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Geotechnical Report

TPHERIS ISRAEL CHEVRA KADISHA SYNAGOGUE IMPROVEMENTS
CHESTERFIELD, MISSOURI

July 2006

TPHERIS ISRAEL CHEVRA KADISHA SYNAGOGUE
Owner

THE CLAYTON ENGINEERING COMPANY
Civil Engineer/Surveyor

STATE OF MISSOURI
ALLEN G. MINKS
NUMBER
E-22438
REGISTERED PROFESSIONAL ENGINEER

SCI No. 2006-2182.10

7-27-06

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July 27, 2006

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

Mr. David Colvin, P.E., President
The Clayton Engineering Company
11920 Westline Industrial Drive
St. Louis, Missouri 63146

RE: Geotechnical Report
Tpheris Israel Chevra Kadisha Synagogue Improvements
Chesterfield, Missouri
SCI No. 2006-2182.10

Dear Mr. Colvin:

Enclosed is our *Geotechnical Report*, dated July 2006. It should be read in its entirety, and our recommendations applied to the design and construction of the project. Selected excerpts are highlighted below:

- Shallow foundations bearing on natural soil or newly placed structural fill are appropriate for support of the proposed addition and can be designed using maximum net allowable soil bearing pressures of 2,000 pounds per square foot (psf) for continuous wall footings and 2,400 psf for isolated, square, column footings.

- A factor of safety (FS) of 1.1 was calculated for the natural slope located east of the existing building, which is less than the standard of practice FS of 1.3 typical for parking lots. It is recommended that the parking locations be kept away from the slope if modification of the slope is not desired.

- The dam and the area surrounding the eroded locations below the dam should be cleared of vegetation, and the eroded areas backfilled with structural fill.

- Three feet of existing fill was encountered in the proposed building location. However, this fill generally appears suitable to support the floor slab, and should only need to be removed from below the footings.

We appreciate the opportunity to be of service, and look forward to working with you during the construction phase of the project. We should be included as participants in a formal preconstruction meeting with the Owner's Representative, Civil Engineer, and Contractor, prior to site clearing. Such meetings are valuable in reviewing and clarifying project requirements and responsibilities.

If you have any questions or comments, please call.

Respectfully,

SCI ENGINEERING, INC.

JW Copeland

JW Copeland, E.I.T.
Staff Engineer

Allen G. Minks

Allen G. Minks, P.E.
Chief Engineer

JWC/SLE/AGM/tdw

Enclosure: Geotechnical Report

Two additional copies submitted.

C: Scott Andrew, President of Tpheris Israel Chevra Kadisha Synagogue

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Geotechnical Report

TPHERIS ISRAEL CHEVRA KADISHA SYNAGOGUE IMPROVEMENTS CHESTERFIELD, MISSOURI

1.0 INTRODUCTION

At the request of Mr. David Colvin of The Clayton Engineering Company, Inc., SCI Engineering, Inc. (SCI) performed a geotechnical study for the above project. The purpose of our study was to characterize and evaluate the subsurface conditions, provide recommendations for foundations, and address other geotechnical aspects in regards to the proposed addition to the synagogue. The existing earth dam located northeast of the existing building was evaluated regarding the stability of the downstream slope as well as to provide recommendations for erosion protection. Also included in this report is an evaluation of the slope located east of the existing building and its suitability for support of a proposed parking lot. Our services were provided in general accordance with our proposal dated May 11, 2006, and authorized by Mr. Scott Andrew, President of Tpheris Israel Chevra Kadisha Synagogue, on June 13, 2006.

2.0 SITE AND PROJECT DESCRIPTION

SCI understands that an addition to the Tpheris Israel Chevra Kadisha Synagogue will be constructed at 14550 Ladue Road within Green Trails Village, a subdivision located in Chesterfield, Missouri. The site is currently improved with an existing synagogue and asphaltic concrete parking lot. The location of the site is shown on a Vicinity and Topographic Map, Figure 1.

Based on the plan prepared by The Clayton Engineering Company, provided July 5, 2006, the addition will be located south of the existing building. The proposed addition will be a single-story, slab-on-grade structure, with a footprint of approximately 5,500 square feet. The site was previously graded and is relatively flat; therefore, required cut and fills are assumed to be minimal. For the purpose of this report, we have anticipated that the building will be lightly loaded, with estimated maximum wall and column loads of 4 kips per lineal foot and 100 kips, respectively. If these loads will be exceeded, then SCI should be retained to review our recommendations. Parking is proposed along the church to all sides. The proposed construction is shown on the Site Plan, Figure 2.

Parking is proposed at the crest of the existing slope east of the church. According to the topographic information provided on the site plan, the slope is approximately 2 horizontal to 1 vertical (2H:1V) with 48 feet of relief. The topography varies from about elevation 528 along the draw to about elevation 576 near the crest of the slope.

The dam is located northeast of the existing structure. While the dam face remains intact, several areas of erosion have compromised the outfall structure of the dam. Slopes along the dam are as steep as 2H:1V with 24 feet of relief.

Correspondence between the City of Chesterfield and the synagogue, as well as correspondence between the Missouri Department of Natural Resources (MDNR) and the City of Chesterfield, have been reviewed. We are not aware of any other previous studies on this specific site, by SCI or others, that would affect the preparation of this report.

3.0 SUBSURFACE CONDITIONS

Eight borings, designated B-1, B-2, and B-5 thru B-10, were staked at the approximate locations shown on the Site Plan by SCI personnel measuring from existing site features. Borings B-3 and B-4 were not accessible to the drill rig and were hand augered instead, designated as HA-1 and HA-2. The borings were then drilled to a depth of 10 feet within the parking area; 20 feet within the building area; 25 feet along the slope; and 50 feet and auger refusal at 43.5 feet for the dam. Borings into the dam were backfilled with bentonite grout upon completion. 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. A Boring Log Legend and Nomenclature Sheet, to aid with interpretation of the boring logs, are also provided in Appendix A. The Boring Log Legend is also applicable for the hand auger logs.

3.1 Existing Fill

Existing fill was encountered in the dam to a depth of 23 feet in B-1 and 36 feet in B-2. In the building area, existing fill was observed in B-6 to 2.5 feet and B-7 to 3 feet; and for parking, B-9 had 10 feet. The fill typically consisted of low plastic silty clay, though a layer of high plastic clay was encountered in B-2 from 5 to 8 feet. Crushed rock and/or asphaltic concrete pavement was encountered to 12 inches in B-1, 20 inches in B-2, 9 inches in B-7 and B-8, and 3 inches in B-9.

3.2 Natural Soils

The natural soils consisted of low plastic silty clay (CL in accordance with the Unified Soil Classification System and ASTM D 2487), low plastic silt (ML), and high plastic clay (CH). In HA-2, silty sand (SM) was encountered at a depth of 8 feet. Atterberg limits tests performed on samples from B-1, B-2, and B-6 resulted in liquid limits of 29, 39, and 47, and plasticity indices of 6, 16, and 21 respectively, which would classify the first specimen as silty clay–clayey silt (CL-ML), and the others as low plastic silty clay (CL). However, SCI would consider the last sample medium plastic as the delineation between high and low plastic is a liquid limit of 50. Atterberg limits tests performed on the samples from HA-1 and HA-2 resulted in liquid limits of 36 and 41, with plasticity indices of 10 and 14, respectively, which would classify these soils as low plastic silty clay. Dry density and unconfined compressive strengths obtained on the Shelby tube samples from B-1, B-2, and B-5 were measured at 92, 93, and 91 pounds per cubic foot (pcf), and 1.0, 0.9, and 2.7 kips per square foot (ksf) respectively. A dry unit weight of 89 pcf was obtained from the Shelby tube in B-7, although the sample was too disturbed for meaningful laboratory strength testing. These soils were generally medium stiff in consistency.

Auger refusal was encountered in B-1 at a depth of 43.5 feet. Auger refusal is a designation applied to any material that cannot be further penetrated by the power auger without extraordinary effort, and is indicative of a very hard or very dense material, usually boulders or bedrock. Hand auger refusal, likely on gravel, was encountered in HA-2 at a depth of 9.25 feet.

3.3 Bedrock

Documented geology, including the Bedrock Geology Map of the St. Louis Quadrangle, Missouri and Illinois indicates that bedrock at the site consists of the Salem Formation. The Salem Formation consists of sandstone, limestone, chert, and evaporites. A distinctive chert layer occurs near the surface of the rock in the St. Louis area.

3.4 Groundwater

Groundwater was observed at the time of drilling in the borings on the dam at a depth of 22 feet in B-1 and 28 feet in B-2. Groundwater was also observed in HA-1 at 7.5 feet and 9 feet in HA-2. We do not anticipate that groundwater will influence the construction of building foundations. However, it should be noted that the groundwater level is subject to seasonal and climatic variations, and other factors; 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.

