GEOCAL, INC.
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30% DESIGN
SOIL AND FOUNDATION INVESTIGATION

Proposed
6th Avenue Freeway over BNSF Bridge Replacement
City and County of Denver, Colorado

Prepared For

Wilson & Company, Inc.
Attn: Scott Waterman, P.E.
999 18th Street, Suite 2600
Denver, Colorado 80202

December 19, 2011

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December 19, 2011
G10.1354.003

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1.0 PURPOSE AND SCOPE

This 30% design level report contains the results of a preliminary soil and foundation investigation conducted for the proposed replacement of the existing three-span 6th Avenue Bridge over Burlington Northern Santa Fe (BNSF) Railroad in Denver, Colorado. The project is being conducted under the Colorado Bridge Enterprise Program administered under the direction of Colorado Department of Transportation (CDOT). Affected rights-of-way are controlled by City and County of Denver, BNSF Railroad, and CDOT.

A field subsurface investigation was conducted to obtain information on pavement, soil, bedrock, and ground water conditions. Soil and bedrock samples were visually classified, and selected samples were laboratory tested to evaluate strength, compressibility or swell characteristics, classification, chemical properties, and other engineering properties.

The results of the field and laboratory investigations were analyzed to develop preliminary recommendations for foundations, retaining walls, and pavement sections for the approach areas to the bridge. We understand that this project will be continued under a design/build procedure and that the design/build contractor will be responsible for final design. The investigation was conducted in general accordance with our Subconsultant Agreement/Subcontract No. 001 (WCI File No. 11-100-30102) with Wilson & Co., Inc, dated March 11, 2011. The investigation is identified by CDOT as "Task Order #2 "Preliminary Design of 6th Ave Bridge over BNSF Railroad".

This report has been prepared to summarize the data obtained and to present our preliminary conclusions and recommendations based on the proposed construction and the subsurface conditions encountered. Design parameters and a discussion of geotechnical engineering considerations related to construction of the proposed structures are included. Limited environmental monitoring and sampling was conducted by others (Pinyon Environmental) during Geocal's drilling operations; however, investigations of general environmental issues related to possible hazardous materials at the site are beyond the scope of this study.

2.0 PROPOSED CONSTRUCTION

The proposed construction is expected to consist principally of replacing the existing bridge with a similar two-span structure with abutments, pier and major wingwalls near their current locations. Changes to the grade, approach alignments, approach embankment fills and new deck alignment are expected to be minor. The project may include construction of new pavements in the approach areas to the bridge. The pavements for the bridge approaches are expected to be at or relatively close to existing site grades.

3.0 SITE CONDITIONS

The project site is situated on the transition between lower terraces the eastern pre-controlled floodplain of the South Platte River and an area of upper terraces associated with slightly elevated ground (sometimes refered to as Lincoln Park Uplands) between the South Platte River and Cherry Creek Valley. The original natural terraces have been modified during the development of transportation, industrial-commercial, and drainage control projects in the area. The north-flowing South Platte River is about one-half mile west of the site and separated from the site by the 6th Avenue/I-25 Interchange. 6th Avenue within the project area is elevated on constructed embankments and bridge structures continuously from west of the river (about three-quarters mile west of the project site) to about one-half mile east (North Klamath Street) along which it crosses over, west to east, the South Platte River, I-25 Freeway, BNSF Rail Corridor, Osage Street, Consolidated Main Line and Light Rail Corridor, and other city streets.

The bridge crosses over a rail corridor having two mainline and two siding heavy rail tracks. Tracks are all under the wider west span with rail beds about 25-feet below bottom of girders. The east span is underlain by steep concrete slope pavement extending from near track level to just below base of girders. Embankment slopes near the bridge are covered with sparse grass, weeds, brush and scattered deciduous trees. Areas east of the bridge are occupied by light industrial and warehouse-type businesses in low-rise structures; land to the west is essentially dedicated to I-25 right-of-way.

4.0 SITE GEOLOGY

Standard quadrangle-scale published geologic mapping indicates that natural (pre-construction) unconsolidated surficial and shallow deposits include:

1. near river floodplain soil assigned to the Post-Piney Creek Alluvium generally as interbeds and mixtures of humic clay, silt, sand, and occasional small gravel. Thicknesses (where not removed by construction) of 5-feet to 10-feet are typical. Local, but significantly thick lenses of highly humic bog clays and silt have been noted. Mapping indicates Post-Piney Creek soil covered the surface west of the current track corridor.

2. The upper floodplain terrace soil identified as Piney Creek Alluvium and typified as well stratified clay, silt and sand (including mixtures of) that are commonly humic in the uppermost and gravelly near the base. The Piney Creek Alluvium has been reported as 5-feet to 10-feet in thickness and mapped as originally covering the surface east of the tracks and indicated as extending under portions of Post-Piney Creek deposits.

3. Older upper terrace deposits assigned to Broadway Alluvium, as moderately well-graded sand and gravel with generally limited fines. These deposits are mapped on higher terraces east and west of the South Platte River and interpreted as commonly extending under Piney Creek and Post-Piney Creek soils in the project area.

These soils are indicated to lie on well-stratified sedimentary bedrock assigned to the Denver-Arapahoe Formations (undifferentiated). At depths associated with potential construction in the area, members of the formations are typically dominated by claystone and siltstone interbeds with lesser interbeds and lenses of sandstone. Outcrops or construction excavated exposures of this material are mapped within a mile of the project site and are refered to in published reports of previous nearby soil test borings and water wells. Published mapping indicates bedrock to have about 20-feet of natural alluvium cover (excluding embankment fills) in the vicinity of the bridge and to be flat to very gently dipping.

5.0 SUBSURFACE INVESTIGATION

The subsurface investigation for this project was conducted from October 31st through November 9th, 2011 by drilling six exploratory borings at the approximate locations shown on Figure 1, Locations of Exploratory Borings. Additional borings were planned within the BNSF right-of-way, near track level, but at the time of this report had not been permitted by BNSF. The borings were advanced with a truck-mounted CME-75 drill rig equipped with 3¼ -inch inside diameter (ID) hollow stem augers, and were logged by a Geocal representative. Subsurface soil and bedrock samples were obtained using 2-inch ID California liner samplers and 1-3/8 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 hardness of bedrock. Drive samples were taken at approximately five to ten foot intervals. Larger bulk samples of auger cuttings were collected from about the upper 1-foot to 10-feet of selected borings. Depths at which samples were taken, penetration resistance values and groundwater levels encountered are shown on Figure 2, Logs of Exploratory Borings. Description of the materials encountered and symbols used on the logs are presented on Figure 3, Legend and Notes for Exploratory Borings.

During drilling of portions of Borings 1 and 6, a representative of Pinyon Environmental, Inc. (Pinyon) conducted limited environmental monitoring and sampling, outside of Geocal's scope, but part of total project design program. While drilling approximately 15-feet above and below groundwater level in these borings, open hole air, auger cuttings, and drive samples were monitored for total organic compounds and explosive limits using a field-portable photo ionization detector. Additionally, bailed samples of groundwater were collected once groundwater was encountered in the borings. The results of the field and laboratory investigations are being provided by Pinyon Environmental and are reported elsewhere.

6.0 SUBSURFACE CONDITIONS

As shown on the Figure 2, subsurface conditions varied slightly between the borings. In general, the drilled intervals included relatively thick sections of man-placed embankment fill (artificial fill) followed by natural mostly granular soils over sedimentary (claystone) bedrock. Five of the six borings were drilled through roadway or shoulder pavement consisting of 6-inches to 7-inches of asphalt: no specifically identified aggregate base course material was encountered in the borings. Boring B-1 (near the northwest corner of the bridge) was drilled in off-road right-of-way covered with sparse grass.

The borings encountered man-placed embankment fill (artificial fill) from below pavement or at the surface to depths of about 35-feet to 39-feet deep. The fill generally consisted of loose to medium dense slightly clayey to silty sand to gravelly sand that graded to medium stiff to stiff clayey sand to sandy clay. The fill was generally medium to coarse grained, had low to high plasticity for clay portions, small to large gravel where present, was moist, and light to dark brown. Asphalt, construction debris, and pieces of glass were found in the lower portions of the fill in some of the borings.

Below the artificial fill, the borings encountered natural soils comprised of medium dense (with some loose and very dense zones) of gravel with sand, silt and some clayey zones. In Boring 1, natural soil consisting of medium dense sand with some gravel was encountered below the fill. A thin layer (4-feet) of silt with sand and some organics was encountered in Boring 6 below the fill. The natural granular soils extended to the bedrock surface. The gravel generally was comprised small to medium sized gravel that were rounded to subrounded, had coarse to medium grained sand, was wet, and brown to grey. Some organic silt layers were encountered within the gravel.

Sedimentary bedrock was encountered at about 48-foot to 53-foot depths and extended to the maximum depth explored, 85 feet. The bedrock was comprised of mostly of claystone that was very hard, had medium to high plasticity, contained varying amounts of silt and fine grained sand, was moist, and blue to dark grey. The claystone did contain some small interbeeded lenses of sandstone.

Ground water was measured at about 32-foot to 40-foot depths in the borings immediately after drilling. Boring 1 was left temporarily covered for approximately 24-hours; and the water level was about 40-feet. Groundwater levels may fluctuate significantly depending on seasonal precipitation and levels of South Platte River flow. Borings were backfilled with gravel and cement mixture after drilling (with the exception of Boring 1) and compacted with the weight of the drill rig. The borings conducted in 6th Avenue were patched with a minimum of 9-inches of Transpatch© High Strength Early set grout that was mixed on site.

7.0 LABORATORY TESTING

Laboratory tests conducted on selected soil and bedrock samples consisted of natural moisture contents, dry densities, liquid and plastic limits (Atterberg Limits), grain size distribution (gradation), swell-compression, unconfined compression, R-value, water-soluble sulfate concentrations, and chemical analysis. Laboratory test results are shown on Figures 4 through 21 and summarized on Tables 1 and 2.

Swell-Compression Tests: Swell-compression tests are a direct measurement of compressive or expansive potential for a particular sample when wetted. Measurements were made by loading the sample in a consolidometer to a light surcharge pressure, subjecting the sample to wetting, then allowing the specimen to swell or compress. After stabilization, additional loads were applied with each load increment given the opportunity to stabilize. Swell-compression tests were performed in accordance with local practice on samples of the fill soils consisting of clay and clayey sand and claystone bedrock.

Results are shown on Figures 4 through 7 and indicate no to low swell potential under light load and wetting for the samples of soil and bedrock. Relatively low swell pressures were measured for samples that swelled. The samples (soil and bedrock) showed low to moderate compressibility under increased loading.

Atterberg Limits and Gradations: Atterberg limits and gradation analyses were used to classify the soils according to the American Association of State Highway and Transportation Officials (AASHTO) classification system. These tests also help provide a qualitative assessment of engineering properties. Gradation analysis and Atterberg Limits test results are presented on Figures 8 through 10 and summarized in Table 1.

The Atterberg Limits tests indicate that the fines content of the man-placed (artificial) fill generally had low to high plasticity and underlying granular natural soils had no plasticity. An elastic silt was measured for a sample from Boring 6 at 34 feet. Tests on the underlying claystone bedrock indicate generally medium plasticity.

The combined gradation and Atterberg Limits indicate that most of the embankment fill soils encountered classified as A-6 soils with some A-1-a material encountered in the upper portion of the embankment. Lower natural granular soils typically classified as A-1-b type soils.

R-Value: Selected bulk samples from the upper embankment fill were tested for R-value. The R-value is an indication of the ability of the soil to transfer traffic loading laterally. Figures 11 through 13 show R-values of 60, 62 and below 5 which indicate relatively high strength (and quality) to very low strength indicating highly variable pavement support characteristics for the near surface embankment fill materials encountered.

Unconfined Compressive Strength: The unconfined strength is a measurement of compressive strength under axial loading without lateral confinement. The test is useful in evaluating soil or bedrock strength and bearing capacities and the results are shown on Figures 14 through 21 and summarized on Table 1. The values ranged from 3,910 pounds per square foot (psf) to 17,860 psf for the samples of claystone and sandstone bedrock tested, and 1,750 psf to 3,000 psf for the clay fill soils tested.

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 and bedrock. Sulfate solutions react with tri-calcium aluminate hydrate, which is a normal constituent of Portland Cement concrete, forming calcium sulfo-aluminate hydrate with an accompanying substantial volume expansion which causes cracking. Sulfate expansion problems will typically exist when the soils have concentrations in excess of 0.10%.

The concentrations of water-soluble sulfates measured on selected samples of soil and bedrock ranged from 0.05 to 0.18%. The test results indicate a Class 1 "Severity of Sulfate Exposure" in accordance with Table 601-2 of the Colorado Department of Transportation (CDOT) Standard Specifications for Road and Bridge Construction (2011 Edition). For preliminary design, Class 1 requirements as defined in Section 601.04 Sulfate Resistance should be used for concrete exposed to the near surface soils and bedrock encountered within the project area. During the final design, additional sulfate concentration tests should be performed, as needed. Water soluble sulfate test results are summarized in Table 2.

Other Chemical Tests: Laboratory test results on selected samples of soil and bedrock indicate electrical resistivities in the range of approximately 460 ohm-cm to 4,500 ohm-cm, pH values in the range of 6.3 to 7.6, and chloride concentrations in the range of approximately 0.0015 percent to 0.067 percent. Sulfides were varied from positive to negative detection. A summary of the chemical tests conducted are presented in Table 2.

