GEOCAL, INC.
GEOSCIENCES & ENGINEERING
7290 South Fraser Street
Centennial, Colorado 80112-4286

SOIL & FOUNDATION INVESTIGATION AND PAVEMENT DESIGN

Proposed

RTD-Sheridan Parking Structure
Northwest Corner of Sheridan Boulevard & West 10th Avenue
City and County of Denver, Colorado

Prepared For

Swinerton Builders
Attn: Dennis Ashley, AIA
6890 West 52nd Avenue
Suite 100
Arvada, CO 80002

April 4, 2012

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Prepared For

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1.0 PURPOSE AND SCOPE

This report contains the results of a soil and foundation investigation and pavement design conducted for the proposed Sheridan Parking Structure for the RTD West Corridor LRT, to be located northwest of the intersection of West 10th Avenue and Sheridan Boulevard in Denver, Colorado. A subsurface investigation was conducted to obtain information on soil, bedrock, and ground water conditions. Soil and bedrock samples collected were visually classified by our project engineer and selected samples were laboratory tested to evaluate engineering properties. A preliminary geotechnical investigation was conducted for the Parking Structure by Rocksol Consulting Group (dated July 8, 2011). The findings of that report have been used to help develop the recommendations contained herein.

The results of the field and laboratory investigations were evaluated to develop recommendations for foundation types, depths, and allowable pressures for the proposed parking structure, and pavement designs for the parking structure access roadway, the reconstruction of the Sheridan Boulevard and West 10th Avenue intersection, and the portion of West 10th Avenue extending west of the intersection to the parking structure access roadway. This report has been prepared to summarize the data obtained and to present our conclusions and recommendations based on the proposed construction and subsurface conditions encountered. Environmental considerations related to hazardous materials are beyond the scope of this study. The investigation was conducted in general accordance with our proposal to Swinerton Builders, dated August 30, 2011.

2.0 PROPOSED CONSTRUCTION

Based on information provided by Swinerton Builders, we understand that the proposed construction will include the following: construction of a 4-story parking garage, utilizing cast-in-place (post-tensioned) slabs and concrete columns and beams, supported by a drilled shaft foundation bearing on the underlying bedrock. The structure will accommodate approximately 800 parking spaces and will be about 38,000 square feet in plan area. Column loading is not expected to exceed 1,200 kips and slab-on-grade loads are not anticipated to exceed 200 pounds per square foot (psf).

Access to the parking structure will be at or near existing grade at the south end, but the structure will have a lower level (designated Level 0). Additional site improvements will include a storm water detention basin located north of the structure, landscaping, and pedestrian access from the RTD West Corridor Line platform to the north. Maximum excavation cut depths below the existing ground surface are expected to about 25 feet deep to accommodate the basement level parking. Ames Street will be repositioned about 100 feet west of the existing alignment, and a driveway off Ames Street will service the structure. Ames Street is not expected to extend north to 11th Street. West 10th Avenue may be widened and the intersection of West 10th Avenue and Sheridan Boulevard will be improved. Planned pavement grades are expected to be similar to existing.

3.0 SITE CONDITIONS

The project site is located at the northwest corner of the intersection of West 10th Avenue and Sheridan Boulevard, about 400 feet south of Dry Gulch. The site is situated on a gently sloping uplands terrace that has been modified by grading for the construction of roadways and commercial developments. At the time of our field work, the site has been stripped of most vegetation although there was a sparse growth of weeds and trees to the north end. Stockpiles of soil and aggregate also existed. The surrounding neighborhoods mainly consisted of apartment buildings and single family residences. We understand that the site had previously been occupied by a large apartment complex that was demolished as part of the West Corridor LRT project.

Published quadrangle scale geologic mapping assigns the original unconsolidated surficial soils at the southern portion of the site as upper terrace deposits of Verdos Alluvium (variably mixed clay, silt, and sand with gravel). These native soils in the area are now mostly covered by artificial fill (from site development). Surficial soils in the northern portion of the site are Piney Creek Alluvium, described as well stratified interbeds of sand, silt, and clay, frequently humic, with gravels near the base. Bedrock is indicated to be at relatively shallow depth and assigned to sedimentary members of the Denver-Arapahoe Formations undifferentiated; predominantly interbedded claystone, siltstone, and sandstone, dipping very gently to the east.

4.0 SUBSURFACE INVESTIGATION

The subsurface investigation for this project was conducted on March 1st and 2nd, 2012, by drilling ten exploratory borings at the approximate locations shown on Figure 1, Locations of Exploratory Borings. Five of those borings (S1 through S5) were drilled within the proposed building area and were extended into bedrock. The remaining borings were drilled to 10 feet deep for pavement design purposes. Drilling was conducted using a truck-mounted D-50 drill-rig equipped with 4¼ inch inside diameter hollow-stem augers and 4 inch outside diameter solid stem augers. The borings were logged by a Geocal representative.

Subsurface soils were obtained using 2 inch ID California liner samplers and 1⅜ inch ID split-spoon (Standard Penetration Tester) samplers. The samplers were driven into the various strata with blows from a 140 pound hammer, similar to ASTM D1586 test standard. Penetration resistance values, when properly evaluated, indicate the relative consistency or density of the soils, or bedrock hardness. Drive samples were taken at approximately five foot intervals. Bulk samples of auger cuttings were collected from about the upper 1 foot to 5 feet of each boring. Logs of the conditions encountered are shown on Figure 2 and description of the materials encountered and symbols used are presented on Figure 3.

5.0 SUBSURFACE CONDITIONS

In general, the structural borings (Borings S1 through S5) encountered 5 feet to 15 feet of artificial fill, consisting of variable clay with sand and gravel, to silty or clayey sand that was stiff to very stiff or medium dense to dense, with fine to coarse grained sand and low plasticity fines. Below artificial fill, the borings encountered mixed natural sand that was silty to clayey with some gravel in parts, over natural medium to high plasticity sandy clays and clayey sands. The natural clays were stiff to very stiff, whereas the sands were dense. The soils were moist and ranged from brown or dark brown for the fill to light brown or brown for the natural soils. Claystone bedrock was encountered from about 10 feet to 28 feet below the ground surface at the north and south ends of the site, respectively, and ranged from approximate elevations 5,327 to 5,336. Bedrock was mostly hard to very hard, moist, olive gray to reddish brown, and high plasticity. The upper 1 to 4 feet of claystone bedrock was weathered in a number of the borings. Ground water was not encountered in the borings during drilling. However, three structural borings (Borings S3 through S5), were left open for a follow up water level check three days after drilling, and ground water was measured in Boring S3 at a depth of 38 feet. Ground water levels can be expected to fluctuate with varying seasonal and weather conditions.

The borings drilled for pavement design purposes (Borings P1 through P5) were extended to a total depth of 10 feet. Borings P3 through P5, encountered between 5 inches to 8 inches of asphalt pavement overlying a thin (2 to 3 inch) section of aggregate base course. In general, the borings encountered about 1 foot to 3 feet of artificial fill under the pavement section or at the surface. The artificial fill was generally comprised of sandy clay with some gravel that was stiff to very stiff, moist, with low plasticity and was light brown to brown. Below the artificial fill were natural clay soils which extended to total depth explored, 10 feet. The clay was sandy, very stiff with isolated soft areas, moist, low to medium plasticity, and light brown. Borings drilled in West 10th Avenue and Sheridan Boulevard were backfilled with a mix of pea-gravel and sand and compacted by the weight of the drill rig, then patched with 8 inches of compacted cold mix asphalt. All other holes were backfilled with compacted auger cuttings.

The conditions encountered in the exploratory borings were generally consistent with those described in the Rocksol preliminary report and from borings conducted by Geocal for the West Corridor geotechnical study for the Sheridan Bridge.

6.0 LABORATORY TESTING

Laboratory tests conducted on soil and bedrock samples consisted of natural moisture and density, Atterberg Limits, swell-compression, gradation, moisture-density relationship, unconfined compressive strength and, water-soluble sulfate concentrations. The results of the laboratory testing are shown on Figures 4 through 19 with a summary on Table 1.

Swell-Consolidation Tests: Swell-compression tests (ASTM D4546) were conducted on samples of the sandy clay soils and claystone bedrock to evaluate compressibility or swell characteristics under loading and wetting. The samples were placed in an odometer ring between porous discs and light surcharge load was applied. After stabilization, the samples were submerged and the percent volume change was measured. Subsequent loads were applied and the change monitored until deformation practically ceased under each load. The swell-compression test results are shown on Figures 4 through 8. The results indicate that the natural soil samples tested had low to moderately high swell potential under light load and wetting, and moderate compressibility under increased loading.

The claystone bedrock samples tested showed low swell potential under light loading and wetting, and low to moderate compressibility under increased loading. Based on previous work in the area, and the RockSol test results from the preliminary report, the claystone bedrock can be expected to have moderate to high expansive potential under light load and wetting.

Gradation Analyses and Atterberg Limits: These tests were used to classify the soils in accordance with the American Association of State Highway and Transportation Officials (AASHTO) classification system and Unified Soil Classification System (USCS). These classifications provide qualitative information on the soil suitability for use in engineering application. The results indicate low to high plasticity for the clay soils with Unified Soil Classifications of SC (clayey sand), CL (sandy lean clay), or CH (sandy fat clay). In general, the AASHTO soil classifications ranged from A-6(3) for the clayey sands, to A-7-6 for the sandy lean clays, with Group Indices from 3 to 20. The soils tested had medium to high plasticity. Samples of the claystone also indicate high plasticity and high fines content.

Test results indicate that the soils encountered are highly variable, with some high plasticity and potentially expansive clays overlying claystone bedrock that is also high plasticity and known to be expansive. Gradation and Atterberg Limits test results are shown on Figure 9.

Moisture-Density Relationship: This test is done to evaluate the density variation that occurs with a particular sample under different moisture contents using the same compaction effort. The results (Figures 10 through 12) indicate sandy lean clays encountered have maximum dry densities (Proctor) and optimum moisture contents values of: 101.3 pcf at 18.7%, and 103.4 pcf at 17.1%. The results from a sample of clayey sand indicate a maximum dry density of 113.3 pcf at an optimum moisture content of 13.6%, which has been corrected for gravel content to a maximum dry density of 119.0 pcf with an optimum moisture content of 11.4%. The sandy lean clay test results were used to provide remolding criteria for unconfined strength measurement.

Unconfined Compressive Strength: Unconfined compressive strength testing was conducted on remolded samples of the sandy lean clay (A-7-6 soils), clayey sand with gravel (A-6(4)), and California liner samples of claystone bedrock. Remolded soil samples were compacted to about 95% of the maximum dry density and near 2% above optimum moisture content. Loads were applied continuously and without shock to produce a constant rate of deformation so that failure occurred within 5 minutes to 15 minutes of loading. The unconfined strength test results, and corresponding stress-strain curves, are shown on Figures 13 through 19. The results indicate unconfined strength values of 1,660 psf to 2,540 psf for the remolded sandy lean clay samples. Unconfined compressive strength of the claystone varied from 3,920 psf to 18,060 psf.

Water-Soluble Sulfates: The water-soluble sulfate test is a measurement of the potential degree of sulfate attack on concrete exposed to the onsite soils. Sulfate solutions react with tricalcium aluminate hydrate, a normal constituent of Portland Cement concrete, forming calcium sulfo-aluminate hydrate with an accompanying expansion in volume. Sulfate expansion problems are typical of soils with a sulfate concentration in excess of 0.10%. The levels of water soluble sulfates contained in samples of soils tested were 0.01% to 0.03%, indicating negligible to low level of potential sulfate attack on concrete exposed to the onsite soil. Based on Table 4.3.1 of the American Concrete Institute (ACI) 318-05 "Requirements of Concrete Exposed to Sulfate Bearing Solutions", the potential sulfate exposure indicates that Type I/II cement should be considered for concrete exposed to the onsite soils.

7.0 FOUNDATION RECOMMENDATIONS

7.1 Drilled Shafts

Based on the proposed construction and the subsurface conditions encountered, a drilled shaft foundation is recommended for support of the parking garage structure. The following design and construction recommendations should be observed.

1. Drilled shafts should be designed for a maximum allowable end bearing pressure of 35,000 psf and side shear of 3,500 psf for that portion of shaft in unweathered claystone bedrock. For drilled shafts that extend below an elevation of 5,320 feet and meet the minimum bedrock penetration and minimum length requirements, a maximum allowable end bearing pressure of 45,000 psf and side shear capacity of 4,500 psf may be used for design. An allowable side shear value of 3,000 psf, plus the weight of drilled shaft, may be assumed for uplift resistance.

2. Due to potential weathering, the upper three feet of bedrock penetration should be neglected for side shear resistance.

3. Settlement of properly constructed drilled shafts is expected to be ½ inch or less.

4. Drilled shafts should be designed for a minimum dead load pressure of 10,000 psf, based on the shaft cross section area only. If the minimum dead load requirement cannot be achieved, then the shaft length should be extended beyond the minimum bedrock penetration to make up the dead load deficit. This can be accomplished by assuming the uplift resistance value given above acts in the direction to resist uplift.

5. Some variation in the bedrock surface should be anticipated. Drilled shafts should penetrate at least 8 feet into competent bedrock and have a minimum length of 12 feet. These are geotechnical parameters. Greater penetration depths may be needed based on the structural requirements.

6. Drilled shafts should be designed with additional reinforcement over their full length to resist an unfactored net tensile force from expansive soils/bedrock of 20 kips. The net tensile force is from expansive bedrock in the upper 8 feet of shaft length which represents the estimated zone of influence for the expansive materials. The tensile force is for a one foot diameter shaft, and is applied along the circumference of the shaft along the upper 8 feet of bedrock penetration due to expansion of the bedrock. The value should be corrected for other shaft diameters. The tensile force may be reduced by the dead load on each shaft.

7. The minimum spacing requirements between drilled shafts should be 3 diameters from center to center. At this spacing, no reduction in axial design parameters is required. Drilled shafts grouped less than 3 diameters center to center should be studied on an individual basis to evaluate the appropriate reduction in axial capacity. Lateral capacity parameters are provided in Section 7.2.

8. Drilled shaft holes should be properly cleaned prior to placement of reinforcing steel or concrete. A maximum length to diameter ratio of 25 is recommended to facilitate cleaning and observation of the shaft hole.

9. Concrete utilized in the drilled shafts should be a fluid mix with sufficient slump so it will fill the voids between reinforcing steel and the shaft hole. Concrete with a slump in the range of 5 inches to 7 inches is recommended.

10. The presence of water and some isolated granular soils encountered in the exploratory borings indicates that casing may be required to reduce water infiltration and to help control caving in some of the shafts. In some cases, the requirements for casing can sometimes be reduced by placing concrete immediately upon cleaning and observing the pier hole. If water cannot be removed prior to placement of concrete, then concrete should be placed using an approved tremie method. In no case should concrete be placed through more than 2 inches of water and only after the hole has been well cleaned and approved.

11. If shaft holes are cased, a sufficient head of concrete should be maintained inside the casing during casing extraction to help reduce the potential for voids being formed in the concrete upon casing removal. The concrete should not be allowed to rise during the casing removal. If it becomes apparent that voids may have formed during shaft installation, the contractor should be required to perform non-destructive tests to evaluate the continuity and integrity of the shaft. Tests may include sonic echo tests or other tests.

12. Bedrock penetration should be measured down from the bottom of the casing or top of competent bedrock, whichever is the lower elevation.

13. Concrete should be placed in the holes the same day they are drilled and the presence of water will require that concrete be placed immediately after the shaft hole is completed. Failure to place concrete the day of drilling will result in degradation of the bedrock capacity and a requirement for additional bedrock penetration. The amount of additional bedrock penetration will be a function of how long the hole is left open and whether or not water accumulates during the inactive period. If holes are drilled into bedrock and left open over-night, this office should be contacted for additional bedrock penetration requirements.

14. Care should be taken to prevent forming mushroom shapes at the top of the drilled shafts. If caving is excessive, the contractor should be required to use slurry, sonotube, or other methods to protect the integrity of the hole.

15. The drilling contractor should mobilize equipment of sufficient size and operating condition to penetrate the materials and to achieve the required bedrock penetration.

16. To help reduce potential differential movement between the main structure and elevator pit, we recommend that the elevator pit also be supported by drilled shafts.

17. Installation of drilled shaft operations should be observed by Geocal personnel on a full-time basis.

7.2 Lateral Capacity Parameters

The following recommended lateral capacity parameters are based on the structural engineer using the computer program LPILE for lateral load analysis. Data presented below is based on our judgment and the user and technical manuals for LPILE Plus 4.0.

Lateral Capacity Parameters for Drilled Shaft Foundation

Material Type	Total Unit Weight (pcf)	Cohesion C (psf)	Friction angle degrees θ	k-static (pci)	k-cyclic (pci)	ε_{50}
Onsite sandy clays to clayey sand soil	115	500	0	100	--	0.010
Claystone Bedrock	120	4,000	0	2,000	600	0.005

ε_{50} = strain at 50% of peak strength

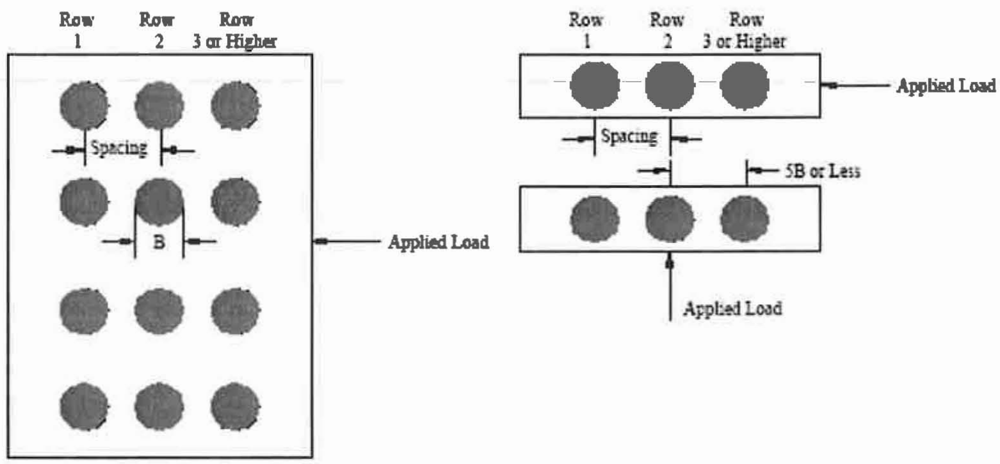
The unit weight of water should be subtracted from the total weight for the submerged or potentially submerged condition. Bedrock may be modeled as hard clay.

Reductions in lateral capacity for loading perpendicular to the line of shafts will not be required if center to center spacing of 5 shaft diameters or more between adjacent drilled shafts is maintained. For lateral loads parallel to the line of shafts, reduction in lateral capacity is necessary at a spacing less than 6 diameters. LPILE uses p-multipliers to account for reduced capacity of closely spaced drilled shafts or piles for loading in either direction. Data presented below are from the 2007 AASHTO LRFD Bridge Design Specifications 4th Edition Manual. A sketch of the loading and how the rows are referenced is shown.

P-Multipliers
Drilled Shaft Foundation

	p-multiplier for LPILE		
Center to Center Spacing	Row 1	Row 2	Row 3 and Higher
3B	0.7	0.5	0.35
4B	0.85	0.67	0.52
5B	1	0.85	0.70

B= Diameter of Shaft



8.0 RETAINING STRUCTURES

Earth pressures against below grade walls are a function of the material type, compaction, moisture, drainage, and lateral movement. Foundation walls, basement walls, elevator pit walls, and retaining structures that are laterally supported and can be expected to undergo only a slight amount of deflection should be designed for lateral earth pressures based on the "at-rest" earth pressure condition. Cantilevered or gravity retaining structures which rotate and/or deflect sufficiently to mobilize the internal soil strength of the wall backfill may be designed for the "active" earth pressure condition.

The onsite soils encountered at the Sheridan Parking Structure site consisted of artificial fill and natural clayey sands or sandy clays having medium to high plasticity. Fine grained soils (clays and soils with high clay content) typically produce excessive earth pressures on walls and are not considered suitable for use as retaining wall backfill. Therefore the majority of the onsite soils may not be suitable for use as wall backfill.

The following ultimate earth pressure coefficients are recommended for the imported granular material and onsite soils. The following values assume backfill placement and compaction to at least 95% of the maximum standard Proctor density at moisture contents within 2% of optimum.

Material or location	Active (K_a)	At-Rest (K_o)	Passive (K_p)	γ_T – Unit Weight (pcf)	Friction Angle (ϕ), degrees
Imported Granular Soils	0.28	0.44	3.54	130	34
Onsite Soils	0.70	0.83	1.42	125	10

Imported granular soils should meet the following gradation:

Sieve Size	Percent Passing
2 inch	100
No. 4	30 - 100
No. 50	10 - 60
No. 200	5 - 20

For building wall backfill, the backfill should be capped with onsite relatively impervious clays in the upper three feet to help reduce the infiltration of surface water.

Lateral wall movements or rotation of at least 0.5% of the wall height is typically required to develop the full active case, whereas lateral movement of at least 1% of the wall height is normally required to establish the full passive case assuming granular backfill. Suitable factors of safety should therefore be applied to the above ultimate values to limit strain needed to reach ultimate strength, particularly with passive resistance where large strains are needed to mobilize full resistance. Equivalent fluid unit weights should be taken as follows:

Above ground water: $\gamma_{eq} = \gamma_T \times K_{a,o,p}$

Below ground water: $\gamma_{eq} = (\gamma_T - 62.4) \times K_{a,o,p}$

where: γ_T = soil total unit weight

$K_{a,o,p}$ = appropriate earth pressure coefficient

The above parameters are for a horizontal backfill and no surcharge loading. Foundation and retaining structures should be designed for appropriate surcharge pressures such as from traffic, upsloped backfill, water buildup behind the wall, or other external loadings that will increase the lateral pressure on the wall. An under-drain should be provided to prevent hydrostatic pressure buildup unless the wall is designed to accommodate the additional pressure. Care should be taken not to over compact the backfill or use large equipment adjacent to the wall which could cause excessive lateral loading. Retaining walls and other major structures should be supported by drilled shafts as described in Section 7.0, Foundation Recommendations.