4.0 DESIGN RECOMMENDATIONS

4.1 BUILDING ADDITION

4.1.1 Existing Fill

Although the existing fill generally appears to be of similar consistency to the natural soils, the composition and consistency of the fill could vary significantly away from the boring locations. Presently, there are no records to document that the existing fill was placed and compacted in a controlled manner. Based on present knowledge of the site, the engineering properties and performance of the existing fill cannot be predicted with certainty. As a result, there is some risk of settlement or other performance problems if the foundations, floor slabs or pavements are supported on the fill material. In order to eliminate this risk, all of the existing fill would have to be excavated and either recompacted or replaced.

It is recommended that where existing fill will underlie the foundations, that the fill be excavated and either recompacted, or replaced. Alternately, the 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 is most desirable. Although the fill encountered in B-6 and B-7 extended only to 3 feet below existing grade, the potential does exist for the fill to be deeper at other locations across the site. With a fill depth of only 3 feet, the fill would not extend more than a foot below the base of the foundations, and deepening the footing through the fill may be more expedient and cost effective. If 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. In these areas, it may be more cost effective to perform a general overexcavation and recompaction or replacement of the existing fill prior to installing the foundations.

The disposition of the existing fill beneath the building floor slab and pavements should also be considered. In order to reduce potential settlement and cracking of the new floor slab that would overlie the existing fill, the fill should be removed and replaced. However, the cost of entirely removing and replacing the fill beneath the floor slab may not justify the potential benefit gained; and some risk of settlement of the floor slab may be acceptable to the Owner. Considering the probable length of time that the fill has been in place, and the anticipated light load on the building floor slab and pavements, the risk of supporting the floor slab and pavements on the existing fill is judged to be low, with proper proofrolling and treatment as described later in this report.

4.1.2 Shallow Foundations

Spread footing foundations bearing on natural soil or newly placed structural fill are appropriate for support of the proposed addition. Based on the soils encountered during our exploration, shallow foundations can be sized for maximum net allowable bearing pressures of 2,000 pounds per square foot (psf) for continuous wall footings and 2,400 psf for isolated, square, column footings. Some localized areas of inadequate bearing materials may be encountered during construction; therefore, we recommend that an allowance be made in the construction budget for selected footing overexcavations.

Isolated column footings should have a minimum dimension of 30 inches and continuous wall footings should be at least 24 inches wide. Exterior footings and foundations in unheated areas should be provided with at least 30 inches of soil cover for frost protection. Interior footings in heated portions 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.

Special attention must be given to designing the foundations immediately adjacent to the existing building. It is advisable to place the foundations for the proposed addition at the same level as those of the existing building. If the foundations of the new addition bear at a different elevation, either the new, or existing foundation walls should be structurally evaluated, to determine if they could accommodate the lateral stresses imposed by the adjacent shallower foundations. In spite of these precautions, some minor differential settlement between the existing building and the addition should be expected. Accordingly, we recommend that construction joints be provided and other measures taken, as needed, between the existing building and the addition. Even so, small differential movements may occur and future leveling of the floor slab between the existing and new construction may be necessary.

4.1.3 Seismic Considerations

The 2003 International Building Code (IBC) requires the design of buildings and their structural components to withstand seismic forces. Site coefficients, which are a function of the soil or rock type and consistency, are required for the calculation of minimum earthquake design forces. Based on the soils encountered and the depth to bedrock, Site Class D should be used for foundation design, with site coefficients F_a of 1.4 and F_v of 2.0 for short periods (S_s) and 1-second periods (S_1), respectively. This is based on our geotechnical explorations for the subject site, including the borings through predominantly medium stiff to stiff cohesive soil, a boring terminating on bedrock, our knowledge of the area bedrock from other local projects, and the *Geologic Map of St. Louis and St. Louis County, Missouri,* by R.G. Brill, 1991, published by MDNR. Using the procedures outlined in Section 1615 of the 2003 IBC, the calculated undrained shear strength (average of the top 100 feet) is on the order of 1,600 pounds per square foot (psf), well in excess of the 1,000 psf required to be classified as Site Class D.

Even with proper seismic design, some vertical and horizontal movement should still be expected during a major earthquake event, particularly if the earthquake occurs during a period of elevated groundwater.

4.1.4 Floor Slab

We recommend that the floor slab be designed using a modulus of subgrade reaction (k) of 100 pounds per square inch per inch of deflection (pci) if the floor slab will be supported on the existing fill material. A modulus of 150 pci may be used if supported on natural soils. The floor slab should be supported on a minimum 4-inch-thick layer of free-draining crushed stone. This will help distribute concentrated loads and equalize moisture conditions beneath the slab.

It is generally preferable to maintain structural separation between the floor slab and the foundation walls and column pads, using isolation joints. We also suggest that joints be placed in the floor slab on no more than 15-foot intervals in any direction. Such joints permit movement 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 floor slab and above the crushed rock or gravel to reduce the transfer of capillary moisture to the slab. However, without careful attention to curing of the floor slab, the polyethylene sheet can cause excessive shrinkage cracking and "curling."

4.1.5 Below - Grade Walls

Below-grade walls required at this site may include minor retaining or wing walls designed to accommodate surface grade changes around the building and paved areas. The maximum toe pressure for below-grade walls should not exceed the bearing pressure given previously 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 or traffic loads. We recommend the equivalent fluid unit weights tabulated 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. 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 4.1 – 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

Note: 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 to prevent the build-up of hydrostatic pressures behind the wall. A minimum 4-inch-diameter, perforated drainpipe should be placed at foundation level. Granular drainage material, consisting of 1-inch clean crushed rock (GP) with less than 5 percent passing the No. 200 sieve, should be placed a minimum of 6 inches in all directions around the drainpipe. Synthetic filter fabric, such as Mirafi 140N or equivalent, should encapsulate the drainpipe and granular backfill. Where practical, the pipe should drain by gravity to daylight or to a sump with a pump. Otherwise, the pipe may drain through weepholes located on approximately 10-foot centers for above grade retaining walls. Alternately, drainage can be provided directly through the weepholes without a drainpipe, provided that filter fabric is used or other measures are taken to prevent the granular backfill from migrating out through the weepholes.

When information is available regarding wall locations, configuration, and heights, SCI should be retained to evaluate global stability based on developed strength parameters for the subsurface soils and backfill. At your request, we can provide a global stability study, or we can work with the wall designer to provide coordinated internal and global stability studies. These services are beyond our current scope.

4.1.6 Pavements

Selection of the pavement section is dependent on the design life, traffic loads, subgrade strength, drainage characteristics, and the desired level of maintenance. Neither CBR testing nor formal pavement design was a part of our scope for this project. However, for planning purposes, the following recommendations typically result in pavements that perform satisfactorily on similar soil subgrades under automobile and pickup truck loads. They are intended to roughly provide a pavement requiring routine maintenance for a 5-year period, minor repair and maintenance during the 5- to 10-year life of the pavement, and possibly major repairs and restoration after a 10-year service life.

A flexible pavement section may be used for the parking lot and driveways. Parking areas for automobiles and light trucks should consist of a minimum 6-inch-thick, crushed stone base with a minimum 3-inch-thick, asphaltic concrete, wearing surface. The crushed stone base should be thickened to at least 8 inches in drive areas. Care should be taken to provide drains or drainable transition at locations where pavement sections of varying thickness abut, so as not to trap water within the crushed stone base, which could saturate and soften the subgrade.

Alternately, a rigid concrete pavement section may be used, with less anticipated long-term maintenance. Parking areas for automobiles and light trucks should consist of a minimum 6-inch-thick, non-reinforced concrete pavement. Crushed stone base is not required under this light-duty pavement section. For more heavily trafficked areas, we recommend that the section consist of an 8-inch-thick, non-reinforced concrete pavement, over 4 inches of compacted base rock. This concrete pavement section should also be used to support concentrated wheel loads for trash dumpster pads, approaches, and other areas where trucks will maneuver. To provide resistance against salt and freeze-thaw cycles, we recommend the concrete have a minimum 28-day compressive strength of 4,000 pounds per square inch and air entrainment of 5 to 7 percent by volume. We also recommend that the maximum joint spacing be approximately 15 feet.

4.2 DAM AT VILLAGE OF GREEN TRAILS

4.2.1 Site Observations

The dam appears to be retaining the lake water as no seepage through the dam was observed at the time of our site visits and the lake was at the level of the spillway on each site visit. The outflow pipes appeared to be functioning properly, although the 60-inch emergency overflow pipe is corroding along the flowline and will need to be replaced, as mentioned in the letter from MDNR. The discharge of water reported by MDNR appears to be from another, smaller diameter, spillway pipe.

Rock bedding should not be used for the new spillway pipe backfill. Instead, the outlet pipe should be placed on a cohesive soil subgrade, shaped to fit the pipe barrel, and the trench backfilled with properly compacted cohesive soil. Alternately, the trench can be backfilled to the springline of the pipe with lean concrete or flowable fill. Three concrete anti-seepage collars, extending 3 feet beyond the pipe, should also be used to reduce seepage around the pipe. The seepage collars should be located at the third points of the length through the dam.

