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Geotechnical Report

MAJESTIC DENTAL – COTTLEVILLE PARKWAY
COTTLEVILLE, MISSOURI

February 2023

MD REAL PROPERTY, LLC
Owner

BAALMAN ARCHITECTS
Architect

BAX ENGINEERING COMPANY, INC.
Civil Engineer/Surveyor

SCI No. 2023-0091.10

STATE OF MISSOURI
TIMOTHY J. BARRETT
2/28/2023
NUMBER
PE-2011015698
PROFESSIONAL ENGINEER

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Tim J Barrett
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February 28, 2023

Dr. Michael Lear
MD Real Property, LLC
7100 Castle Cliff Court
St. Charles, Missouri 63304

RE: Geotechnical Report
Majestic Dental – Cottleville Parkway
Cottleville, Missouri
SCI No. 2023-0091.10

Dear Dr. Michael Lear:

Attached is our *Geotechnical Report*, dated February 2023. 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.

- We calculate that up to 1 inch of settlement will occur due to the weight of the new fill in the southern portion of the clinic, where fills of up to 11 feet is planned. As such, this may result in excessive differential settlement compared to the northern portion of the clinic that will be near the existing grade. To reduce the risk of excessive differential settlement, it is recommended the fill be placed to the planned subgrade elevation as soon as possible during the site grading. Based on the soils encountered, as well as our experience with similar soils in the area, we anticipate that the majority of the settlement will occur during fill placement, with the remainder occurring within approximately 4 to 6 weeks upon completion of fill placement.

- Undocumented existing fill was encountered in the site and will likely be present below the footings and floor slab on the northern portion of the clinic footprint. To eliminate the risk of excessive movement, the fill would need to be removed from within the building footprint at its entirety and extending a minimum of 5 feet beyond the clinic footprint laterally. However, the fill appears to be compacted with some effort and based on the results of field and laboratory testing, the age of the fill, and the anticipated structural loads, the risk of supporting the improvements on the existing fill is judged to be low with proper proofrolling.

- Expansive clays were encountered in the borings. Where the bearing soils consist of fat clay, we recommend that they be removed to minimum depths of 2 feet beneath the bearing level of the footings and 3 feet beneath the bottom of the floor slab. We estimate remediation will be required for the footings and for the floor slab at the northernmost wall.

- Shallow foundations can be sized for maximum net allowable bearing pressure of 2,500 pounds per square foot (psf) and 3,000 psf for strip and column footings, respectively.

- Seismic Site Class C should be used for the project, with seismic design parameters as follows: $F_a = 1.20$, $F_v = 1.66$, $S_{DS} = 0.27$, $S_{D1} = 0.16$, indicating that a Seismic Design Category C may be used with a Risk Category of I through III.

We appreciate the opportunity to be of service and look forward to working with you during the construction phase of the project.

If you have any questions or comments, please do not hesitate to contact me.

Respectfully,

SCI ENGINEERING, INC.

Prakash Paudel, E.I.
Staff Engineer

Timothy J. Barrett, P.E., CFM
Senior Engineer

Enclosure
Geotechnical Report

C: Mike Baalman, Baalman Architects

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COTTLEVILLE, MISSOURI

1.0 INTRODUCTION

At the request of Dr. Michael Lear and Sherry Lear of MD Real Property, LLC, SCI Engineering, Inc. (SCI) conducted a geotechnical exploration for the proposed dental clinic. 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 24, 2023, and accepted on January 27, 2023, by Dr. Michael Lear.

2.0 SITE AND PROJECT DESCRIPTION

A dental clinic is planned for Lot 5 of Harmony Ridge Commercial Park in Cottleville, Missouri. The location of the site is shown on the *Vicinity and Topographic Map*, Figure 1. The site is an undeveloped, grass-covered lot, with sparse trees on the south. The topography slopes from the north downward to the south with approximately 20 feet of relief. The existing site conditions are shown on the *Aerial Photograph*, Figure 2.

Based on the *Grading Plans*, dated January 20, 2023, and prepared by Bax Engineering Company, Inc. (BAX), the clinic will be a single-story, slab-on-grade structure with a footprint of 5,473 square feet (sf), and a finished floor elevation (FFE) of approximately 547. Asphaltic parking and drive lanes are planned north and west of the clinic. A tall foundation wall with maximum exposed height of 5 feet is planned along the southern edge of the building and a concrete retaining wall will extend from the western edge of the tall foundation wall with a maximum height on the order of 4 feet. The proposed development is shown on the *Site Plan*, Figure 3.

Cuts on the order of 2 to 3 feet will be required in the northern portion of the building footprint while fills up to 11 feet will be required in the southern portion as shown on the *Grading Plan*, Figure 4. Structural loads were not available at the time of this proposal; however, we assume that the dental clinic will have column loads of less than 100 kips and wall loads of less than 4 kips per linear foot.

3.0 SUBSURFACE CONDITIONS

Four borings (B-1 through B-4) were drilled at the approximate locations shown on Figures 2, 3, and 4. The boring locations were staked in the field by SCI personnel using a global positioning system. Ground surface elevations were interpolated from the above-mentioned Grading Plans, prepared by Bax. 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* contained in Appendix A.