9.0 Seismic Design Parameters

Due to the presence of relatively shallow bedrock that is hard and generally expected to increase in hardness with depth, the Site Class can be increased to a C (dense/very stiff soil – soft rock). Utilizing 2009 International Building Code requirements, the following site factors may be utilized for design:

Site Class	C
S_s, Site Class B (0.2 Second Period)	0.224 g
S_1, Site Class B (1.0 Second Period)	0.057 g
SM_s, Site Class C (0.2 Second Period)	0.269 g
SM_1, Site Class C (1.0 Second Period)	0.098 g
SD_s, Site Class C (0.2 Second Period)	0.179 g
SD_1, Site Class C (1.0 Second Period)	0.065 g
Fa	1.2
Fv	1.7

We have included printouts of the USGS seismic design parameter determination program in Appendix A.

10.0 Slab-on-Grade Construction

The onsite soils and bedrock encountered in the proposed parking structure area are expected to have high expansive potential when the materials are subjected to light load and wetting. The materials expected to be exposed at the bottom of the excavation for the lower parking level (near elevation 5331 feet) are expansive claystone bedrock and clays. The claystone has the potential to have high volume change characteristics depending on the depth of wetting that occurs, and very high swell pressures. The amount of dead load pressure imposed by the slab to the bedrock will not be sufficient to resist the uplift pressure generated when the bedrock becomes wet and expands. Slab differential and total movements could be many inches.

On this site, slab-on-grade construction carries high risk that the amount of slab total and differential movement caused by expansion of the bedrock will be unacceptable. A positive way to reduce the risk of slab movement is to construct a structurally supported floor over a well-ventilated crawl space. A structural floor system is therefore recommended.

However, if the high risk of distress resulting from floor slab movement is recognized and acceptable to the owner, then slab-on-grade construction may be considered in lieu of a structural floor. The following recommendations are provided for slab-on-grade construction.

1) Floor slabs should be supported by at least 3 feet of relatively impervious, non-expansive structural fill. Relatively impervious non-expansive structural fill material should meet the following specification:

   30% to 60% passing the No. 200 sieve
   Liquid Limit of 20 or less
   Plasticity Index of 10 or less

   The majority of the on-site soils are not expected to meet the above specification and the material will have to be imported. New imported structural fill should be placed in uniform lifts, moisture conditioned to within 2% of optimum moisture, and compacted to at least 95% of the maximum Proctor density as defined by ASTM D 698.

2) Prior to placing new structural fill the bottom of the excavation should be uniformly scarified, moisture conditioned to within 2% of optimum moisture, and compacted to at least 95% of the maximum Proctor density as defined by ASTM D 698.

3) Bedrock material excavated should be wasted or used in non-structure areas, such as in the detention pond or in landscaping.

4) Floor slabs should be separated from bearing walls and columns with an expansion joint which allows unrestrained vertical movement.

5) Interior partitions resting on floor slabs should be provided with a slip joint at the bottom so that if the slabs move, the movement cannot be transmitted to the upper structure. This detail is also important for wallboards and doorframes. A slip joint which will allow at least 4 inches of vertical movement is recommended. Floor slabs should be provided with control joints to reduce damage due to shrinkage cracking, and the slabs should be adequately reinforced. Joints should be provided based on the design and intended slab use.

The precautions outlined above will not prevent movement of the slab-on-grade if the underlying expansive bedrock or soils are subjected to wetting, but will help reduce the amount of damage if slab heave occurs.

11.0 Under-drain System

Below grade structures should be provided with an under-drain system which will help prevent the buildup of hydrostatic pressures. The under-drain system should consist of a perforated PVC pipe surrounded by free draining granular material placed at the bottom of the wall backfill and sloped at a minimum 1% grade to a suitable gravity outlet or to a sump with pump to remove the water. Granular material should meet the following gradation.

Sieve Size	Percent Passing
1½ Inch	100
No. 4	20-60
No. 16	10-30
No. 50	0-10
No. 200	0-3

12.0 Surface Drainage

The following drainage precautions should be observed during construction and maintained at all times after the facility has been completed:

- Excessive wetting or drying of the building excavation, exterior flatwork and pavement areas should be avoided.
- The ground surface surrounding the exterior of the parking structure should be sloped to drain away from the building in all directions. A minimum slope of 6 inches in the first 10 feet for unpaved areas and a minimum slope of 3 inches in the first 10 feet for paved areas are recommended.
- Roof downspouts and drains should discharge well beyond the limits of building backfill.
- Landscaping that requires excessive watering and lawn sprinkler heads should be located at least 10 feet from the foundation walls.
- Plastic membranes should not be used to cover the ground surface adjacent to foundation walls.

13.0 Site Grading

The contractor should remove all existing artificial fill from within the building area to a distance of at least five (5) feet beyond the planned building limits. If slab-on-grade is proposed, then the bottom of the building excavation should be sub-excavated at least 3 feet below the bottom of the slab as described in Section 10 Slab-on-Grade Construction. Determination of the extent of the artificial fill should be made by a representative of Geocal. The sub-excavation should be filled with new structural fill meeting requirements outlined in Section 10. Some sub-excavation of the existing artificial fill may be needed for access drives and paved areas to meet the pavement subgrade strength requirements and to help reduce the effects of expansive soils.

The project is anticipated to have maximum cuts depths of 23 feet to 25 feet to achieve the first floor level of the parking structure. Excavation of the onsite materials should be possible with conventional heavy duty excavating equipment. For shallow excavations, the majority of the material expected to be encountered are clayey sands and sandy clays, both with variable amounts of gravel. Because of the age and history of the site however, some construction or other debris may also be included in the old fill. For deeper excavations such as in the building area, claystone bedrock will likely be intercepted at the bottom. In the building area, the excavated soils are expected to have medium to high plasticity and the bedrock is expected to have high plasticity. Some of the soils may be suitable for use in exterior flat-work and pavement areas. However the quantity of re-useable soils is expected to be limited. Claystone bedrock and/or high plasticity soils should not be used in structure or pavement areas, but may be suitable for use in constructing the detention pond and/or in landscape areas. Debris and debris ridden soils should be wasted. During excavation a representative of this firm should be on-site to help identify the suitability of the material encountered.

Permanent un-retained cuts in the overburden soils or fill slopes up to 5 feet high should be constructed no steeper than 3:1 horizontal to vertical grade unless evaluated individually. The risk of slope instability will be significantly increased if seepage is encountered in cuts. Good surface drainage should be provided around permanent cuts to direct surface runoff away from the slope face. Cut slopes and other stripped areas should be protected against erosion by vegetation or other methods.

If sloped excavations are used for utility trenches, stockpiled material should be kept at least a distance equal to the height of the cut away from the top of the excavation. Sloped excavations should conform to applicable OSHA regulations, and the contractor should assume responsibility for excavations that are safe for workers. Soils encountered in the area are classified as Type C by OSHA requirements. Excavations exceeding 4 feet in depth should be designed and monitored by the contractor's competent personnel.

Ground water was encountered in the bedrock during our field work, but may be higher or lower at the time of construction. The contractor should be prepared to dewater excavations that are expected to approach the groundwater table such as for deep utility trenches. Ground water was encountered during our field work at about elevation 5,318 feet. The ground water level can be expected to fluctuate and could be higher during construction.

Compaction Recommendations: The following compaction specifications are recommended based on the percentages of the maximum Standard Proctor Density (ASTM D698). Fill should be moisture conditioned to within 2 percent of optimum moisture content.

Soil Use	Minimum Compaction Requirements
Bottom of sub-excavated areas	95%
Fill to support building slab-on-grades	95%
Foundation wall backfill (non-structural areas)	95%
Exterior flatwork subgrade	95%
Utility trenches beneath slabs/pavements	95%
Utility trenches in landscaping and other areas	90%
Detention Pond	95%

Exterior Flatwork Areas: The subgrade for exterior flatwork (sidewalks and pedestrian areas) should be scarified a minimum of 12 inches, moisture conditioned to within 2 percent of optimum moisture content, and compacted to at least 95% of the Maximum Standard Proctor density. New fill should be compacted to the same specification. Any debris, soft or loose soils, or high plasticity soils should be removed areas and replaced with non-expansive granular soils. Prepared subgrade should be proof-rolled with at least a 40,000 pound water or dump truck prior to placing concrete or other pavement materials. Loose or soft zones identified should be sub-excavated and replaced with compacted fill, then proof-rolled again.

Storm Water Detention Pond: The soils and bedrock encountered near the detention pond consisted of sandy clay with gravel, extending to claystone bedrock at a depth of about 9½ feet. The bottom of the proposed pond is expected to be near elevation 5,330 feet. At that elevation, the bottom of the pond and side slopes are expected to be artificial fill (clays). The artificial fill and underlying claystone bedrock are expected to be relatively impervious and have very low percolation rates. Embankments constructed of the onsite soils are also expected to be relatively impervious.

Slopes for the detention pond should be no steeper than 3:1 horizontal to vertical grade. Material used to construct the pond slopes may consist of the onsite clays. Prior to placement of new embankment the area should be prepared by removing vegetation, uniformly scarifying the onsite soil, moisture conditioning to within 2% of optimum and compacting to at least 95% of the maximum Standard Proctor Density, as defined by ASTM D 698. New fill shall be placed in uniform lifts not exceeding 8 inches thick, moisture conditioned, and compacted to the same specifications.

14.0 Pavement Design

We understand that new pavements will be constructed for the intersection of Sheridan Boulevard and West 10th Avenue, for West 10th Avenue west of Sheridan, and the access driveway southwest of the structure. Peak hour traffic counts for 2010, 2013, and 2035 for Sheridan Boulevard and side streets in the project area were obtained from the traffic study performed by Apex Design for RTD and were provided by the client. Printouts from traffic study are included in Appendix B.

For the intersection of Sheridan Boulevard and West 10th Avenue, right turn traffic was neglected and only through traffic and left turn traffic counts were used in Average Daily Traffic (ADT) calculations. The corresponding ADTs for Sheridan Boulevard and West 10th Avenue were added together to obtain a representative ADT value for the intersection.

For design of the parking structure driveway pavement section, it was assumed that traffic would consist of passenger vehicles only (i.e. cars, SUVs, pickup trucks). We understand that the peak traffic counts for the AM and PM hours were estimated to be 272 and 314 for initial year of operation (2013) and that the peak traffic counts for horizon year (2035) were estimated to be 362 and 419, respectively. Using the average peak hour estimates, we assumed roughly 80% of the traffic volume per hour for non-peak hours and determined Average Daily Traffic (ADT) value for the initial year (2013) and the horizon year (2035), summarized in the table on the following page.

Location (Year)	Average Daily Traffic (ADT)
Parking Structure Driveway (2013)	5,626
Parking Structure Driveway (2035)	7,489
West 10th Avenue (2013)	10,800
West 10th Avenue (2035)	12,624
Sheridan & West 10th Ave. Intersection (2013)	69,552
Sheridan & West 10th Ave. Intersection (2035)	78,000

From the above data and for a 22 year period, we determined annual growths rates of 1.3% for the parking structure access drive, 0.7% for West 10th Avenue, and 0.5% for the intersection at Sheridan Boulevard and West 10th Avenue. It was assumed that construction would be complete and the improvements to West 10th Avenue, Sheridan Boulevard and the new driveway will be put into service by 2013. The 20 year Design ADT is the average ADT over the 20 year design life of the pavement and is used to calculate the 18 kip Equivalent Single Axle Load (ESAL). The following ADT values were used for the pavement designs, calculations are provided in Appendix B.

Location	20 yr Design Average Daily Traffic (ADT)
Parking Structure Driveway	6,463
West 10th Avenue	11,621
Sheridan Boulevard	73,353

The distribution of passenger vehicles, single unit and combination unit trucks for Sheridan Boulevard was obtained from the Colorado Department of Transportation (CDOT) website, and is summarized below. The same truck traffic distribution was assumed for West 10th Avenue.

Cars & Pickups	97%
Single Unit Trucks	2%
Combination Unit Trucks	1%

For Metropolitan Government Pavement Engineers Council (MGPEC) pavement design, combination unit truck traffic was equally distributed between trash/concrete trucks, RTD and School Busses, and light delivery trucks. The assumed traffic distribution is summarized in the following table.

Location	Vehicle Type	(% Traffic)
Parking Structure Driveway	Cars & Pickups	100%
West 10th Avenue	Cars & Pickups	97.00%
	Single Unit Truck	2.00%
	Trash / Concrete Truck	0.25%
	RTD Bus	0.25%
	Light Delivery Truck	0.25%
	School Bus	0.25%

Sheridan Boulevard	Cars & Pickups	97.00%
	Single Unit Truck	2.00%
	Trash / Concrete Truck	0.25%
	RTD Bus	0.25%
	Light Delivery Truck	0.25%
	School Bus	0.25%

For MGPEC pavement design, truck traffic was distributed and factored by the following vehicle equivalency factors.

Vehicle Type	MGPEC Equivalency Factor
Cars & Pickups	0.0045
Single Unit Trucks	1.587
Trash / Concrete Trucks	1.693
RTD Bus	3.848
Light Delivery Truck	0.617
School Bus	2.578

A design lane factor of 60% was applied to West 10th Avenue, and 45% was applied to the intersection of Sheridan Boulevard and West 10th Avenue. Applying MGPEC vehicle factors, the 20 year ESALs are summarized as follows (calculations are in Appendix B):

Location	20 yr Design ESAL ($ESAL_{20}$)
Parking Structure Driveway	127,376
West 10th Avenue	2,947,940
Intersection at Sheridan Boulevard & West 10th Avenue	13,965,747

Subgrade Soil Strength: Based on the laboratory test results, the subgrade materials classify as AASHTO A-7-6 for the clay soils. The unconfined compressive strength for the remolded sample obtained near the proposed Ames street was 1,660 psf, which was applied to the design of the driveway pavement. The unconfined compressive strength of the remolded sample obtained near the intersection was 2,540 psf, which was applied to the design of the pavement sections for West 10th Avenue and for the intersection of West 10th Avenue and Sheridan Boulevard. The resilient modulus (Mr) for design was determined by the following MGPEC equation: $Mr = 3.13 \times q_u$, where q_u is the remolded, unconfined strength. The resilient modulus was reduced by 25% because the subgrade soils are relatively impermeable, and we assumed that a pavement sub-drain will not be used. The corresponding resilient modulus of 5,963 psi was applied to the intersection and to West 10th Avenue, 3,897 psi was used for the driveway pavement design.

Pavement Thickness Recommendations: The pavement sections are based on laboratory test results and Metropolitan Government Pavement Engineers Council (MGPEC) design criteria and guidelines. Using the relevant parameters, MGPEC software calculated the following Portland Cement Concrete Pavement (PCCP) and Hot Mix Asphalt Pavement (HMAP) sections. Software printouts are in Appendix B along with the MGPEC Form #9.

Location	Pavement Type	Thickness (inches)
Parking Structure Driveway	PCCP doweled & tied	6.0
	PCCP no reinforcement	6.5
	HMAP full depth	6.5
	HMAP Composite (layered)	4.0 inches HMAP over 8 inches CSS
West 10th Avenue	PCCP doweled & tied	8.5
	PCCP no reinforcement	10.5
	HMAP full depth	11.0 (not recommended)
	HMAP Composite (layered)	7.5 inches HMAP over 12 inches CSS
Intersection at Sheridan Boulevard & West 10th Avenue	PCCP doweled & tied	10.5
	HMAP full depth	14.5 (not recommended)
	HMAP Composite (layered)	11.0 inches HMA over 12 inches CSS

Where: CSS is chemically stabilized subgrade

MGPEC design procedures do not provide specific recommendations for intersection pavement design. Due to the accelerating and decelerating flow of traffic surrounding intersections, the Colorado Department of Transportation (CDOT) pavement design manual recommends that an intersection pavement section be extended 300 linear feet away from the intersection, for each roadway carrying two-way traffic.

Hot Mix Asphalt Pavement (HMAP): HMAP materials should consist of a bituminous plant mix composed of a mixture of aggregate and bituminous material that meets the requirements of a job-mix formula established by a qualified engineer. The following grading and binder types are recommended for this project:

Top Lift	Grading SX (75) PG 64-22
Lower Lifts	Grading S (75) PG 64-22

Grading SX (75) PG 64-22 has a finer aggregate gradation and may be used for the top lift. This layer may help reduce surface water penetration and oxidation of the asphalt surface, which in turn may help reduce long-term maintenance. Mix design and construction should be performed in accordance with Item 9 of the MGPEC, Volume I - Pavement Design Standards & Construction Specifications.

Portland Cement Concrete Pavement (PCCP): PCCP pavements should also consist of an approved mix design by a qualified engineer, and in accordance with Item 11 of the MGPEC, Volume I - Pavement Design Standards & Construction Specifications.

Chemically Stabilized Subgrade (CSS): Chemically stabilized subgrade design and construction should meet Item 5 of the MGPEC Specifications.

Subgrade Sub-excavation and Replacement: Based on the soils encountered, some high plasticity soils may be exposed in the pavement subgrade. These soils may require sub-excavation and replacement with non-expansive soils. New non-expansive granular fill should have a minimum resilient modulus of 6,000 psi, be moisture conditioned to within 2% of optimum moisture content and be compacted to at least 95% of the maximum standard Proctor density (AASHTO T-99). Debris and any otherwise unsuitable materials should be removed from the pavement subgrade.

Proof-Roll: Prior to paving, the subgrade should be thoroughly proof-rolled with pneumatic-tired vehicle weighing at least 40,000 pounds. Areas that deform (rut or deflect) excessively under the wheel loads should be repaired prior to paving. If precipitation occurs after the proof-roll and prior to paving, then the subgrade should be proof-rolled again and repaired as needed.

Drainage, Frost Potential, and Utilities: The collection and diversion of surface drainage away from paved areas is extremely important for the satisfactory performance of the pavement. The design of surface drainage should be carefully considered to remove all water from paved areas. The predominant soil types are sandy clay that is moderately to severe frost susceptible. Frost heave potential can be reduced through proper surface drainage and construction control.

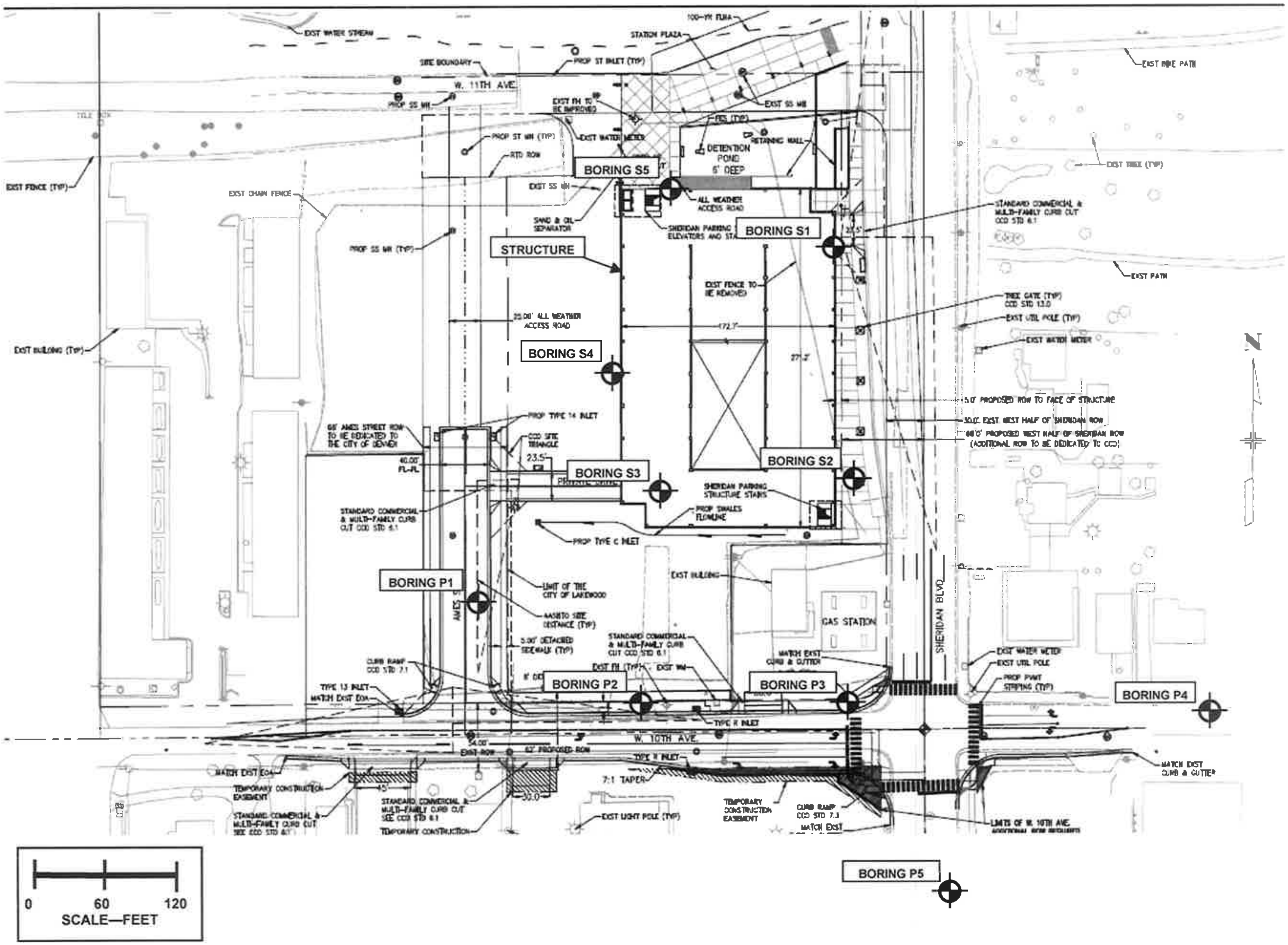
Maintenance: Periodic maintenance of paved areas will extend pavement life. The scheduled maintenance programs as listed Section 5 of the MGPEC Specifications should be followed for HMA and PCC pavements.