8.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

Two foundation types, driven H-piles and drilled shafts, both supported by the underlying bedrock appear to be suitable for use at this site. Driven H-piles will likely encounter refusal within a few feet of the bedrock surface and may be designed for the structural capacity of the piles. Drilled shafts will likely have to be installed with slurry and casing to control ground water, caving, and potentially flowing material. The two foundation types are discussed in the following sections.

8.1 Driven Piles

Preliminary recommendations presented in this section are based on the "AASHTO LRFD Bridge Design Specifications" manual, the subsurface data obtained, our experience, and local geotechnical engineering practice. Installation of driven piles should be in accordance with Section 502 "Piling" of the *Standard Specifications for Road and Bridge Construction (2011)*, by the Colorado Department of Transportation (CDOT standard specifications) and applicable Standard Special Provisions.

1. Piles may consist of heavy steel H-sections consisting of Grade A50 steel or higher and driven to refusal in the underlying bedrock. Refusal criteria should be determined during construction using the Pile Driving Analyzer (PDA) in accordance with Section 502 of the CDOT specifications, latest edition.

2. The pile driving contractor should provide the results of a GRLWeap drivability analysis for the pile driving equipment proposed for use, and the type of pile in accordance with the CDOT specifications prior to pile driving operations.

3. Due to the presence of granular soils underlying the embankment, use of a driving shoe or pre-drilling may be required to drive the pile through the granular soils and into the underlying bedrock.

4. A combined side shear and end bearing nominal capacity of 45 kips per square inch (ksi) times the cross sectional area may be used for grade A50 steel for preliminary design. Load and resistnace factors used for final design should be consistent with correct LFRD procedures, as established by AASHTO.

8.2 Drilled Shafts

Drilled shafts also appear feasible from a geotechnical consideration. Casing and slurry installation methods will be required to control caving and ground water. The design and construction criteria presented below should be observed for a drilled shaft foundation system. Installation should be in accordance with Section 503 – Drilled Caissons of the CDOT *Standard Specifications for Road and Bridge Construction (2011)*, by the Colorado Department of Transportation (CDOT standard specifications) and applicable Standard Special Provisions.

1) For preliminary design, drilled shafts may be designed for an ultimate end bearing pressure of 115,000 psf and ultimate side shear value of 11,500 psf for that portion of the foundation in competent bedrock. Load and resistance factors used for final design should be consistent with current LFRD procedures as established by AASHTO.

2) The presence of water and caving soils encountered in the exploratory borings indicates that casing and slurry construction methods will be required to reduce water infiltration and caving. If water cannot be removed, or if it is impractical to remove the water prior to placement of concrete, then concrete should be placed using an approved tremie method. The contractor should be advised that water bearing sandstone layers may be encountered.

8.3 Lateral Load Capacity

The following preliminary recommendations are based on the structural engineer using the computer program LPILE for the lateral load analysis. We recommend that the granular soils be modeled as dense sand and bedrock as hard clay. A rangemodulus values are presented below to allow the structural engineer to evaluate possible soil-structure responses under varying conditions and assumptions.

Lateral Capacity Parameters
For Drilled Shaft or Driven Pile Foundations

Soil Type	Total Unit Weight (pcf)	Cohesion, c (psf)	Friction Angle (ϕ)	k-static (pci)	ε_{50}
Artificial Fill (Embankment Soils)	125	0	28	75-100	0.020
Natural Granular Soils (Submerged)	65	0	32	---	---
Bedrock	125	5,000	0	2,000-3000	0.003

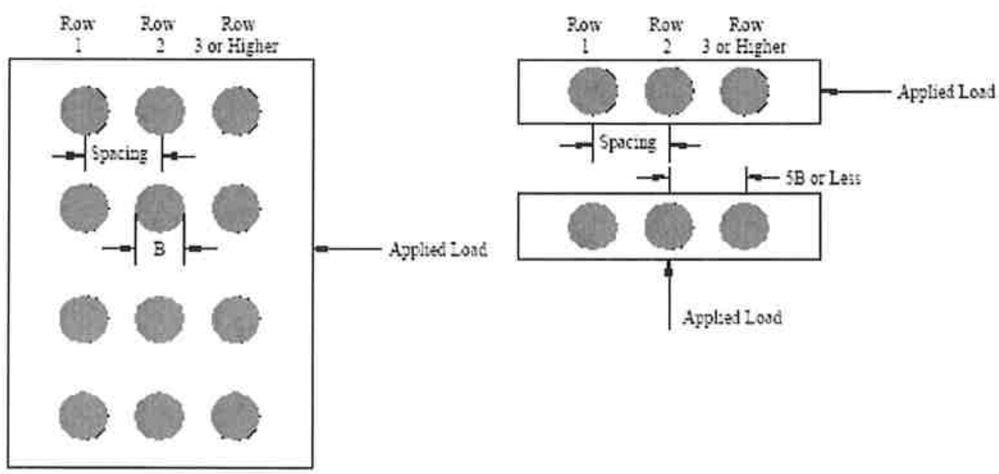
Reductions in lateral capacity for loading perpendicular to the line of shafts or piles will not be required if center to center spacing of 5 shaft or pile diameters or more between adjacent drilled shafts or piles is maintained.

For lateral loads parallel to the line of shafts/piles, 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 Article 10.7.2.4 of the 2007 AASHTO LRFD Bridge Design Specifications 4th Edition Manual. A sketch of the loading and how the rows are referenced is also shown.

P-Multipliers
Drilled Shaft or Driven Pile 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 or Pile



9.0 RETAINING STRUCTURES

The recommendations presented below should be considered preliminary. Additional explorations, analysis, and design recommendations will be required once retaining wall types, locations, and geometries have been determined. We have assumed that new retaining walls will likely be required for the abutments and wing walls and will be constructed either within the embankment or near the base of the existing embankment.

9.1 Gravity and Cantilever Walls

Gravity or cantilevered retaining walls should be supported by the same foundation system as the bridge foundations (driven piles or drilled shafts) and designed based on the recommendations provided in the previous sections. 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. For preliminary design, the following ultimate earth pressure coefficients may be used for imported Class 1 material.

Material	Active (K_a)	At-Rest (K_o)	Passive (K_p)	γ_T – Unit Weight (pcf)	Friction Angle (ϕ), degrees
Imported Class 1	0.28	0.44	3.53	130	34

Lateral wall movements or rotation of at least 0.1% of the wall height is typically required to develop the full active case, whereas lateral movement of at least 2% 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. Imported material should meet CDOT Class 1 structure backfill grading requirements. Equivalent fluid unit weights should be taken as follows:

Above ground water:	γ_{eq}	=	$\gamma_T \times K_{a,o,p}$
Below ground water:	γ_{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, etc. The buildup of water behind a wall or an upward sloping backfill surface will increase the lateral pressure imposed on retaining structures. An under-drain or weep holes 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 because this could cause excessive lateral wall loading.

10.0 UNDERDRAIN SYSTEM

Below grade structures should be provided with an underdrain system which will help prevent buildup of hydrostatic pressure. The underdrain 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. Free draining granular material used in the drain system should conform to the requirements for Class B filter material as specified in the CDOT standard specifications.

11.0 SITE GRADING

Based on the limited explorations and the anticipated construction activities, excavation of the onsite materials should be possible with conventional heavy duty excavating equipment. Most of the embankment material is expected to vary between granular (sand and gravel) and fine grained (clay) soils and vary in relative quality. Below the embankment fill, the natural soils are anticipated to be mostly granular soils. Soils used for support of pavements should be granular and meet the minimum strength requirements as determined during final design. Additional subsurface investigations should be conducted to better define material properties and suitability of use.

12.0 PRELIMINARY PAVEMENT DESIGN

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade without overstressing the subgrade soils. Performance of the pavement structure is a function of a number of factors including but not limited to the physical properties of the subgrade soils, drainage, climate, and traffic loading. The preliminary pavement sections presented in this section are based on laboratory test results and CDOT and AASHTO design procedures, and apply to the 6th Avenue approaches to the bridge.

General Design Parameters: The following parameters were used:

General	
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability	95%
Drainage Coefficient	1.0
Growth Factor	3.0%
Concrete	
Overall Standard Deviation	0.34
Loss of Support	1.0
Modulus of Rupture	650 psi
Concrete Modulus of Elasticity	3.4 million psi
Load Transfer Coefficient (doweled and tied)	2.8
Asphalt	
Structural Coefficient (HMAP)	0.44
Structural Coefficient (ABC)	0.12

Traffic Loading and ESAL Calculations: The CDOT web site was used to obtain Annual Average Daily Traffic Volumes for 6th Avenue from near the intersection of Sheridan Boulevard. These volumes were then utilized to determine the 20-year 18-kip Equivalent Single Axle Loadings (ESAL) for asphalt pavement and 30-year ESALs for concrete pavements based on CDOT. The CDOT website presents the route and reference points (mile posts) and provides the traffic data for those points and year the data was gathered. The data is displayed as annual average daily traffic (AADT), the breakdown of single unit and combination unit trucks. The information from the website is presented in Appendix A of this report. However these preliminary values will need to be evaluated and adjusted for final design.

The total number of traffic lanes (6) was used for the design. This allows the site to apply a lane factor (30% of the total traffic) to account for a roadway with 3 lanes in each direction, and truck factors are applied to the volume of different types of vehicles (passenger, single unit and combination unit trucks) with 60% of the truck traffic applied to the design lane. We assumed a 3 percent growth rate (a 1.806 Traffic Factor) for the project and calculated traffic volumes for 20 years and 30 years based on the 2010 traffic volumes (115,000 ADT). Vehicle distribution used was: 96.5% passenger vehicles, 2.4% single unit trucks, and 1.1% combination unit trucks.

An 18-kip Equivalent Single Axle Load (ESAL) is the equivalent 18,000 pound axle loading for the different vehicle types, and the design period ESALs are the total number of equivalent loadings to asphalt and concrete pavements for the design period. The following ESAL values were calculated.

New Construction

$ESAL_{20}$ = 4,825,027 (HMAP)

$ESAL_{30}$ = 11,011,767 (PCCP)

Subgrade Soil Strength Coefficients: The pavement subgrade soils encountered classified between A-1-a and A-6(9) in accordance with the AASHTO classification system with laboratory R-values measured from 62 to less than 5, indicating a high variability for the pavement subgrade soil in the approach areas. These values also indicate good to very poor subgrade support characteristics. For design purposes, we assigned an R-value of 50, indicating that any poor subgrade (R-value less than 50) encountered within the pavement areas will need to be subexcavated a minimum of 3 feet and replaced with R-value 50 or better material. A resilient modulus of 13,168 was determined based on the CDOT equations 2.1 and 2.2 in the 2012 Pavement design manual. For rigid pavement thickness calculations, a k-value (modulus of vertical subgrade reaction) of 175 pounds per cubic inch (pci) was chosen based on Table 2.3 of the CDOT 2012 Pavement Design Manual. The values utilized for design are:

R-Value	Resilient Modulus (psi)	k-value (pci)
50	13,168	175

Pavement Thickness Recommendations: Hot Mix Asphalt Pavement (HMAP) thickness sections were calculated using AASHTOWare DARWin software, following CDOT and AASHTO guidelines. Portland Cement Concrete Pavement (PCCP) thickness sections were calculated using the AASHTO 1998 Rigid Pavement Design Guide software provided by the FHWA. The recommended pavement thickness sections are shown below. Design printouts are included in Appendix A.

Subgrade Material	Full Depth HMAP 20-year (in)	HMAP Over ABC 20-year (in)	Full Depth PCCP 30-Year (in)
R- Value = 50	8½	7 (HMAP)/6(ABC)	10½ /6(ABC)

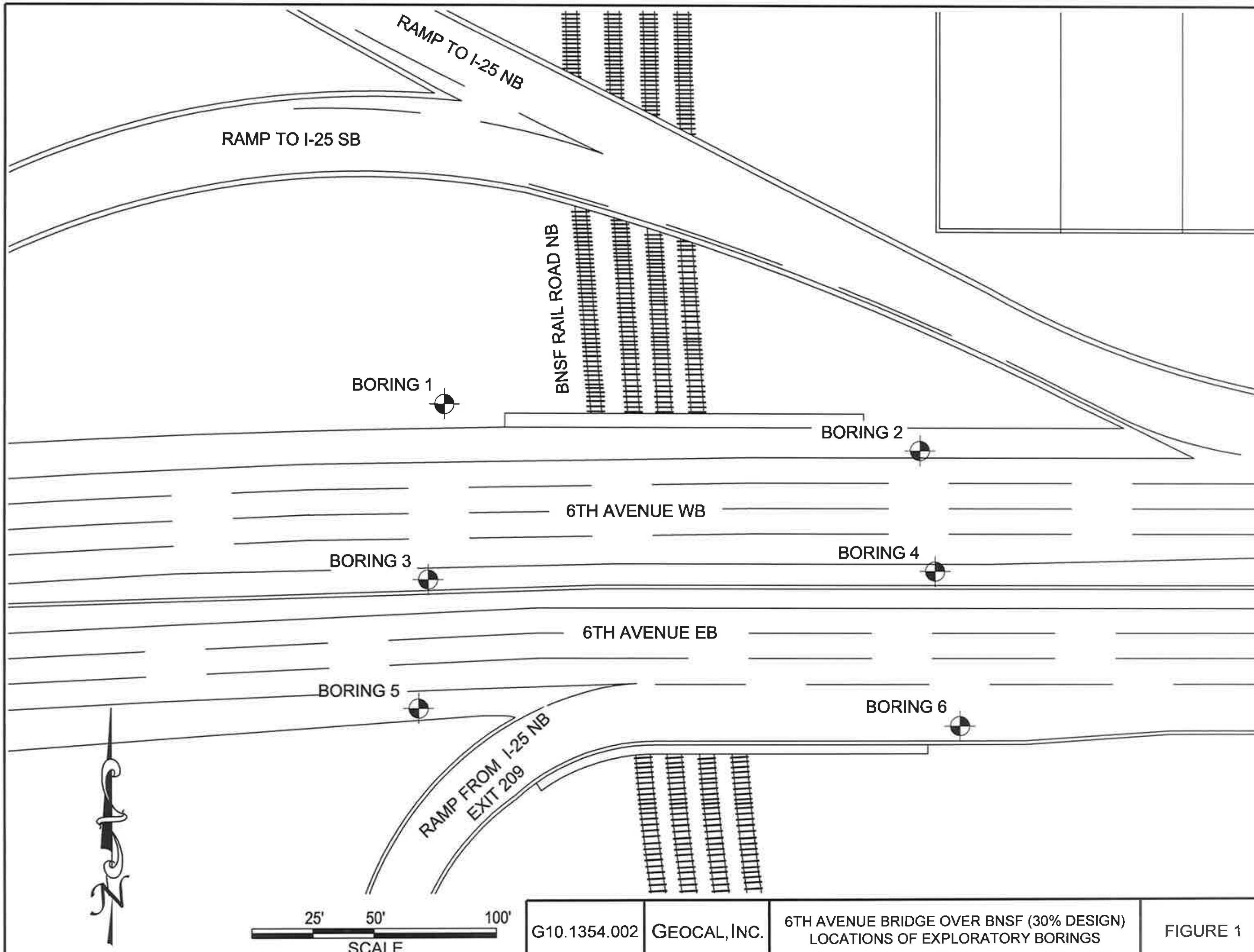
HMAP = Hot Mix Asphalt Pavement
ABC = Aggregate Base Course (CDOT Class 6)
PCCP = Portland Cement Concrete Pavement