3.1 Existing Fill

Existing fill consisting of soft to medium stiff lean clay (CL in accordance with the Unified Soil Classification System and ASTM D 2487) and fat clay (CH) was encountered in B-1 to a depth of 3 feet (approximate elevation (El.) 546) and B-4 to a depth of 5.5 feet (approximate El. 540.5). Existing fill was not encountered in the remaining borings. Standard Penetration Test (SPT) N-values within the existing fill ranged from 5 to 8 blows per foot (bpf). Moisture contents ranged from 17 to 24 percent. Documentation regarding the placement and compaction of the existing fill was not available at the time of this report; however, the fill appears to have been placed during the general grading of the existing Harmony Ridge Estates in 2004 or 2005.

3.2 Native Soil Profile

The native soils predominantly consisted of medium stiff to stiff fat clay (CH) with varying amounts of gravel to the depths of auger refusal in B-2, B-3, and B-4, which ranged from 5 to 14 feet, or planned termination depth in B-1, at 10 feet. As an exception, lean clay (CL) was encountered at depths of 5.5 feet to 11 feet in B-4. SPT N-values ranged from 7 to 15 bpf, with an average of approximately 9 bpf. Moisture contents ranged from 10 to 34, averaging approximately 23 percent.

3.3 Groundwater

Groundwater was not observed during drilling in any of the borings. 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.

3.4 Bedrock Geology

Documented geology including the *Bedrock Geologic Map of the St. Louis Quadrangle, Missouri and Illinois*, the site falls on the boundary of Warsaw Formation, and undivided Keokuk and Burlington Limestones, and Fern Glen Formation. The Warsaw Formation consists of dark, fissile shale and intercalated argillaceous and silty dolomite or dolomitic limestone in the upper half; shaley to argillaceous, cherty, very fossiliferous, finely crystalline dolomitic limestone in the lower half. The thickness of this formation ranges from 60 to 100 feet.

The Keokuk limestone is medium crystalline and lesser finely and coarsely crystalline limestone while Burlington limestone is light-colored, medium to coarsely crystalline limestone. The Fern Glen Formation consists of red and green calcareous shale, shaley limestone, and a basal bed of massive, dolomitic limestone.

Auger refusal was encountered in all borings except B-1 as shown in Table 3.1. Auger refusal is a designation applied to any material that cannot be further penetrated by the drill rig without extraordinary effort and is indicative of a very hard or very dense material, usually boulders or bedrock.

Table 3.1 – Bedrock Summary

Boring	Approximate Ground Surface Elevation (feet)	Approximate Auger Refusal Depth (feet)	Approximate Auger Refusal Elevation (feet)
B-1	549	NE	--
B-2	537	5	532
B-3	536	7	529
B-4	546	14	532

NE – Not Encountered

4.0 DESIGN RECOMMENDATIONS

4.1 Settlement

We calculate that up to 1 inch of settlement will occur due to the weight of the new fill in the southern portion of the clinic where fills of up to 11 feet is planned, while the northern portion of the clinic footprint is near the proposed grade. As such, this may result in excessive differential settlement.

To reduce the risk of excessive total settlement, it is recommended the fill be placed to the planned subgrade elevation as soon as possible during the site grading. In addition, sufficient time should be added to the construction schedule to allow the settlement to reach tolerable limits prior to the construction of the building and installation of utilities in these areas.

Based on the soils encountered, as well as our experience with similar soils in the area, we anticipate that the majority of the settlement will occur during fill placement, with the remainder occurring within approximately 4 to 6 weeks upon completion of fill placement. The actual rate of settlement should be measured in the field during, and upon, completion of site grading to ensure that settlement is nearly complete prior to construction.

SCI can monitor the settlement of the fill by periodically surveying settlement plates installed prior to grading or points installed upon completion of site grading. SCI would then notify the construction team when the observed rate of settlement has reached an acceptable level to continue construction in this area.

4.2 Existing Fill

Based on the FFE of 547, existing fill will likely be present below the footings and floor slab on the northern portion of the building footprint. The fill appears to have been placed during the general grading of the existing Harmony Ridge Estates in 2004 or 2005; however, documentation regarding its placement and compaction was not available at the time of this report. As such, there is a risk of excessive movement if the building is supported on the fill material. To eliminate this risk, the fill would need to be removed from within the building footprint at its entirety and extending a minimum of 5 feet beyond the clinic footprint laterally. The overexcavation should be backfilled with engineered fill in accordance with Section 5.2. Alternately, the foundations could be extended through the fill to bear on native soils.

However, the fill appears to be compacted with some effort, and based on the results of field and laboratory testing, the age of the fill, and the anticipated structural loads, the risk of supporting the improvements on the existing fill is judged to be low with proper proofrolling. At a minimum, SCI should observe the fill in the footing excavations and any unsuitable areas should be selectively undercut and replaced in accordance with Section 5.1 and 5.2.