15.0 Limitations

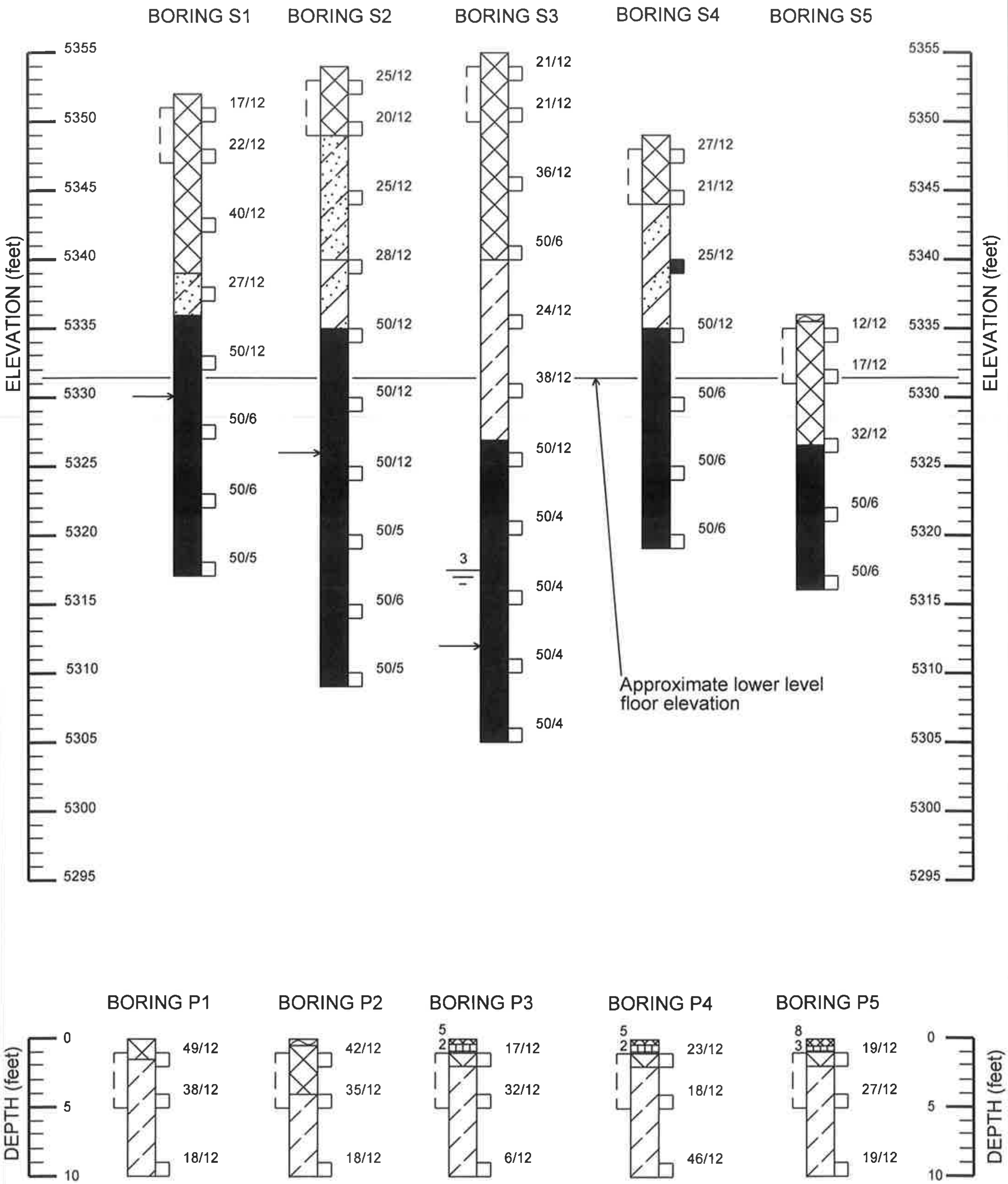
This report has been prepared in accordance with generally accepted geotechnical engineering practices used in this area, and has been prepared for design purposes. The conclusions and recommendations are based upon the data obtained from the borings drilled at the approximate locations shown on Figure 1. The nature and extent of the variations between borings may not become evident until excavation is performed. If during construction, soil, bedrock, fill, or groundwater conditions appear to be different from those described, this office should be advised so that re-evaluation of our recommendations may be made. Onsite observation and testing of construction materials is recommended.

Our professional services were performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in this or similar localities. No warranty expressed or implied is made. We prepared the report as an aid in the design of the proposed project. This report is not a bidding document. Any contractor reviewing this report must draw his or her own conclusions regarding site conditions and specific construction techniques to be used on this project.

This report is for the exclusive purpose of providing geotechnical engineering information and recommendations. The scope of services for this project does not include environmental assessment of the site or identification of contaminated or hazardous materials or conditions. If the owner is concerned about the potential for such contamination, other studies should be undertaken.



GEOCAL, INC. G11.1423.000	SHERIDAN PARKING STRUCTURE LOCATIONS OF EXPLORATORY BORINGS	FIGURE 1



G11.1423.000	GEOCAL, INC.	SHERIDAN PARKING STRUCTURE LOGS OF EXPLORATORY BORINGS	FIGURE 2

LEGEND

TOP SOIL

5 ASPHALT, approximate thickness in inches shown to the left of the logs.

7 AGGREGATE BASE COURSE, sand and gravel, approximate thickness in inches shown to the left of the logs.

FILL, variable clay with sand to mixed silty to clayey sand, trace gravel, medium dense to dense or stiff to very stiff, moist, brown to dark brown, fine to coarse grained sand, low plasticity fines.

SAND with CLAY, dense, moist, brown, fine to coarse grained sand, medium to high plasticity.

CLAY, sandy, stiff to very stiff, moist, light brown, low to medium plasticity.

SAND, silty to clayey, some gravel, dense, fine to coarse grained, moist, low plasticity, gravel to $\frac{1}{2}$ inch maximum dimension.

CLAYSTONE BEDROCK, mostly hard to very hard with depth, moist, olive gray to reddish brown, very fine sand, medium plasticity, weathered in upper 1 to 4 feet.

17/12 Drive sample blow count, Indicates that 17 blows from a 140 pound hammer falling 30 inches were required to drive the California or SPT sampler 12 inches.

2 inch I.D. California liner drive sample.

Standard Penetration Test, $1\frac{3}{8}$ inch I.D., split spoon drive sample.

3 Indicates depth to water level and number of days after drilling measurement was made.

Indicates depth at which caved material accumulated.

Indicates disturbed bulk sample.

NOTES

1. Borings were drilled on March 1 and March 2, 2012 with a D-50 drill rig equipped with $3\frac{1}{4}$ inch inside diameter hollow-stem and 4 inch diameter solid stem augers.

2. Location of borings shown on Figure 1 are approximate.

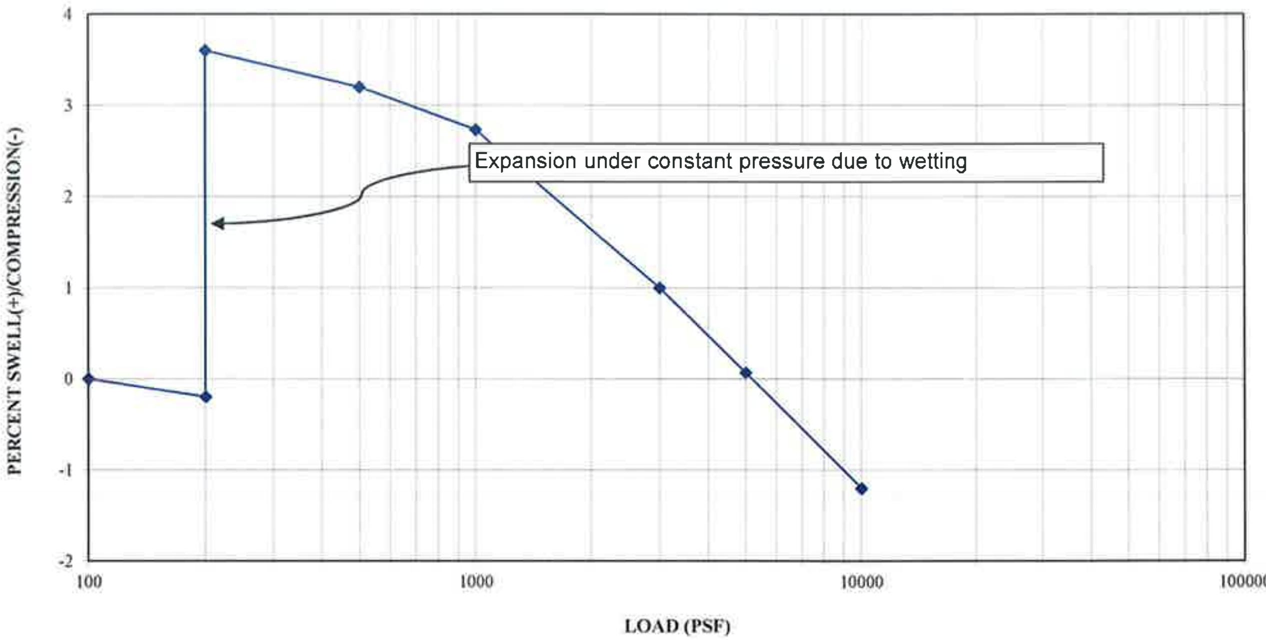
3. The lines between strata represent approximate boundaries between material types. Transitions between materials may actually be gradual.

4. Structure boring logs (S1 through S5) are drawn to elevation. Pavement boring logs (P1 through P5) are drawn to depth.

5. Water level readings shown on the logs were made at the time and under conditions indicated, fluctuations in the water level may occur with time.

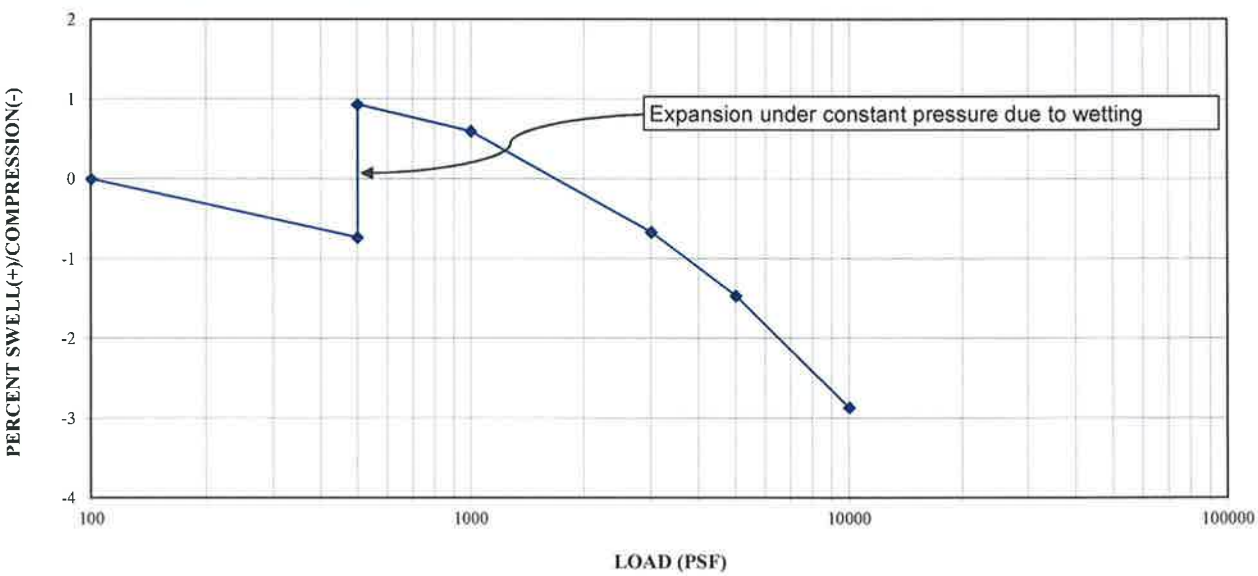
G11.1423.000	GEOCAL, INC.	SHERIDAN PARKING STRUCTURE LEGEND AND NOTES FOR EXPLORATORY BORINGS	FIGURE 3

SWELL-COMPRESSION TEST



Sample Location	P-1
Sample Depth	4 feet
Sample Description	Sandy fat clay
USCS Classification	CH
AASHTO Classification	A-7-6(20)

Dry Density	103 pcf
Moisture Content	20.7 %
Volume Change	3.8 %
Swell Pressure	5,100 psf

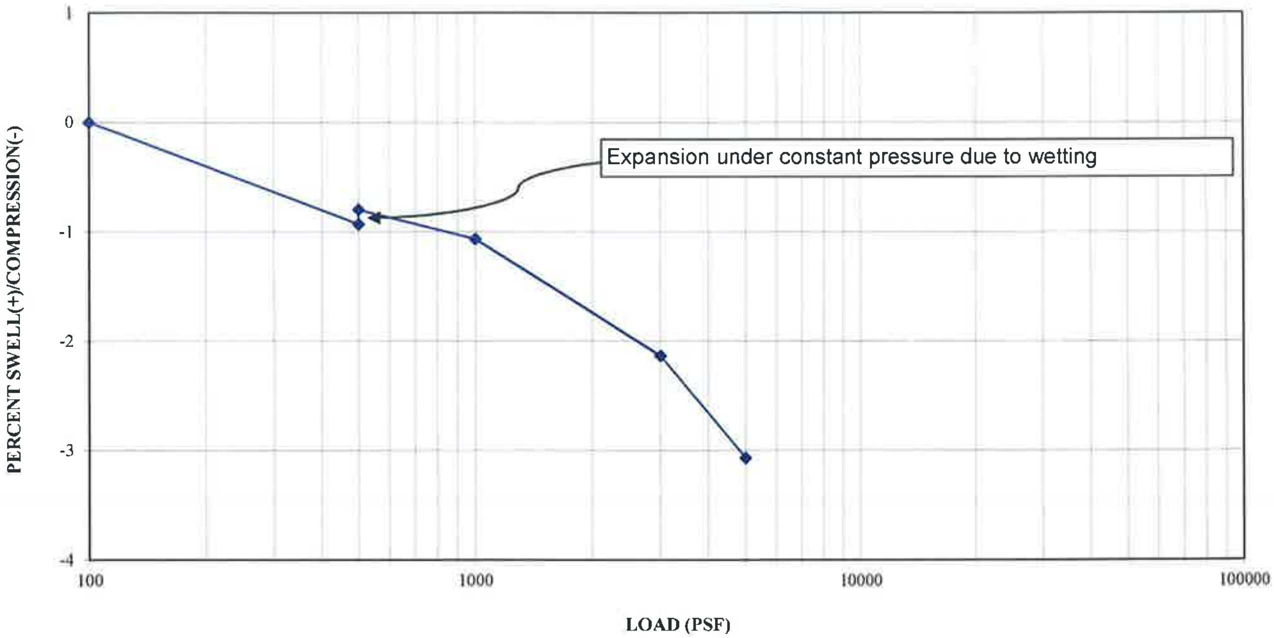


Sample Location	P-3
Sample Depth	4 feet
Sample Description	Sandy lean clay
USCS Classification	CL
AASHTO Classification	A-7-6(13)

Dry Density	112 pcf
Moisture Content	18.2 %
Volume Change	1.7 %
Swell Pressure	1,750 psf

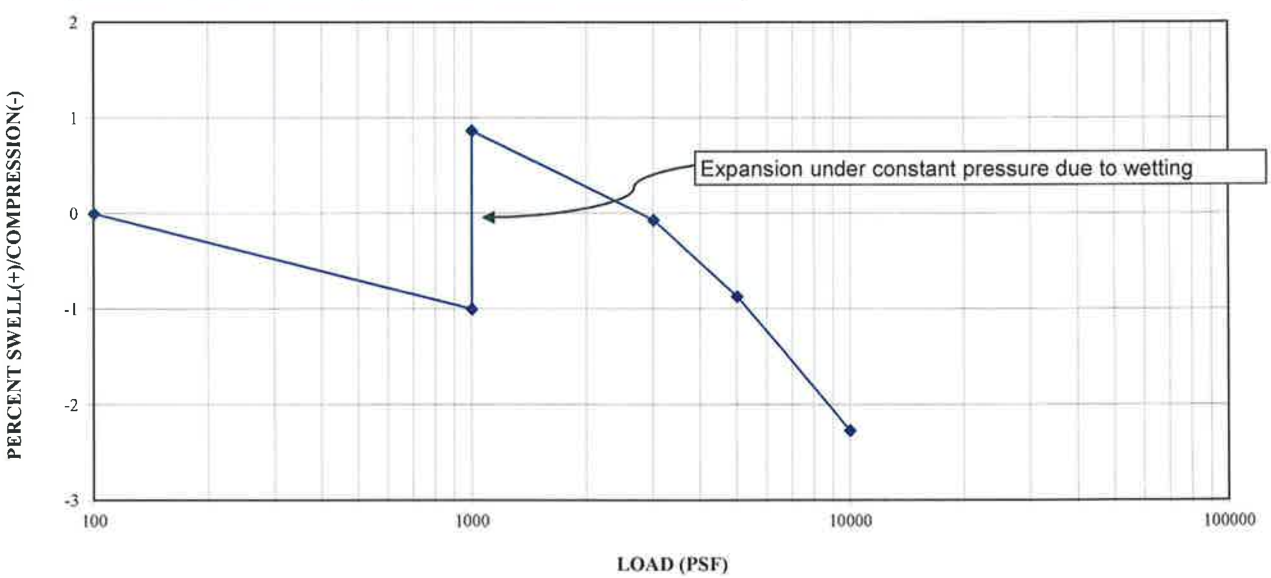
GEOCAL, INC.	Sheridan Parking Structure	JOB NO.	G11.1423.000
	SWELL - COMPRESSION TEST RESULTS	FIGURE NO.	4

SWELL-COMPRESSION TEST



Sample Location	S-1
Sample Depth	4 feet
Sample Description	Clayey sand with gravel, fill
USCS Classification	SC
AASHTO Classification	A-7-6(9)

Dry Density	95 pcf
Moisture Content	26.4 %
Volume Change	0.1 %
Swell Pressure	0 psf

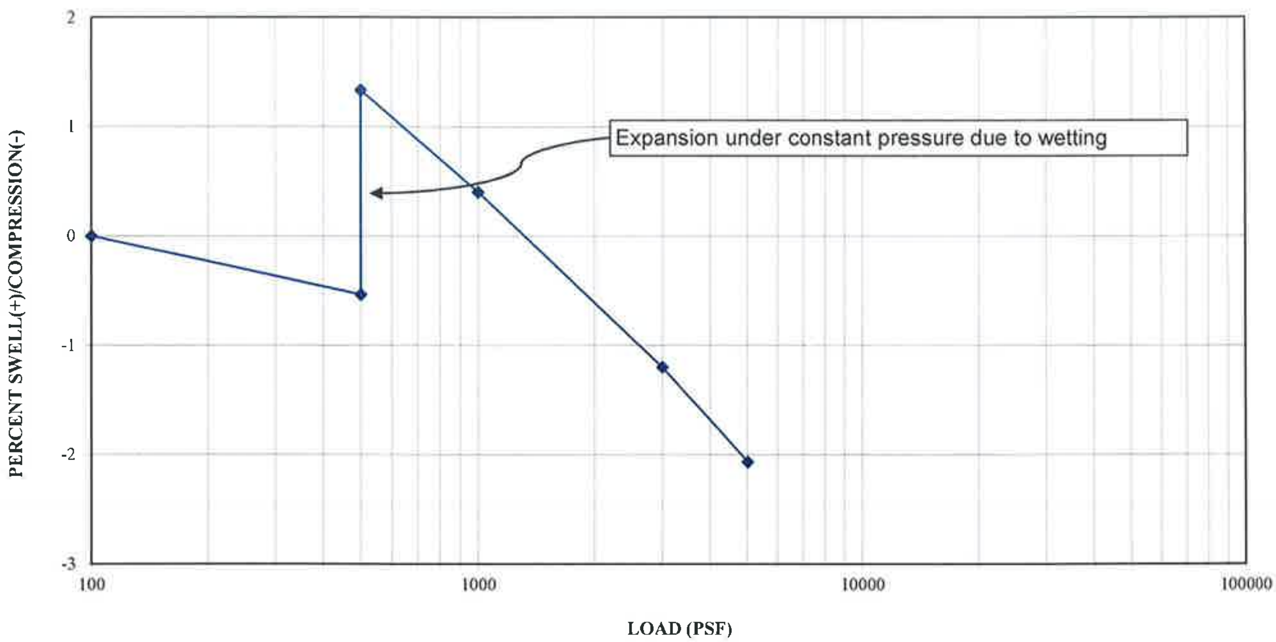


Sample Location	S-1
Sample Depth	19 feet
Sample Description	Claystone bedrock
USCS Classification	
AASHTO Classification	

Dry Density	100 pcf
Moisture Content	25.1 %
Volume Change	1.9 %
Swell Pressure	2,900 psf

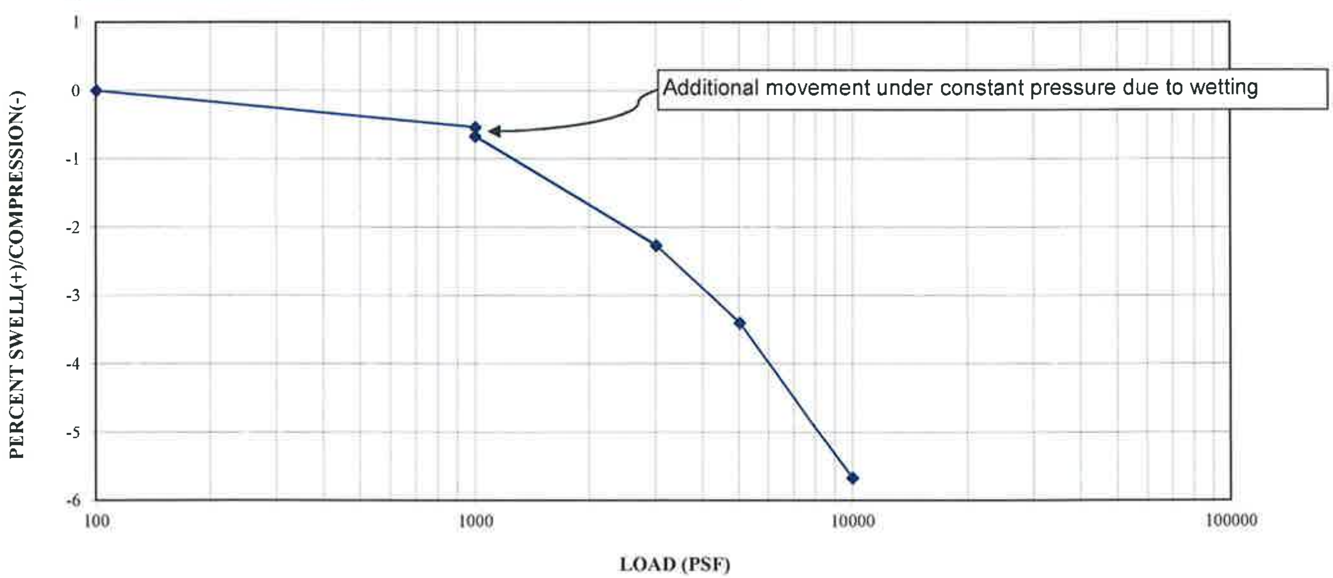
GEOCAL, INC.	Sheridan Parking Structure	JOB NO.	G11.1423.000
	SWELL - COMPRESSION TEST RESULTS	FIGURE NO.	5