13.0 LIMITATIONS

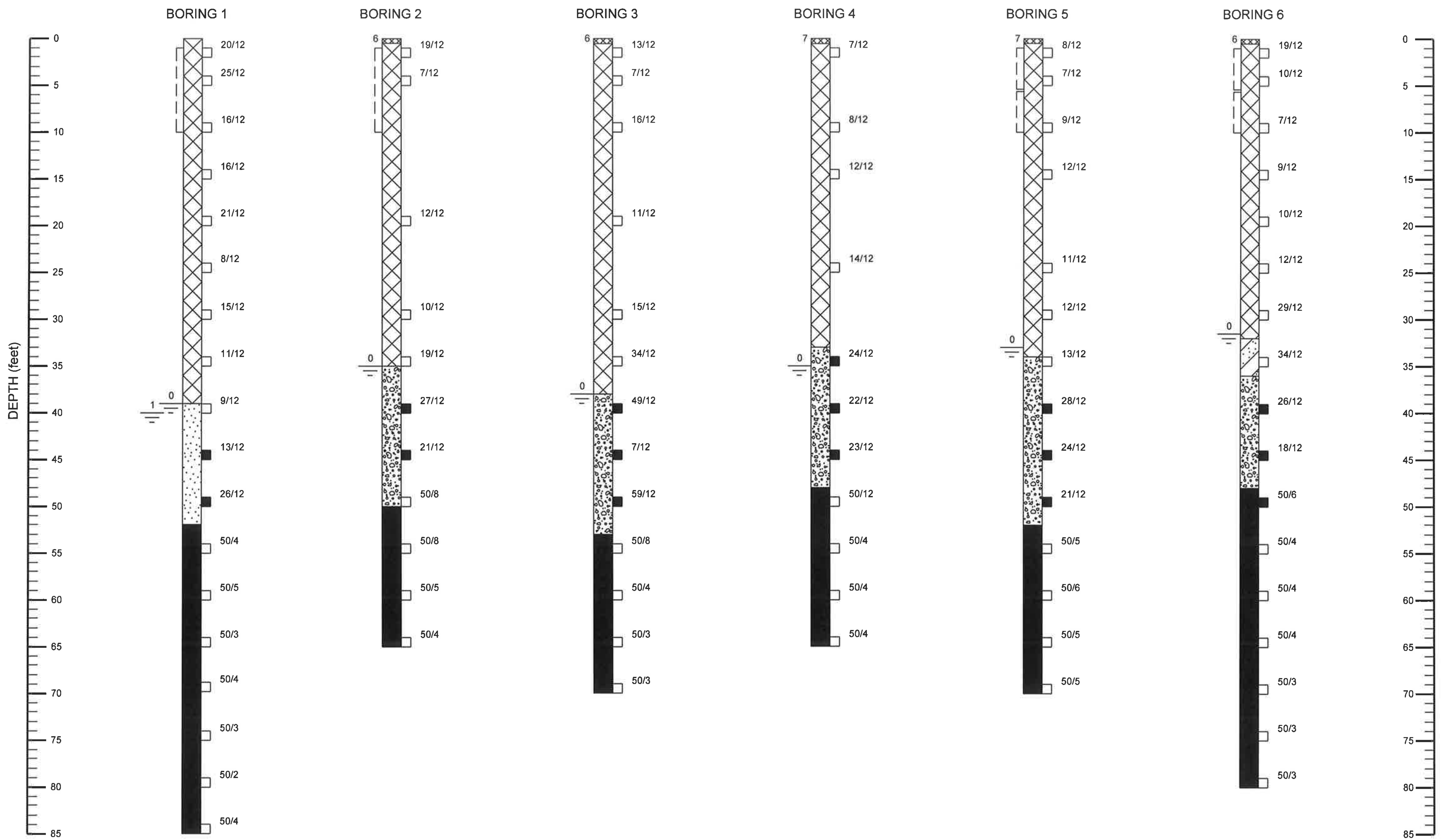
This 30% design level report has been prepared in accordance with generally accepted geotechnical engineering practices in this area, and is provided for use by the client for preliminary design purposes. The conclusions and recommendations submitted in this report are based upon the data obtained from the exploratory borings drilled at the approximate locations shown on Figure 1. Additional explorations for the structures, walls, and pavements recommended for final design. The nature and extent of variations between the borings may not become evident until excavation is performed

Geocal's professional services were performed using the degree of care and skill ordinarily exercised by reputable geotechnical engineers practicing in this or similar environments. No warranty expressed or implied is made. Geocal is not responsible for the interpretation of the site surface and subsurface conditions by others that are not consistent with the contents of this report.

Investigations into the occurrence or potential occurrence of hazardous materials, or other environmental assessments that may be applicable to the site are beyond the scope of services represented by this report. On-site observation of excavations and foundation bearing strata and testing of geotechnical materials by a representative of this office is recommended.



G10.1354.002	GEOCAL, INC.	6TH AVENUE BRIDGE OVER BNSF (30% DESIGN) LOCATIONS OF EXPLORATORY BORINGS	FIGURE 1



G10.1354.002	GEOCAL, INC.	6TH AVENUE BRIDGE OVER BNSF (30% DESIGN) LOGS OF EXPLORATORY BORINGS	FIGURE 2

LEGEND

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

FILL, gravel and sand with silt, trace clay, occassionally cobbley, medium dense with some loose zones, moist, fine to coarse grained sand, small to medium gravel, light to medium brown, fill grades to clayey sand to sandy clay with depth, medium dense, medium stiff to stiff, low to high plasticity, fine to coarse grained sand, some asphalt and glass debris encountered near base.

SAND with GRAVEL, medium dense, medium to coarse grained sand, small gravel, wet, brown.

SAND and SILT, dense, fine grained, low plasticity, wet, black to dark brown, some organic material.

GRAVEL, medium dense to very dense some loose zones, small to medium gravel, rounded to sub-rounded, wet, brown to dark brown, some clay and sand seams.

CLAYSTONE BEDROCK, mostly with slight sand and slight silt to silty, very hard, slightly moist to moist, blue to dark gray, very fine to fine grained sand, contains some sandstone lenses.

20/12 Drive sample blow count, Indicates that 20 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.

0,1 Indicates depth to water level and number of days after drilling measurement was made.

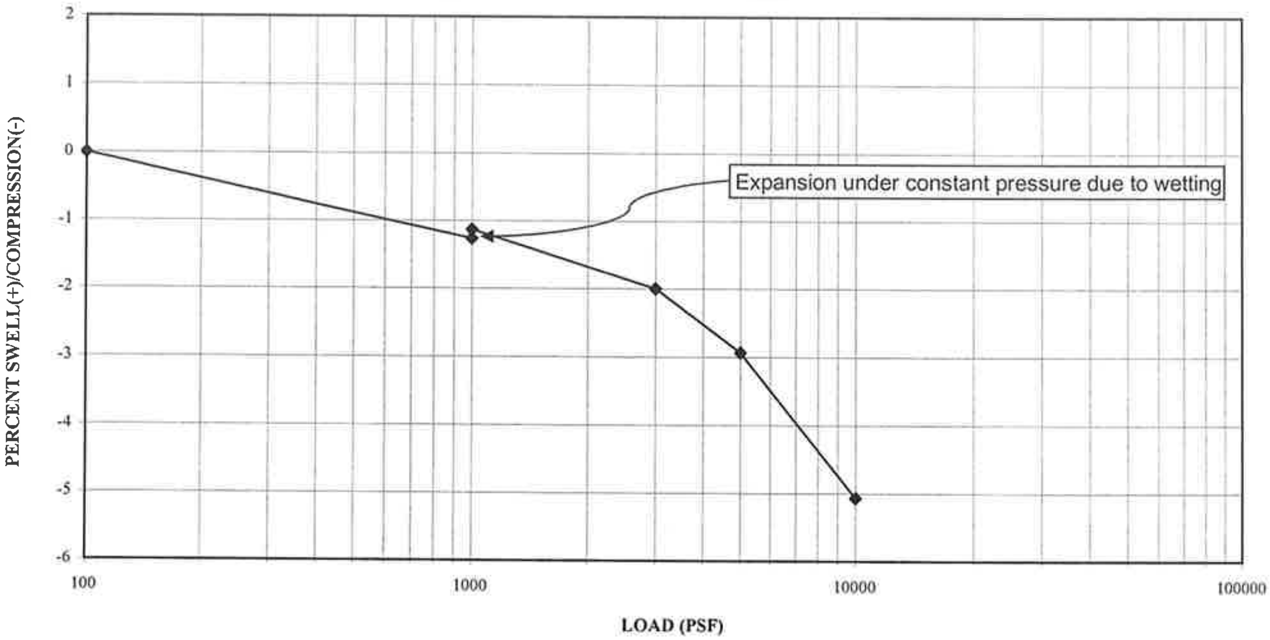
Indicates disturbed bulk sample.

NOTES

1. Borings were drilled on October 31 to November 9 of 2011 with a CME-75 drill rig and $3\frac{1}{4}$-inch inside diameter hollow-stem augers.
2. Locations 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. Boring logs 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.

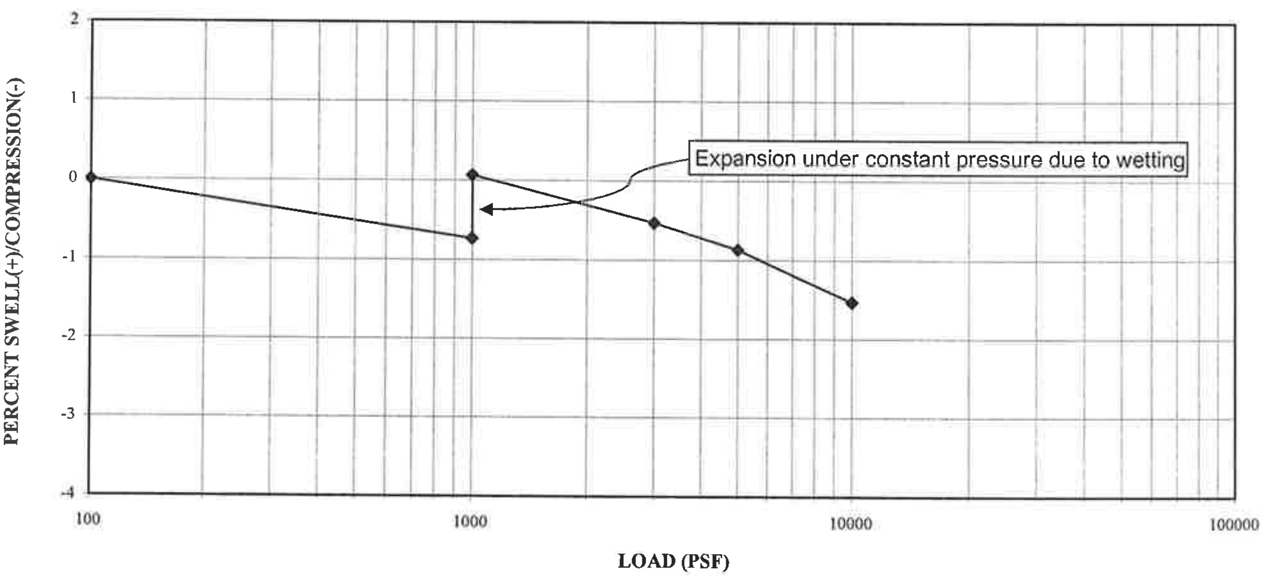
G10.1354.002	GEOCAL, INC.	6TH AVENUE OVER BNSF (30% DESIGN) LEGEND AND NOTES FOR EXPLORATORY BORINGS	FIGURE 3

SWELL-COMPRESSION TEST



Sample Location	Boring 1
Sample Depth	24 feet
Sample Description	Sandy lean clay, fill
USCS Classification	CL
AASHTO Classification	

Dry Density	107 pcf
Moisture Content	20.7 %
Volume Change	0.1 %
Swell Pressure	0 psf