4.3 Expansive Clays

Expansive clay was encountered near the proposed finished floor elevation and anticipated foundation bearing elevation near the northern wall of the proposed clinic. Where the bearing soils consist of fat clay, we recommend that they be removed to minimum depths of 2 feet beneath the bearing level of the footings and 3 feet beneath the bottom of the floor slab**. Based on a proposed FFE of 547, expansive clay remediation will likely be along the northern wall of the clinic below footings where less than 4.5 feet of new fill is planned and below the floor slab, where less than 3 feet of fill is planned.** Ultimately, the need for, and extent of, remediation should be delineated by SCI personnel in the field during construction.

The overexcavation should extend at least 2 feet beyond the outside edge of the footings and building footprint to facilitate uniform compaction of the replacement materials and may require additional widening at the building 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 alternate, the footing overexcavation may be backfilled with lean concrete. With this option, widening of the footing excavation is not required. If clean rock is used as backfill material, it must be drained to daylight or to a sump with a pump. The footings and floor slab would then be constructed on the newly placed fill.

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 in accordance with Section 5.2.

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

4.4 Shallow Foundations

Shallow spread footing foundations bearing on native lean clay, remediated expansive clay and suitable existing fill, or newly placed, low plastic structural fill can be considered for support of the proposed clinic. Shallow foundations can be sized for a maximum net allowable bearing pressure of 2,500 pounds per square foot (psf) for strip footings and 3,000 psf for column footings. A one-third increase in the net allowable bearing pressures may be used for transient loads, such as wind and earthquake.

We anticipate that some localized areas of inadequate bearing materials may be encountered during construction, and subsequently, require remediation to achieve these capacities. Therefore, we recommend that an allowance be made in the construction budget for selected footing overexcavations. If encountered, inadequate bearing materials should be undercut to firmer soils and the overexcavation backfilled with lean concrete or flowable fill.

Exterior footings and foundations in unheated areas of the building 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, the total settlement should be less than 1 inch, and differential settlement between adjacent footings should be less than ¾ inch.

4.5 Seismic Considerations

Ground shaking at the foundation of structures and liquefaction of the soil under the foundations 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.

4.5.1 Design Earthquake

According to International Building Code (2015 edition) (IBC 2015), structures such as those proposed for this project are required to be designed to a design earthquake with a 2 percent Probability of Exceedance over a 50-year exposure period (i.e., a 2,475-year design earthquake).

4.5.2 International Building Code Site Classification

Based on procedures outlined in the IBC 2015, our geotechnical exploration, and the depth to bedrock, the site can be classified as Site Class C. Using the procedures outlined in Section 1613 of the IBC 2015, the calculated weighted average undrained shear strength (s_u) is more than 2,000 psf, required to be classified as a Site Class C. Seismic design parameters for the site as determined from data provided by the IBC 2015 and the United States Geological Survey National Seismic Hazard Mapping Project are shown in Table 4.1.

Table 4.1 - Seismic Design Parameters

Site Class	C
M_w	7.52
PGA	0.17
F_{PGA}	1.20
Site Modified PGA_M	0.20
S_S	0.33
S_1	0.14
F_a	1.20
F_v	1.66
S_{DS} (Design Spectral Acceleration at 0.2 sec)	0.27
S_{D1} (Design Spectral Acceleration at 1.0 sec)	0.16
Seismic Design Category (Risk Category I, II, and III)	C

4.5.3 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 plasticity index 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, some vertical and horizontal movement may be experienced during a major earthquake event, particularly if the earthquake occurs during a period of elevated groundwater, even with the proper seismic design.

4.6 Floor Slab

The following recommendations assume a floor slab load of 150 psf or less. If sections of the floor slab will support loads greater than the assumed floor slab loads, underlying subgrade soils below these sections may need to be removed and replaced with compacted/engineered fill. SCI should be provided the opportunity to review the final design plans and specifications to determine if the underlying subsurface soils can adequately support the loads. Proofrolling, as discussed in Section 5.1, should be accomplished to identify soft or unstable soils that should be removed from the floor slab areas prior to fill placement and/or floor slab construction.

We recommend that the floor slab be designed using a modulus of subgrade reaction, *k* value, of 150 pounds per cubic inch based on values typically obtained from 1-foot by 1-foot plate load tests. This value assumes the slab will bear on native lean clay, remediated existing fill and expansive clay, or newly placed, low plastic structural fill. Depending on how the slab load is applied, the value will have to be geometrically modified. The value should be adjusted for larger areas using the following expression for cohesive and cohesionless soil:

Modulus of Subgrade Reaction, $k_s = k/B$ for cohesive soil; and

$k_s = k \times ((B+1)/2B)^2$ for cohesionless soil.

where: k_s = coefficient of vertical subgrade reaction for loaded area;
k = coefficient of vertical subgrade reaction for 1x1 square foot area; and
B = width of area loaded, in feet.

The floor slab should be supported on a minimum 4-inch-thick layer of crushed stone. 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 with spacing (in feet) equal to approximately three times the thickness of the slab (in inches) in both directions. 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.