SWELL-COMPRESSION TEST



Sample Location	S-2
Sample Depth	9 feet
Sample Description	Clayey sand
USCS Classification	SC
AASHTO Classification	A-7-6(11)

Dry Density	104 pcf
Moisture Content	20.0 %
Volume Change	1.9 %
Swell Pressure	1,350 psf

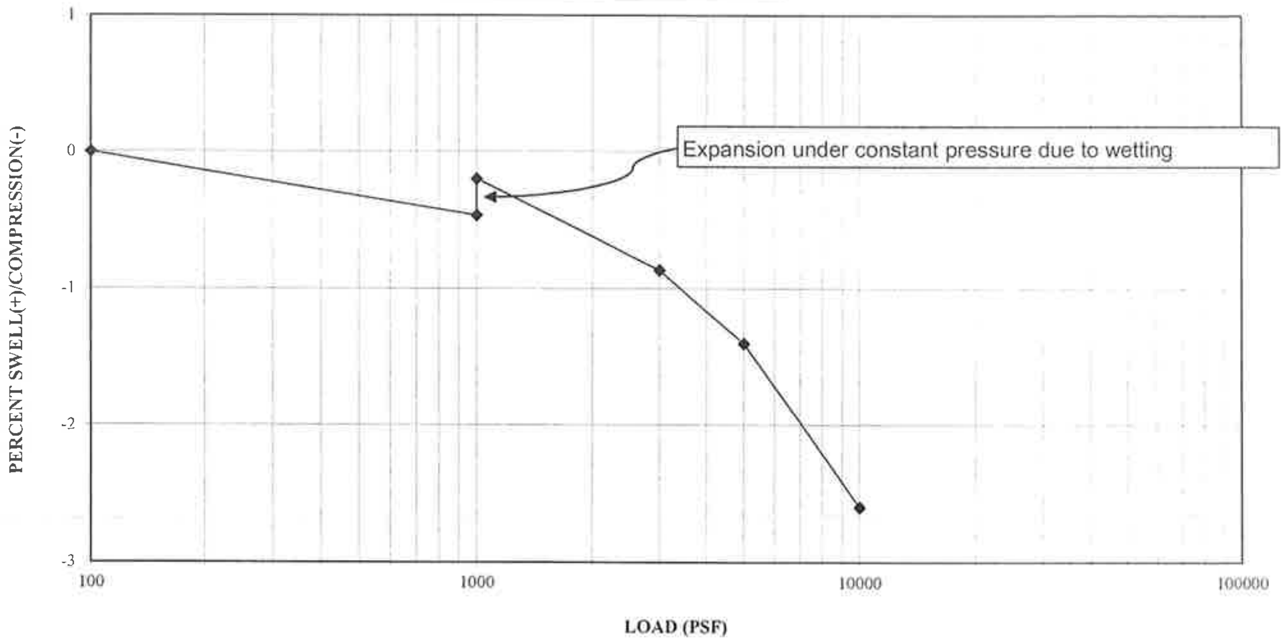


Sample Location	S-3
Sample Depth	19 feet
Sample Description	Sandy lean clay
USCS Classification	CL
AASHTO Classification	A-7-6(10)

Dry Density	101 pcf
Moisture Content	15.6 %
Volume Change	-0.1 %
Swell Pressure	0 psf

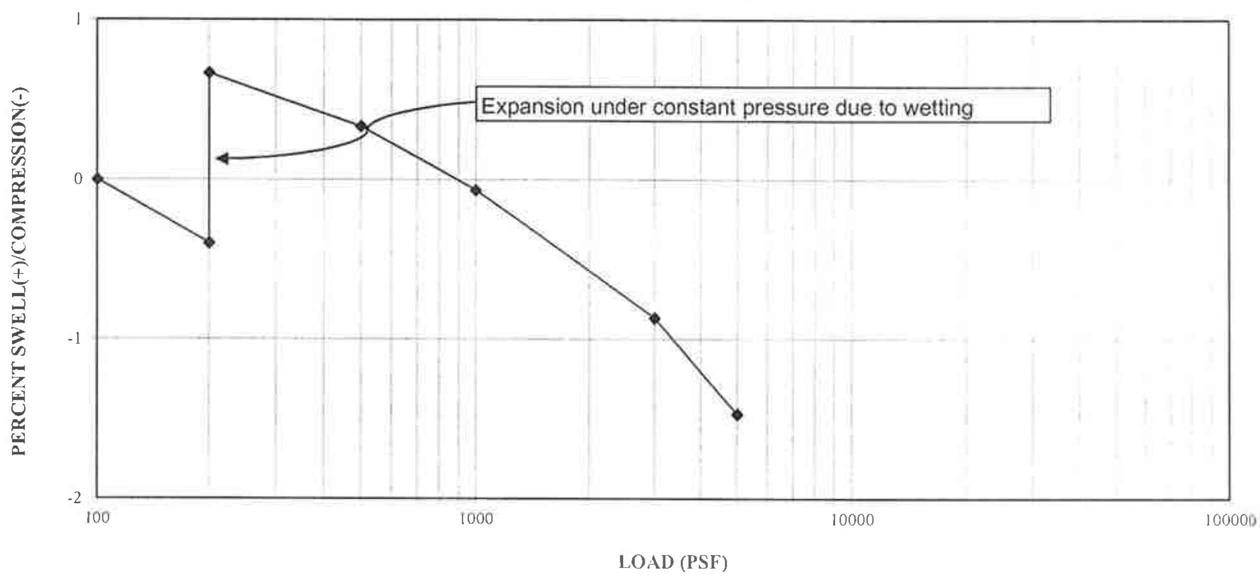
GEOCAL, INC.	**Sheridan Parking Structure**	**JOB NO.**	**G11.1423.000**
	SWELL - COMPRESSION TEST RESULTS	**FIGURE NO.**	**6**

SWELL-COMPRESSION TEST



Sample Location	S-3
Sample Depth	34 feet
Sample Description	Claystone bedrock
USCS Classification	
AASHTO Classification	

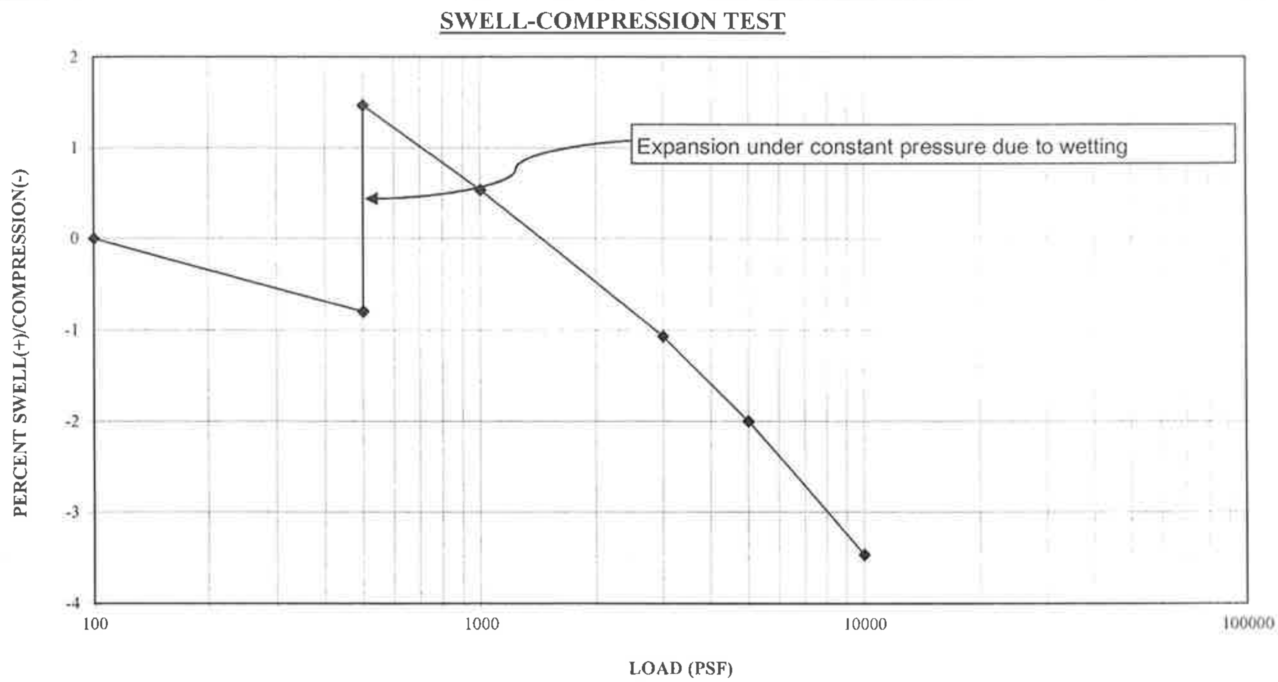
Dry Density	96 pcf
Moisture Content	25.7 %
Volume Change	0.3 %
Swell Pressure	0 psf



Sample Location	S-5
Sample Depth	9 feet
Sample Description	Clayey sand with gravel
USCS Classification	SC
AASHTO Classification	A-6(3)

Dry Density	113 pcf
Moisture Content	15.6 %
Volume Change	1.1 %
Swell Pressure	900 psf

GEOCAL, INC.	**Sheridan Parking Structure**	**JOB NO.**	**G11.1423.000**
	SWELL - COMPRESSION TEST RESULTS	**FIGURE NO.**	**7**

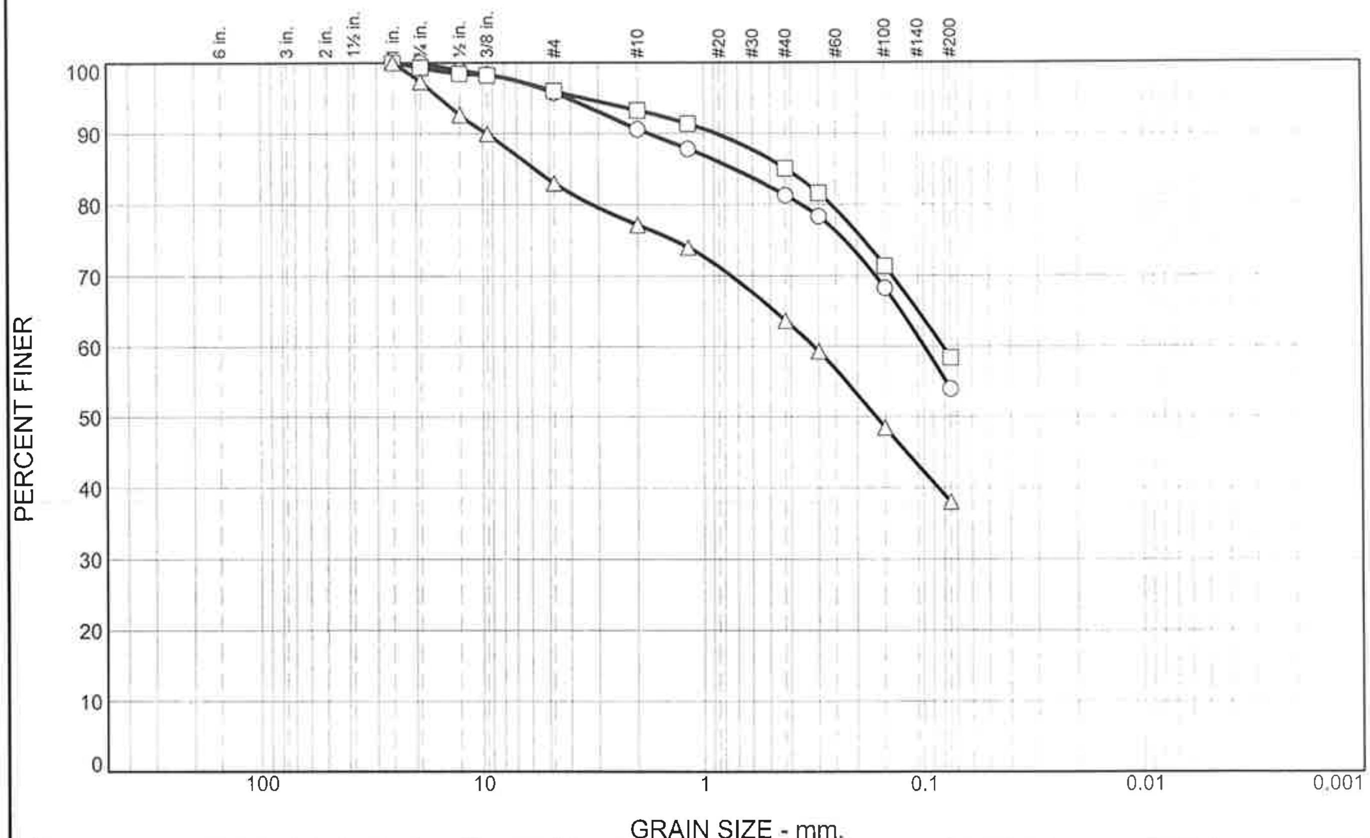


Sample Location	S-5
Sample Depth	14 feet
Sample Description	Claystone bedrock
USCS Classification	
AASHTO Classification	

Dry Density	105 pcf
Moisture Content	22.7 %
Volume Change	2.3 %
Swell Pressure	1,500 psf

GEOCAL, INC.	**Sheridan Parking Structure**	**JOB NO.**	**G11.1423.000**
	SWELL - COMPRESSION TEST RESULTS	**FIGURE NO.**	**8**

Gradation Test Results



	% +3"	% Gravel	% Sand	% Silt	% Clay
○	0	4	42	54	
□	0	4	38	58	
△	0	17	45	38	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	43	18	0.7250	0.0993						
□	43	17	0.4215	0.0814						
△	38	15	5.8254	0.3157	0.1651					

	Material Description	USCS	AASHTO
○	sandy lean clay	CL	A-7-6(10)
□	sandy lean clay	CL	A-7-6(12)
△	clayey sand with gravel	SC	A-6(4)

Project No. G11.1423.000 **Client:** Swinerton Builders

Project: Sheridan Parking Structure

○ **Location:** Boring P-1 **Depth:** 1-5 feet **Sample Number:** 5979-2

□ **Location:** Boring P-3 **Depth:** 1-5 feet **Sample Number:** 5979-4

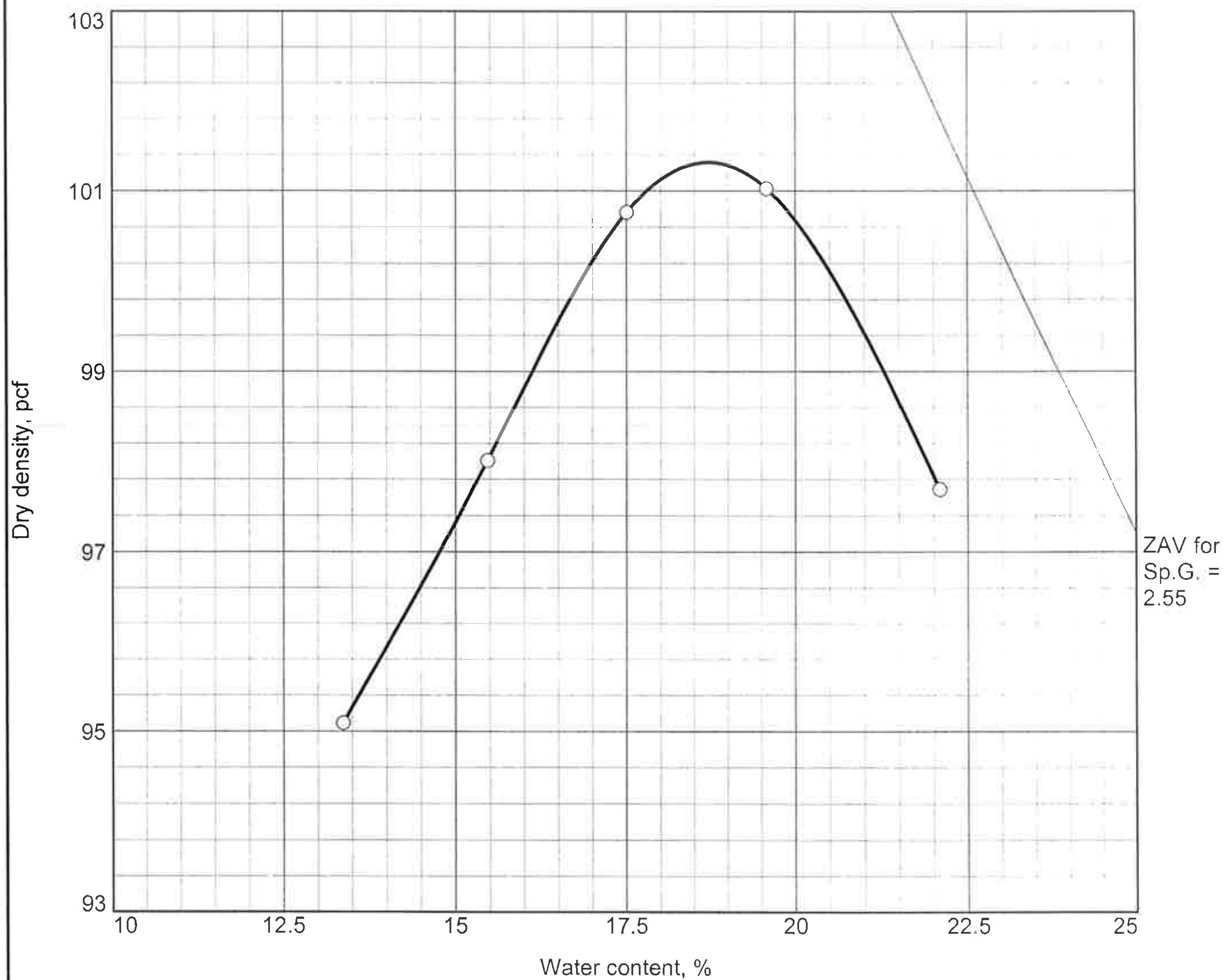
△ **Location:** Boring P-4 **Depth:** 1-5 feet **Sample Number:** 5979-5

Remarks:

GEOCAL, INC.

Figure 9

Moisture-Density Relationship Test Results



Test specification: AASHTO T 99 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
1-5 feet	CL	A-7-6(10)			43	25	4.0	54

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 101.3 pcf Optimum moisture = 18.7 %	sandy lean clay

Project No. G11.1423.000 **Client:** Swinerton Builders

Project: Sheridan Parking Structure

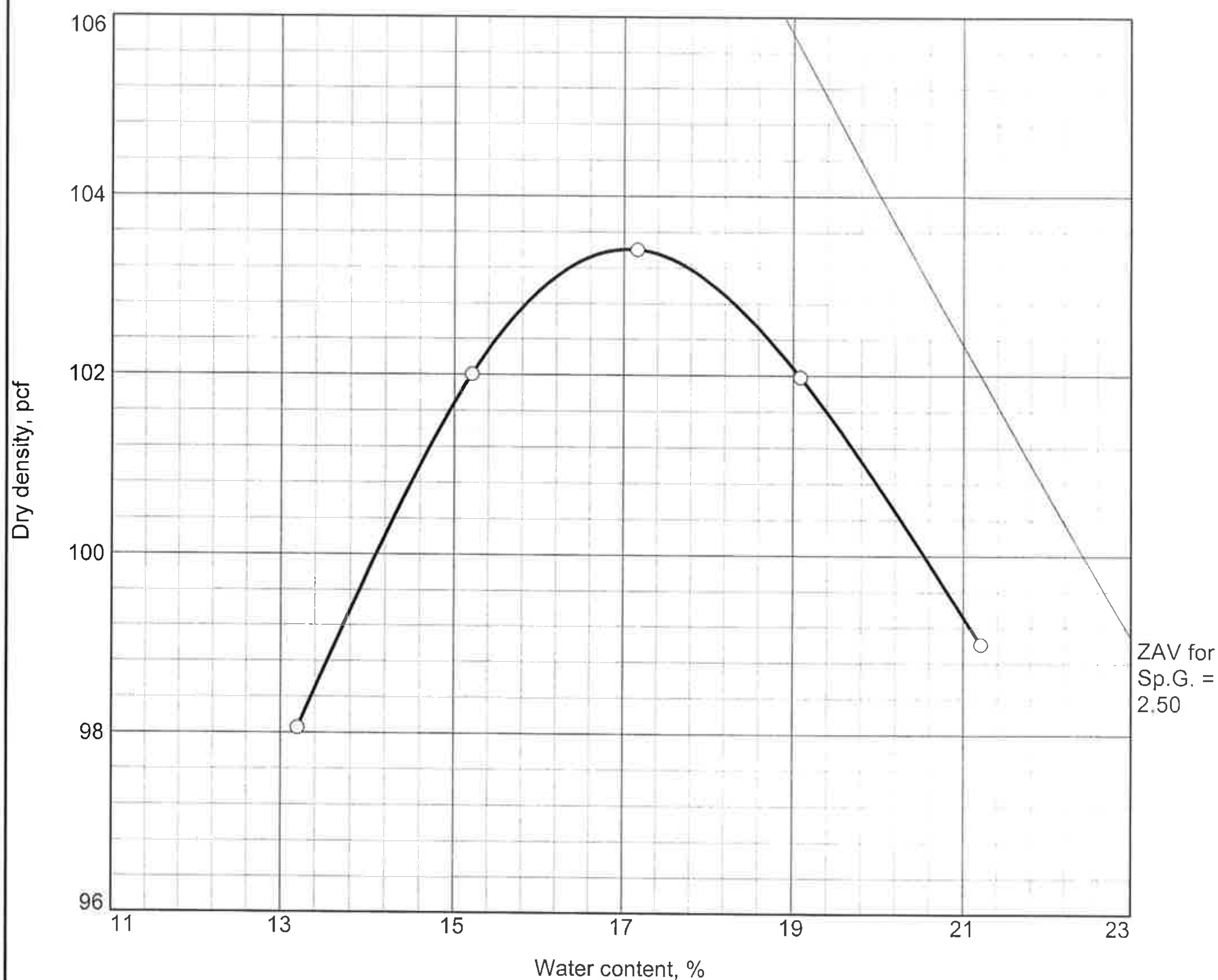
○ **Loc.:** Boring P-1 **Depth:** 1-5 feet **Sample No.:** 5979-2

Remarks:

GEOCAL, INC.