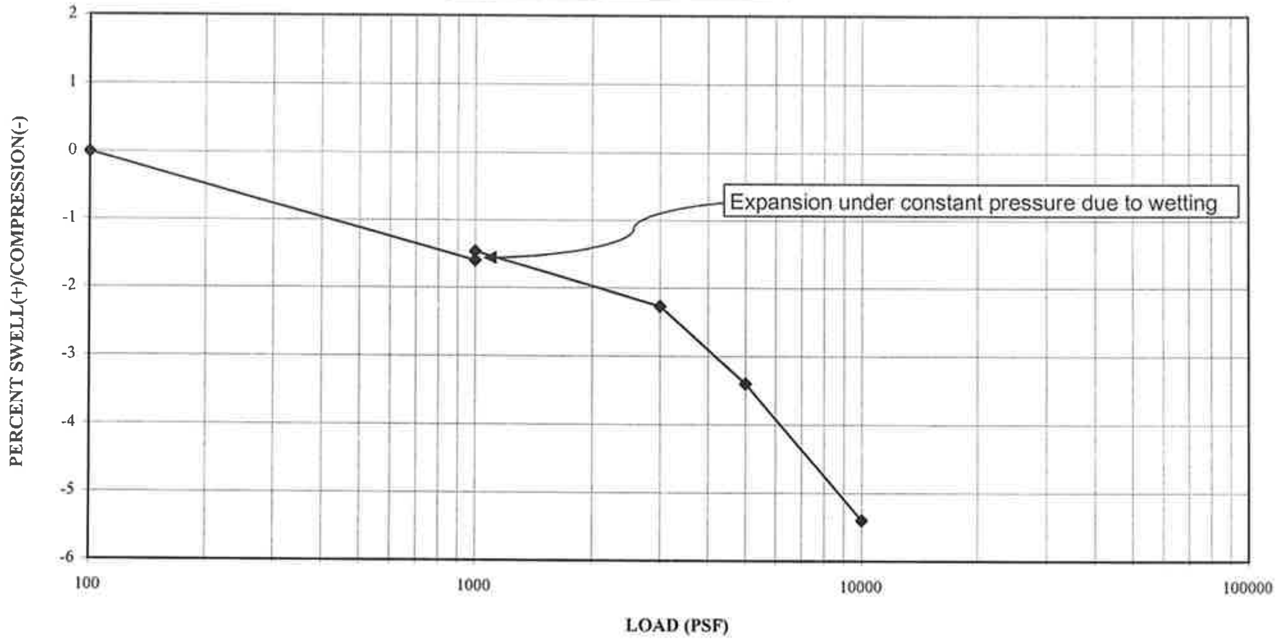


Sample Location	Boring 1
Sample Depth	59 feet
Sample Description	Claystone bedrock
USCS Classification	
AASHTO Classification	

Dry Density	112 pcf
Moisture Content	17.3 %
Volume Change	0.8 %
Swell Pressure	1,030 psf

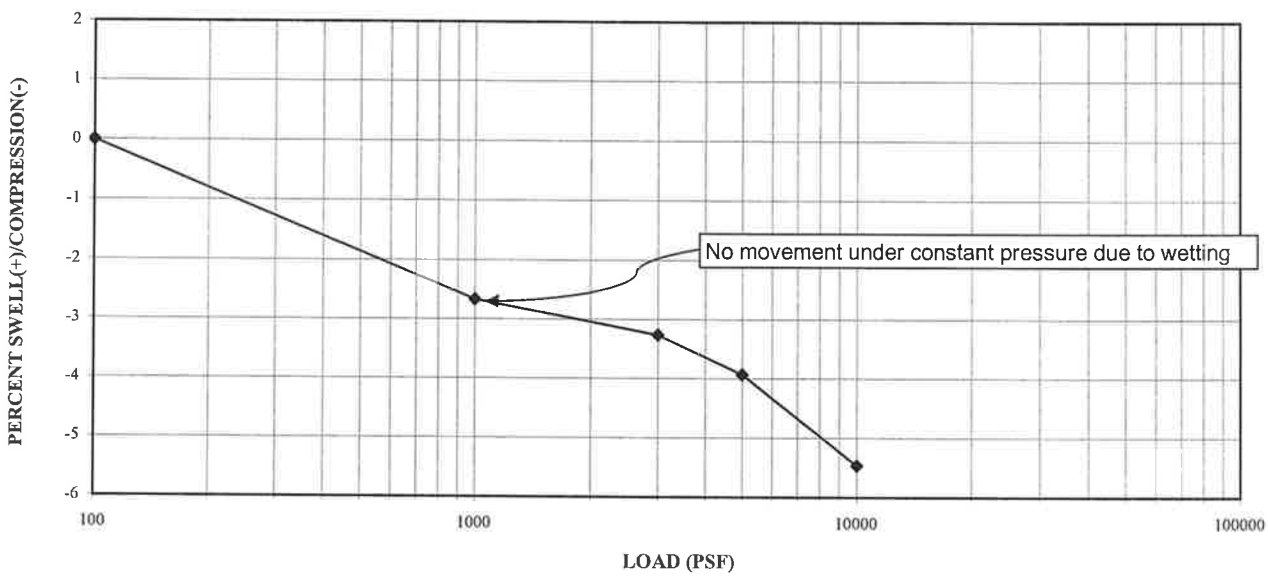
GEOCAL, INC.	**6th Avenue over BNSF**	**JOB NO.**	**G10.1354.002**
	SWELL - COMPRESSION TEST RESULTS	**FIGURE NO.**	**4**

SWELL-COMPRESSION TEST



Sample Location	Boring 3
Sample Depth	34 feet
Sample Description	Fat clay with sand, fill
USCS Classification	CH
AASHTO Classification	A-7-6(25)

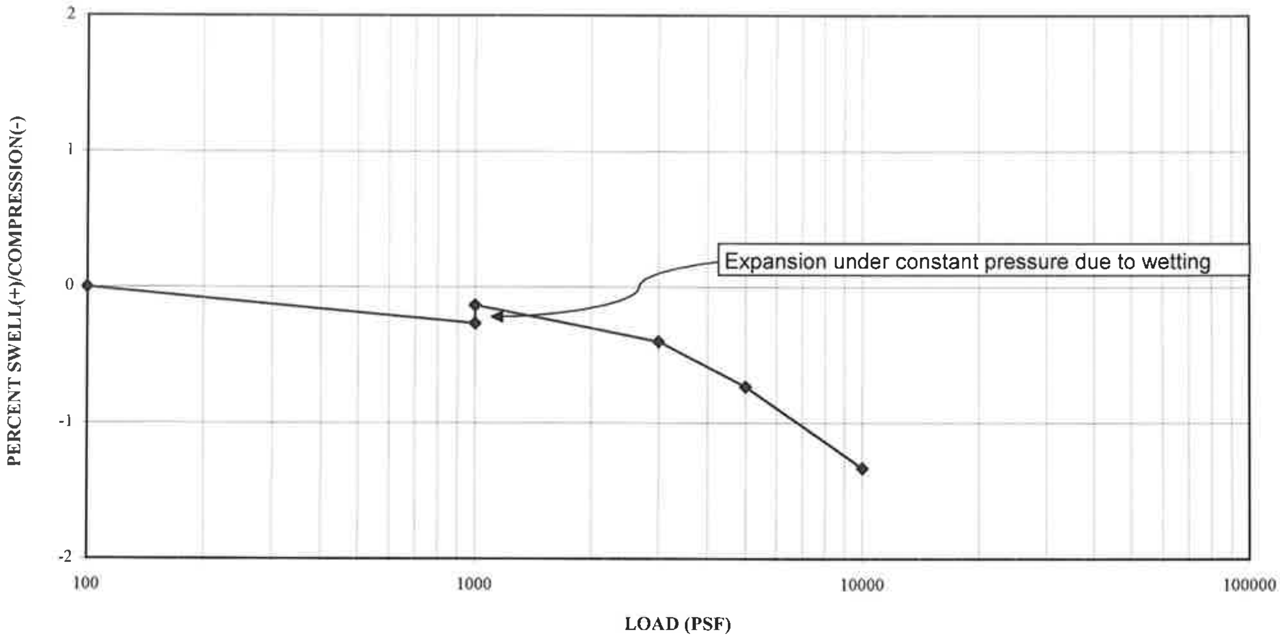
Dry Density	92 pcf
Moisture Content	32.9 %
Volume Change	0.1 %
Swell Pressure	0 psf



Sample Location	Boring 4
Sample Depth	24 feet
Sample Description	Clayey sand with gravel, fill
USCS Classification	SC
AASHTO Classification	

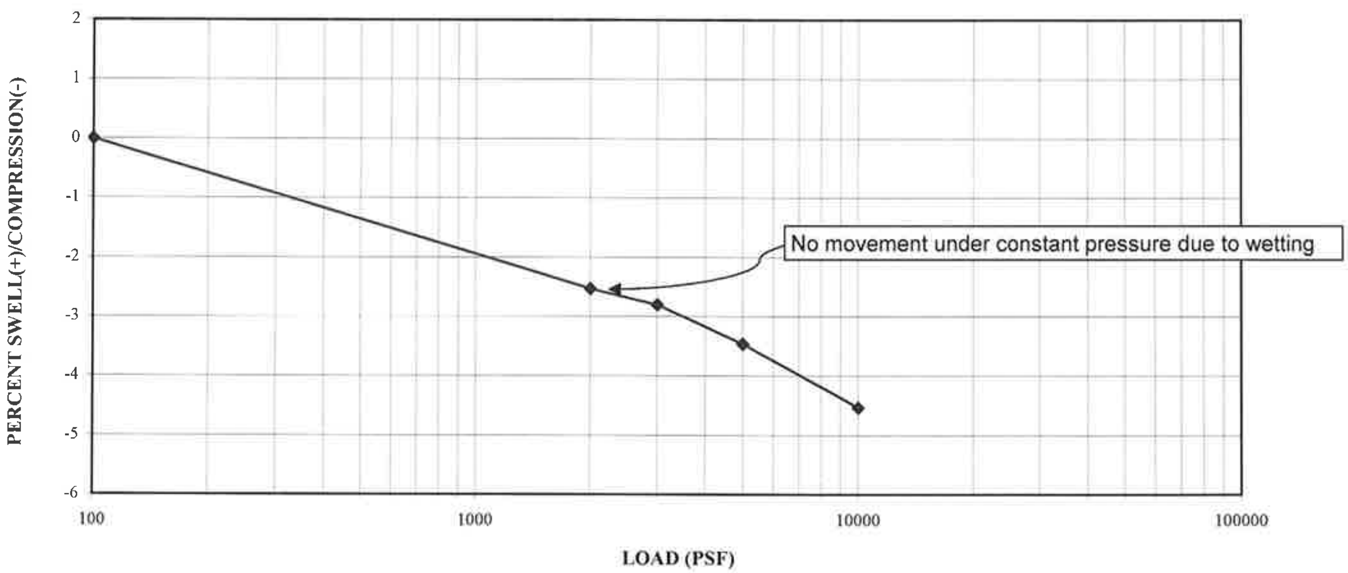
Dry Density	110 pcf
Moisture Content	17.4 %
Volume Change	0.0 %
Swell Pressure	0 psf

SWELL-COMPRESSION TEST



Sample Location	Boring 4
Sample Depth	54 feet
Sample Description	Sandstone bedrock
USCS Classification	
AASHTO Classification	

Dry Density	112 pcf
Moisture Content	16.3 %
Volume Change	0.1 %
Swell Pressure	0 psf

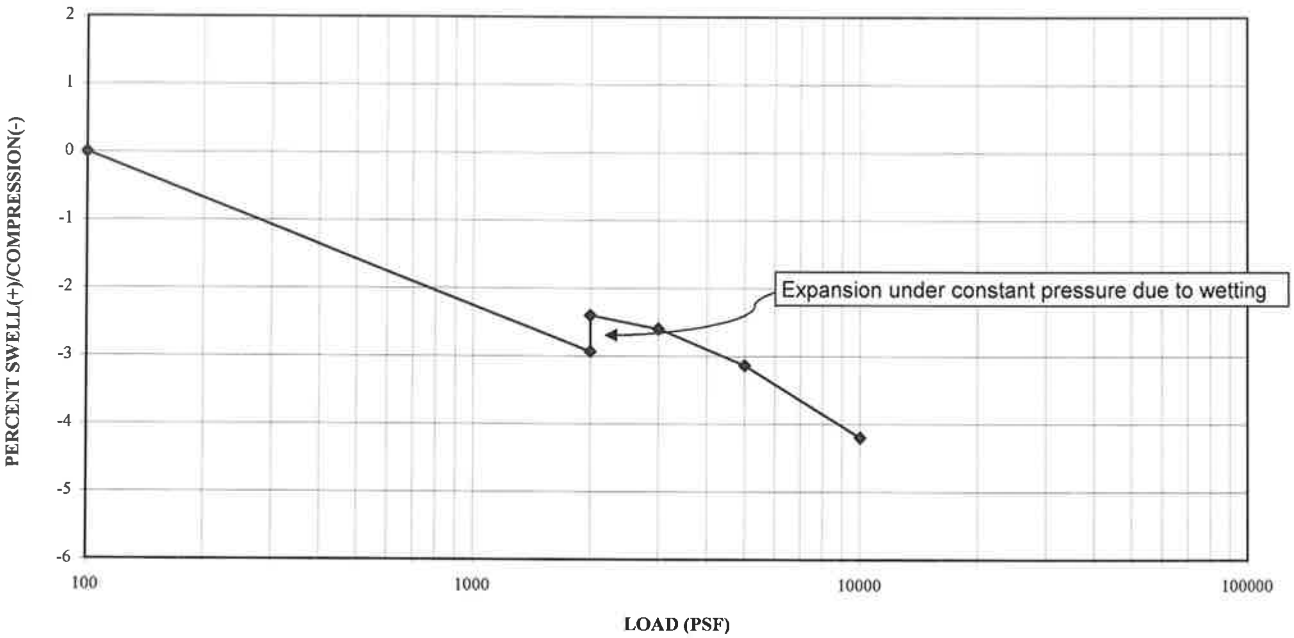


Sample Location	Boring 5
Sample Depth	54 feet
Sample Description	Claystone bedrock
USCS Classification	
AASHTO Classification	