Where occupied space or moisture sensitive floor coverings are planned, we recommend a 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 sheeting can cause excessive shrinkage cracking and "curling."

The precautions listed below should be followed for construction of slab-on-grade pads. These details will not reduce the amount of movement, but are intended to reduce potential damage, should some settlement of the supporting subgrade take place. Some increase in moisture content is inevitable as a result of development and associated landscaping. However, extreme moisture content increases can be largely controlled by proper and responsible site drainage, building maintenance and irrigation practices.

- Cracking of slab-on-grade concrete is normal and should be expected. Cracking can occur not only as a result of heaving of the supporting soil, but also as a result of concrete curing stresses. The occurrence of concrete shrinkage cracking, and problems associated with concrete curing may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement, finishing, curing, and by the placement of crack control joints at frequent intervals, particularly where re-entrant slab corners occur.

- Areas supporting slab should be properly moisture conditioned and compacted. Backfill in all interior and exterior water and sewer line trenches should be carefully compacted to reduce the shear stress in the concrete extending over these areas.

Exterior slabs should be isolated from the building. These slabs should be reinforced to function as independent units. Movement of these slabs should not be transmitted to the building foundation or superstructure.

4.7 Below-Grade Walls

Below-grade walls required at this site will include a tall foundation wall along the southern edge of the building and the concrete retaining wall extending from the western edge of the tall foundation wall. 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 ultimate 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 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 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-head 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-head 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 native 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 to each side and above 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 to daylight 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 weepholes, spaced on approximately 10-foot centers, 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. 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

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

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 floor slab 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 clinic before the surface slopes down and away. Slopes of less than 15 feet in total height should perform satisfactorily at this inclination, or flatter. If steeper or taller slopes are proposed, they should be brought to our attention and individually addressed and evaluated on a case-by-case basis.

4.9 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 clinic. 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 clinic 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.

4.10 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 California Bearing Ratio 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 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 as summarized in Table 4.3. 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.

Table 4.3 - Recommended Flexible Pavement Thickness

Pavement Layer	Thickness (inches)	
	Parking Stalls	Drive Lanes for Automobiles and Light Trucks
Asphaltic Concrete Surface Course (BP-1)	3	3
Aggregate Base (Type 5)	6	8

Alternately, a rigid concrete pavement section may be used with less anticipated long-term maintenance. The recommended rigid pavement sections are shown in Table 4.4. 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 (psi) and air entrainment of 5 to 7 percent by volume. We also recommend that the maximum joint spacing be approximately 15 feet.

Table 4.4 - Recommended Rigid Pavement Thickness

Pavement Layer	Parking Stalls		Drive Lanes for Automobiles and Light Trucks	Loading Docks, Dumpster Pads and Approaches
Non-Reinforced Portland Cement Concrete	5	6	6	8
Aggregate Base Course (Type 5)	4	--	4	4

The fat clay could cause some distress in the pavements. To reduce long-term maintenance, consideration could be given to remediating the upper 12 inches of subgrade. Alternatives include removal and replacement with crushed stone or low plastic soil, lime treatment, or geogrid reinforcement. At a minimum, the pavement subgrade should be proofrolled and any soft areas identified should be selectively undercut to firmer soils and backfilled with engineered fill material in accordance with Sections 5.1 and 5.2.

5.0 SITE DEVELOPMENT AND CONSTRUCTION CONSIDERATIONS

5.1 Site Preparation

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. Trees and brush may be burned on-site if approved by local ordinances. Burn pits should be located in cut areas such that the ashes are completely removed during site grading. If this is not practical, burn pits must be located outside of building, street, and areas designated as slopes steeper than 5H:1V. Stumps that cannot be burned should be removed from the site.

After stripping, the clinic footprint should be checked for unsuitable existing fill and expansive clay, and undercut/remediated in accordance with Sections 4.2 and 4.3 respectively. The clinic footprint should then 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 crushed rock or lime 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.

Slopes to receive fill which are steeper than 5H:1V 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 height of the cut at the up-slope end of the bench is less than 5 feet.

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 should be placed in maximum 8-inch-thick loose lifts and mechanically compacted in accordance with Table 5.1. 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 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, GP, and GW by ASTM D 2487-11. Fat clay (CH) can be lime stabilized and placed as structural fill as discussed in Section 4.4.

Table 5.1 - Typical Compaction Requirements for Fill

Material Tested	Proctor Type	Minimum Percentage Dry Density
Structural Fill (Cohesive)	Modified (ASTM D 1557)	90
	Standard (ASTM D 698)	95
Structural Fill (Granular)	Modified	95
	Standard	98
Landscaped Areas (non-load bearing)	Modified	88
	Standard	92
Utility Trench Backfill	Modified	90
	Standard	95

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 lean clay or 1-inch minus crushed limestone or controlled low-strength material. We advise performing field density tests on at least every other lift to monitor compaction. As an alternate, 1-inch clean crushed limestone could be used 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 should be drained to daylight or a sump with a pump. Clean rock 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 slab, or pavements. Field density tests should be performed on each lift of fill to document that proper compaction is achieved.