Figure 10

Moisture-Density Relationship Test Results



Test specification: AASHTO T 99 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
1-5 feet	CL	A-7-6(12)			43	26	4.0	58

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 103.4 pcf Optimum moisture = 17.1 %	sandy lean clay

Project No. G11.1423.000 **Client:** Swinerton Builders
Project: Sheridan Parking Structure

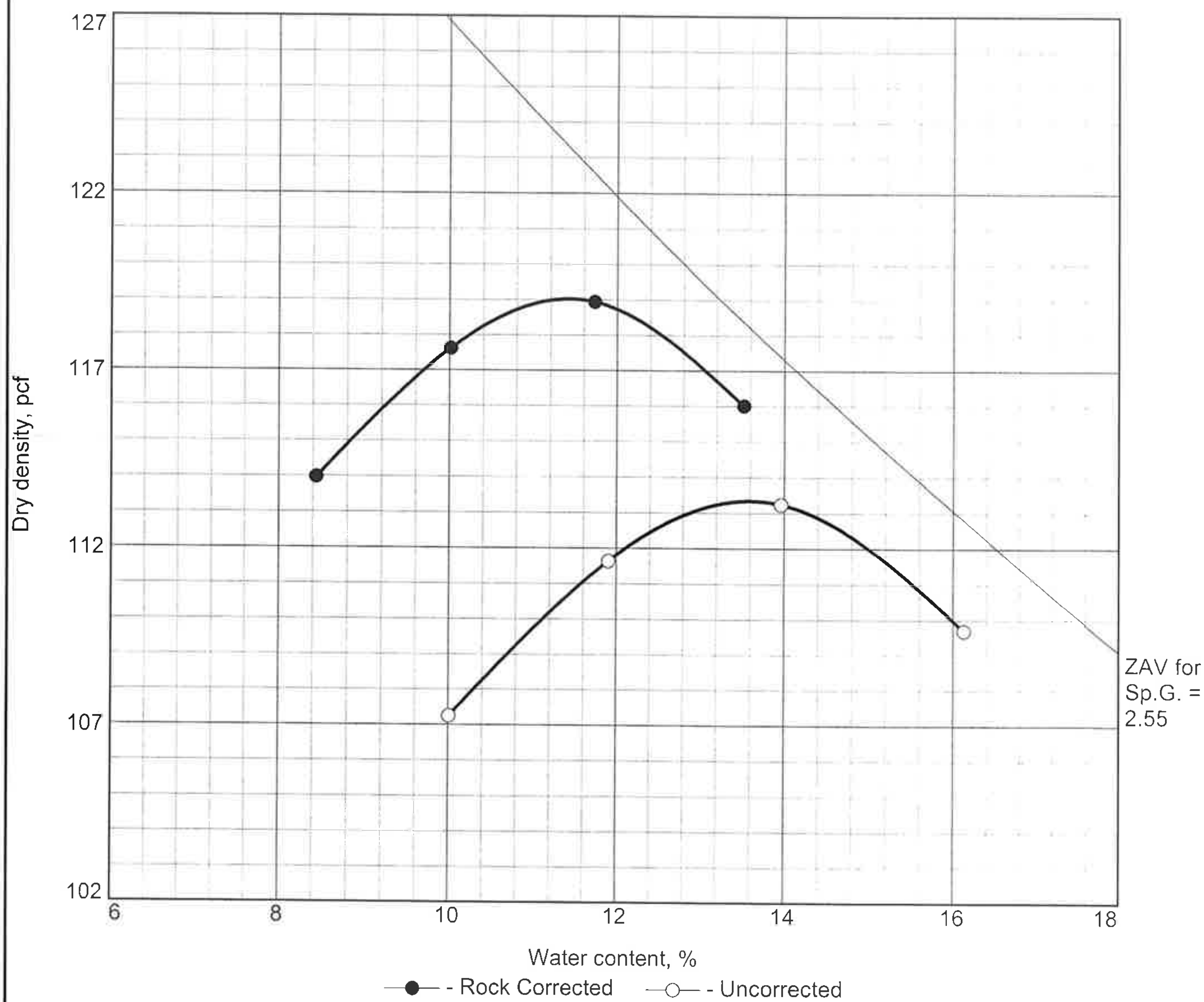
○ **Loc.:** Boring P-3 **Depth:** 1-5 feet **Sample No.:** 5979-4

GEOCAL, INC.

Remarks:

Figure 11

Moisture-Density Relationship Test Results



Test specification: AASHTO T 99 Method A Standard
Oversize corr. applied to each test point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
1-5 feet	SC	A-6(4)			38	23	17.0	38

ROCK CORRECTED TEST RESULTS	UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 119.0 pcf	113.3 pcf	clayey sand with gravel
Optimum moisture = 11.4 %	13.6 %	

Project No. G11.1423.000 **Client:** Swinerton Builders
Project: Sheridan Parking Structure

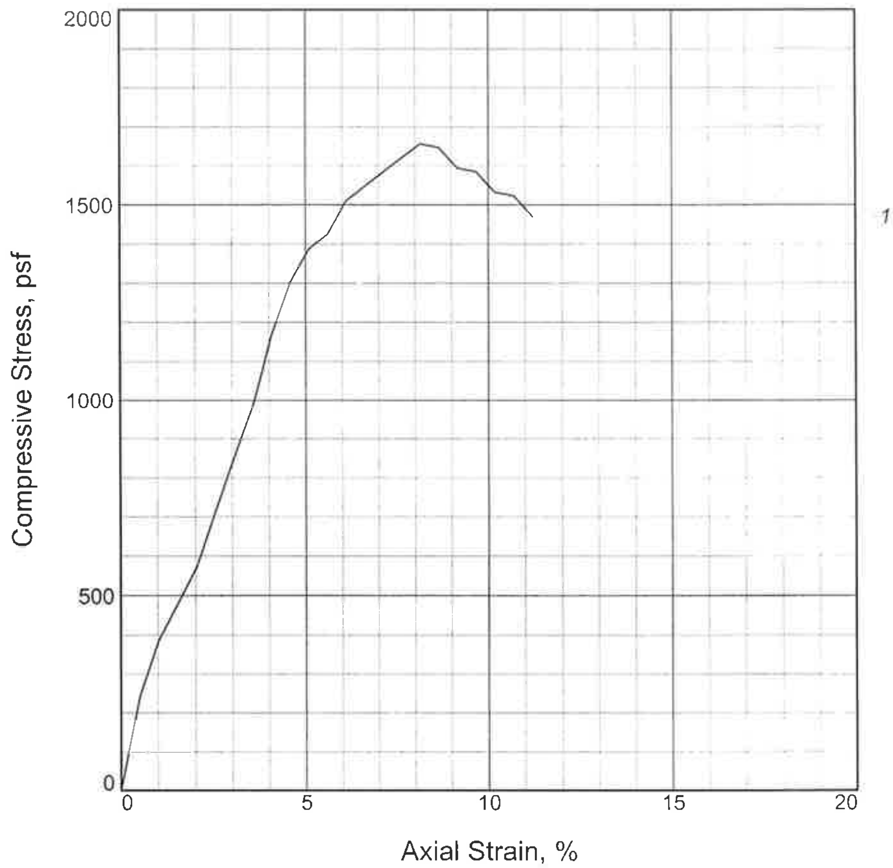
○ **Loc.:** Boring P-4 **Depth:** 1-5 feet **Sample No.:** 5979-5

GEOCAL, INC.

Remarks:
Aggregate bulk specific gravity = 2.613,
Aggregate absorption = 0.8%.

Figure 12

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	1656			
Undrained shear strength, psf	828			
Failure strain, %	8.1			
Strain rate, in./min.	0.03			
Water content, %	20.4			
Wet density, pcf	116.1			
Dry density, pcf	96.4			
Saturation, %	75.6			
Void ratio	0.7168			
Specimen diameter, in.	1.94			
Specimen height, in.	3.93			
Height/diameter ratio	2.03			

Description: sandy lean clay

LL = 43	**PL =** 18	**PI =** 25	**Assumed GS=** 2.65	**Type:**

Project No.: G11.1423.000

Date Sampled:

Remarks:
Remolded at 95% of MDD & 2% over OMC.

Figure 13

Client: Swinerton Builders

Project: Sheridan Parking Structure

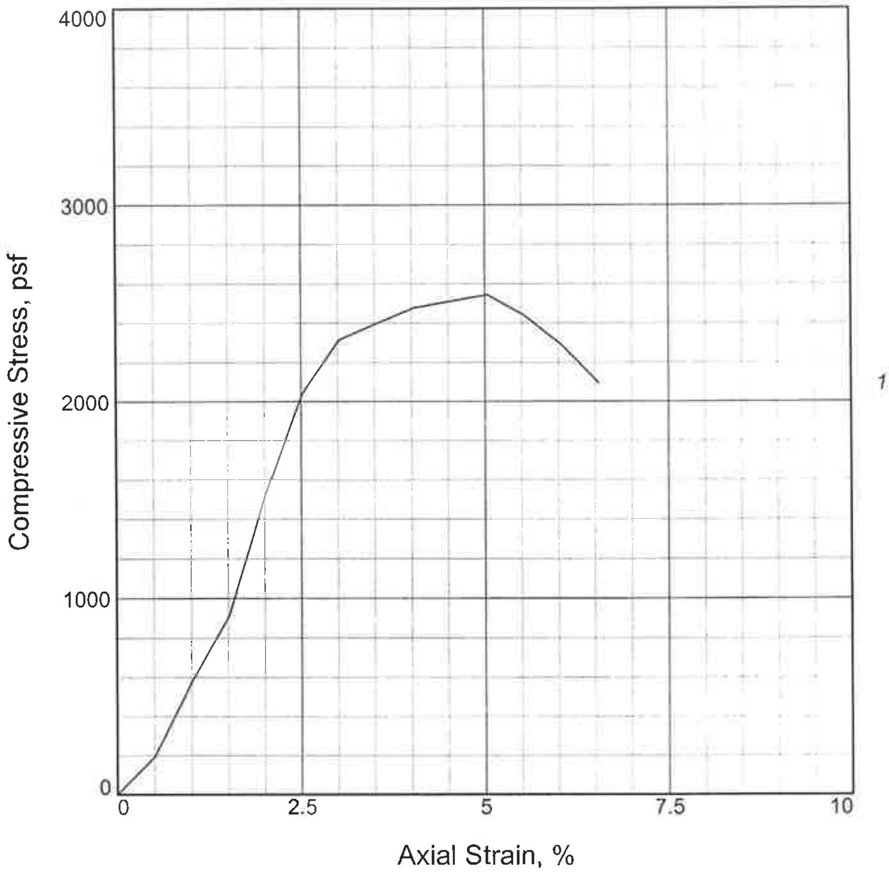
Location: Boring P-1

Sample Number: 5979-2 **Depth:** 1-5 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	2544			
Undrained shear strength, psf	1272			
Failure strain, %	5.0			
Strain rate, in./min.	0.03			
Water content, %	19.1			
Wet density, pcf	117.0			
Dry density, pcf	98.2			
Saturation, %	74.0			
Void ratio	0.6846			
Specimen diameter, in.	1.94			
Specimen height, in.	3.97			
Height/diameter ratio	2.05			

Description: sandy lean clay

LL = 43	PL = 17	PI = 26	Assumed GS= 2.65	Type:

Project No.: G11.1423.000

Date Sampled:

Remarks:
Remolded at 95% of MDD & 2% over OMC.

Figure 14

Client: Swinerton Builders

Project: Sheridan Parking Structure

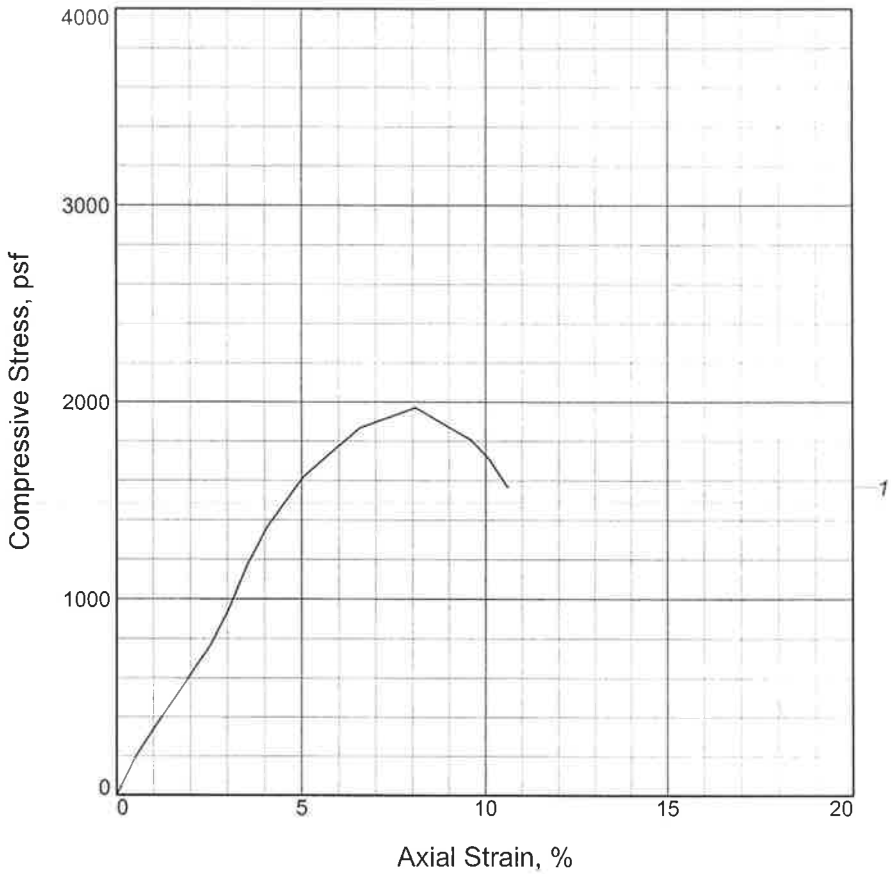
Location: Boring P-3

Sample Number: 5979-4 **Depth:** 1-5 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	1970			
Undrained shear strength, psf	985			
Failure strain, %	8.1			
Strain rate, in./min.	0.03			
Water content, %	15.7			
Wet density, pcf	124.5			
Dry density, pcf	107.6			
Saturation, %	77.3			
Void ratio	0.5371			
Specimen diameter, in.	1.94			
Specimen height, in.	3.96			
Height/diameter ratio	2.04			

Description: clayey sand with gravel

LL = 38	**PL** = 15	**PI** = 23	**Assumed GS=** 2.65	**Type:**

Project No.: G11.1423.000

Date Sampled:

Remarks:
Remolded at 95% of MDD & 2% over OMC.

Figure 15

Client: Swinerton Builders

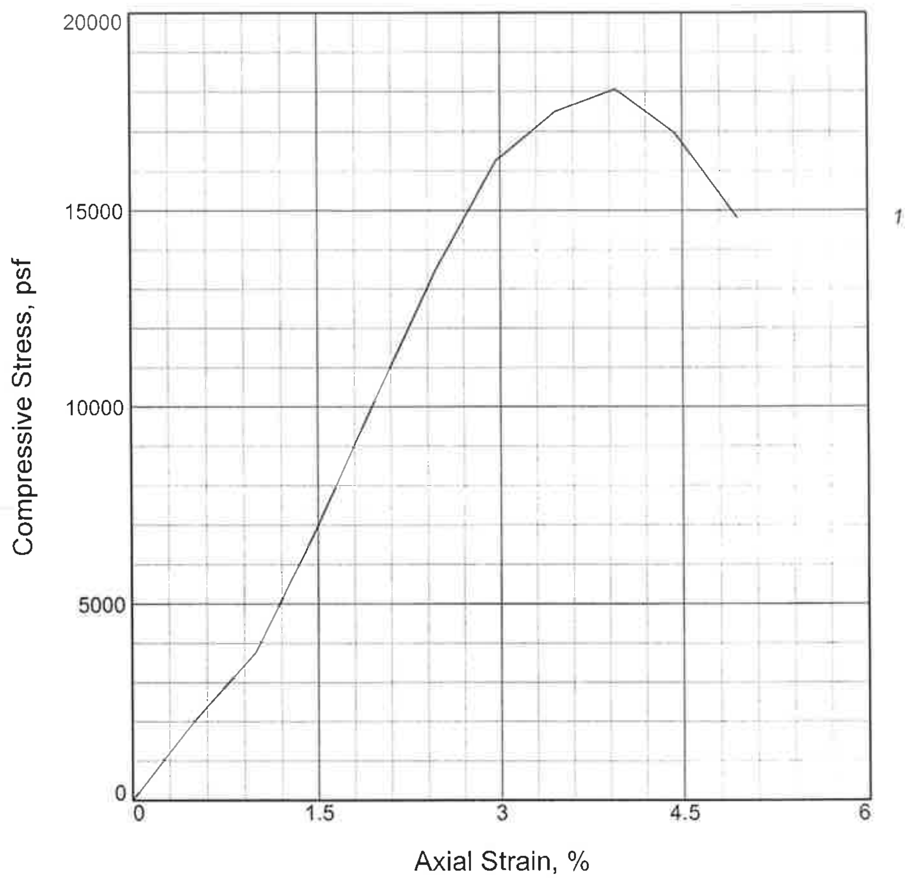
Project: Sheridan Parking Structure

Location: Boring P-4
Sample Number: 5979-5 **Depth:** 1-5 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	18061			
Undrained shear strength, psf	9031			
Failure strain, %	4.0			
Strain rate, in./min.	0.03			
Water content, %	15.9			
Wet density, pcf	133.7			
Dry density, pcf	115.4			
Saturation, %	97.1			
Void ratio	0.4332			
Specimen diameter, in.	1.94			
Specimen height, in.	4.05			
Height/diameter ratio	2.09			

Description: claystone bedrock

LL = 40	**PL =** 18	**PI =** 22	**Assumed GS=** 2.65	**Type:**

Project No.: G11.1423.000

Date Sampled:

Remarks:

Figure 16

Client: Swinerton Builders

Project: Sheridan Parking Structure

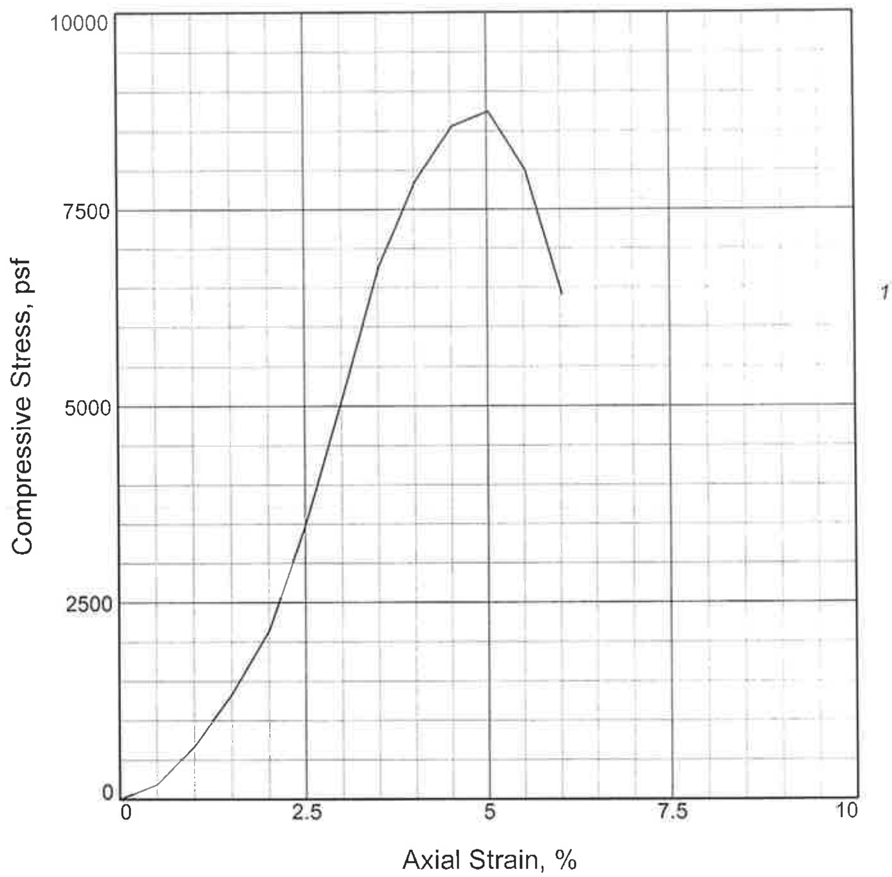
Location: Boring S-1

Sample Number: 5979-8 **Depth:** 24 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	8745			
Undrained shear strength, psf	4372			
Failure strain, %	5.0			
Strain rate, in./min.	0.03			
Water content, %	21.6			
Wet density, pcf	125.2			
Dry density, pcf	103.0			
Saturation, %	97.5			
Void ratio	0.5766			
Specimen diameter, in.	1.94			
Specimen height, in.	3.98			
Height/diameter ratio	2.05			