Once removal of existing vegetation and regrading of the dam has been completed as discussed below, the existing roadway, including pavements, curbs, and stormwater manholes should be repaired or replaced. Care will need to be taken to avoid discharge of pavement runoff onto the slopes of the dam.

Trees and vegetation are located on the dam. The trees must be removed from the dam face and below the dam to reduce the risk of seepage through the dam due to the extensive root systems of the trees, and to allow the downstream area of the dam to be regraded in order to fill the eroded areas. Precautions must be taken when removing the trees as the excavation of the root system may be below the water level. If this becomes the case, then the lake should be temporarily drained to a level at least 3 feet below the deepest roots prior to proceeding with tree removal. High plastic clay should be used when filling the excavations caused by the removal of the trees to reduce further erosion. SCI should be on-site during removal of the trees and regrading of the dam in order to observe any seepage which may occur. If seepage is apparent or lowering of the lake is required, then seepage cutoffs with bentonite amended soil or other means of reducing flow will need to be considered. This could potentially be a major concern in consideration of the large trees that have been allowed to grow on the dam.

After regrading has been performed, the downstream slope of the dam should be vegetated with appropriate grasses, limited to 1 foot in height or less. To prevent further erosion, a concrete apron and/or riprap should be placed at the outfall structure along the steeper portions of the discharge channel after grading.

The silty soils on site are highly erodible when subject to flowing water, such as rainfall or streams. Care must be taken when working with these soils to prevent future erosion. Vegetation and silt fences are appropriate to help limit the transport of soil.

4.2.2 Slope Stability Analysis

Stability analysis was performed on a cross section of the dam (B-B'), to calculate the factor of safety (FS) on the downstream slope for steady-state seepage conditions. The upstream slope of the dam was not analyzed as no information was available relative to existing grades. Additionally, in consideration of the age of the dam, the upstream slope is assumed to remain stable provided proper erosion measures are taken, as discussed later, and a rapid drawdown does not occur. We are unaware of the means being available to cause a rapid drawdown. The stability analyses were performed using the computer program STEDWIN. A modified Bishop's Method was used to search for a circular mode of failure to calculate the FS for the slope. Soil parameters were based on the soils recovered from the site exploration.

SCI's analysis of the modeled section of the earth dam indicates that corrective measures are required to achieve the MDNR required FS of 1.5. An FS of approximately 1.4 was calculated for the model at steady-state seepage at an existing slope close to 2H:1V. Based on the results of this analysis, it appears that the downstream slope must be flattened to at least 2.5H:1V to achieve the required FS. Output from the computer runs, are included in Appendix B.

While the dam has performed satisfactorily, other than erosion, at an inclination of nearly 2H:1V, we recommend that all final slopes have an inclination of 3H:1V or flatter. This will allow placement of a greater quantity of high plastic fill along the dam face to better control future erosion, as well as allowing easier access to the slope for maintenance and inspection.

The existing dam embankment should be benched prior to the placement of fill. Benching will provide level surfaces for compaction and reduce the potential for development of inclined planes of weakness between the natural soil and compacted fill. The benches should be spaced such that the maximum height of cut at the up-slope end of the bench is 3 feet.

4.2.3 Erosion Control Considerations

It appears that discharge from the outflow pipe, lack of vegetation and proper maintenance, and natural silty soils have contributed to the erosion observed on the downstream face of the dam and the severe erosion in the discharge channel. The spillway soils should be reworked and riprap added to the spillway to reduce further erosion in this area. We recommend that the emergency spillway be extended beyond the toe of the dam, to reduce the erosion in that area. The size, thickness, and limits of the riprap should be based on the design velocities the riprap is to resist. SCI can assist with rip-rap design, if needed, once the design velocities are available.

We recommend that the remaining erosion areas be backfilled with a material consisting primarily of clayey soils. These soils will likely have to be transported in from off-site, as the material on-site is too silty in nature. The new fill placed within the erosion areas should be benched into the existing slope at 1-foot vertical intervals.

Riprap should be installed along the upstream face of the dam, extending at least 2 feet vertically below and 3 feet vertically above the normal water level in the lake. The riprap should be crushed limestone, ranging from 4 inches to 12 inches in size, with an average diameter of 8 inches. The riprap thickness should be at least 18 inches. All riprap for the project should be underlain by filter fabric, such as Mirafi 180N or equivalent, properly anchored at its upper and lower edges.

In addition, appropriate erosion control measures, such as proper contouring, the installation of siltation fences, or the placement of staked straw bales, should be utilized during construction until a proper vegetative cover can be established. 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 silt fence or straw bales. Timely sodding and/or seeding of sloped surfaces is highly recommended.

4.3 EAST SLOPE

4.3.1 Discussion and Recommendations

The east slope at the site is heavily vegetated, and the upper portion of the slope is covered with debris that appears to have been pushed over the edge during previous site clearing. Also included in the debris are large quantities of grass and leaves.

SCI reviewed the apparent critical maximum height cross section (A-A') of the slope for stability using the computer program STEDWIN. The cross section was developed for the slope with a maximum height of approximately 48 feet. Borings B-5 and HA-1 were reviewed and used to model the anticipated subsurface profile and soil properties for the slope.

SCI's analysis of the modeled section indicates that flattening of the existing slope to at least 3H:1V will be required to standard of practice FS of 1.3, as a FS of approximately 1.1 was calculated for the model at the existing inclination of 2H:1V. However, we recognize that this slope has existed in its present state for considerable time, and has performed satisfactorily. Accordingly, if the parking lot is maintained at least 20 feet away from the edge of the slope without additional fill placement, and proper erosion control measures are implemented, then the slope will likely perform satisfactorily. However, removal of the vegetation from the slope would likely be detrimental and allow significant erosion to occur. If the natural soils within the slope were to become saturated, the FS would be reduced and could lead to failure. Therefore, if grading of the slope is required, or if vegetation is removed from the slope when regrading to fill the draw, proper surficial drainage and vegetative cover of the slope should be provided in a timely manner to reduce surface water infiltration on the slope face and the potential for saturation of the underlying soils.

A 3H:1V slope would achieve a FS of 1.3, and allow additional fill placement in the parking lot area up to 2 feet, and extension of the parking lot to the crest of the slope.

4.4 EXCAVATION BRACING REQUIREMENTS

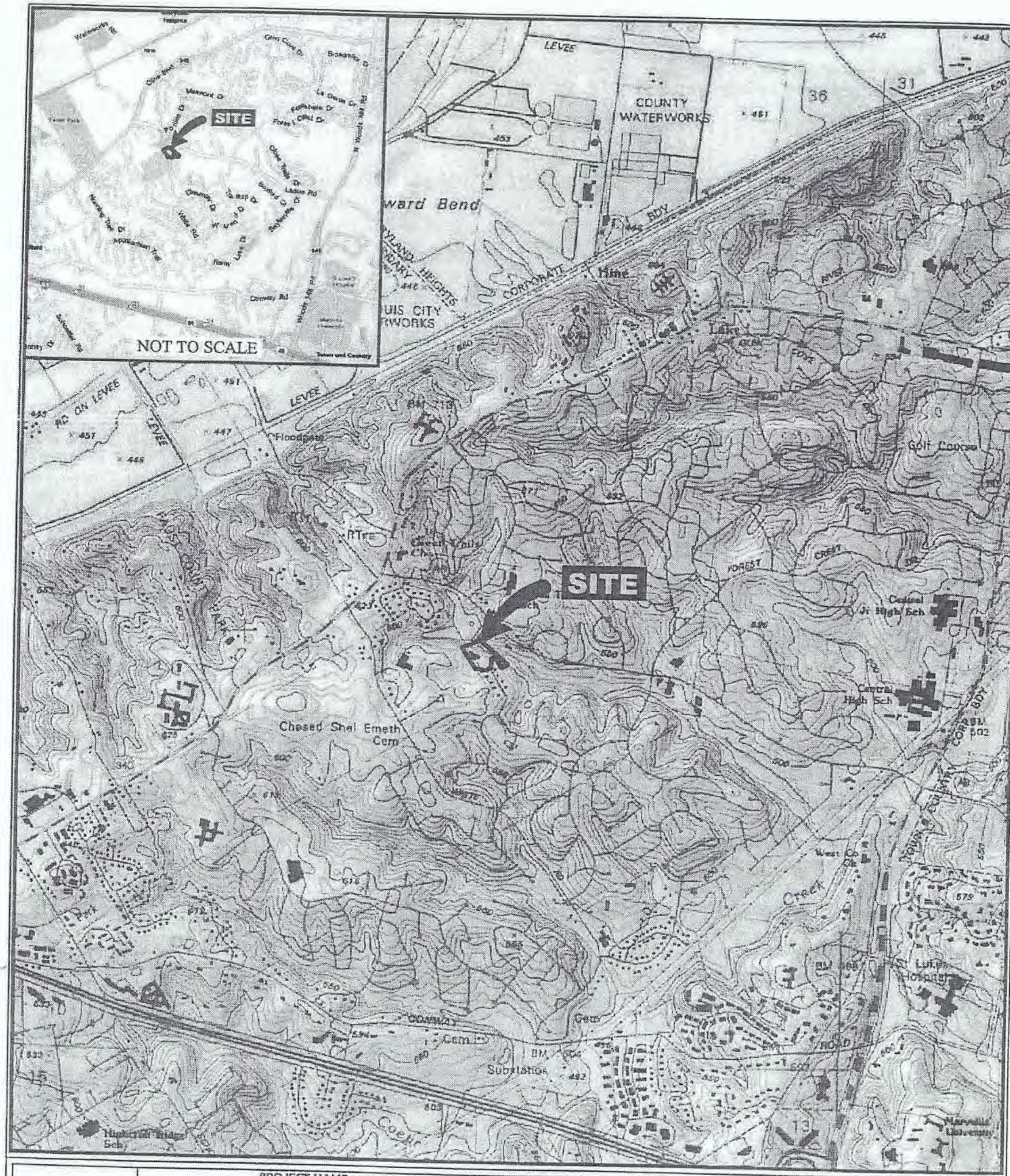
In the *Federal Register*, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, Part 1926, Subpart P." This document was issued to provide for the safety of workers entering 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.