5.3 Shallow Foundation Excavations

SCI should observe all footing and floor slab excavations for problem areas, such as soft zones or areas of expansive clay 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. 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, 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 significantly influence foundation construction. 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.

5.4 Subgrade Considerations

Floor slab and pavement subgrades may be subjected to construction traffic and exposure to weather for an extended period and significant problems may be incurred. It will likely 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 slab. 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 floor slab 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 TriAx-140, or equivalents, may be used to help stabilize particularly soft areas. Where possible, the subgrade should be sloped to provide drainage.

5.5 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.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations, as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should the slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

SCI is providing this information solely as a service to our client. SCI does not assume responsibility for construction site safety or the contractor's or other party's compliance with local, state, and federal safety or other regulations.

5.6 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. 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:

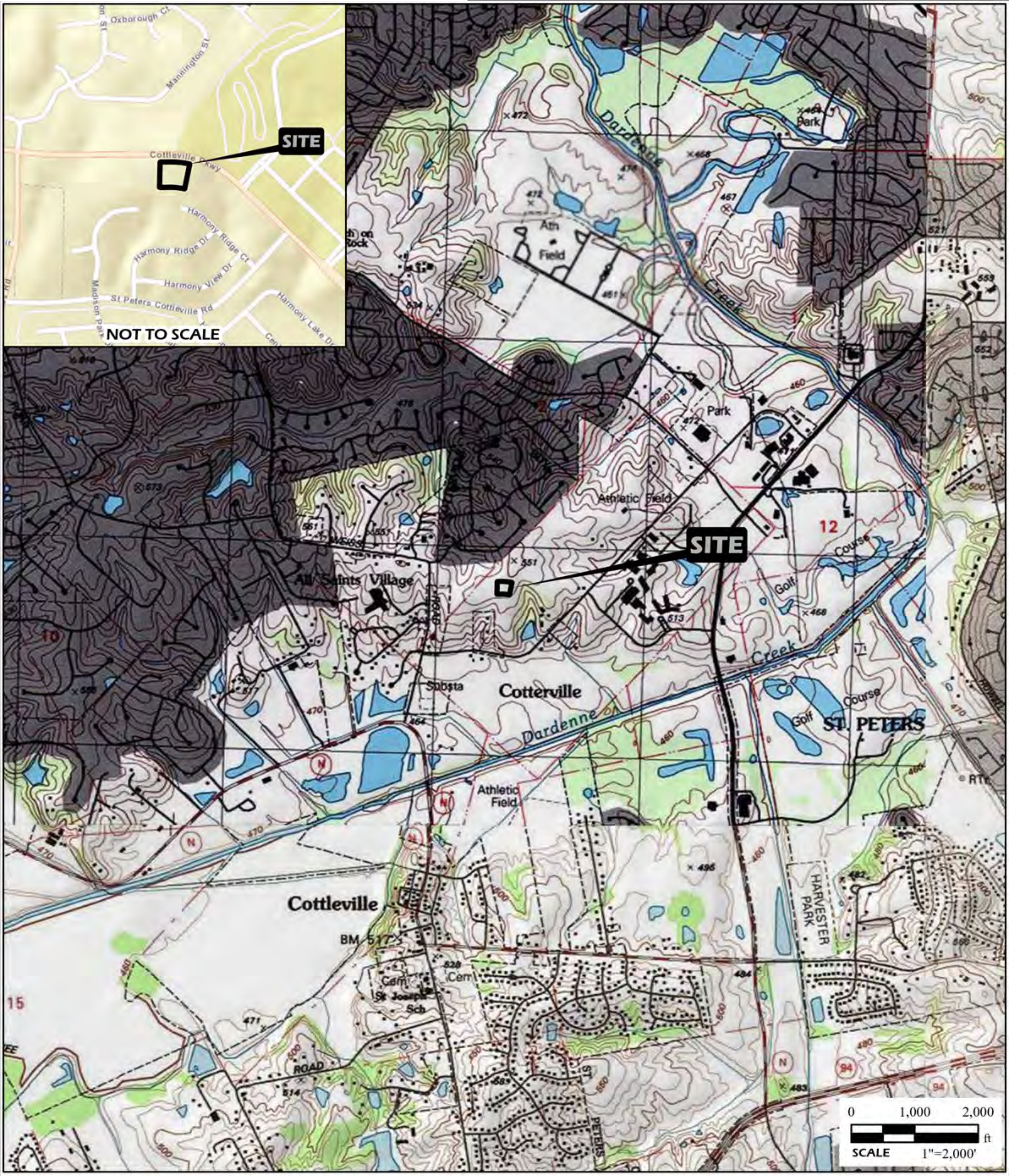
- Participate in a formal preconstruction meeting with the Owner's Representative, Civil Engineer, and Contractor, prior to construction at the site;
- Observe site preparation activities prior to construction, including stripping, remediation of expansive clay, and proofrolling;
- Conduct and document 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 impact of existing fill, expansive clay, and soft soils, and to recommend the extent of remedial measures;
- Observe footing excavations for adequacy of bearing materials;
- Observe the floor slab and pavement subgrades prior to placing base rock;
- Observe backfilling of below-grade utility excavations; and
- Provide quality assurance testing of structural concrete and paving.