Dry Density	110 pcf
Moisture Content	16.0 %
Volume Change	0.0 %
Swell Pressure	0 psf

GEOCAL, INC.	6th Avenue over BNSF	JOB NO.	G10.1354.002
	SWELL - COMPRESSION TEST RESULTS	FIGURE NO.	6

SWELL-COMPRESSION TEST

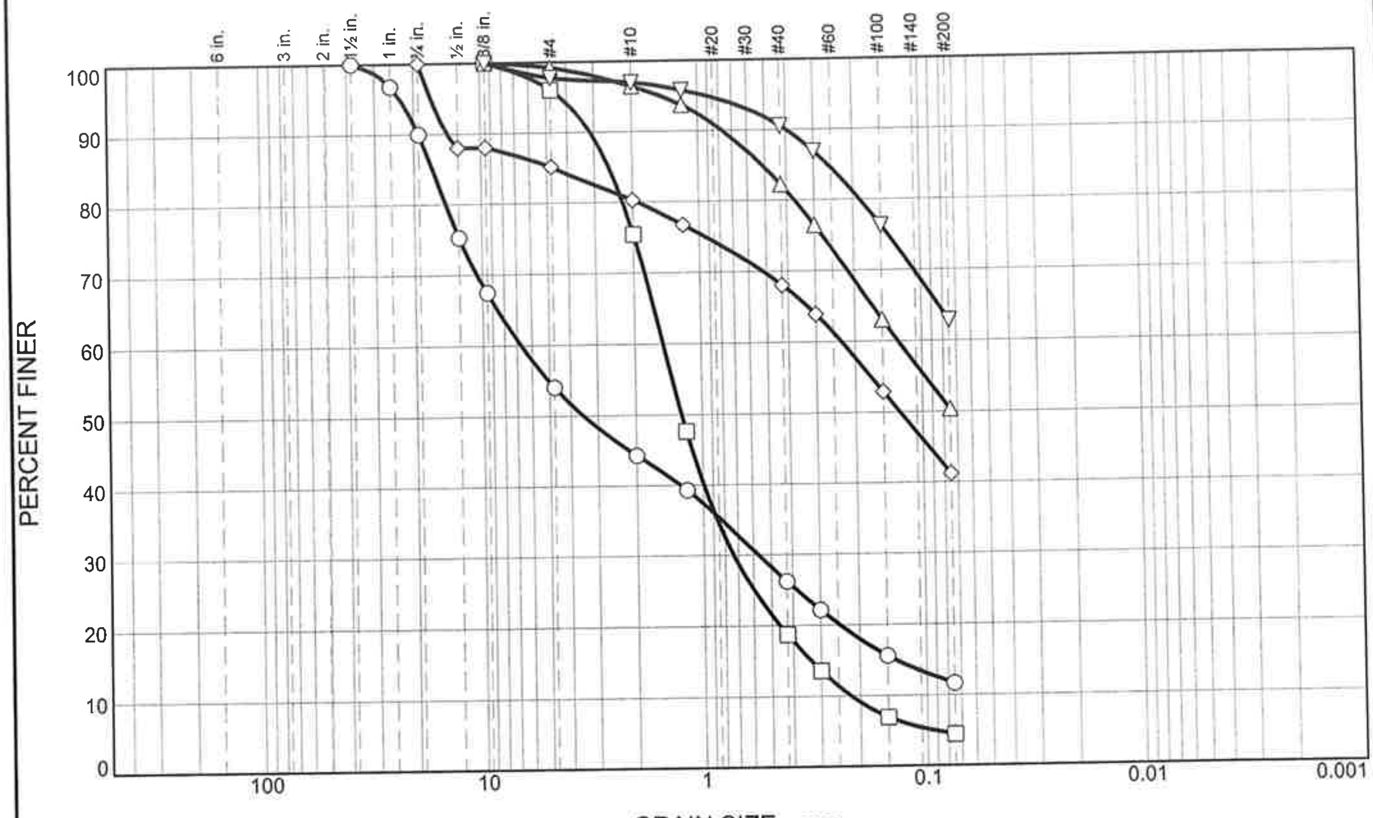


Sample Location	Boring 6
Sample Depth	49 feet
Sample Description	Claystone bedrock
USCS Classification	
AASHTO Classification	

Dry Density	110 pcf
Moisture Content	15.6 %
Volume Change	0.5 %
Swell Pressure	0 psf

GEOCAL, INC.	**6th Avenue over BNSF**	**JOB NO.** G10.1354.002
	SWELL - COMPRESSION TEST RESULTS	**FIGURE NO.** 7

Gradation Test Results



	% +3"	% Gravel	% Sand	% Silt	% Clay
○	0	46	42	12	
□	0	4	92	4	
△	0	1	49	50	
◇	0	15	44	41	
▽	0	2	35	63	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	21	15	16.5413	6.7271	3.5052	0.5704	0.1380			
□	NV	NP	2.5776	1.4893	1.2374	0.7341	0.3329	0.2202	1.64	6.76
△	37	16	0.5136	0.1281						
◇	37	15	4.6349	0.2307	0.1268					
▽	44	18	0.2588							

Material Description	USCS	AASHTO
○ poorly graded gravel with silty clay and sand, fill	GP-GC	A-1-a
□ well-graded sand	SW	A-1-b
△ sandy lean clay, fill	CL	A-6(7)
◇ clayey sand with gravel, fill	SC	A-6(4)
▽ sandy lean clay, fill	CL	A-7-6(14)

Project No. G10.1354.002 **Client:** Wilson & Company

Project: 6th Avenue over BNSF

○ **Location:** Boring 1 **Depth:** 1-10 feet **Sample Number:** 5815-1

□ **Location:** Boring 1 **Depth:** 49 feet **Sample Number:** 5815-4

△ **Location:** Boring 2 **Depth:** 4 feet **Sample Number:** 5830-1

◇ **Location:** Boring 2 **Depth:** 34 feet **Sample Number:** 5830-2

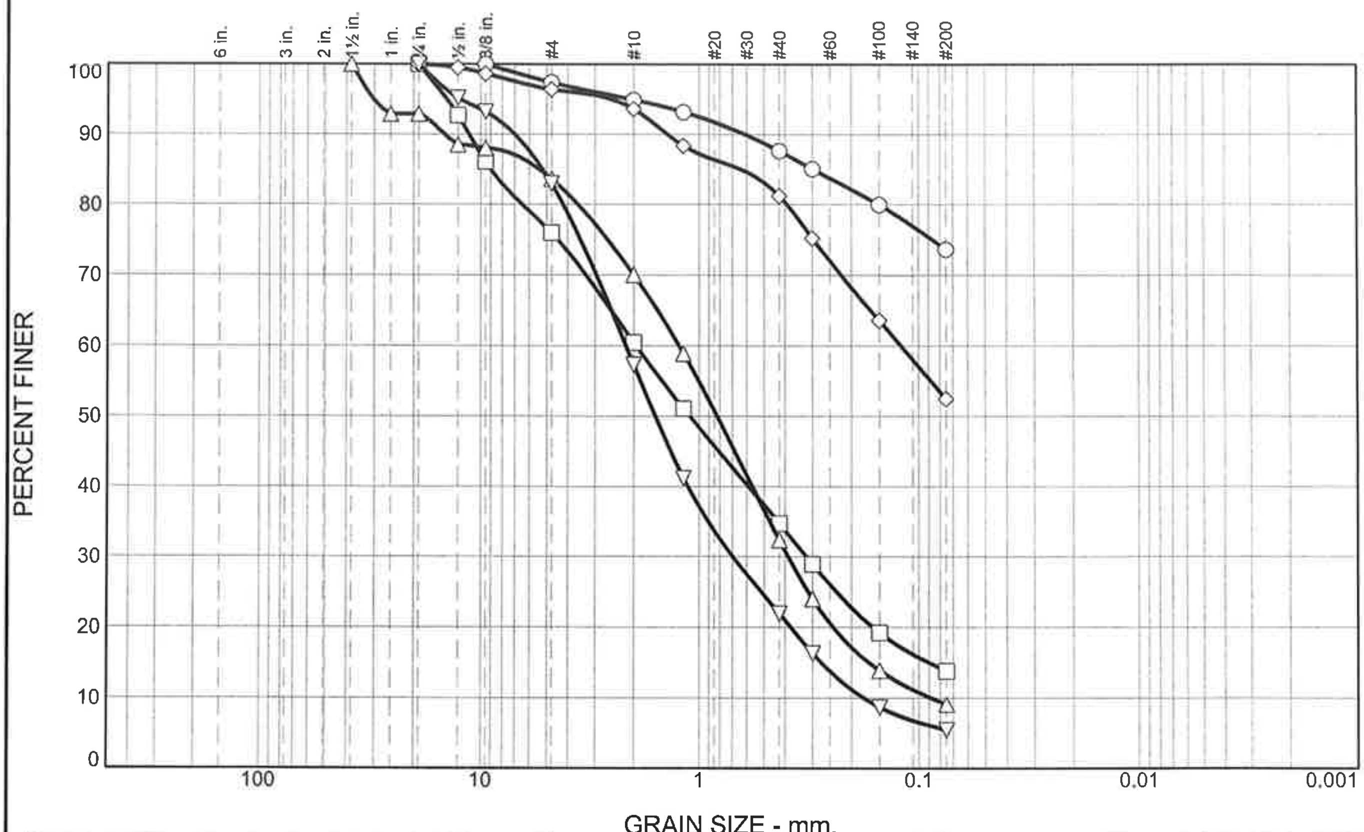
▽ **Location:** Boring 3 **Depth:** 4 feet **Sample Number:** 5820-1

Remarks:

GEOCAL, INC.

Figure 8

Gradation Test Results



	% +3"	% Gravel	% Sand	% Silt	% Clay
○	0	3	23	74	
□	0	24	62	14	
△	0	16	75	9	
◇	0	4	44	52	
▽	0	17	78	5	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	57	23	0.2980							
□	NV	NP	9.0089	1.9535	1.1056	0.3205	0.0898			
△	NV	NP	5.3989	1.2371	0.8253	0.3876	0.1670	0.0879	1.38	14.08
◇	40	16	0.6410	0.1200						
▽	NV	NP	5.2220	2.1705	1.5958	0.6967	0.2735	0.1762	1.27	12.32

Material Description	USCS	AASHTO
○ fat clay with sand, fill	CH	A-7-6(25)
□ silty sand with gravel	SM	A-1-b
△ well-graded sand with silt and gravel	SW-SM	A-1-b
◇ sandy lean clay, fill	CL	A-6(9)
▽ well-graded sand with silt and gravel	SW-SM	A-1-b

Project No. G10.1354.002 **Client:** Wilson & Company

Project: 6th Avenue over BNSF

○ **Location:** Boring 3 **Depth:** 34 feet **Sample Number:** 5820-3

□ **Location:** Boring 3 **Depth:** 44 feet **Sample Number:** 5820-4

△ **Location:** Boring 4 **Depth:** 39 feet **Sample Number:** 5820-8

◇ **Location:** Boring 5 **Depth:** 1-5 feet **Sample Number:** 5830-4

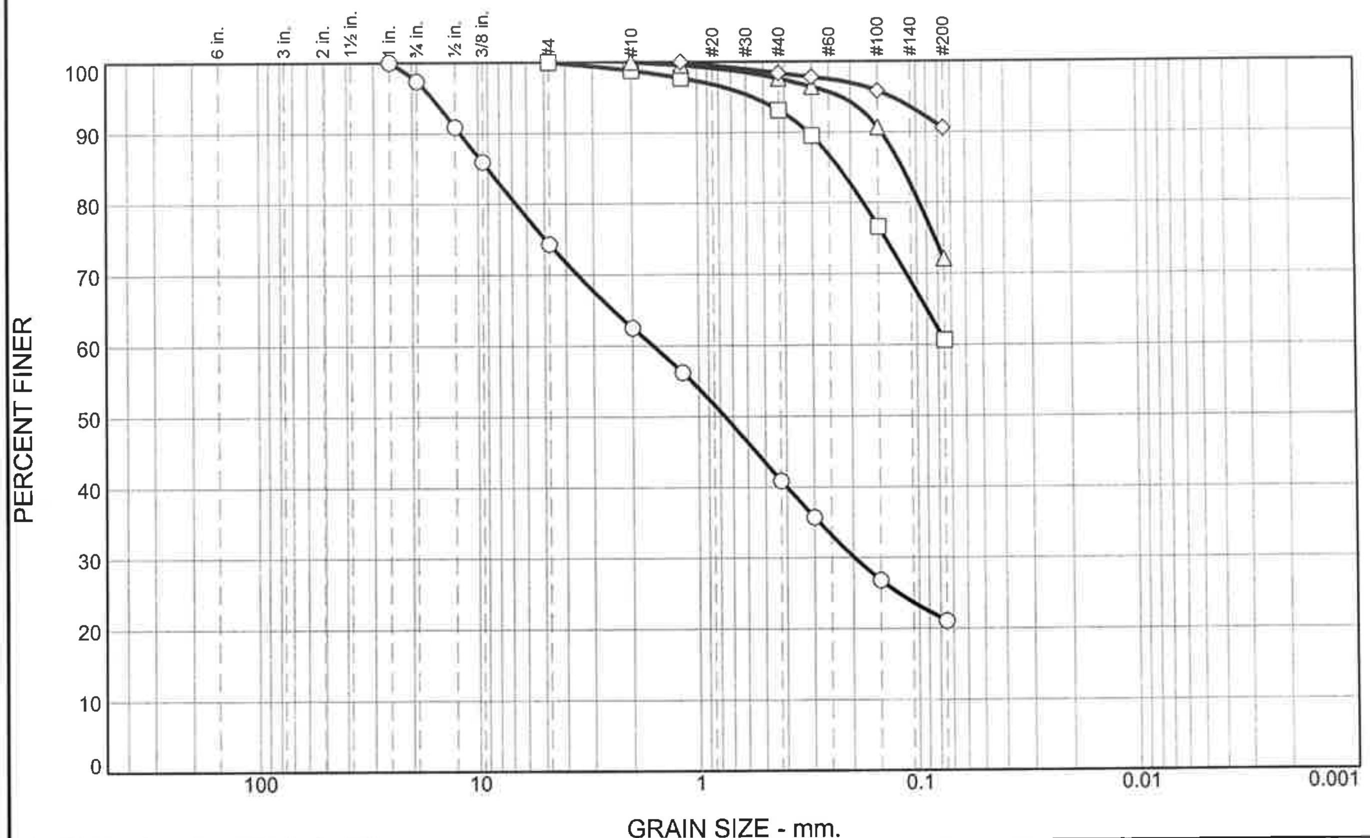
▽ **Location:** Boring 5 **Depth:** 44 feet **Sample Number:** 5830-5

Remarks:

GEOCAL, INC.