5.0 LIMITATIONS

The recommendations provided herein are for the exclusive use of our client. They are specific only to the project described, and are not meant to supercede more stringent requirements of local ordinances. They are based on subsurface information obtained at ten, 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 construction plans and specifications, once they are available, to review whether our recommendations have been understood and applied correctly. 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 analyses and 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

TPHERIS ISRAEL CHEVRA KADISHA
SYNAGOGUE IMPROVEMENTS
CHESTERFIELD, MISSOURI

VICINITY AND TOPOGRAPHIC MAP

DRAWN BY	DKM	DATE	JOB NUMBER
CHECKED BY	JWC	07/2006	2006-2182.10

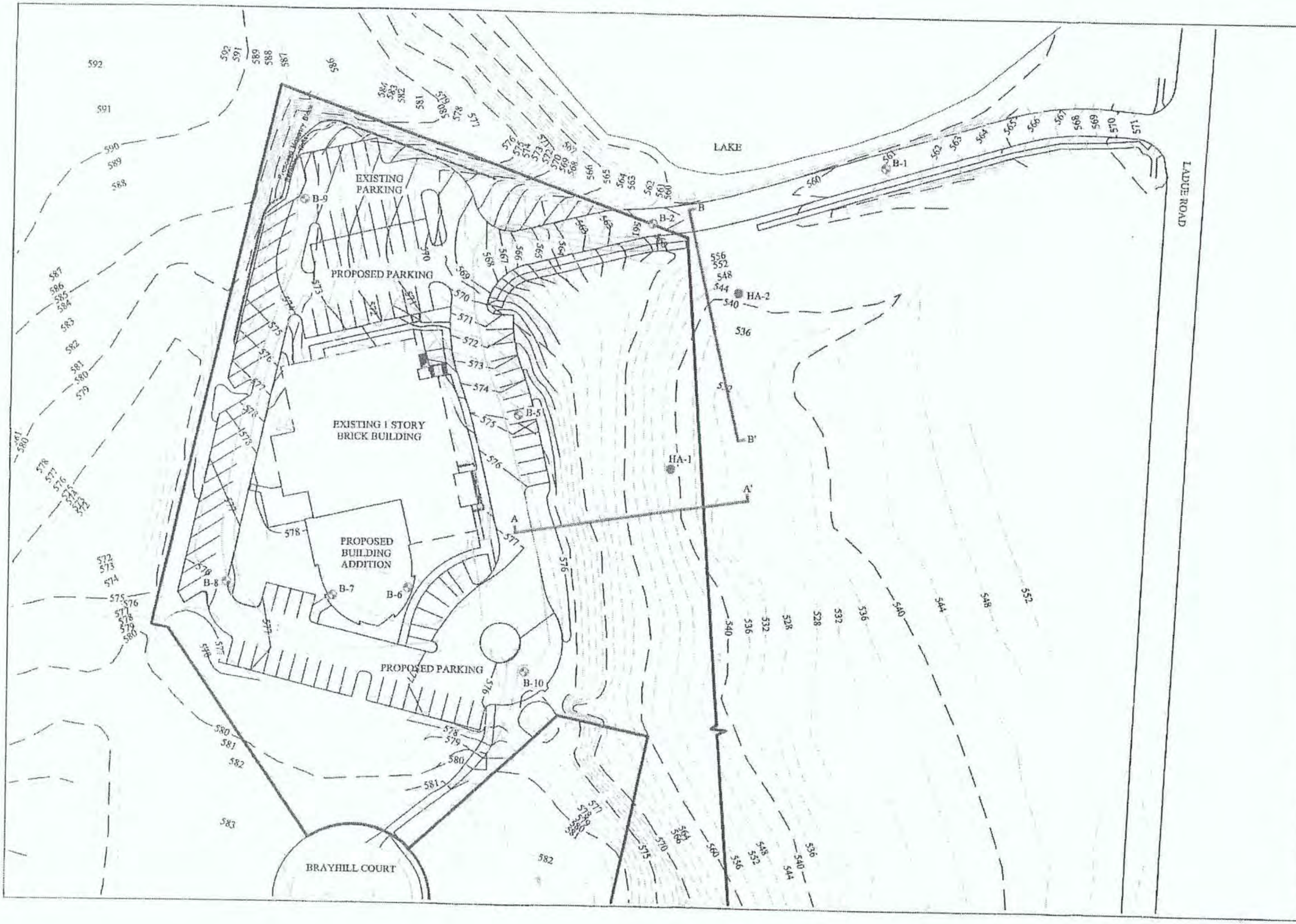
General Notes/Legend

USGS TOPOGRAPHIC MAP
CHESTERFIELD, MISSOURI QUADRANGLE
DATED 1994
10' CONTOURS

MICROSOFT STREETS AND TRIPS 2005

N
W E
S

SCALE	1" = 2000'
FIGURE	1



PROJECT NAME
TPHERIS ISRAEL CHEVRA KADISHA
SYNAGOGUE IMPROVEMENTS
CHESTERFIELD, MISSOURI

SITE PLAN

General Notes/Legend
INDICATES APPROXIMATE SOIL BORING LOCATIONS
INDICATES APPROXIMATE HAND AUGER LOCATIONS
BASED ON UNDATED PLAN PROVIDED ELECTRONICALLY ON 07/05/2006 FROM THE CLAYTON ENGINEERING COMPANY
DIMENSIONS AND LOCATIONS ARE APPROXIMATE; ACTUAL MAY VARY.
DRAWING SHALL NOT BE USED OUTSIDE THE CONTEXT OF THE REPORT FOR WHICH IT WAS GENERATED.

SCALE	1" = 50'
JOB NUMBER	2006-2182.10
DATE	07/2006
DRAWN BY	DKM
CHECKED BY	JWC
FIGURE	2

APPENDIX A

SCI ENGINEERING, INC.
130 POINT WEST BOULEVARD
ST. CHARLES, MISSOURI 63301
636•949•8200 FAX 636•949•8269
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) divided by 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	0 to 10
Some	10 to 35
By Modifier	35 to 50

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 Tpheris Israel Chevra Kadisha Synagogue Improvements
LOCATION Chesterfield, Missouri
DRILLER Midwest Drilling, Inc. HAMMER Automatic
EQUIPMENT CME-75 w/CFA ELEVATION 561±
BORING NUMBER B-1
SHEET 1 of 2
PROJECT NO. 2006-2182.10
DATE DRILLED 06/27/06

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	PLASTIC LIMIT	ELEVATION (ft)
					3" ASPHALTIC CONCRETE									
					9" CRUSHED ROCK									560
					FILL: Brown, low plastic silty clay to clayey silt									
	1	SS		3 5 9				17		9.0				
	2	SS		4 6 7				19		6.0				
5														555
	3	SS		3 3 2				19		4.5		29	23	
	4	SS		5 6 8	Becomes brown and gray			22		6.5				
10														550
	5	SS		1 1 2				32		2.0				
15	6	ST	22/24					30 26	92		1.0			545
	7	SS		2 4 3				24		7.0				
20														540
	8	SS		2 2 2	SILTY CLAY (CL): Brown and gray, low plastic			38		2.0				