7.0 LIMITATIONS

The recommendations provided herein are for the exclusive use of Majestic Dental. 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 Majestic Dental. 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 four specific, widely spaced, 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 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

MAJESTIC DENTAL – COTTLEVILLE PARKWAY
COTTLEVILLE, MISSOURI

VICINITY AND TOPOGRAPHIC MAP

DRAWN BY	ACV	DATE	JOB NUMBER
CHECKED BY	PP	02/2023	2023-0091.10

GENERAL NOTES/LEGEND

USGS TOPOGRAPHIC MAP
O'FALLON MISSOURI QUADRANGLE
DATED 2002
10' CONTOURS
WELDON SPRING MISSOURI QUADRANGLE
DATED 1994
10' CONTOURS

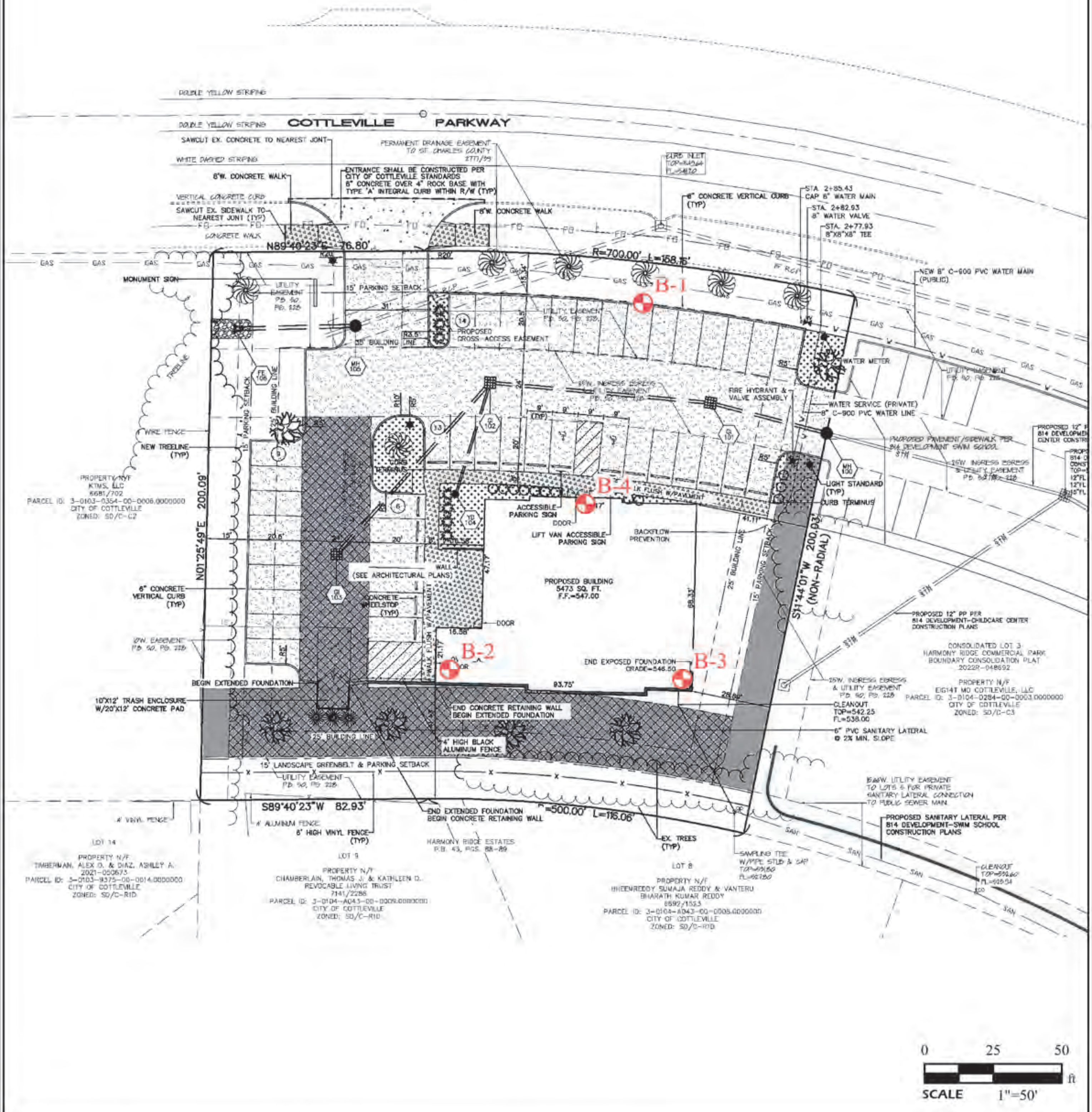
USGS TOPOGRAPHIC MAP
KAMPERVILLE MISSOURI QUADRANGLE
DATED 1994
10' CONTOURS
CHESTERFIELD MISSOURI QUADRANGLE
DATED 1994
10' CONTOURS