Figure 9

Gradation Test Results



	% +3"	% Gravel	% Sand	% Silt	% Clay
○	0	26	53	21	
□	0	0	39	61	
△	0	0	28	72	
◇	0	0	9	91	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	26	16	9.0144	1.6148	0.7630	0.1975				
□	41	16	0.2258							
△	52	31	0.1159							
◇	43	23								

Material Description	USCS	AASHTO
○ clayey sand with gravel, fill	SC	A-2-4(0)
□ sandy lean clay, fill	CL	A-7-6(12)
△ elastic silt with sand	MH	A-7-5(16)
◇ claystone bedrock	CL	A-7-6(20)

Project No. G10.1354.002 **Client:** Wilson & Company

Project: 6th Avenue over BNSF

○ **Location:** Boring 6 **Depth:** 1-5 feet **Sample Number:** 5830-7

□ **Location:** Boring 6 **Depth:** 14 feet **Sample Number:** 5830-8

△ **Location:** Boring 6 **Depth:** 34 feet **Sample Number:** 5830-9

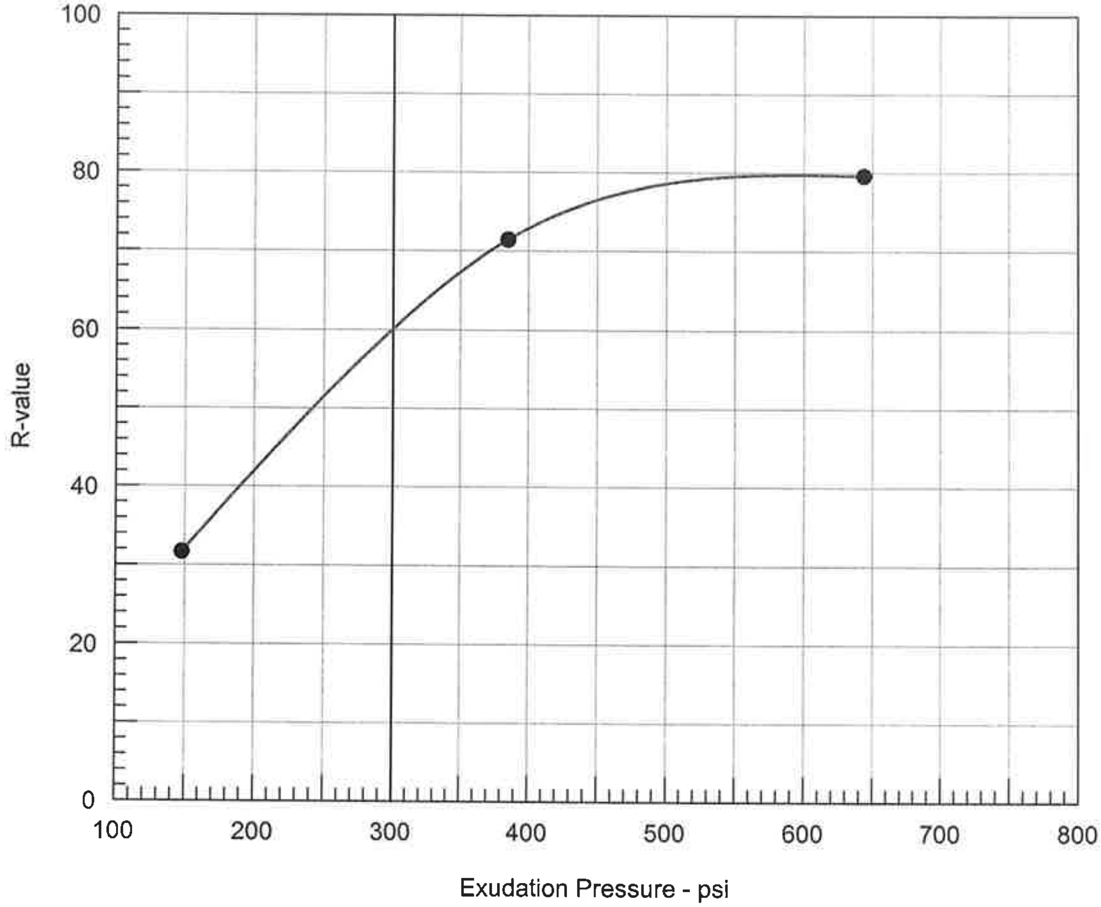
◇ **Location:** Boring 6 **Depth:** 49 feet **Sample Number:** 5830-10

Remarks:

GEOCAL, INC.

Figure 10

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - AASHTO T 190

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	300	127.9	8.2	9	102	2.41	148	34	32
2	350	129.0	7.3	17	40	2.45	385	71	71
3	350	129.2	6.3	26	26	2.50	644	80	80

Test Results	Material Description
R-value at 300 psi exudation pressure = 60	poorly graded gravel with silty clay and sand, fill

Project No.: G10.1354.002

Project:6th Avenue over BNSF

Location: Boring 1

Sample Number: 5815-1 **Depth:** 1-10 feet

Date: 12/9/2011

Tested by: H. Redzic

Checked by: G. Burgess, P.E.

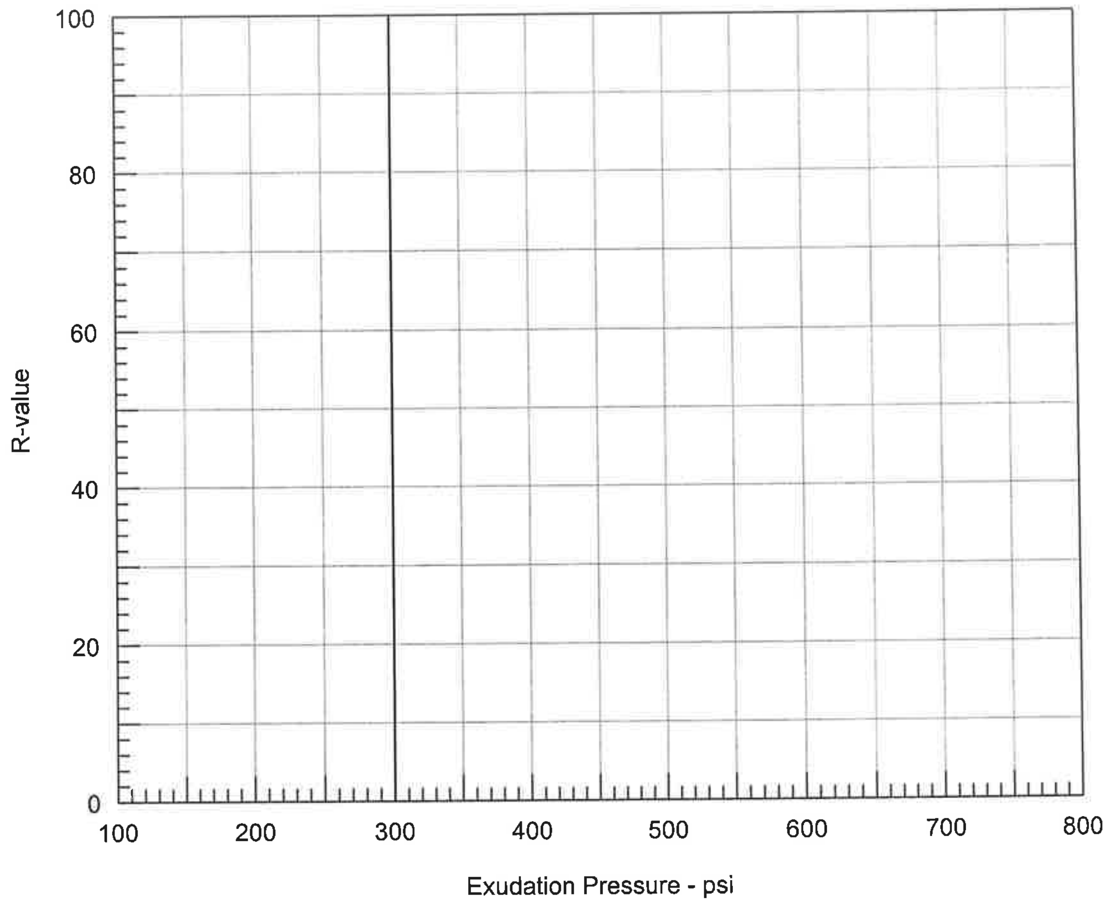
Remarks:
Test performed in accordance with Colorado procedures CP-L 3101 & 3102

R-VALUE TEST REPORT

Geocal, Inc.

Figure 11

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - AASHTO T 190

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.

Test Results	Material Description
R-value at 300 psi exudation pressure = n/a	sandy lean clay, fill

Project No.: G10.1354.002
Project:6th Avenue over BNSF
Location: Boring 5
Sample Number: 5830-4 **Depth:** 1-5 feet
Date: 12/19/2011

R-VALUE TEST REPORT

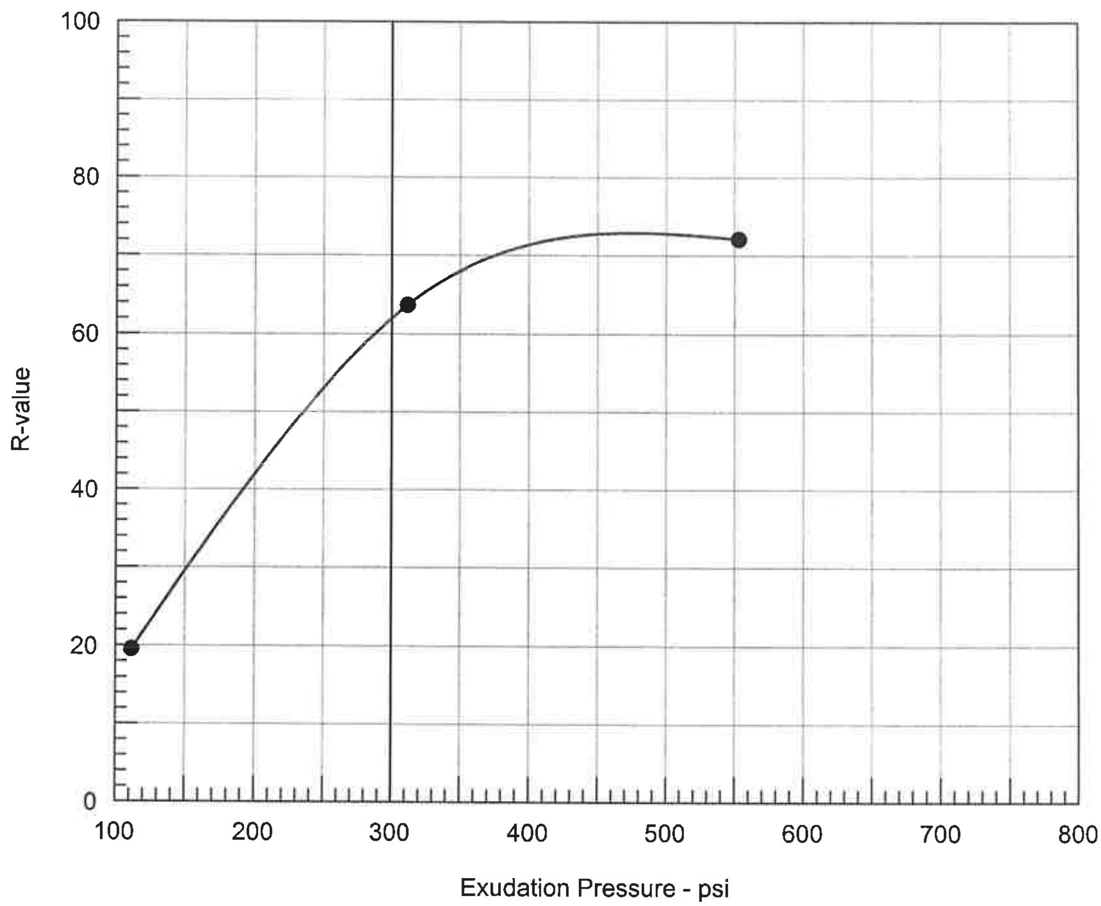
Geocal, Inc.