WATER LEVEL:
____ NONE OBSERVED WHILE DRILLING
22.0 ft WHILE DRILLING
____ ft ____ HRS AFTER DRILLING
____ ft ____ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Tpheris Israel Chevra Kadisha Synagogue Improvements BORING NUMBER B-1
LOCATION Chesterfield, Missouri SHEET 2 of 2
DRILLER Midwest Drilling, Inc. HAMMER Automatic PROJECT NO. 2006-2182.10
EQUIPMENT CME-75 w/CFA ELEVATION 561± DATE DRILLED 06/27/06

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	PLASTIC LIMIT	ELEVATION (ft)
					SILTY CLAY (CL): Brown and gray, low plastic *(Continued)*									535
	9	SS		2 1 3	Becomes light gray			32		3.0				
30														530
	10	SS		2 2 2				32		6.0				
35														525
	11			2 2 3				33		2.5				
40					SILT (ML): Dark gray, low plastic									
					SILTY CLAY (CL): Brown and gray, low plastic, some rock fragments									520
	12	SS		50/1"	SHALE: Gray									
					Auger Refusal at 43.75 feet									
45														515

WATER LEVEL:

_____ NONE OBSERVED WHILE DRILLING
22.0 ft WHILE DRILLING
_____ ft ___ HRS AFTER DRILLING
_____ ft ___ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Tpheris Israel Chevra Kadisha Synagogue Improvements
LOCATION Chesterfield, Missouri
DRILLER Midwest Drilling, Inc. HAMMER Automatic
EQUIPMENT CME-75 w/CFA ELEVATION 561±

BORING NUMBER B-2
SHEET 1 of 2
PROJECT NO. 2006-2182.10
DATE DRILLED 06/29/06

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	PLASTIC LIMIT	ELEVATION (ft)
					2" ASPHALTIC CONCRETE									
					18" CRUSHED ROCK									560
					FILL: Brown, low plastic silty clay, some rock fragments									
5					FILL: Brown, high plastic clay									555
	1	SS		2 2 3				24		8.0				
					FILL: Brown, low plastic silty clay									
	2	SS		3 4 5				6		6.0		39	23	
10					Becomes brown and gray									
	3	ST	17/24					28	93	3.0	0.9			550
	4	SS		1 2 2	Trace organics			28		2.0				
15														545
	5	SS		3 4 5				25		7.5				
20														540
	6	SS		3 5 5				22		7.0				

WATER LEVEL:

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

REMARKS:

BORING LOG

PROJECT Tpheris Israel Chevra Kadisha Synagogue Improvements
LOCATION Chesterfield, Missouri
DRILLER Midwest Drilling, Inc.
EQUIPMENT CME-75 w/CFA
HAMMER Automatic
ELEVATION 561±
BORING NUMBER B-2
SHEET 2 of 2
PROJECT NO. 2006-2182.10
DATE DRILLED 06/29/06

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	PLASTIC LIMIT	ELEVATION (ft)
					FILL: Brown, low plastic silty clay *(Continued)*									535
30	7	SS		4 8 9				25		6.5				530
35	8	SS		4 6 6				24		6.0				
					SILTY CLAY (CL): Brown and gray, low plastic									525
40	9	SS		2 3 3				32		2.0				520
45	10	SS		3 4 3	Becomes light brown and light gray			30		3.0				515
	11	SS		3 4 1				27		3.0				

Boring terminated at 50 feet.

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

REMARKS:

BORING LOG

PROJECT Tpheris Israel Chevra Kadisha Synagogue Improvements
LOCATION Chesterfield, Missouri
DRILLER Midwest Drilling, Inc.
EQUIPMENT CME-75 w/CFA
HAMMER Automatic
ELEVATION 574±
BORING NUMBER B-5
SHEET 1 of 1
PROJECT NO. 2006-2182.10
DATE DRILLED 06/28/06

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	PLASTIC LIMIT	ELEVATION (ft)
					SILTY CLAY (CL): Grayish brown, low plastic									
	1	SS		5 5 5				20		6.5				
	2	ST	15/24					16	91	9.0	2.7			570
5														
					SILT (ML): Brown, low plastic									
	3	SS		2 1 2				12		5.0				
	4	SS		3 3 4				13						565
10														
	5	ST	24/24				1	21						
					SILTY CLAY (CL): Brown, low plastic									
	6	SS		3 5 5				26		5.0				560
15														
	7	SS		4 6 7	Trace roots			23		6.0				555
20														
					CLAY (CH): Light brown, high plastic									
	8	SS		2 3 5				27		5.0				550

Boring terminated at 25 feet

WATER LEVEL:

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

REMARKS:

1) Unable to extrude the sample in one piece from the tube, therefore only a moisture was obtained.

BORING LOG

PROJECT Tpheris Israel Chevra Kadisha Synagogue Improvements
LOCATION Chesterfield, Missouri
DRILLER Midwest Drilling, Inc.
EQUIPMENT CME-75 w/CFA
HAMMER Automatic
ELEVATION 578±
BORING NUMBER B-6
SHEET 1 of 1
PROJECT NO. 2006-2182.10
DATE DRILLED 06/27/06

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	PLASTIC LIMIT	ELEVATION (ft)
					FILL: Brown, low plastic silty clay									
	1	SS		4 6 6				17		9.0				
					CLAY (CL/CH): Brown, medium plastic									575
	2	ST	0/24	3 5 6				25		9.0		47	26	
5														
	3	SS		3 4 4				25		7.0				
					SILT (ML): Brown, low plastic									570
	4	SS		2 3 3				23		2.0				
10														
					SILTY CLAY (CL): Brown, low plastic									
														565
	5	SS		2 2 2				29		1.0				
15														
														560
	6	SS		2 3 4				23		3.0				
20					Boring terminated at 20 feet.									
														555

WATER LEVEL:

X NONE OBSERVED WHILE DRILLING
____ ft WHILE DRILLING
____ ft ____ HRS AFTER DRILLING
none ft 1 DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Tpheris Israel Chevra Kadisha Synagogue Improvements — BORING NUMBER B-7
LOCATION Chesterfield, Missouri — SHEET 1 of 1
DRILLER Midwest Drilling, Inc. — HAMMER Automatic — PROJECT NO. 2006-2182.10
EQUIPMENT CME-75 w/CFA — ELEVATION 577± — DATE DRILLED 06/27/06

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	PLASTIC LIMIT	ELEVATION (ft)
					9" CRUSHED ROCK, trace brown, low plastic silty clay									
	1	SS		4 6 7	FILL: Brown and gray, low plastic silty clay			18		5.0				575
	2	SS		3 2 4	SILTY CLAY (CL): Brown, low plastic			23		5.0				
5														
	3	SS		2 3 4				26		3.0				570
	4	ST	24/24					22	89					
10														
														565
	5	SS		2 3 3				24		2.5				
15														
														560
	6	SS		2 2 3				23		1.0				
20					Boring terminated at 20 feet.									
														555

WATER LEVEL:

X NONE OBSERVED WHILE DRILLING
___ ft WHILE DRILLING
___ ft ___ HRS AFTER DRILLING
none ft 1 DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Tpheris Israel Chevra Kadisha Synagogue Improvements
LOCATION Chesterfield, Missouri
DRILLER Midwest Drilling, Inc. HAMMER Automatic
EQUIPMENT CME-75 w/CFA ELEVATION 577±
BORING NUMBER B-8
SHEET 1 of 1
PROJECT NO. 2006-2182.10
DATE DRILLED 06/27/06

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	PLASTIC LIMIT	ELEVATION (ft)
					9" CRUSHED ROCK									
	1	SS		4 5 6	SILTY CLAY (CL): Brown, low plastic			24		5.0				575
	2	SS		2 3 3				25		3.0				
5														
	3	SS		2 2 3				26		3.5				570
	4	SS		1 2 2				30		2.0				
10					Boring terminated at 10 feet.									
														565
15														
														560
20														
														555

WATER LEVEL:
X NONE OBSERVED WHILE DRILLING
ft WHILE DRILLING
ft HRS AFTER DRILLING
ft DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Tpheris Israel Chevra Kadisha Synagogue Improvements
BORING NUMBER B-9
LOCATION Chesterfield, Missouri
SHEET 1 of 1
DRILLER Midwest Drilling, Inc.
HAMMER Automatic
PROJECT NO. 2006-2182.10
EQUIPMENT CME-75 w/CFA
ELEVATION 578±
DATE DRILLED 06/27/06

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	PLASTIC LIMIT	ELEVATION (ft)
					1" ASPHALTIC CONCRETE									
					2" CRUSHED ROCK									
					FILL: Brown, low plastic silty clay, trace roots									
	1	SS		4 4 5				12		4.0				
														575
	2	SS		4 5 6				16		4.0				
5														
	3	SS		3 5 7				18		4.0				
														570
	4	SS		6 12 12				12		4.0				
10					Boring terminated at 10 feet.									
														565
15														
														560
20														
														555