STREET MAP
HTTP://GOTO.ARCGISONLINE.COM/MAPS/WORLD_STREET_MAP

N
W E
S

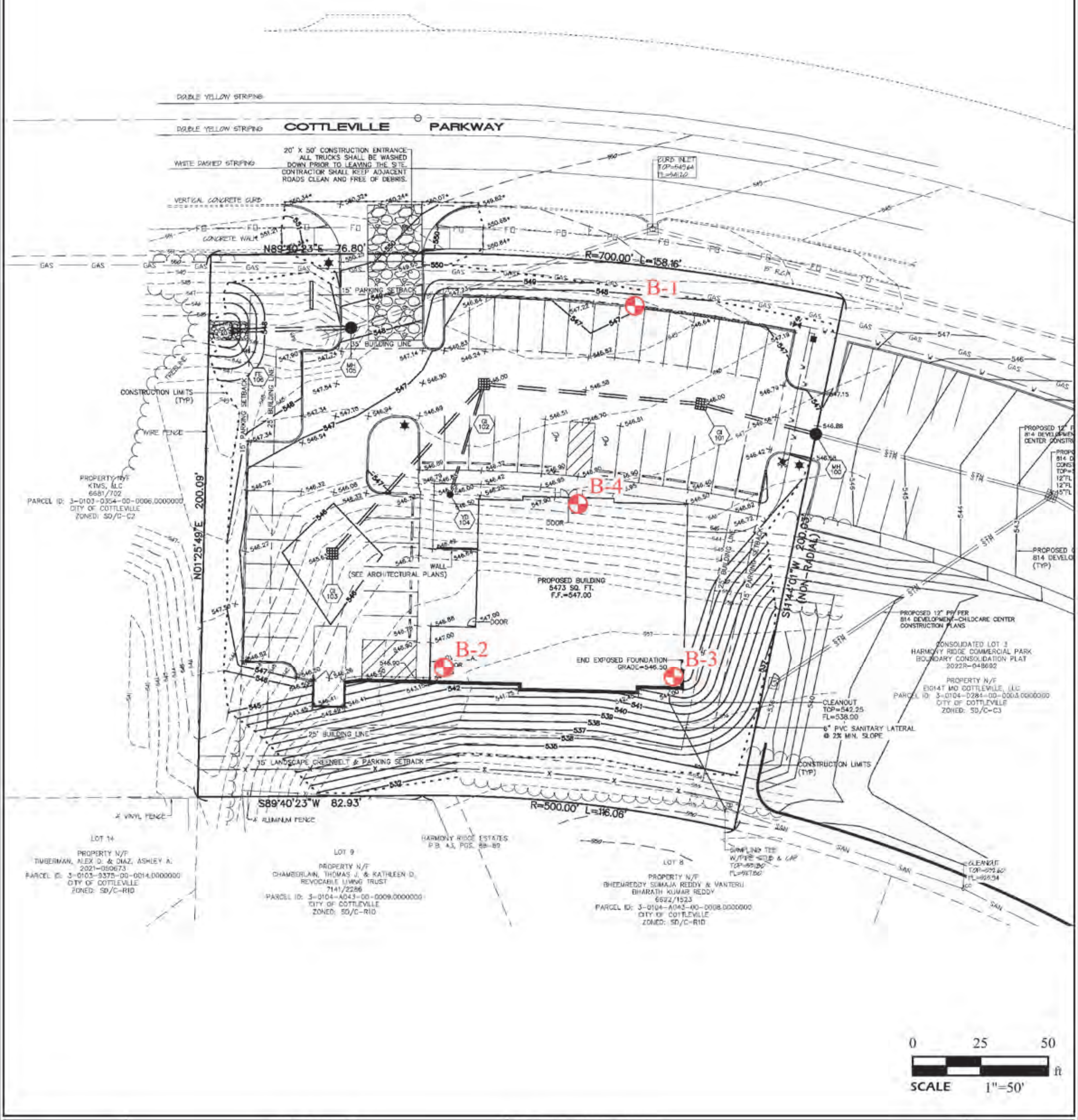
FIGURE
1

PROJECT NAME			GENERAL NOTES/LEGEND	
MAJESTIC DENTAL – COTTLEVILLE PARKWAY COTTLEVILLE, MISSOURI			APPROXIMATE SOIL BORING LOCATIONS	
AERIAL PHOTOGRAPH			AERIAL PHOTOGRAPH OBTAINED FROM ARCGIS ONLINE, WORLD IMAGERY.	
DRAWN BY ACV	DATE	JOB NUMBER	DIMENSIONS AND LOCATIONS ARE APPROXIMATE; ACTUAL MAY VARY. DRAWING SHALL NOT BE USED OUTSIDE THE CONTEXT OF THE REPORT FOR WHICH IT WAS GENERATED.	FIGURE
CHECKED BY PP	02/2023	2023-0091.10		2