Tested by: H. Redzic
Checked by: G. Burgess, P.E.

Remarks:
Sample extruded from under the mold during the exudation portion of test prior 800 psi.
R Value < 5.

Figure 12

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - AASHTO T 190

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	150	123.9	8.5	9	120	2.42	112	21	20
2	350	125.2	7.9	22	51	2.47	312	64	64
3	350	124.3	7.1	44	39	2.50	553	72	72

Test Results	Material Description
R-value at 300 psi exudation pressure = 62	clayey sand with gravel, fill

Project No.: G10.1354.002
Project:6th Avenue over BNSF
Location: Boring 6
Sample Number: 5830-7 **Depth:** 1-5 feet
Date: 12/9/2011

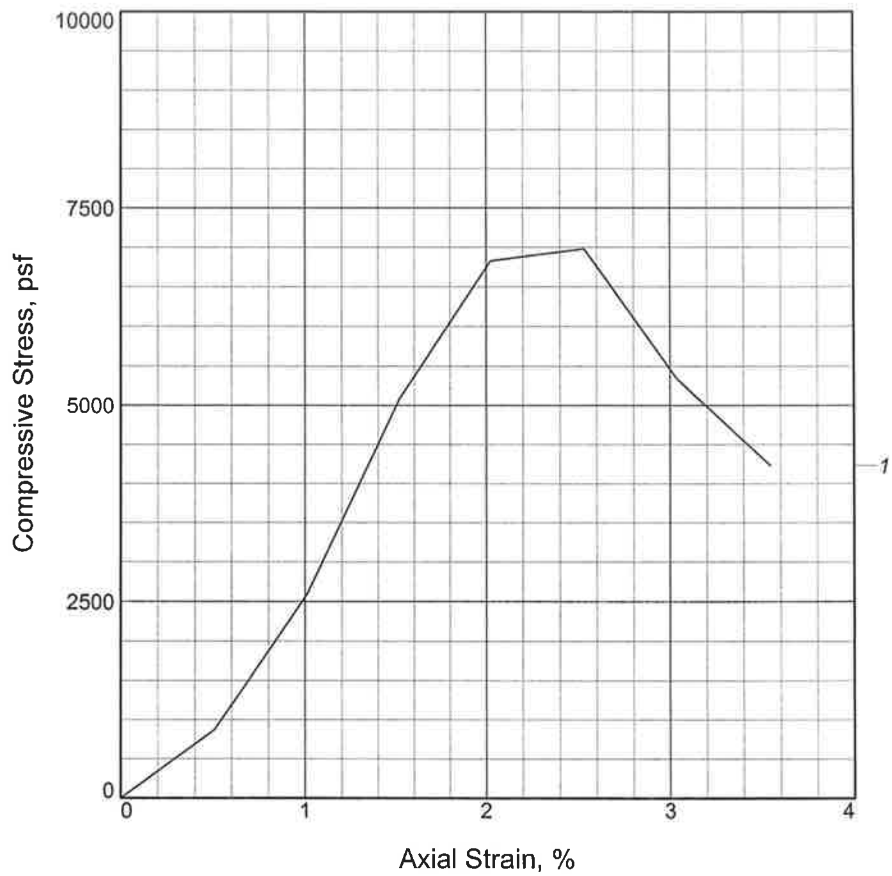
Tested by: H. Redzic
Checked by: G. Burgess, P.E.
Remarks:
Test performed in accordance with colorado procedures CP-L 3101 & 3102.

R-VALUE TEST REPORT

Geocal, Inc.

Figure 13

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	6980			
Undrained shear strength, psf	3490			
Failure strain, %	2.5			
Strain rate, in./min.	0.05			
Water content, %	16.2			
Wet density, pcf	134.0			
Dry density, pcf	115.3			
Saturation, %	98.6			
Void ratio	0.4346			
Specimen diameter, in.	1.94			
Specimen height, in.	3.95			
Height/diameter ratio	2.04			

Description: claystone bedrock

LL = 42	**PL =** 23	**PI =** 19	**Assumed GS=** 2.65	**Type:**

Project No.: G10.1354.002

Date Sampled:

Remarks:

Client: Wilson & Company

Project: 6th Avenue over BNSF

Location: Boring 1

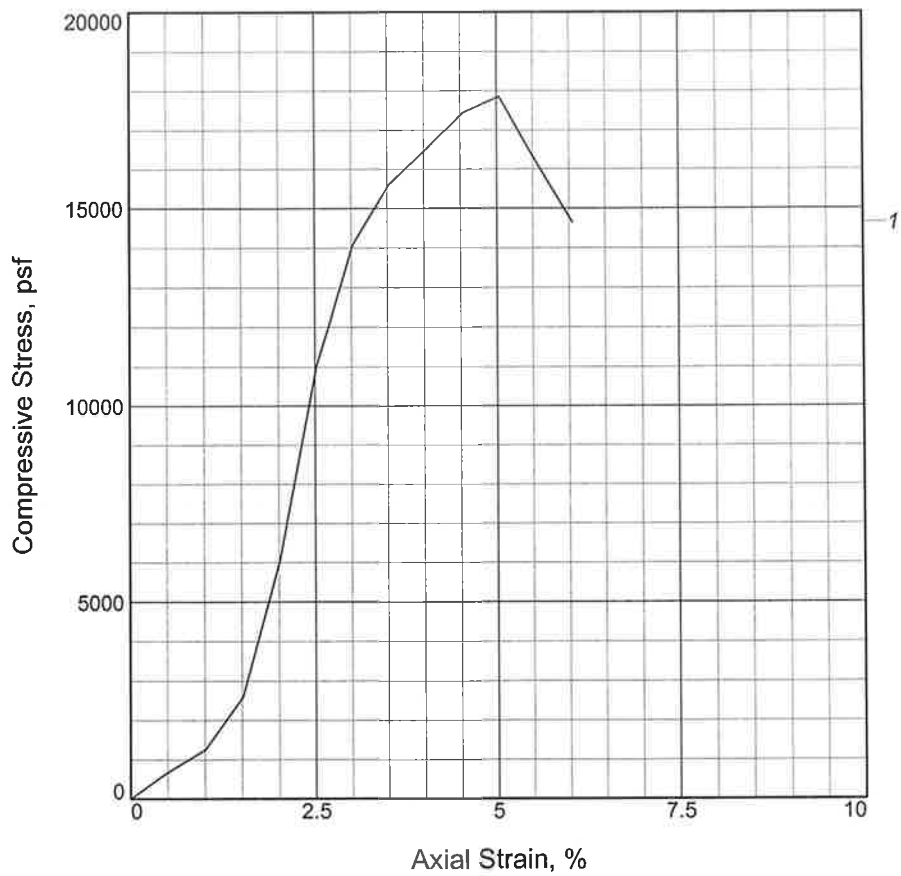
Sample Number: 5815-5 **Depth:** 54 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

Figure 14

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	17857			
Undrained shear strength, psf	8928			
Failure strain, %	5.0			
Strain rate, in./min.	0.05			
Water content, %	16.3			
Wet density, pcf	133.6			
Dry density, pcf	114.8			
Saturation, %	98.2			
Void ratio	0.4411			
Specimen diameter, in.	1.94			
Specimen height, in.	3.97			
Height/diameter ratio	2.05			

Description: claystone bedrock

LL = 46	PL = 24	PI = 22	Assumed GS= 2.65	Type:

Project No.: G10.1354.002

Date Sampled:

Remarks:

Figure 15

Client: Wilson & Company

Project: 6th Avenue over BNSF

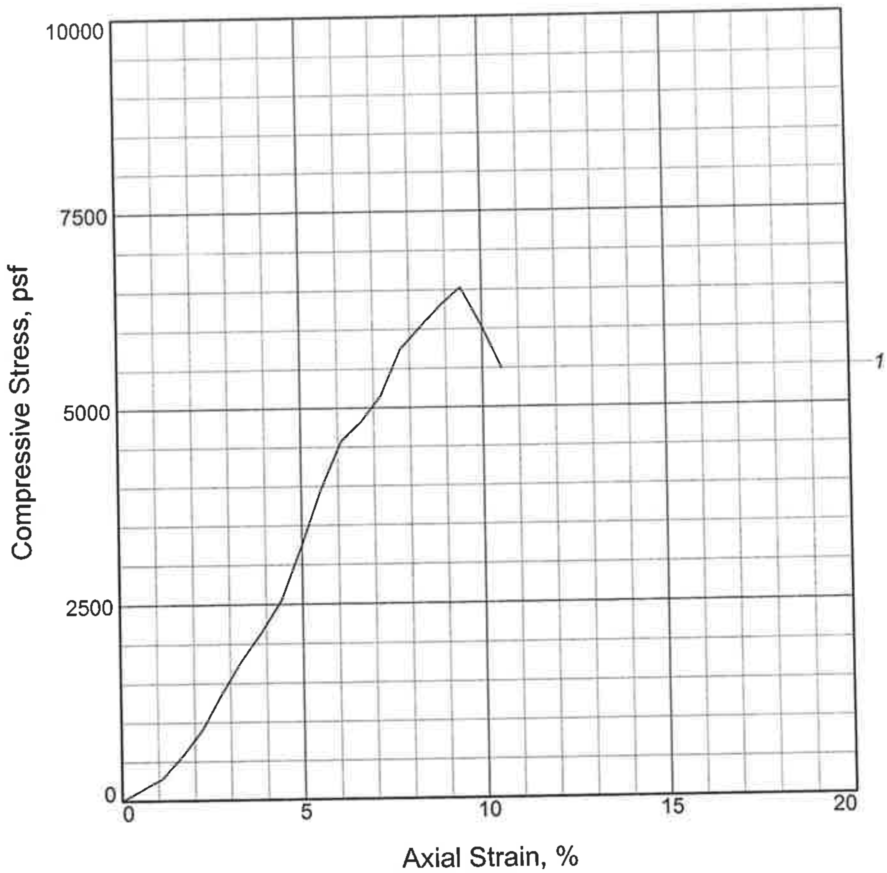
Location: Boring 1

Sample Number: 5815-7 **Depth:** 64 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	6531			
Undrained shear strength, psf	3265			
Failure strain, %	9.4			
Strain rate, in./min.	0.05			
Water content, %	15.9			
Wet density, pcf	121.1			
Dry density, pcf	104.5			
Saturation, %	72.2			
Void ratio	0.5828			
Specimen diameter, in.	1.94			
Specimen height, in.	3.61			
Height/diameter ratio	1.86			

Description: claystone bedrock

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

Project No.: G10.1354.002

Date Sampled:

Remarks:

Client: Wilson & Company

Project: 6th Avenue over BNSF

Location: Boring 2

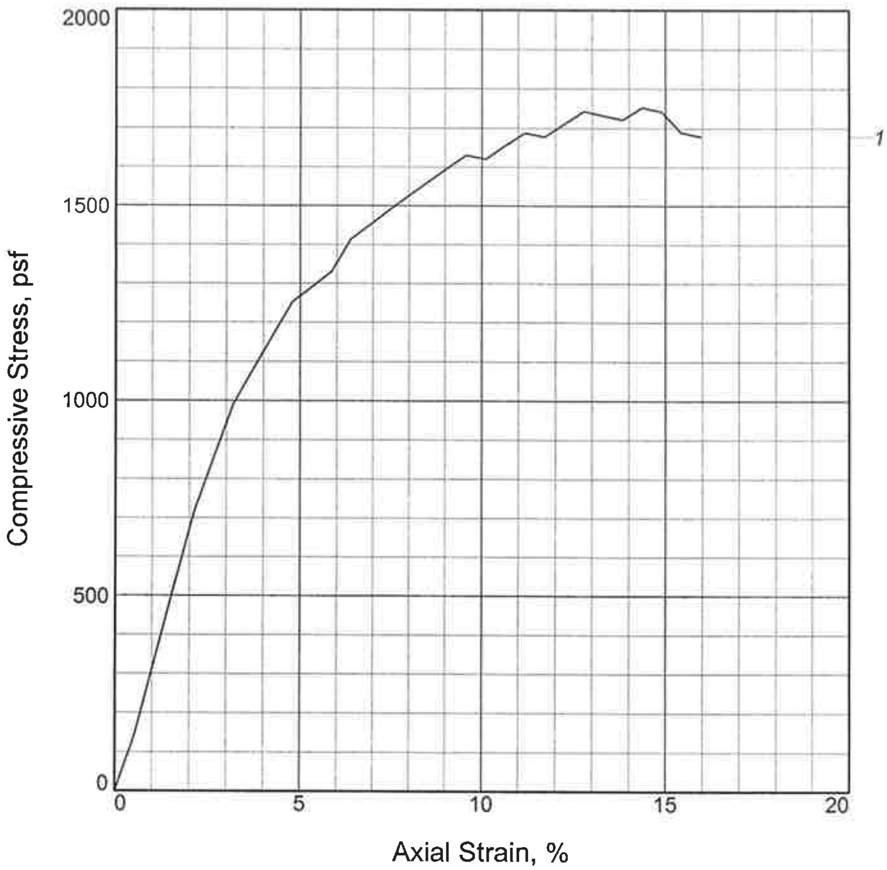
Sample Number: 5830-3 **Depth:** 54 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

Figure 16

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	1752			
Undrained shear strength, psf	876			
Failure strain, %	14.4			
Strain rate, in./min.	0.05			
Water content, %	22.6			
Wet density, pcf	122.0			
Dry density, pcf	99.5			
Saturation, %	90.5			
Void ratio	0.6629			
Specimen diameter, in.	1.94			
Specimen height, in.	3.76			
Height/diameter ratio	1.94			