WATER LEVEL:
X NONE OBSERVED WHILE DRILLING
____ ft WHILE DRILLING
____ ft ____ HRS AFTER DRILLING
____ ft ____ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Tpheris Israel Chevra Kadisha Synagogue Improvements
LOCATION Chesterfield, Missouri
DRILLER Midwest Drilling, Inc. HAMMER Automatic
EQUIPMENT CME-75 w/CFA ELEVATION 576±

BORING NUMBER B-10
SHEET 1 of 1
PROJECT NO. 2006-2182.10
DATE DRILLED 06/27/06

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	PLASTIC LIMIT	ELEVATION (ft)
					SILTY CLAY (CL): Brown, low plastic, trace roots									575
	1	SS		2 2 2				26		3.5				
5	2	SS		2 3 3				24		4.0				570
	3	SS		2 2 2				27		2.0				
10	4	SS		2 1 2	No roots			29		2.0				565
15	5	SS		2 3 4				26		4.0				
					Boring terminated at 15 feet.									560
20														555

WATER LEVEL:
X NONE OBSERVED WHILE DRILLING
____ ft WHILE DRILLING
____ ft ____ HRS AFTER DRILLING
____ ft ____ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Tpheris Israel Chevra Kadisha Synagogue Improvements
LOCATION Chesterfield, Missouri
DRILLER SCI Engineering Inc. HAMMER n/a
EQUIPMENT Hand Auger ELEVATION 542±

BORING NUMBER HA-1
SHEET 1 of 1
PROJECT NO. 2006-2182.10
DATE DRILLED 07/07/06

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	PLASTIC LIMIT	ELEVATION (ft)
					SILTY CLAY (CL): Brown and gray, low plastic, trace roots									
	1	BS						21		3.0				540
					Some fine sand									
	2	BS			Becomes gray, some organics and roots			33		<0.5				
5					CLAYEY SILT (ML): Gray with brown, low plastic									
	3	BS						41		<0.5		36	26	535
	4	BS						32		<0.5				
10					SILTY CLAY (CL): Gray, low plastic									
	5	BS						29		2.0				530
					CLAY (CH): Brown and gray, high plastic									
15	6	BS						28		1.0				
					Boring terminated at 15 feet.									525
20														520

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

REMARKS:

BORING LOG

PROJECT Tpheris Israel Chevra Kadisha Synagogue Improvements BORING NUMBER HA-2
LOCATION Chesterfield, Missouri SHEET 1 of 1
DRILLER SCI Engineering Inc. HAMMER n/a PROJECT NO. 2006-2182.10
EQUIPMENT Hand Auger ELEVATION 543± DATE DRILLED 07/07/06

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	PLASTIC LIMIT	ELEVATION (ft)
					SILTY CLAY (CL): Brown and gray, low plastic, trace roots									
	1	BS						17		3.0				
	2	BS						17		1.5		41	27	
														540
	3	BS			Becomes gray, some organics			26		1.0				
5					No roots or organics					<0.5				
	4	BS						25		1.5				
					Some fine sand									
	5	BS						27		0.5				535
					SILTY SAND (SM): Gray, fine									
	6	BS			SILTY CLAY (CL): Brown, low plastic, some fine sand, some gravel		1	33		≤ 0.5				
10					Boring terminated at 9.25 feet.									
														530
15														
														525
20														
														520

WATER LEVEL:

_____ NONE OBSERVED WHILE DRILLING
9.0 ft WHILE DRILLING
_____ ft ____ HRS AFTER DRILLING
_____ ft ____ DAYS AFTER DRILLING

REMARKS:

1) Bucket refusal on gravelly clay or piece of gravel.

APPENDIX B

Tpheris Israel Chevra Kadisha Synagogue Dam - Existing

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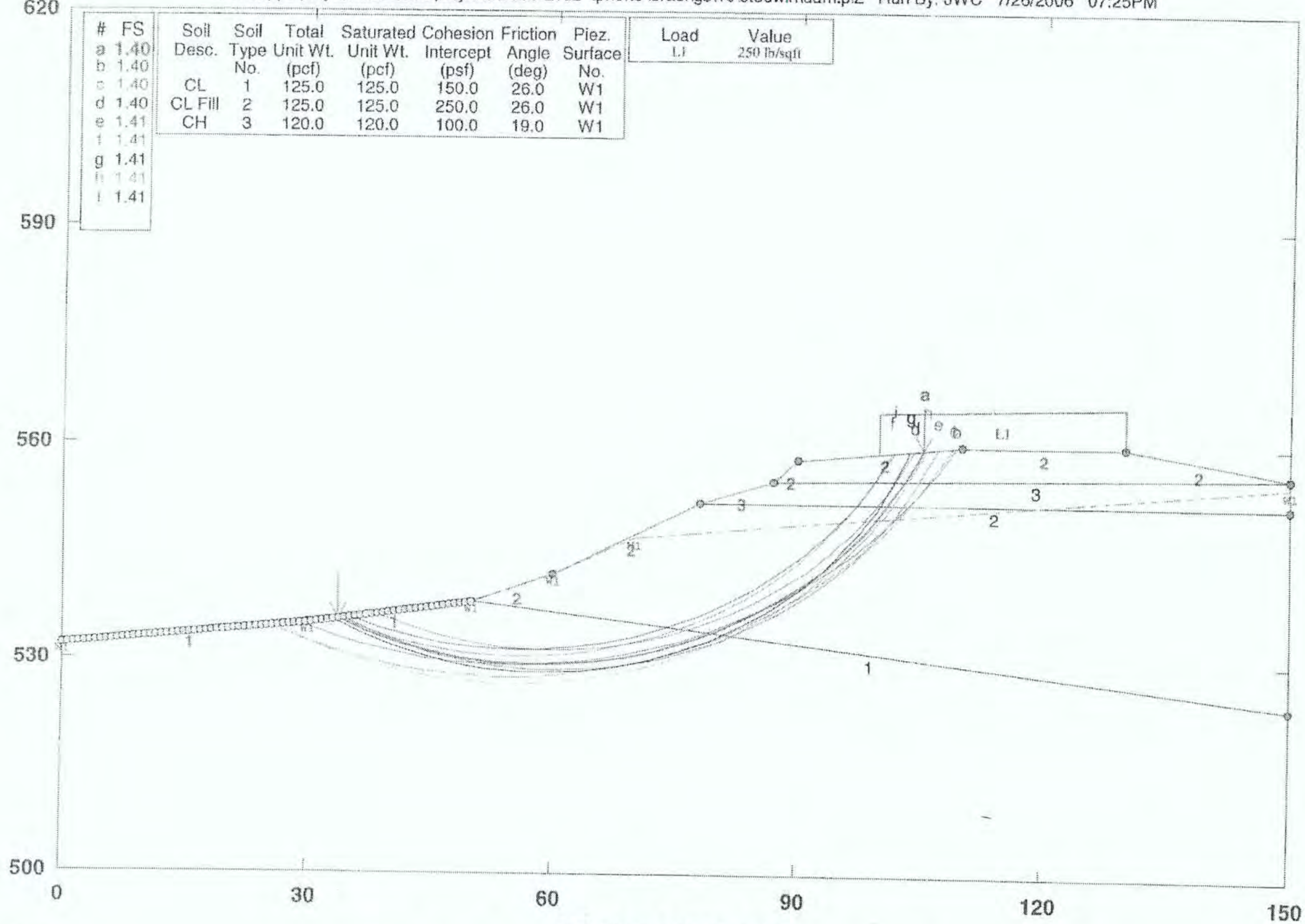