PROJECT NAME			**GENERAL NOTES/LEGEND**
MAJESTIC DENTAL – COTTLEVILLE PARKWAY COTTLEVILLE, MISSOURI			APPROXIMATE SOIL BORING LOCATION
SITE PLAN			PLAN DATED 01/20/2023 BY RCK.
DRAWN BY ACV	**DATE**	**JOB NUMBER**	DIMENSIONS AND LOCATIONS ARE APPROXIMATE; ACTUAL MAY VARY. DRAWING SHALL NOT BE USED OUTSIDE THE CONTEXT OF THE REPORT FOR WHICH IT WAS GENERATED.
CHECKED BY PP	02/2023	2023-0091.10	

FIGURE
3



PROJECT NAME			GENERAL NOTES/LEGEND
MAJESTIC DENTAL – COTTLEVILLE PARKWAY COTTLEVILLE, MISSOURI			APPROXIMATE SOIL BORING LOCATION
GRADING PLAN			PLAN DATED 01/20/2023 BY RCK.
DRAWN BY ACV	DATE	JOB NUMBER	DIMENSIONS AND LOCATIONS ARE APPROXIMATE; ACTUAL MAY VARY. DRAWING SHALL NOT BE USED OUTSIDE THE CONTEXT OF THE REPORT FOR WHICH IT WAS GENERATED.
CHECKED BY PP	02/2023	2023-0091.10	

FIGURE 4

Appendix A

SCI ENGINEERING, INC.
130 Point West Boulevard
St. Charles, Missouri 63301
636-949-8200
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 Majestic Dental - Cottleville Parkway **BORING NUMBER** B-1
LOCATION Cottleville, Missouri **SHEET** 1 **of** 1
DRILLER Midwest Drilling, Inc. **HAMMER** Automatic **PROJECT NO.** 2023-0091.10
EQUIPMENT CME-750 w/CFA **ELEVATION** 549± **DATE DRILLED** 02/09/2023

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									
	1	SS	15/18	2 2 3				24		3.5				
3					FAT CLAY (CH): Brown									546
	2	SS	15/18	2 3 5				18		8.0		60	38	
6														543
	3	SS	18/18	2 3 4				26		5.0				
9	4	SS	18/18	2 3 4				22		4.5				540
					Boring terminated at 10 feet.									
12														537
15														534
18														531

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

REMARKS:

BORING LOG

PROJECT Majestic Dental - Cottleville Parkway
LOCATION Cottleville, Missouri
DRILLER Midwest Drilling, Inc. **HAMMER** Automatic
EQUIPMENT CME-750 w/CFA **ELEVATION** 537±
BORING NUMBER B-2
SHEET 1 **of** 1
PROJECT NO. 2023-0091.10
DATE DRILLED 02/09/2023

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)
					FAT CLAY (CH): Brown									
	1	SS	18/18	4 4 4				24		7.0				
3					With fine to coarse gravel									534
	2	SS	10/17	3 5 50/5"				29		3.0				
					Auger refusal on limestone at 5 feet.									
6														531
9														528
12														525
15														522
18														519

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

REMARKS:

BORING LOG

PROJECT Majestic Dental - Cottleville Parkway
BORING NUMBER B-3
LOCATION Cottleville, Missouri
SHEET 1 **of** 1
DRILLER Midwest Drilling, Inc.
HAMMER Automatic
PROJECT NO. 2023-0091.10
EQUIPMENT CME-750 w/CFA
ELEVATION 536±
DATE DRILLED 02/09/2023

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)
					FAT CLAY (CH): Brown									
	1	SS	18/18	2 3 4				24		3.5				534
3														
	2	SS	18/18	3 4 5				29		7.5				531
					With fine to coarse chert gravel									
6	3	SS	4/7	8 50/1"				34		5.5				
					Auger refusal on limestone at 7 feet.									
														528
9														
														525
12														
														522
15														
														519
18														

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

REMARKS:

BORING LOG

PROJECT Majestic Dental - Cottleville Parkway **BORING NUMBER** B-4
LOCATION Cottleville, Missouri **SHEET** 1 **of** 1
DRILLER Midwest Drilling, Inc. **HAMMER** Automatic **PROJECT NO.** 2023-0091.10
EQUIPMENT CME-750 w/CFA **ELEVATION** 546± **DATE DRILLED** 02/09/2023

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, trace organics									
	1	SS	18/18	2 3 5				20		6.0		37	12	
3					FILL: Brown, fat clay, trace organics									543
	2	ST	24/24					17	90		3.4	58	36	
6					LEAN CLAY (CL): Brown									540
	3	SS	10/18	6 7 8				20		5.0				
9	4	SS	12/18	5 7 7				10		7.5				537
					FAT CLAY (CH): Reddish-brown, trace fine to coarse chert gravel									
12														534
	5	SS	2/2	50/2"	With coarse gravel			21		5.0				
					Auger refusal on limestone at 14 feet.									
15														531
18														528

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.

ASFE
THE BEST PEOPLE ON EARTH

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