Description: claystone bedrock

LL =	PL =	PI =	Assumed GS= 2.6	Type:

Project No.: G11.1423.000

Date Sampled:

Remarks:

Figure 17

Client: Swinerton Builders

Project: Sheridan Parking Structure

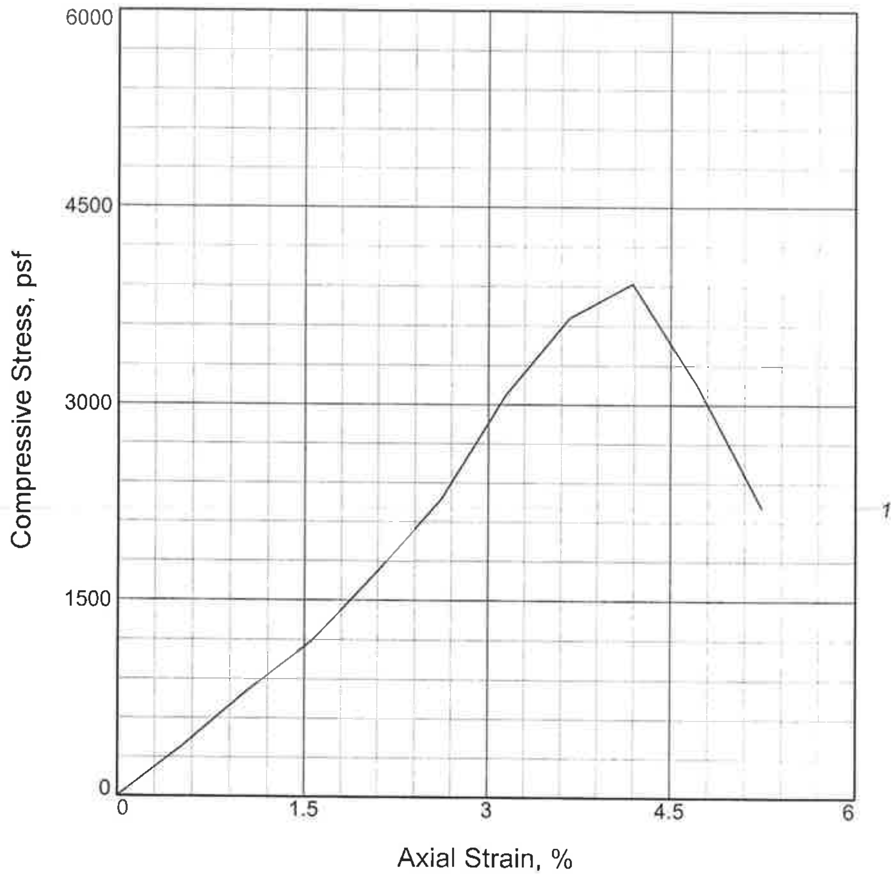
Location: Boring S-2

Sample Number: 5979-10 **Depth:** 34 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	3921			
Undrained shear strength, psf	1960			
Failure strain, %	4.2			
Strain rate, in./min.	0.03			
Water content, %	19.9			
Wet density, pcf	117.2			
Dry density, pcf	97.8			
Saturation, %	78.2			
Void ratio	0.6597			
Specimen diameter, in.	1.94			
Specimen height, in.	3.82			
Height/diameter ratio	1.97			

Description: claystone bedrock

LL =	PL =	PI =	Assumed GS= 2.6	Type:

Project No.: G11.1423.000

Date Sampled:

Remarks:

Client: Swinerton Builders

Project: Sheridan Parking Structure

Location: Boring S-3

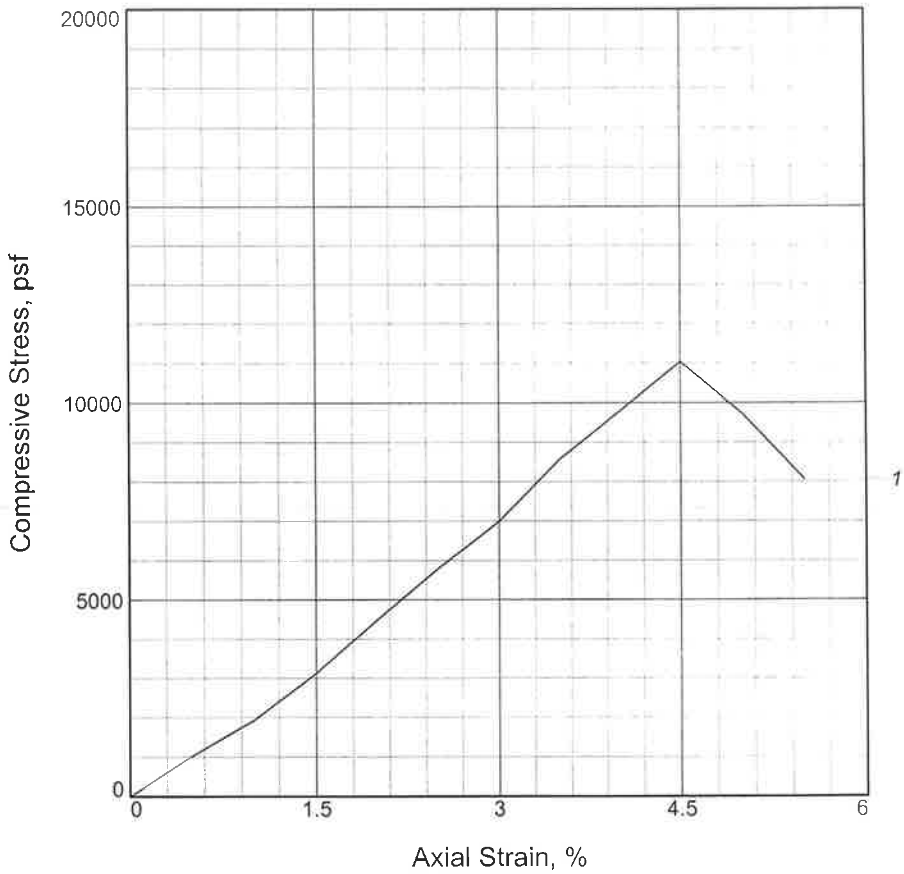
Sample Number: 5979-13 **Depth:** 44 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

Figure 18

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	11026			
Undrained shear strength, psf	5513			
Failure strain, %	4.5			
Strain rate, in./min.	0.30			
Water content, %	15.2			
Wet density, pcf	128.9			
Dry density, pcf	111.9			
Saturation, %	87.9			
Void ratio	0.4511			
Specimen diameter, in.	1.94			
Specimen height, in.	4.00			
Height/diameter ratio	2.06			

Description: claystone bedrock

LL =	**PL =**	**PI =**	**Assumed GS=** 2.6	**Type:**

Project No.: G11.1423.000

Date Sampled:

Remarks:

Figure 19

Client: Swinerton Builders

Project: Sheridan Parking Structure

Location: Boring S-4

Sample Number: 5979-14 **Depth:** 19 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

Project #: **G11.1423.000**

TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

Client: **Swinerton**
Project Name: **Sheridan Parking Structure**

Sample Location		Natural	Natural	Gradation			Atterberg Limits		Swell	Swell	Unconfined	Water	AASHTO	USCS	
Boring No.	Depth (feet)	Moisture Content (%)	Dry Density (pcf)	Gravel (%)	Sand (%)	Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	Pressure (psf)	w/.2,.5 or 1 ksf Surcharge (%)	Compressive Strength (psf)	Soluble Sulfates (%)	Class. (Group Index)	Class.	Soil or Bedrock Description
P-1	4	20.7	103			67	51	32	5,100	3.8			A-7-6(20)	CH	Sandy fat clay
P-1	1-5			4	42	54	43	25			1660*	0.02	A-7-6(10)	CL	Sandy lean clay
	Standard Proctor AASHTO T 99, Method A: MDD = 101.3 pcf; OMC = 18.7%.														
P-3	4	18.2	112			59	44	27	1,750	1.7			A-7-6(13)	CL	Sandy lean clay
P-3	1-5			4	38	58	43	26			2540*	0.01	A-7-6(12)	CL	Sandy lean clay
	Standard Proctor AASHTO T 99, Method A: MDD = 103.4 pcf; OMC = 17.1%.														
P-4	1-5			17	45	38	38	23			1970*	0.01	A-6(4)	SC	Clayey sand with gravel
	Standard Proctor AASHTO T 99, Method A: Corrected: MDD = 119.0 pcf; OMC = 11.4%.														
S-1	4	26.4	95			44	54	31	0	0.1		0.02	A-7-6(9)	SC	Clayey sand with gravel, fill
S-1	19	25.1	100			93	54	32	2,900	1.9					Claystone bedrock
S-1	24	15.4	115			84	40	22			18,060				Claystone bedrock
S-2	9	20.0	104			49	51	32	1,350	1.9		0.02	A-7-6(11)	SC	Clayey sand
S-2	34	21.6	103								8,750				Claystone bedrock
S-3	19	15.6	101			57	41	22	0	-0.1		0.03	A-7-6(10)	CL	Sandy lean clay
S-3	34	25.7	96			59	50	21	0	0.3					Claystone bedrock
S-3	44	19.9	98								3,920				Claystone bedrock
S-4	19	15.2	112								11,030	0.01			Claystone bedrock
S-5	9	15.6	113			36	39	22	900	1.1		0.01	A-6(3)	SC	Clayey sand with gravel, fill
S-5	14	22.7	105			92	50	25	1,500	2.3					Claystone bedrock

*Remolded at 95% of Maximum Dry Density & 2% over Optimum Moisture Content

APPENDIX A

SEISMIC PRINTOUTS FROM USGS BASED ON 2009 IBC REQUIREMENTS

2009 International Building Code
Latitude = 39.733788
Longitude = -105.053893
MCE Response Spectrum for Site Class B
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0

Period (sec)	Sa (g)	Sd (inches)
0.000	0.090	0.000
0.051	0.224	0.006
0.200	0.224	0.088
0.256	0.224	0.144
0.300	0.191	0.168
0.400	0.144	0.224
0.500	0.115	0.280
0.600	0.096	0.336
0.700	0.082	0.393
0.800	0.072	0.449
0.900	0.064	0.505
1.000	0.057	0.561
1.100	0.052	0.617
1.200	0.048	0.673
1.300	0.044	0.729
1.400	0.041	0.785
1.500	0.038	0.841
1.600	0.036	0.897
1.700	0.034	0.953
1.800	0.032	1.009
1.900	0.030	1.066
2.000	0.029	1.122

Conterminous 48 States
2009 International Building Code
Latitude = 39.733788
Longitude = -105.053893
Site Modified Response Spectrum for Site Class C
SMs = FaSs and SM1 = FvS1
Site Class C - Fa = 1.2 ,Fv = 1.7

Period (sec)	Sa (g)	Sd (inches)
0.000	0.108	0.000

0.200	0.269	0.105
0.363	0.269	0.346
0.400	0.244	0.381
0.500	0.195	0.477
0.600	0.163	0.572
0.700	0.139	0.667
0.800	0.122	0.763
0.900	0.108	0.858
1.000	0.098	0.953
1.100	0.089	1.049
1.200	0.081	1.144
1.300	0.075	1.239
1.400	0.070	1.335
1.500	0.065	1.430
1.600	0.061	1.525
1.700	0.057	1.621
1.800	0.054	1.716
1.900	0.051	1.811
2.000	0.049	1.907

Conterminous 48 States
2009 International Building Code
Latitude = 39.733788
Longitude = -105.053893
Design Response Spectrum for Site Class C
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class C - Fa = 1.2 ,Fv = 1.7

Period (sec)	Sa (g)	Sd (inches)
0.000	0.072	0.000
0.073	0.179	0.009
0.200	0.179	0.070
0.363	0.179	0.231
0.400	0.163	0.254
0.500	0.130	0.318
0.600	0.108	0.381
0.700	0.093	0.445
0.800	0.081	0.508
0.900	0.072	0.572
1.000	0.065	0.636
1.100	0.059	0.699
1.200	0.054	0.763

1.400	0.046	0.890
1.500	0.043	0.953
1.600	0.041	1.017
1.700	0.038	1.080
1.800	0.036	1.144
1.900	0.034	1.208
2.000	0.033	1.271

Conterminous 48 States
2009 International Building Code
Latitude = 39.733788
Longitude = -105.053893
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.05 deg grid spacing

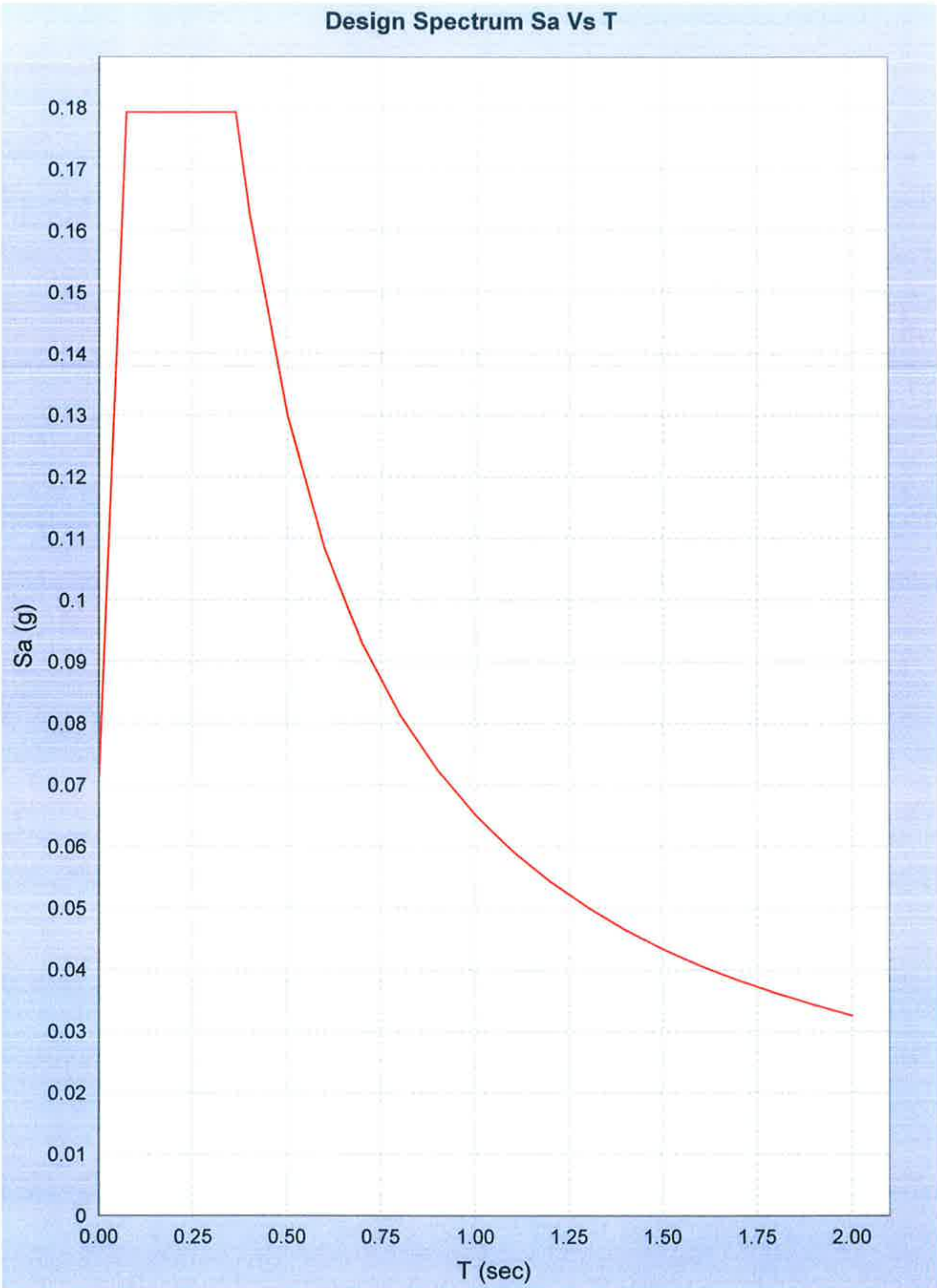
Period (sec)	Sa (g)
0.2	0.224 (Ss, Site Class B)
1.0	0.057 (S1, Site Class B)

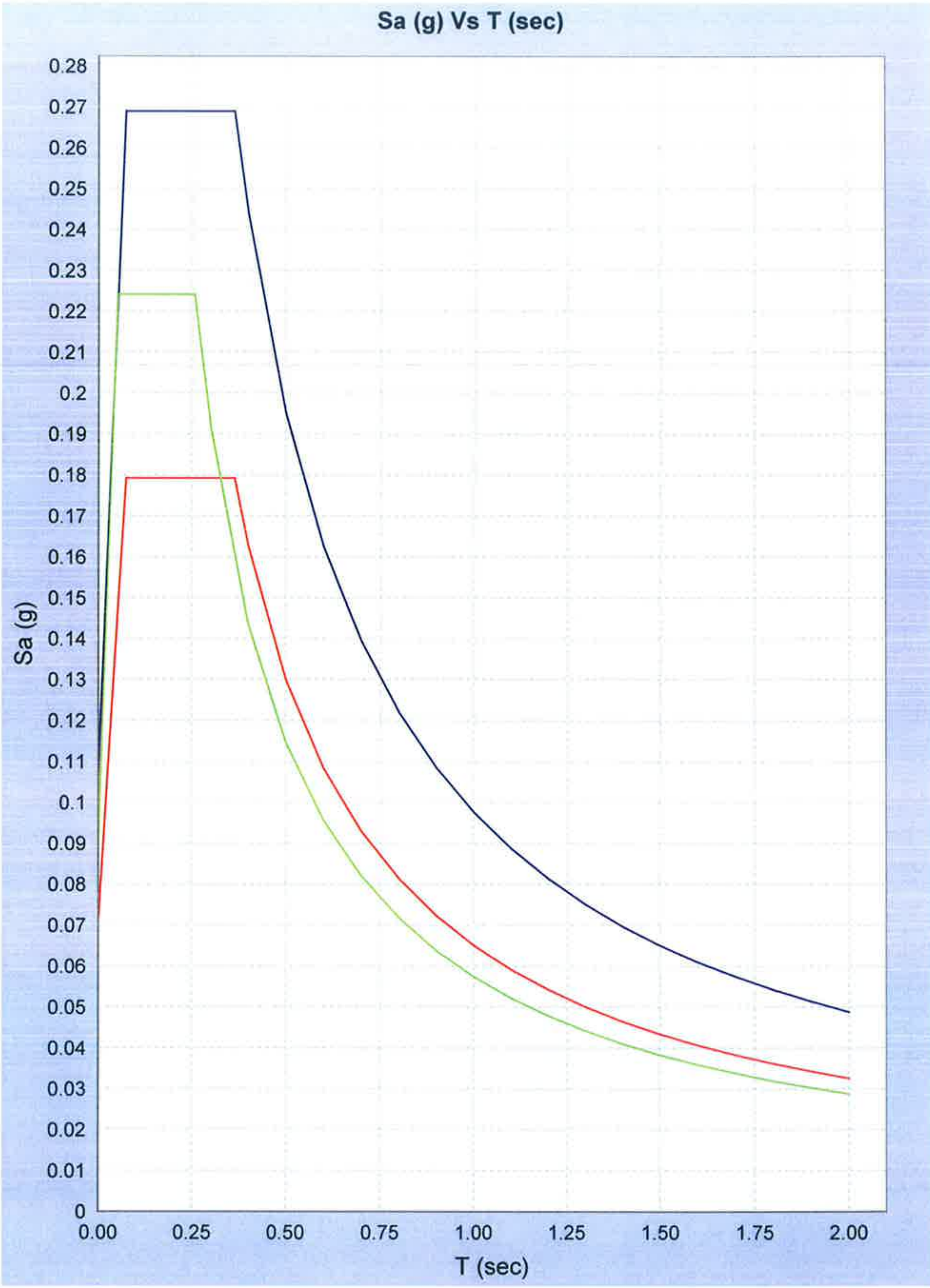
Conterminous 48 States
2009 International Building Code
Latitude = 39.733788
Longitude = -105.053893
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class C - Fa = 1.2 ,Fv = 1.7

Period (sec)	Sa (g)
0.2	0.269 (SMs, Site Class C)
1.0	0.098 (SM1, Site Class C)