STABL6H FSmin=1.40

Safety Factors Are Calculated By The Modified Bishop Method

STED

Tpheris Israel Chevra Kadisha Synagogue Dam - Improved

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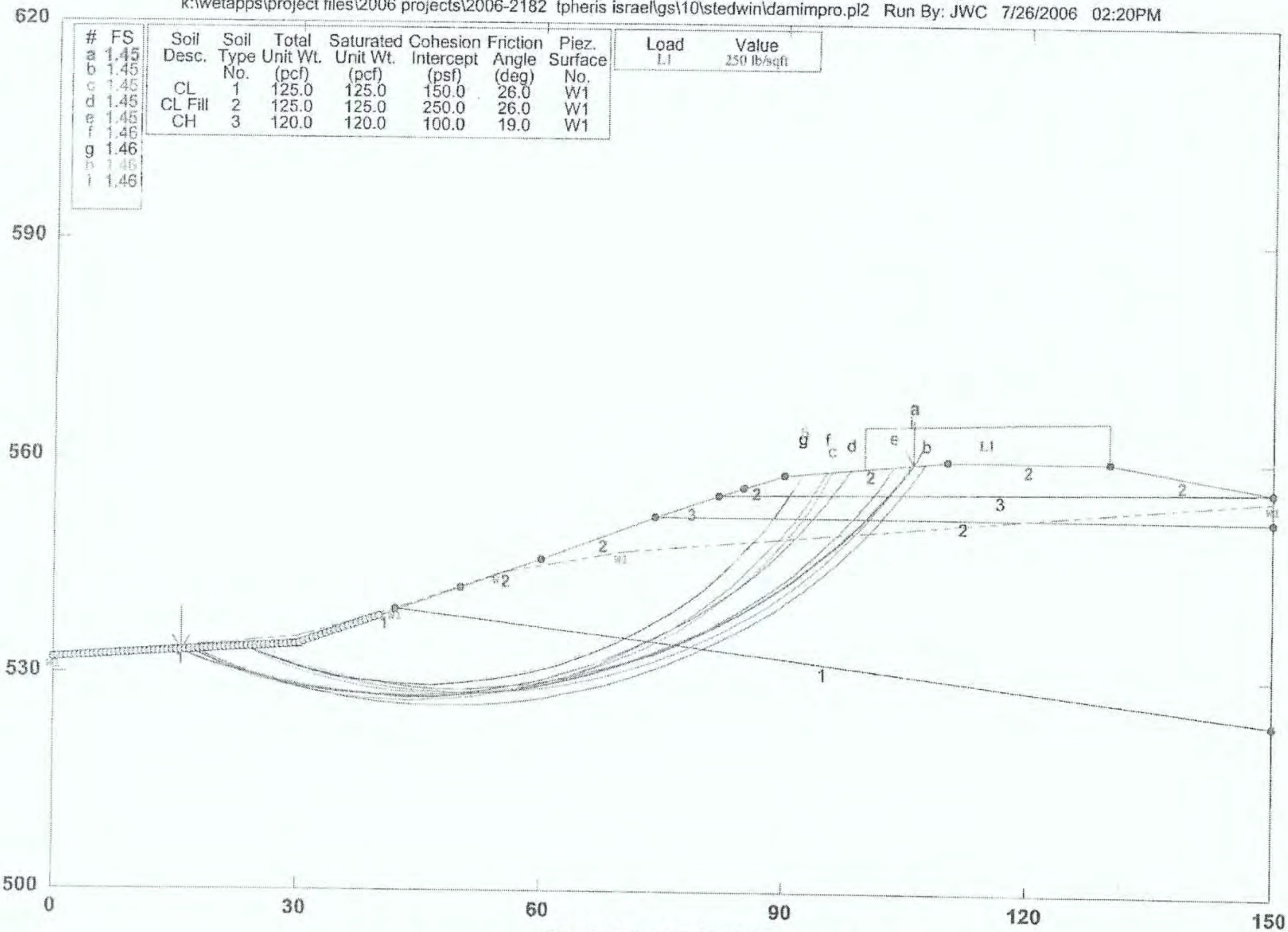


#	FS
a	1.45
b	1.45
c	1.45
d	1.45
e	1.45
f	1.46
g	1.46
h	1.46
i	1.46

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
CL	1	125.0	125.0	150.0	26.0	W1
CL Fill	2	125.0	125.0	250.0	26.0	W1
CH	3	120.0	120.0	100.0	19.0	W1

Load	Value
L1	250 lb/sqft

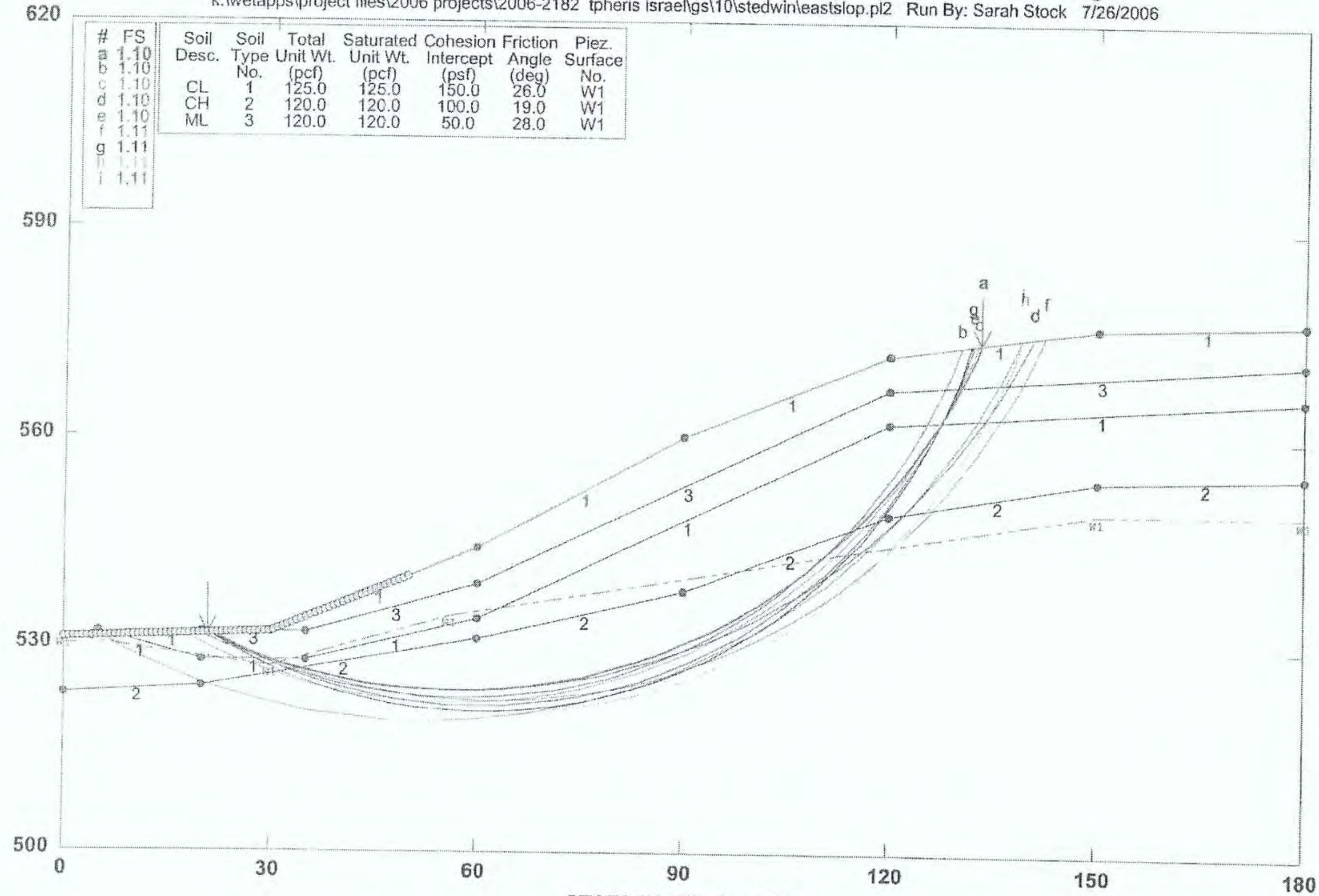
STABL6H FSmin=1.45

Safety Factors Are Calculated By The Modified Bishop Method

STED

Tpheris Israel Chevra Kadisha Synagogue East Slope - Existing

k:\wetapps\project files\2006 projects\2006-2182 tpheris israel\gs\10\stedwin\eastslop.pl2 Run By: Sarah Stock 7/26/2006



