red and gray, trace fine sand									
	2	SS	18/18	4 6 7				23		4.0				942
6	3	SS	18/18	4 5 7				33		5.0				
					FAT CLAY (CH): Gray, trace fine sand									939
9	4	SS	13/15	4 6 50/3"				24		>9.0				
					SHALEY CLAY (CH): Orange, with fine sand and weathered sandstone									936
12														
	5	SS	5/5	50/5"				10		>9.0				933
15					Boring terminated at 15.0 feet.									
														930
18														

WATER LEVEL:

X NONE OBSERVED WHILE DRILLING
ft WHILE DRILLING
ft ___ HRS AFTER DRILLING
ft ___ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Owensville Scenic Regional Library		**BORING NUMBER**	B-3
LOCATION Owensville, Missouri		**SHEET** 1	**of** 1
DRILLER Midwest Drilling, Inc.	**HAMMER** Automatic	**PROJECT NO.**	2015-5027.10
EQUIPMENT CME 750 w/CFA	**ELEVATION** 945.9	**DATE DRILLED**	02/12/15

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE RECOVERY (in/in)	SAMPLE BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					3" TOPSOIL									
					LEAN CLAY (CL): Brown and gray									945
	1	SS	16/18	3 4 7				21		2.5				
3														
	2	SS	18/18	3 4 7	Becomes brownish-gray and red, trace fine sand			22		7.5				942
6					FAT CLAY (CH): Gray with red nodules									
	3	SS	18/18	4 6 6				25		6.0				939
9	4	SS	18/18	5 6 9	Becomes gray, trace fine sand			23		5.0				936
					CLAYEY GRAVEL (GC): Brown and gray, fat clay, with sandstone fragments		1							
12														933
					Auger refusal at 13.5 feet.									
15														930
18														927

WATER LEVEL:

X NONE OBSERVED WHILE DRILLING
____ ft WHILE DRILLING
____ ft ____ HRS AFTER DRILLING
____ ft ____ DAYS AFTER DRILLING

REMARKS:

1) Driller's Observation

BORING LOG

PROJECT Owensville Scenic Regional Library | **BORING NUMBER** B-4
LOCATION Owensville, Missouri | **SHEET** 1 **of** 1
DRILLER Midwest Drilling, Inc. | **HAMMER** Automatic | **PROJECT NO.** 2015-5027.10
EQUIPMENT CME 750 w/CFA | **ELEVATION** 947.7 | **DATE DRILLED** 02/12/15

DEPTH (ft)	SAMPLE NUMBER	TYPE	RECOVERY (in/in)	BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					FILL: Dark gray and red, trace weathered sandstone fragments									
	1	SS	10/18	4 3 4				35		3.5				
3					LEAN CLAY (CL): Gray with brown, trace fine sand		1							945
	2	ST	24/24					23	103		1.8	46	28	
6					FAT CLAY (CH): Gray, with iron nodules, trace fine sand									942
	3	SS	18/18	3 4 5				37		2.0				
9	4	SS	16/18	5 7 8	Becomes brown, with weathered sandstone fragments			28		3.0				939
12														936
					SHALEY CLAY (CH): Light brown, with weathered sandstone fragments									
	5	SS	16/18	15 22 40				15		>9.0				
15					Boring terminated at 15.0 feet.									933
18														930

WATER LEVEL:

X NONE OBSERVED WHILE DRILLING
___ ft WHILE DRILLING
___ ft ___ HRS AFTER DRILLING
___ ft ___ DAYS AFTER DRILLING

REMARKS:

1) pH = 6.8, Resistivity = 7,500 Ohms-cm

BORING LOG

PROJECT Owensville Scenic Regional Library
LOCATION Owensville, Missouri
DRILLER Midwest Drilling, Inc.
EQUIPMENT CME 750 w/CFA
HAMMER Automatic
ELEVATION 945.7
BORING NUMBER B-5
SHEET 1 **of** 1
PROJECT NO. 2015-5027.10
DATE DRILLED 02/12/15

DEPTH (ft)	SAMPLE NUMBER	TYPE	RECOVERY (in/in)	BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					6" TOPSOIL									
					FAT CLAY (CH): Brown with gray and red, trace sand									945
	1	SS	14/18	3 3 4				37		2.5				
3					LEAN CLAY (CL): Brown with gray and red, trace fine sand									942
	2	SS	16/18	4 9 9				19		>9.0				
6					FAT CLAY (CH): Gray and red									
	3	SS	16/18	5 7 8				29		8.0				939
9	4	SS	17/18	4 8 23	Becomes brown, with fine to coarse sand and weathered sandstone			24		5.5				936
12					SANDSTONE: Light brown, slightly weathered									933
	5	SS	5/5	50/5"				9		>9.0				
15					Boring terminated at 15.0 feet.									930
18														927

WATER LEVEL:

X NONE OBSERVED WHILE DRILLING
___ ft WHILE DRILLING
___ ft ___ HRS AFTER DRILLING
___ ft ___ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Owensville Scenic Regional Library		**BORING NUMBER**	B-6
LOCATION Owensville, Missouri		**SHEET** 1	**of** 1
DRILLER Midwest Drilling, Inc.	**HAMMER** Automatic	**PROJECT NO.**	2015-5027.10
EQUIPMENT CME 750 w/CFA	**ELEVATION** 947.9	**DATE DRILLED**	02/12/15

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (in/in)	BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					FILL: Brown with red, fat clay, trace roots									
	1	SS	13/18	2 3 3				36		3.0				
3					FAT CLAY (CH): Brown and gray, trace fine sand									945
	2	SS	17/18	6 5 6				25		4.5				
6					LEAN CLAY (CL): Gray and orange									942
	3	SS	18/18	4 5 7				21		2.5				
					GRAVELLY FAT CLAY (CH): Red, with fine sand, weathered sandstone									
9	4	SS	14/18	12 22 37				15		>9.0				939
12					SHALE: Light orange, with weathered sandstone and chert fragments									936
	5	SS	12/15	16 34 50/3"				17		>9.0				
15					Boring terminated at 15.0 feet.									933
18														930

WATER LEVEL:

X NONE OBSERVED WHILE DRILLING
___ ft WHILE DRILLING
___ ft ___ HRS AFTER DRILLING
___ ft ___ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Owensville Scenic Regional Library
BORING NUMBER B-7
LOCATION Owensville, Missouri
SHEET 1 **of** 1
DRILLER Midwest Drilling, Inc. **HAMMER** Automatic
PROJECT NO. 2015-5027.10
EQUIPMENT CME 750 w/CFA **ELEVATION** 945.5
DATE DRILLED 02/12/15

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (in/in)	BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					4" TOPSOIL									
					LEAN CLAY (CL): Gray, trace roots									945
	1	ST	16/24					31		1.0				
					FAT CLAY (CH): Gray with red, trace fine sand			38	82		1.4			
3					LEAN CLAY (CL): Brown with gray and orange									942
	2	SS	18/18	4 5 7				21		3.5		49	31	
6														939
	3	SS	18/18	3 3 3				21		3.5				
					SANDY FAT CLAY (CH): Brown and gray, with fine to coarse sand, weathered sandstone, and chert fragments									
9	4	SS	15/18	13 20 21				11		6.5				936
12														933
					Auger refusal at 12.5 feet.									
15														930
18														927

WATER LEVEL:

X NONE OBSERVED WHILE DRILLING
___ ft WHILE DRILLING
___ ft ___ HRS AFTER DRILLING
___ ft ___ DAYS AFTER DRILLING

REMARKS:

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

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