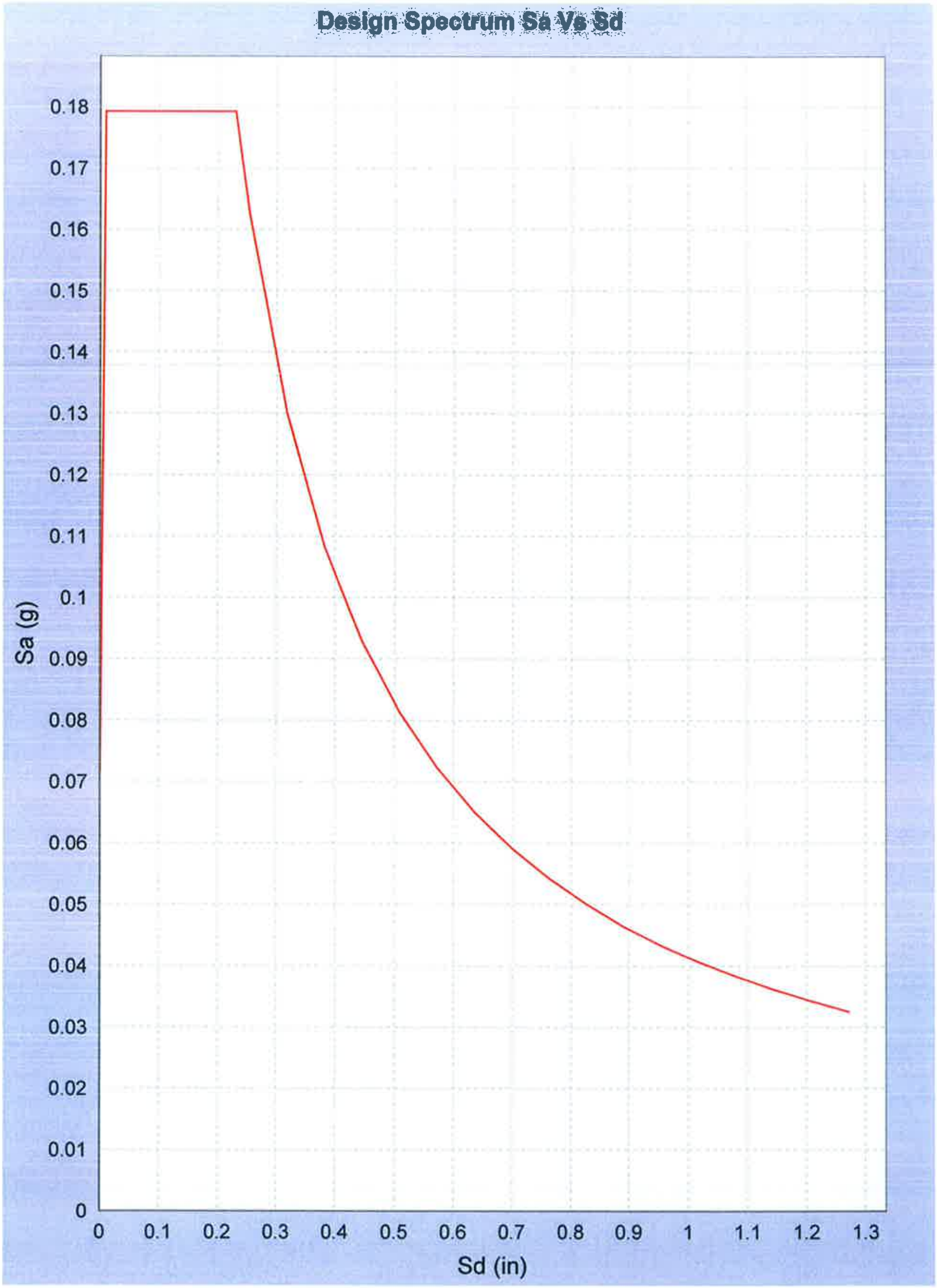
Conterminous 48 States
2009 International Building Code
Latitude = 39.733788
Longitude = -105.053893
Design Spectral Response Accelerations SDs and SD1
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class C - Fa = 1.2 ,Fv = 1.7

Period (sec)	Sa (g)
0.2	0.179 (SDs, Site Class C)
1.0	0.065 (SD1, Site Class C)


Design Spectrum Sa Vs T
Sa (g)
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
T (sec)



Sa (g) Vs T (sec)
Sa (g)
T (sec)
0.28
0.27
0.26
0.25
0.24
0.23
0.22
0.21
0.2
0.19
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00



Design Spectrum Sa Vs Sd
Sa (g)
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
Sd (in)

APPENDIX B

TRAFFIC DATA
AADT CALCULATIONS
ESAL CALCULATIONS
MGPEC SOFTWARE PRINTOUTS
MGPEC FORM # 9 TOP LIFT
MGPEC FORM # 9 INTERMEDIATE & BOTTOM LIFTS

C. EXISTING AND PROJECT TRAFFIC VOLUMES

1. EXISTING VOLUMES

Existing traffic counts are taken from the *West Corridor Environmental Impact Study, Appendix C: Sheridan Boulevard Park-n-Ride and Light Rail Station: Transportation Analysis for the West Corridor Project, January 2003*. Counts were taken in 2002 and projected for 2008 in the study. The Denver Regional Council of Governments' (DRCOG) regional travel model figures were used to compute an annual growth factor to apply to the 2008 values from the report. Thus, all background volumes, both existing and projected, stem from the original 2002 data collected in the Environmental Impact Study (EIS). It is important to note that the EIS 2008 volumes are considerably higher (10-25%) than the existing Sheridan Boulevard counts provided by DRCOG. To be conservative, the higher volumes from the EIS were used as a base. Additionally, the DRCOG counts did not include detailed intersection movement data.

Figure 6 illustrates the existing peak hour volumes, factored, to represent 2010 data.

2. TRIP GENERATION

The number of trips associated with the Sheridan Boulevard parking structure was also based on the *West Corridor Environmental Impact Study, Appendix C: Sheridan Boulevard Park-n-Ride and Light Rail Station: Transportation Analysis for the West Corridor Project, January 2003*. From this report, the year-of-opening was assumed to be 2008; since the opening year is now 2013, the data for 2008 will be used for 2013. Similarly, this referenced report assumed 100% utilization of the parking structure for the year 2025. As such, the trips will be assumed for this report's 2035 horizon year.

Table 1 provides the trip generation information for the AM and PM peak hours related to the opening year 2013 and 2035 horizon year.

Table 1
Trip Generation

Year	Parking Spaces	AM Peak Hour Trips			PM Peak Hour Trips		
		Inbound	Outbound	Total	Inbound	Outbound	Total
2013	800	203	69	272	83	232	314
2035	800	270	92	362	110	309	419

Source: Adopted from West Corridor Environmental Impact Study, Appendix C Transportation Analysis for the West Corridor Project

Figure 6
Existing (2010) Peak Hour Volumes

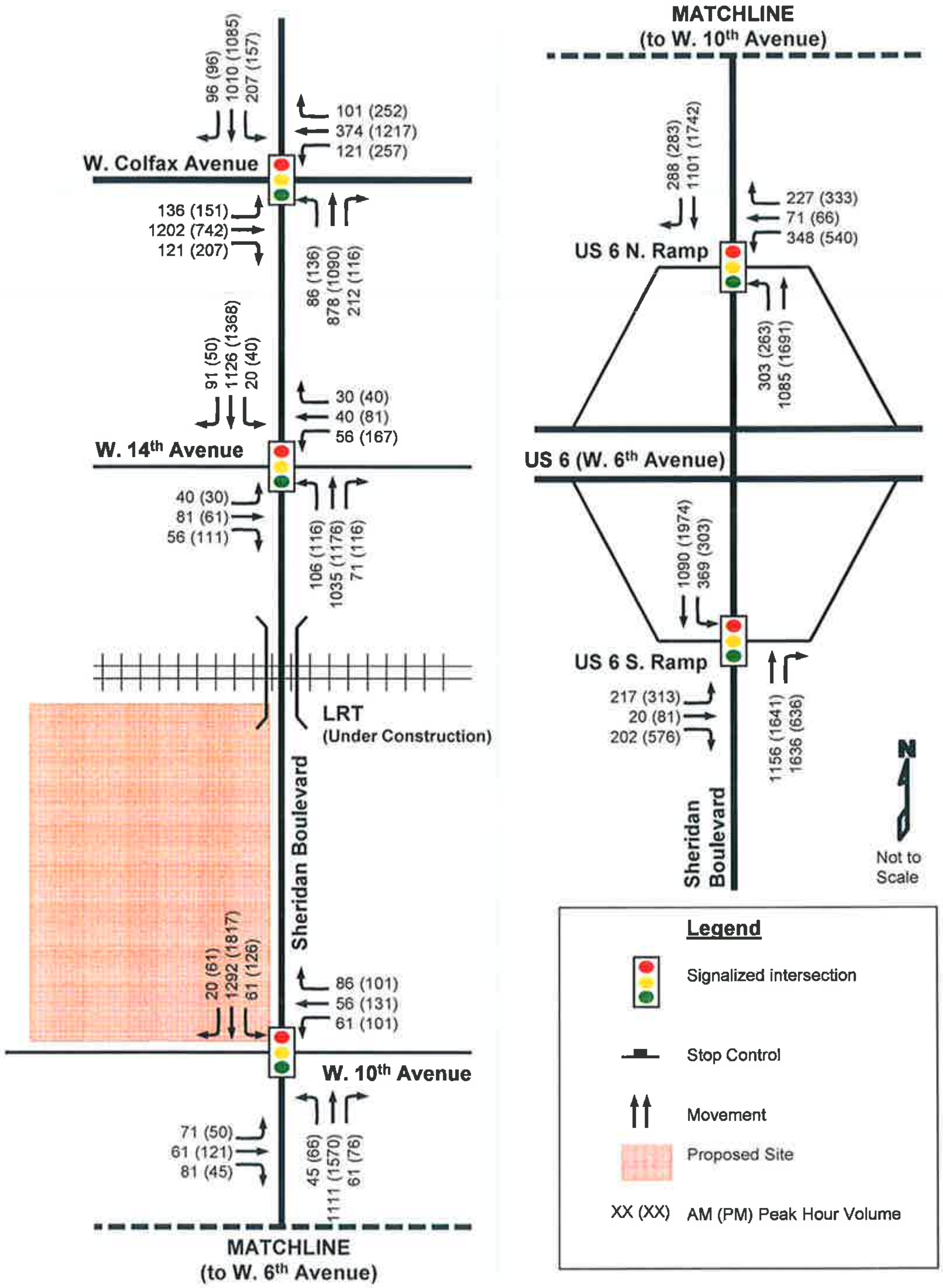


Figure 10
Opening Year (2013) Total Peak Hour Volumes

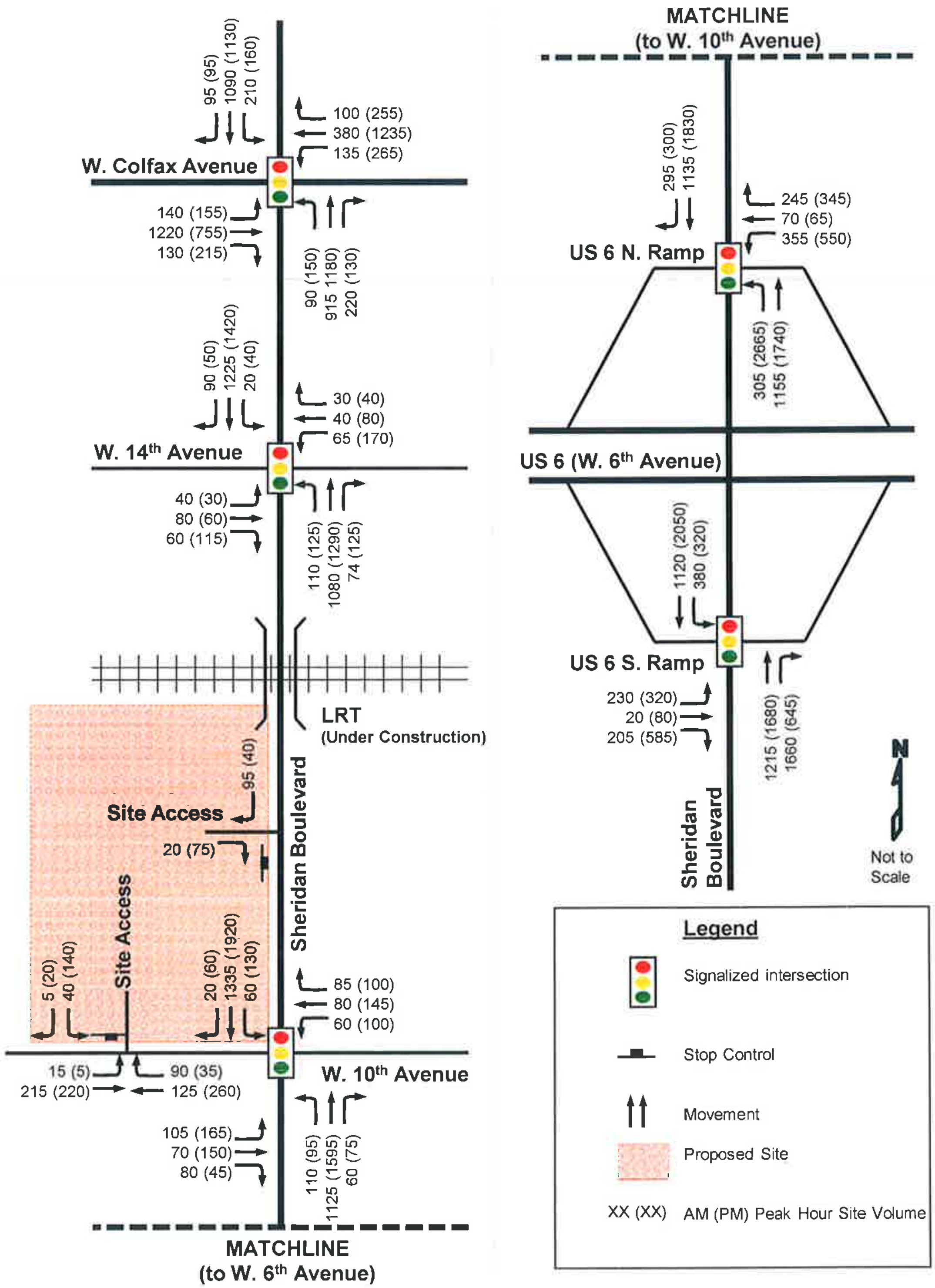
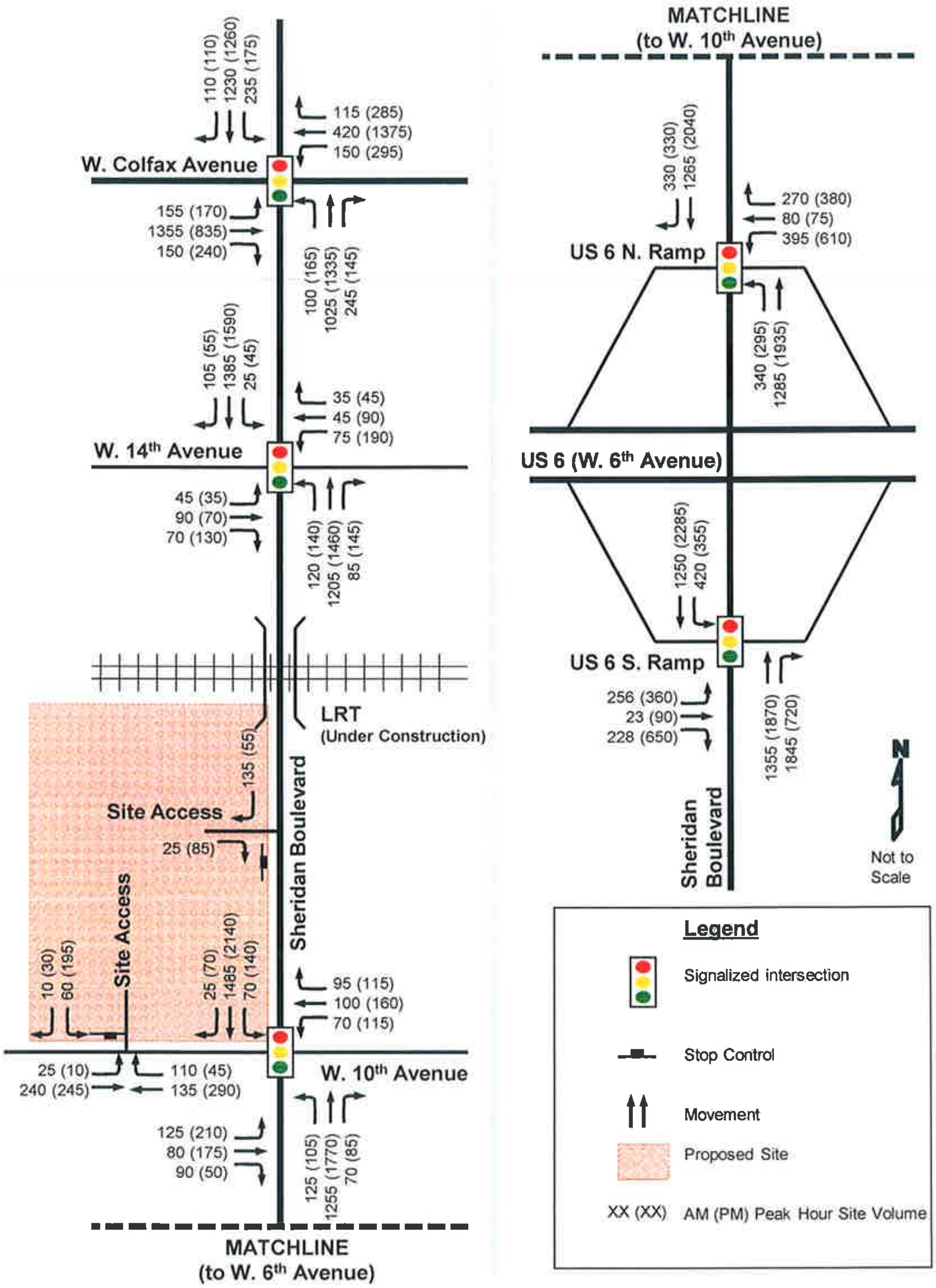


Figure 11
Horizon Year (2035) Total Peak Hour Volumes



Future Traffic Volumes for Highway 095
From RefPoint 4 To RefPoint 6

Route	Ref Point	End Ref Point	Start Point Description	AADT	AADTYR	AADT Single Trucks	AADT Comb. Trucks	Design Hour Volume (% of AADT)	AADT 2030	AADT Single Trucks 2030	AADT Comb. Trucks 2030
095A	4.436	5.018	ON SH 95, SHERIDAN BLVD S/O SH 6, 6TH AVE, DENVER	52,000	2010	940	420	8	60,320	1,090	487
095A	5.018	5.528	ON SH 95, SHERIDAN BLVD N/O SH 6, 6TH AVE, DENVER	39,000	2010	780	350	8	44,850	897	403
095A	5.528	6.031	ON SH 95, SHERIDAN BLVD N/O 10TH AVE, DENVER	32,000	2010	640	260	9	36,800	736	299
095A	6.031	6.534	ON SH 95, SHERIDAN BLVD N/O SH 40, COLFAX AVE, DENVER	33,000	2010	630	300	9	37,620	718	342

If you notice an error, bug or have any questions, Please E-mail us.

W. 10th Ave.

Peak Hour

For Roadway Pavement Design

2010	AM	PM	AVERAGE
NB Sharidan L turn	45	66	56
SB Sharidan R turn	20	61	41
WB W. 10th Ave. Thru	56	131	94
EB W. 10th Ave. R turn	81	45	63
EB W. 10th Ave. L turn	71	50	61
EB W. 10th Ave. Thru	61	121	91
TOTAL Average Peak Hour Volume:			404
80% Peak Hour			323
AADT			7,757

2035	AM	PM	AVERAGE
NB Sharidan L turn	125	105	115
SB Sharidan R turn	25	70	48
WB W. 10th Ave. Thru	100	160	130
EB W. 10th Ave. R turn	90	50	70
EB W. 10th Ave. L turn	125	210	168
EB W. 10th Ave. Thru	80	175	128
TOTAL Average Peak Hour Volume:			658
80% Peak Hour			526
AADT			12,624

2013	AM	PM	AVERAGE
NB Sharidan L turn	110	95	103
SB Sharidan R turn	20	60	40
WB W. 10th Ave. Thru	80	145	113
EB W. 10th Ave. R turn	80	45	63
EB W. 10th Ave. L turn	105	165	135
EB W. 10th Ave. Thru	70	150	110
TOTAL Average Peak Hour Volume:			563
80% Peak Hour			450
AADT			10,800

W. 10th Ave.

Peak Hour

For Intersection Pavement Design

2010	AM	PM	AVERAGE
WB W. 10th Ave. Thru	56	131	94
WB W. 10th Ave. L turn	61	101	81
EB W. 10th Ave. L turn	71	50	61
EB W. 10th Ave. Thru	61	121	91
TOTAL Average Peak Hour Volume:			326
80% Peak Hour			261
AADT			6,259

2035	AM	PM	AVERAGE
WB W. 10th Ave. Thru	100	160	130
WB W. 10th Ave. L turn	70	115	93
EB W. 10th Ave. L turn	125	210	168
EB W. 10th Ave. Thru	80	175	128
TOTAL Average Peak Hour Volume:			518
80% Peak Hour			414
AADT			9,936

2013	AM	PM	AVERAGE
WB W. 10th Ave. Thru	80	145	113
WB W. 10th Ave. L turn	60	100	80
EB W. 10th Ave. L turn	105	165	135
EB W. 10th Ave. Thru	70	150	110
TOTAL Average Peak Hour Volume:			438
80% Peak Hour			350
AADT			8,400

Sheridan Blvd.