STABL6H FSmin=1.10
Safety Factors Are Calculated By The Modified Bishop Method

STED

Tpheris Israel Chevra Kadisha Synagogue East Slope - Improved

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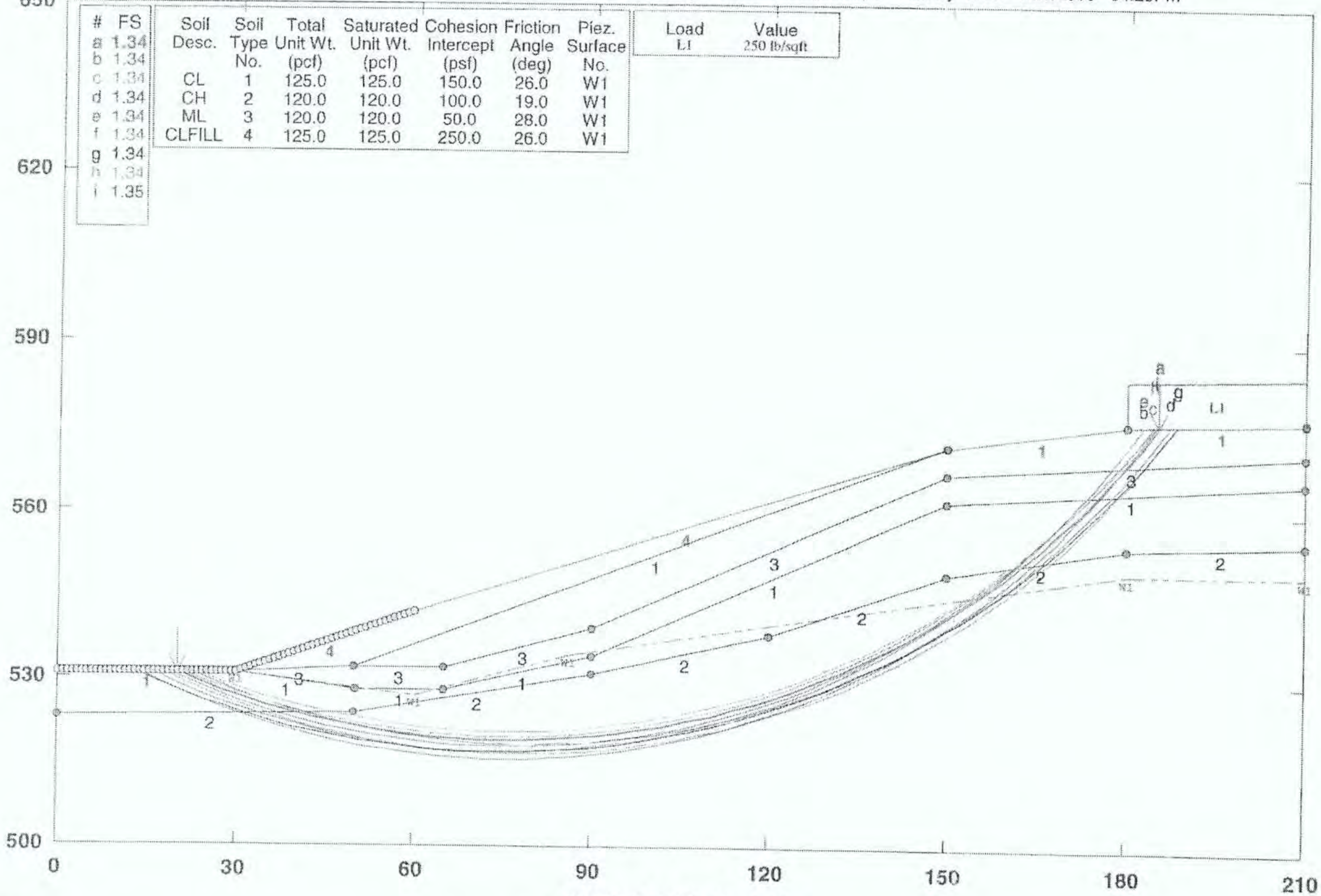

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Safety Factors Are Calculated By The Modified Bishop Method

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APPENDIX C

TPHERIS ISRAEL CHEVRA KADISHA SYNAGOGUE IMPROVEMENTS
CHESTERFIELD, MISSOURI

APPENDIX C

SITE DEVELOPMENT AND CONSTRUCTION CONSIDERATIONS

Site Drainage and Grading

Positive site drainage should be provided to reduce surface water infiltration around the perimeter of the addition and beneath the floor slab. All grades should be sloped away from the building, and roof and surface drainage should be collected and discharged such that water is not permitted to infiltrate the backfill of the building.

The existing dam embankment should be stripped of surface vegetation, brush, and trees. The strippings should be stockpiled for later placement in landscaped areas, as appropriate. We recommend that trees and brush be removed from the fill areas. After clearing the vegetation, the surface of the dam embankment should be carefully checked for holes created by animals or nature. If encountered, the holes should be excavated and then backfilled with bentonite grout, or a combination of bentonite amended soil. For bentonite amended soil, we recommend 5 percent bentonite be added and thoroughly blended with the soil prior to placing as fill or backfill.

Prior to general fill placement, any erosion features should be excavated and backfilled. Excavations for erosion features should be benched into the adjacent soils on 1-foot vertical intervals. The existing dam embankment should also be benched prior to the placement of fill. Benching will provide level surfaces for compaction and reduce the potential development of inclined planes of potential weakness between the natural soil and compacted fill. The benches should be spaced such that the maximum height of cut at the up-slope end of the bench is 3 feet.

We recommend 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 building before the surface slopes down and away. Natural slopes to receive fill which are steeper than 5H:1V should be benched prior to the placement of fill, as previously discussed for the dam

Underground Utilities

Underground utilities can provide a pathway for water to migrate below the floor slab. Drain and utility pipes beneath floors 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 building. 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. Cohesive 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.

Site Preparation

Within the construction area, the asphaltic concrete and any other existing structures must be properly demolished and the debris removed from the site. Existing pavements and utilities, as well as their associated backfill, should be removed from below and at least 10 feet beyond the proposed building footprint. As an exception, deep utilities may be grouted in place rather than being removed. However, the existing backfill associated with deep utilities should be properly proofrolled as discussed later in this report and any soft areas properly recompacted. Excavations resulting from the removal of existing site improvements should be backfilled with properly compacted fill.

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. Topsoil can be reused as fill, if thoroughly mixed with other, acceptable, non-organic fill materials, as approved by SCI.

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. In particular, any isolated soft areas within the existing fill materials to be left in place, should be identified during this operation. 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.

If organic or soft areas are encountered, they should be removed and replaced with properly compacted fill or otherwise stabilized as 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.

Fill Materials and Compaction

Cohesive structural fill 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). Aggregate base course should be compacted to at least 95 percent of the same criterion. We recommend that structural fill placed in the proposed building area have a liquid limit less than 45 and a plasticity index less than 25. If higher plasticity soils are placed within 3 feet of the floor slab subgrade or 2 feet of the bottom of the footings, then remediation will be required. Acceptable non-organic fill soils include materials designated CL, ML, CL-ML, SP, SW, GP, and GW by ASTM D 2487. For the dam, CH fill is recommended.

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 an 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 placed next to walls should be compacted with hand-operated equipment and not large self-propelled or machine operated equipment, which could result in overcompaction and higher lateral pressures than previously recommended for design. 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, or pavements. Field density tests should be performed on each lift of fill to document that proper compaction is achieved.

Significant problems may be incurred if construction takes place in wet or cold weather. Special measures may be required to facilitate construction during these periods. 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.

Unstable subgrade conditions can develop easily in areas of high moisture content or poor drainage. Cohesive soils, particularly those which are more silty, can exhibit a significant loss of strength when they become saturated and disturbed. Disturbance may occur from trafficking of construction equipment, or other sources. **The high moisture contents and silty nature of the soils onsite make the site especially vulnerable to disturbance.** If there is any indication that disturbance has occurred, SCI should be retained to observe these conditions and provide recommendations regarding corrective measures.

Shallow Foundation Excavations

SCI should observe all footing excavations for potential problem areas, such as the presence of existing fill or soft zones, prior to placing concrete. Excessive disturbance of siltier soils in footing excavations should be avoided. The potential for such disturbance will increase during wetter times of the year. Footing 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, relatively dry, and free of loose soil or uncompacted fill. 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 and approved. If the bearing soils have dried significantly, controlled addition of moisture may be desirable.

During construction, existing footings must not be undercut, i.e. no excavation should encroach within an area extending 45 degrees downward and outward from the outside edge of the existing foundations. If this is required, then SCI should be retained to provide specific recommendations to maintain support of the existing foundations and lateral support of the excavations.

Groundwater is not anticipated to be encountered in the footing excavations. However, in most situations, 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.

Floor Slab and Pavement Subgrades

Floor slab and pavement subgrades may be subjected to construction traffic and exposure to weather for an extended period. It may be necessary, therefore, to proofroll the subgrade, in both cut and fill areas, and recompact the subgrade immediately prior to placing base rock for the floor slab or pavement. Soft areas should be selectively undercut and backfilled with properly compacted cohesive soil. Proofroll passes should be limited, particularly on silty subgrades, to reduce the potential for pumping of moisture from deeper within the soil profile. A geotechnical stabilization fabric, such as Mirafi 600X, Tensar BX1100, or equivalent, may be used to help stabilize particularly soft areas. Where possible, the subgrade should be sloped to provide drainage.

Subgrade 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 floor slab or paving as is practical.

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 1 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.

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 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 and the floor slab subgrade to assess the existing fill.
- Observe footing excavations for adequacy of bearing materials.
- Observe removal of the trees and regrading of the dam to observe for seepage and implement appropriate measures
- Observe the floor slab subgrade prior to placing base rock.
- Observe backfilling of below-grade utility excavations.
- Observe pavement subgrade preparation and provide observation and testing services for the base course and pavement section.
- Check the thickness of pavement sections and, for asphaltic concrete, its density.
- Provide quality assurance testing of structural concrete and pavement materials.

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL 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 suggestions and observations are offered 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 study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical study is unique, each geotechnical report is unique, prepared *solely* for the client. *No one except you* should rely on your geotechnical 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 report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical 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 its 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 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 report include those that affect the:

- function and character of the proposed structure,
- elevation, configuration, location, orientation, or loading 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 report is based on conditions that existed at the time the study was performed. Do not rely on a geotechnical 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 way 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 Report is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical 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 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 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 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 such risks, 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 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.*

Rely on Your Geotechnical Engineer for Additional Assistance

Membership in ASFE 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.

The preceding paragraphs are based on information provided by ASFE.

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