Peak Hour

For Intersection Pavement Design

2010	AM	PM	AVERAGE
NB Sheridan L turn	45	66	56
NB Sheridan Thru	1111	1570	1,341
SB Sheridan L turn	61	126	94
SB Sheridan Thru	1292	1817	1,555
TOTAL Average Peak Hour Volume:			3,044
80% Peak Hour			2,435
AADT			58,445

2035	AM	PM	AVERAGE
NB Sheridan L turn	125	105	115
NB Sheridan Thru	1255	1770	1,513
SB Sheridan L turn	70	140	105
SB Sheridan Thru	1485	2140	1,813
TOTAL Average Peak Hour Volume:			3,545
80% Peak Hour			2,836
AADT			68,064

2013	AM	PM	AVERAGE
NB Sheridan L turn	110	95	103
NB Sheridan Thru	1125	1595	1,360
SB Sheridan L turn	60	130	95
SB Sheridan Thru	1335	1920	1,628
TOTAL Average Peak Hour Volume:			3,185
80% Peak Hour			2,548
AADT			61,152

Design Lane ESAL Calculations

Sheridan Parking Access	Vehicle Type/Classification (%)					
	Passenger Vehicles	Single Unit	Combination Unit			
Vehicle Type Load Factor (MGPEC)	0.0045	--	--	--	--	--

	Number of Lanes (per direction) =		1	% in Design Lane		60%
Precent of types	*100.00%*	*100.00%*				
2013 ADT Estimate	*5,626*	*5,626*	*0*	*0*	*0*	*0*	*0*
2035 ADT Estimate	*7,498*	Calculated Average Annual Increase		1.31%	22	Years
Projected 2033 ADT	7,299	7,299	--	--	--	--	--
20-Yr Design ADT	**6,463**	**6,463**	--	--	--	--	--
Roadway ESAL	**212,293**	**212,293**	--	--	--	--	--
Design Lane ESAL	***127,376***						

	AM Peak Tot.	PM Peak Tot	avg	80%
2013 ADT: 5626	272	314	293	234.4
2035 ADT: 7498	362	419	390.5	312.4

Design Lane ESAL Calculations

W. 10th Ave.		Vehicle Type/Classification (%)					
		Passenger Vehicles	Single Unit	Trash / Concrete	RTD Bus	Light Delivery	School Bus
Vehicle Type Load Factor (MGPEC)		0.0045	1.587	1.693	3.848	0.617	2.578
	Number of Lanes (per direction) =			1	% in Design Lane		60%
Precent of types	*100.00%*	*97.00%*	*2.00%*	*0.25%*	*0.25%*	*0.25%*	*0.25%*
2013 Average ADT	*10,800*	*10,476*	*216*	*27*	*27*	*27*	*27*
Projected 2035 ADT	*12,624*	Assumed Average Annual Increase			0.71%	22	Years
Projected 2033 ADT	12,441	12,068	249	31	31	31	31
20-Yr Design ADT	**11,621**	**11,272**	**233**	**29**	**29**	**29**	**29**
Roadway ESAL	**4,913,233**	**370,285**	**2,693,536**	**358,408**	**814,622**	**130,619**	**545,763**
Design Lane ESAL	***2,947,940***						

Design Lane ESAL Calculations

Sheridan Blvd. & W. 10th Ave. Intersection		Vehicle Type/Classification (%)					
		Passenger Vehicles	Single Unit	Trash / Concrete	RTD Bus	Light Delivery	School Bus
Vehicle Type Load Factor (MGPEC)		0.0045	1.587	1.693	3.848	0.617	2.578
	Number of Lanes (per direction) =			2	% in Design Lane		45%
Precent of types	*100.00%*	*97.00%*	*2.00%*	*0.25%*	*0.25%*	*0.25%*	*0.25%*
2013 Average ADT	*69,552*	*67,465*	*1,391*	*174*	*174*	*174*	*174*
Projected 2035 ADT	*78,000*	Assumed Average Annual Increase			0.52%	22	Years
Projected 2033 ADT	77,154	74,839	1,543	193	193	193	193
20-Yr Design ADT	**73,353**	**71,152**	**1,467**	**184**	**184**	**184**	**184**
Roadway ESAL	**31,034,993**	**2,337,343**	**16,995,342**	**2,267,858**	**5,154,588**	**826,502**	**3,453,360**
Design Lane ESAL	***13,965,747***						

PAVEMENT DESIGN TO MGPEC STANDARDS

SUBDIVISION

Subdivision	Sheridan Parking Structure						
Street	Driveway						
From	W. 10th Street						
To	Parking Structure						
Formation	Qs - Colluvium						
Township	Lakewood, CO.	*Range*		*Section*	0	*Quarter*	NW

TRAFFIC

Classification	Commercial	*Speed Limit*	15	Entered ***ESALS***	***127,376***
Residential Lots	0	*Commercial Acres*	0	*Industrial Acres*	0

SUBGRADE

Soil Type	Clay	*AASHTO*	A-7-6			*Subsurface Drainage*	No
R Value	0	*UNC*	1660	***Resilient Modulus***	***3897***		
Swell	3.8 %	*Liquid Limit*	43 %	*Plasticity Index*	25 %	*Passing 200*	54 %
Std Proctor	No	*Mod Proctor*	No	*Optimum Moisture*	19 %	*Max Density*	101 *pcf*
				Load Transfer	2.8 Doweled and Tied		

MATERIALS COSTS

Hot Mix Asphalt Concrete	1.80	*$/sqyd/in*	*Crack Seal - HMAP*	0.32	*$/sqyd*
Portland Cement Concrete	3.00	*$/sqyd/in*	*Milling - HMAP*	1.25	*$/sqyd/in*
Aggregate Base Course	0.59	*$/sqyd/in*	*Annual - HMAP Maintenance*	0.05	*$/sqyd*
Chemical Stablized Subgrad	0.80	*$/sqyd/in*	*Clean/Seal Crack And Joints*	0.72	*$/sqyd*
Moisture Treated Subgrade	0.25	*$/sqyd/in*	*Portland Surface Grinding*	1.50	*$/sqyd/in*
Fog Seal	0.25	*$/sqyd*	*Annual PCCP Maintenance*	0.05	*$/sqyd*
Chip Seal	0.75	*$/sqyd*	*Annual Interest Rate*	7.0	*%*
Slurry Seal	1.25	*$/sqyd*	*Annual Inflation Rate*	3.0	*%*

PAVEMENT DESIGN OPTIONS

Option One	*Portland Cement Concrete Pavement*	6.0	*Inches Thick*
	Construction Cost	$126,720	*Per Lane Mile*
	30 yr Maintenance	$28,469	*Per Lane Mile*
	Total Cost	$155,189	*Per Lane Mile*
Option Two	*Hot Mix Asphalt Pavement*	6.5	*Inches Thick*
	Construction Cost	$82,368	*Per Lane Mile*
	30 yr Maintenance	$78,533	*Per Lane Mile*
	Total Cost	$160,902	*Per Lane Mile*
Option Three	*Hot Mix Asphalt Pavement*	4.0	*Inches Thick*
	Chemical Stabilized Subgrade	8.0	*Inches Thick*
	Construction Cost	$95,744	*Per Lane Mile*
	30 yr Maintenance	$78,533	*Per Lane Mile*
	Total Cost	$174,278	*Per Lane Mile*

PAVEMENT DESIGN TO MGPEC STANDARDS

SUBDIVISION

Subdivision	Sheridan Parking Structure
Street	Driveway
From	W. 10th Street
To	Parking Structure
Formation	Qs - Colluvium
Township	Lakewood, CO. *Range* *Section* 0 *Quarter* NW

TRAFFIC

Classification	Commercial	*Speed Limit*	15	Entered ***ESALS***	***127,376***
Residential Lots	0	*Commercial Acres*	0	*Industrial Acres*	0

SUBGRADE

Soil Type	Clay	*AASHTO*	A-7-6	*Subsurface Drainage*	No		
R Value	0	*UNC*	1660	***Resilient Modulus***	***3897***		
Swell 3.8 %		*Liquid Limit*	43 %	*Plasticity Index*	25 %	*Passing 200*	54 %
Std Proctor	No	*Mod Proctor*	No	*Optimum Moisture*	19 %	*Max Density*	101 *pcf*

Load Transfer 4.2 No Reinforcement

MATERIALS COSTS

Hot Mix Asphalt Concrete	1.80	*$/sqyd/in*	*Crack Seal - HMAP*	0.32	*$/sqyd*
Portland Cement Concrete	3.00	*$/sqyd/in*	*Milling - HMAP*	1.25	*$/sqyd/in*
Aggregate Base Course	0.59	*$/sqyd/in*	*Annual - HMAP Maintenance*	0.05	*$/sqyd*
Chemical Stablized Subgrad	0.80	*$/sqyd/in*	*Clean/Seal Crack And Joints*	0.72	*$/sqyd*
Moisture Treated Subgrade	0.25	*$/sqyd/in*	*Portland Surface Grinding*	1.50	*$/sqyd/in*
Fog Seal	0.25	*$/sqyd*	*Annual PCCP Maintenance*	0.05	*$/sqyd*
Chip Seal	0.75	*$/sqyd*	*Annual Interest Rate*	7.0	*%*
Slurry Seal	1.25	*$/sqyd*	*Annual Inflation Rate*	3.0	*%*

PAVEMENT DESIGN OPTIONS

Option One	*Portland Cement Concrete Pavement*	6.5	*Inches Thick*
	Construction Cost	$137,280	*Per Lane Mile*
	30 yr Maintenance	$28,469	*Per Lane Mile*
	Total Cost	$165,749	*Per Lane Mile*
Option Two	*Hot Mix Asphalt Pavement*	6.5	*Inches Thick*
	Construction Cost	$82,368	*Per Lane Mile*
	30 yr Maintenance	$78,533	*Per Lane Mile*
	Total Cost	$160,902	*Per Lane Mile*
Option Three	*Hot Mix Asphalt Pavement*	4.0	*Inches Thick*
	Chemical Stabilized Subgrade	8.0	*Inches Thick*
	Construction Cost	$95,744	*Per Lane Mile*
	30 yr Maintenance	$78,533	*Per Lane Mile*
	Total Cost	$174,278	*Per Lane Mile*

PAVEMENT DESIGN TO MGPEC STANDARDS

SUBDIVISION

Subdivision	Sheridan Parking Structure				
Street	W. 10th Avenue				
From	Sheridan Blvd.				
To	Depew St.				
Formation	Qs - Colluvium				
Township	Lakewood, CO.	*Range*	*Section* 0	*Quarter*	NW

TRAFFIC

Classification	Commercial	*Speed Limit*	35	Entered ***ESALS***	**2,947,940**
Residential Lots	0	*Commercial Acres*	0	*Industrial Acres*	0

SUBGRADE

Soil Type	Clay	*AASHTO*	A-7-6			*Subsurface Drainage*	No
R Value	0	*UNC*	2540	***Resilient Modulus***	**5963**		
Swell 1.7%		*Liquid Limit*	44 %	*Plasticity Index*	27 %	*Passing 200*	59 %
Std Proctor	No	*Mod Proctor*	No	*Optimum Moisture*	17%	*Max Density*	103 *pcf*
				Load Transfer	2.8 Doweled and Tied		

MATERIALS COSTS

Hot Mix Asphalt Concrete	1.80	*$/sqyd/in*	*Crack Seal - HMAP*	0.32	*$/sqyd*
Portland Cement Concrete	3.00	*$/sqyd/in*	*Milling - HMAP*	1.25	*$/sqyd/in*
Aggregate Base Course	0.59	*$/sqyd/in*	*Annual - HMAP Maintenance*	0.05	*$/sqyd*
Chemical Stablized Subgrad	0.80	*$/sqyd/in*	*Clean/Seal Crack And Joints*	0.72	*$/sqyd*
Moisture Treated Subgrade	0.25	*$/sqyd/in*	*Portland Surface Grinding*	1.50	*$/sqyd/in*
Fog Seal	0.25	*$/sqyd*	*Annual PCCP Maintenance*	0.05	*$/sqyd*
Chip Seal	0.75	*$/sqyd*	*Annual Interest Rate*	7.0	*%*
Slurry Seal	1.25	*$/sqyd*	*Annual Inflation Rate*	3.0	*%*

PAVEMENT DESIGN OPTIONS

Option One	*Portland Cement Concrete Pavement*	8.5	*Inches Thick*
	Construction Cost	$179,520	*Per Lane Mile*
	30 yr Maintenance	$28,469	*Per Lane Mile*
	Total Cost	$207,989	*Per Lane Mile*
Option Two	*Hot Mix Asphalt Pavement*	11.0	*Inches Thick*
Not Recommended	*Construction Cost*	$139,392	*Per Lane Mile*
	30 yr Maintenance	$78,533	*Per Lane Mile*
	Total Cost	$217,926	*Per Lane Mile*
Option Three	*Hot Mix Asphalt Pavement*	7.5	*Inches Thick*
	Chemical Stabilized Subgrade	12.0	*Inches Thick*
	Construction Cost	$162,624	*Per Lane Mile*
	30 yr Maintenance	$78,533	*Per Lane Mile*
	Total Cost	$241,158	*Per Lane Mile*

PAVEMENT DESIGN TO MGPEC STANDARDS

SUBDIVISION

Subdivision	Sheridan Parking Structure
Street	W. 10th Avenue
From	Sheridan Blvd.
To	Depew St.
Formation	Qs - Colluvium
Township	Lakewood, CO.

Range *Section* 0 *Quarter* NW

TRAFFIC

Classification Commercial *Speed Limit* 35 Entered ***ESALS*** **2,947,940**

Residential Lots 0 *Commercial Acres* 0 *Industrial Acres* 0

SUBGRADE

Soil Type Clay *AASHTO* A-7-6 *Subsurface Drainage* No

R Value 0 *UNC* 2540 ***Resilient Modulus*** ***5963***

Swell 1.7% *Liquid Limit* 44 % *Plasticity Index* 27 % *Passing 200* 59 %

Std Proctor No *Mod Proctor* No *Optimum Moisture* 17% *Max Density* 103 *pcf*

Load Transfer 4.2 No Reinforcement

MATERIALS COSTS

Hot Mix Asphalt Concrete	1.80	*$/sqyd/in*	*Crack Seal - HMAP*	0.32	*$/sqyd*
Portland Cement Concrete	3.00	*$/sqyd/in*	*Milling - HMAP*	1.25	*$/sqyd/in*
Aggregate Base Course	0.59	*$/sqyd/in*	*Annual - HMAP Maintenance*	0.05	*$/sqyd*
Chemical Stablized Subgrad	0.80	*$/sqyd/in*	*Clean/Seal Crack And Joints*	0.72	*$/sqyd*
Moisture Treated Subgrade	0.25	*$/sqyd/in*	*Portland Surface Grinding*	1.50	*$/sqyd/in*
Fog Seal	0.25	*$/sqyd*	*Annual PCCP Maintenance*	0.05	*$/sqyd*
Chip Seal	0.75	*$/sqyd*	*Annual Interest Rate*	7.0	*%*
Slurry Seal	1.25	*$/sqyd*	*Annual Inflation Rate*	3.0	*%*

PAVEMENT DESIGN OPTIONS

Option One	*Portland Cement Concrete Pavement*	10.5	*Inches Thick*
	Construction Cost	$221,760	*Per Lane Mile*
	30 yr Maintenance	$28,469	*Per Lane Mile*
	Total Cost	$250,229	*Per Lane Mile*
Option Two	*Hot Mix Asphalt Pavement*	11.0	*Inches Thick*
Not Recommended	*Construction Cost*	$139,392	*Per Lane Mile*
	30 yr Maintenance	$78,533	*Per Lane Mile*
	Total Cost	$217,926	*Per Lane Mile*
Option Three	*Hot Mix Asphalt Pavement*	7.5	*Inches Thick*
	Chemical Stabilized Subgrade	12.0	*Inches Thick*
	Construction Cost	$162,624	*Per Lane Mile*
	30 yr Maintenance	$78,533	*Per Lane Mile*
	Total Cost	$241,158	*Per Lane Mile*

PAVEMENT DESIGN TO MGPEC STANDARDS

SUBDIVISION

Subdivision	Sheridan Parking Structure				
Street	Intersection				
From	Sheridan Blvd.				
To	W. 10th Avenue				
Formation	Qs - Colluvium				
Township	Lakewood, CO.	*Range*	*Section* 0	*Quarter*	NW

TRAFFIC

Classification	Commercial	*Speed Limit*	35	Entered ***ESALS***	***13,965,747***
Residential Lots	0	*Commercial Acres*	0	*Industrial Acres*	0

SUBGRADE

Soil Type	Clay	*AASHTO*	A-7-6	*Subsurface Drainage*	No		
R Value	0	*UNC*	2540	***Resilient Modulus***	***5963***		
Swell	1.7 %	*Liquid Limit*	43 %	*Plasticity Index*	26 %	*Passing 200*	58 %
Std Proctor	No	*Mod Proctor*	No	*Optimum Moisture*	17 %	*Max Density*	103 *pcf*

Load Transfer 2.8 Doweled and Tied

MATERIALS COSTS

Hot Mix Asphalt Concrete	1.80	*$/sqyd/in*	*Crack Seal - HMAP*	0.32	*$/sqyd*
Portland Cement Concrete	3.00	*$/sqyd/in*	*Milling - HMAP*	1.25	*$/sqyd/in*
Aggregate Base Course	0.59	*$/sqyd/in*	*Annual - HMAP Maintenance*	0.05	*$/sqyd*
Chemical Stablized Subgrad	0.80	*$/sqyd/in*	*Clean/Seal Crack And Joints*	0.72	*$/sqyd*
Moisture Treated Subgrade	0.25	*$/sqyd/in*	*Portland Surface Grinding*	1.50	*$/sqyd/in*
Fog Seal	0.25	*$/sqyd*	*Annual PCCP Maintenance*	0.05	*$/sqyd*
Chip Seal	0.75	*$/sqyd*	*Annual Interest Rate*	7.0	*%*
Slurry Seal	1.25	*$/sqyd*	*Annual Inflation Rate*	3.0	*%*

PAVEMENT DESIGN OPTIONS

Option One	*Portland Cement Concrete Pavement*	10.5	*Inches Thick*
	Construction Cost	$221,760	*Per Lane Mile*
	30 yr Maintenance	$28,469	*Per Lane Mile*
	Total Cost	$250,229	*Per Lane Mile*
Option Two	*Hot Mix Asphalt Pavement*	14.5	*Inches Thick*
Not Recommended	*Construction Cost*	$183,744	*Per Lane Mile*
	30 yr Maintenance	$78,533	*Per Lane Mile*
	Total Cost	$262,278	*Per Lane Mile*
Option Three	*Hot Mix Asphalt Pavement*	11.0	*Inches Thick*
	Chemical Stabilized Subgrade	12.0	*Inches Thick*
	Construction Cost	$206,976	*Per Lane Mile*
	30 yr Maintenance	$78,533	*Per Lane Mile*
	Total Cost	$285,510	*Per Lane Mile*

Agency: GEOCAL, INC.
Date: 3/21/2012

Project Number: 611.1423.000
Project Name: SHERIDAN PARKING STRUCTURE

MGPEC Form # 9 (1/26/2012)	• **Mixture Design Requirements for Hot Mix Asphalt Pavements (HMA)**

- Project Special Provision Sheet for Hot Mix Asphalt Pavements (HMA)

This MGPEC Form #9 is a **mandatory part of the bid documents**, and shall be filled out by the AGENCY for each mix specified. The Contractor shall include a copy of this form with each Mix Design submittal after the contract is awarded.

Street Classification: PARKING ACCESS (examples: Residential, Collector, Arterial, Industrial, Parking Lot or actual name for Project)

→ Construction Application: ☒ Top Lift ☐ Intermediate Lift(s) ☐ Bottom Lift
☐ Patching ☐ Other________________

→ Aggregate Gradation:
☐ Grading ST (1.5" or less lifts, 3/8" NMPS)
☒ Grading SX (2.5" or less lifts)
☐ Grading S (2.5+" to 3.5" lifts)
☐ Grading SG*[1] (3.5" or thicker lifts)
SMA (Top lift only) ☐ 3/8" ☐ ½" ☐ ¾"

*[1]Note = Grading SG depends on approved texture of mix, Grading SG lower lift(s) only.

→ RAP Quantity, Maximum: ☐ 0% ☐ 20% ☐ 25%

Notes: - A quality control plan for RAP will be required when RAP is used
- Top lift Maximum RAP content allowed is 20%

→ Superpave Gyratory Mix Design Compaction Level, Recommended usage and Recommend binder(s):

Design Level	Recommended Traffic Levels	Recommended PG Binder(s)
☐ N_{design}=50	Low volume	☐ PG 58-28 or ☒ PG 64-22
☒ N_{design}=75	0 to <3 million ESALs	☐ PG 64-22 or ☐ PG 58-28
☐ N_{design}=100	3 million to <30 million ESALs	☐ PG 64-22 or ☐ PG 76-28

Notes: - The binders are shown in order they should be considered.
- Polymer modified PG Binders are typically used in the top lift only
- PG 58-28 Binder recommended for residential developments with less than 2 million ESAL's

- Target job Mix Optimum Asphalt Content Selection, Choose target % as close to 4.0 as possible (3.5% to 4.5% air voids per MGPEC 2008)
- Target Job Mix optimum Binder content for SMA grading at 3.0% to 4.0% air voids

**Warm mix asphalt (WMA) is allowed as an alternate to hot mix asphalt provided that all material requirements and specification standards are met and as approved by the Agency.

A completed MGPEC Form #9 shall supplement the MGPEC Construction Specifications defining the contract specific requirements of Item 9: Hot Mix Asphalt Pavement (HMA). Refer to the Specifications for details.

Agency: GEOCAL, INC.
Date: 3/21/2012

Project Number: 611.1423.000
Project Name: SHERIDAN PARKING STRUCTURE

MGPEC Form # 9 (1/26/2012)	• Mixture Design Requirements for Hot Mix Asphalt Pavements (HMA)

• Project Special Provision Sheet for Hot Mix Asphalt Pavements (HMA)

This MGPEC Form #9 is a **mandatory part of the bid documents**, and shall be filled out by the AGENCY for each mix specified. The Contractor shall include a copy of this form with each Mix Design submittal after the contract is awarded.

Street Classification: PARKING ACCESS (examples: Residential, Collector, Arterial, Industrial, Parking Lot or actual name for Project)

→ Construction Application: ☐ Top Lift ☒ Intermediate Lift(s) ☒ Bottom Lift ☐ Patching ☐ Other________________

→ Aggregate Gradation:
☐ Grading ST (1.5" or less lifts, 3/8" NMPS)
☐ Grading SX (2.5" or less lifts)
☒ Grading S (2.5+" to 3.5" lifts)
☐ Grading SG*1 (3.5" or thicker lifts)
SMA (Top lift only) ☐ 3/8" ☐ ½" ☐ ¾"

*1Note = Grading SG depends on approved texture of mix, Grading SG lower lift(s) only.

→ RAP Quantity, Maximum: ☐ 0% ☐ 20% ☐ 25%

Notes: - A quality control plan for RAP will be required when RAP is used
- Top lift Maximum RAP content allowed is 20%

→ Superpave Gyratory Mix Design Compaction Level, Recommended usage and Recommend binder(s):

Design Level	Recommended Traffic Levels	Recommended PG Binder(s)
☐ N_{design}=50	Low volume	☐ PG 58-28 or ☒ PG 64-22
☒ N_{design}=75	0 to <3 million ESALs	☐ PG 64-22 or ☐ PG 58-28
☐ N_{design}=100	3 million to <30 million ESALs	☐ PG 64-22 or ☐ PG 76-28

Notes: - The binders are shown in order they should be considered.
- Polymer modified PG Binders are typically used in the top lift only
- PG 58-28 Binder recommended for residential developments with less than 2 million ESAL's

- Target job Mix Optimum Asphalt Content Selection, Choose target % as close to 4.0 as possible (3.5% to 4.5% air voids per MGPEC 2008)
- Target Job Mix optimum Binder content for SMA grading at 3.0% to 4.0% air voids

**Warm mix asphalt (WMA) is allowed as an alternate to hot mix asphalt provided that all material requirements and specification standards are met and as approved by the Agency.

A completed MGPEC Form #9 shall supplement the MGPEC Construction Specifications defining the contract specific requirements of Item 9: Hot Mix Asphalt Pavement (HMA). Refer to the Specifications for details.