Description: sandy lean clay, fill

LL = 44	PL = 18	PI = 26	Assumed GS= 2.65	Type:

Project No.: G10.1354.002

Date Sampled:

Remarks:

Client: Wilson & Company

Project: 6th Avenue over BNSF

Location: Boring 3

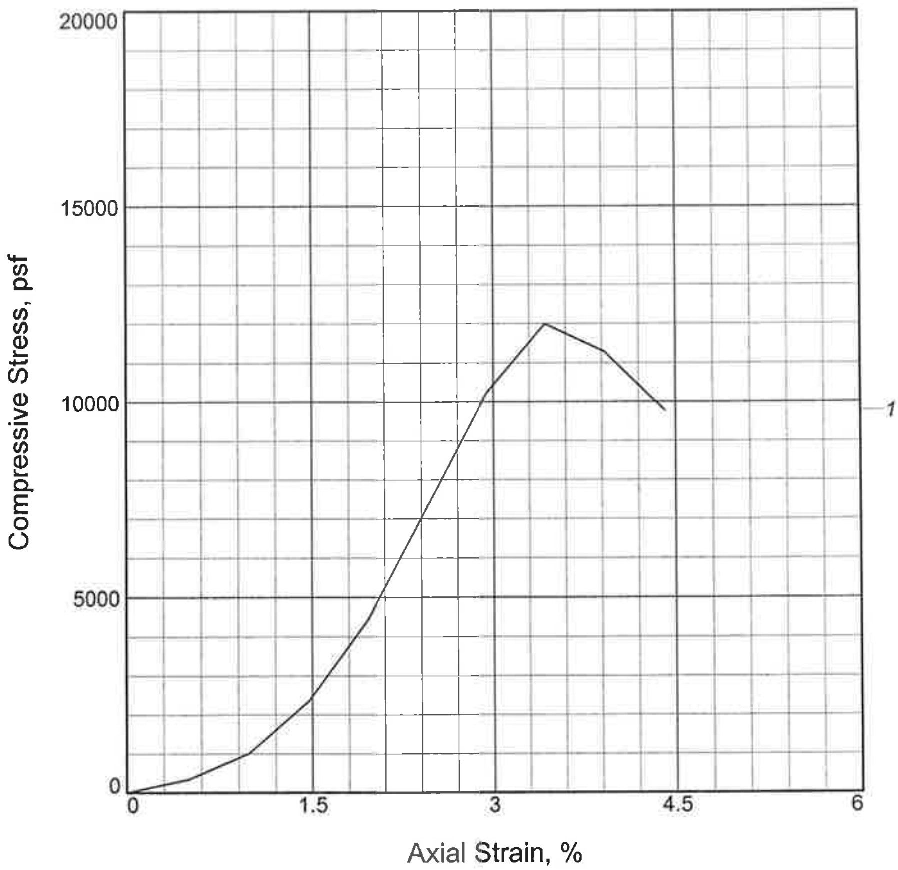
Sample Number: 5820-1 **Depth:** 4 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

Figure 17

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	11996			
Undrained shear strength, psf	5998			
Failure strain, %	3.4			
Strain rate, in./min.	0.05			
Water content, %	17.3			
Wet density, pcf	129.8			
Dry density, pcf	110.7			
Saturation, %	92.6			
Void ratio	0.4947			
Specimen diameter, in.	1.94			
Specimen height, in.	4.08			
Height/diameter ratio	2.10			

Description: claystone bedrock

LL = 47	**PL =** 26	**PI =** 21	**Assumed GS=** 2.65	**Type:**

Project No.: G10.1354.002

Date Sampled:

Remarks:

Client: Wilson & Company

Project: 6th Avenue over BNSF

Location: Boring 3

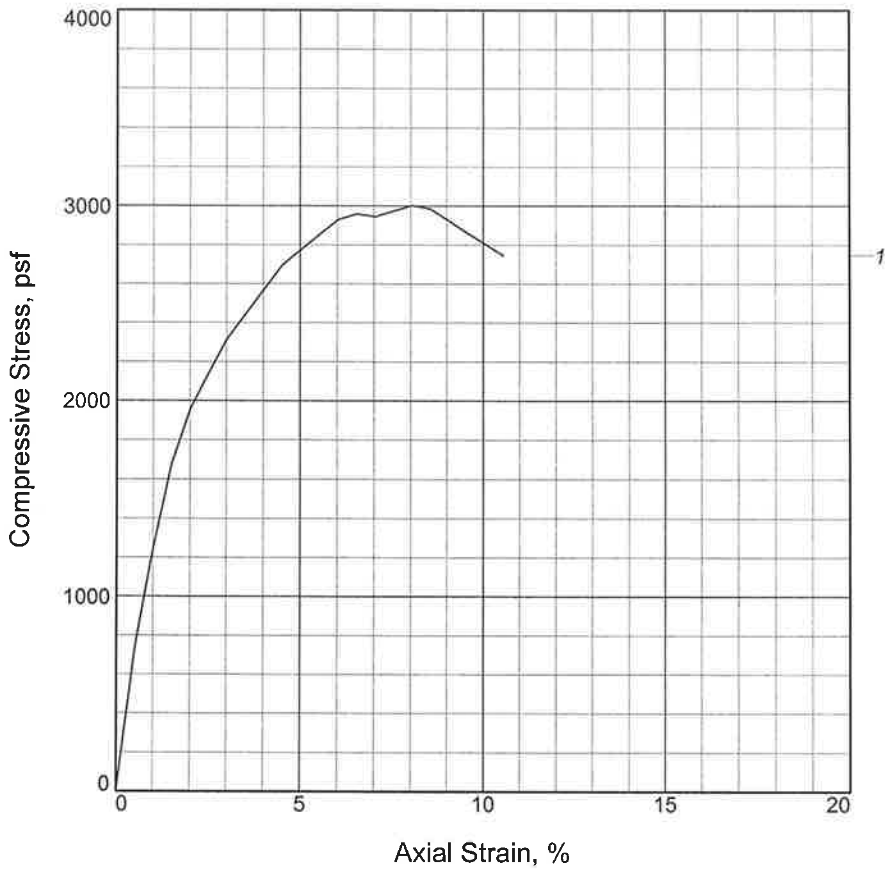
Sample Number: 5820-5 **Depth:** 59 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

Figure 18

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	3002			
Undrained shear strength, psf	1501			
Failure strain, %	8.0			
Strain rate, in./min.	0.05			
Water content, %	21.0			
Wet density, pcf	124.2			
Dry density, pcf	102.7			
Saturation, %	90.8			
Void ratio	0.6113			
Specimen diameter, in.	1.94			
Specimen height, in.	3.98			
Height/diameter ratio	2.05			

Description: sandy lean clay, fill

LL = 47	PL = 18	PI = 29	Assumed GS= 2.65	Type:

Project No.: G10.1354.002

Date Sampled:

Remarks:

Figure 19

Client: Wilson & Company

Project: 6th Avenue over BNSF

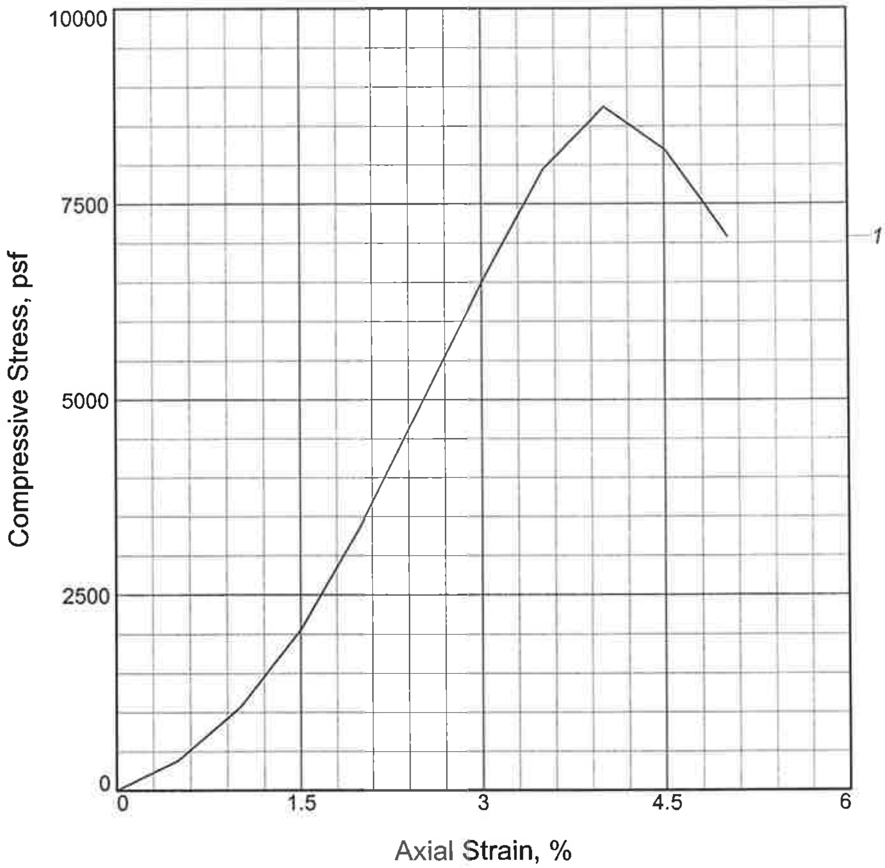
Location: Boring 4

Sample Number: 5820-6 **Depth:** 9 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	8745			
Undrained shear strength, psf	4372			
Failure strain, %	4.0			
Strain rate, in./min.	0.05			
Water content, %	18.5			
Wet density, pcf	129.5			
Dry density, pcf	109.3			
Saturation, %	95.5			
Void ratio	0.5132			
Specimen diameter, in.	1.94			
Specimen height, in.	3.99			
Height/diameter ratio	2.06			

Description: claystone bedrock

LL = 40	PL = 20	PI = 20	Assumed GS= 2.65	Type:

Project No.: G10.1354.002

Date Sampled:

Remarks:

Client: Wilson & Company

Project: 6th Avenue over BNSF

Location: Boring 4

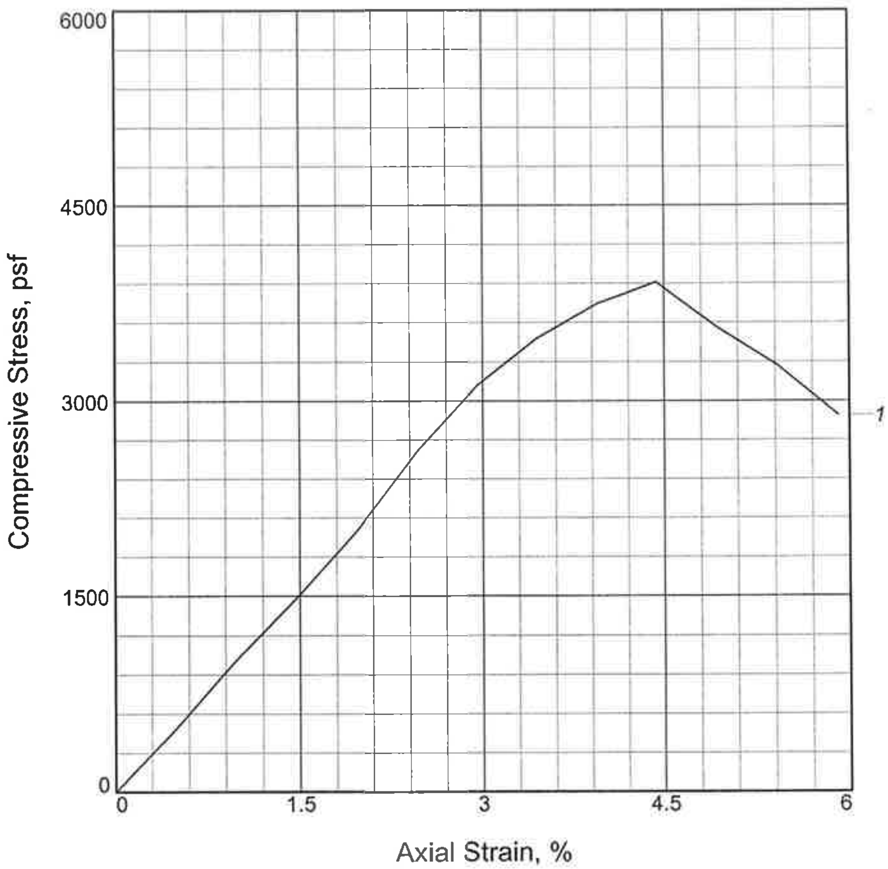
Sample Number: 5820-9 **Depth:** 49 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

Figure 20

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	3911			
Undrained shear strength, psf	1955			
Failure strain, %	4.4			
Strain rate, in./min.	0.05			
Water content, %	17.5			
Wet density, pcf	128.7			
Dry density, pcf	109.5			
Saturation, %	90.8			
Void ratio	0.5110			
Specimen diameter, in.	1.94			
Specimen height, in.	4.06			
Height/diameter ratio	2.09			

Description: claystone bedrock

LL = 48	PL = 24	PI = 24	Assumed GS= 2.65	Type:

Project No.: G10.1354.002

Date Sampled:

Remarks:

Figure 21

Client: Wilson & Company

Project: 6th Avenue over BNSF

Location: Boring 6

Sample Number: 5830-11 **Depth:** 54 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

Project # **G10.1354.002**

Client: **Wilson& Company**
Project Name **6th Avenue over BNSF**

Sample Location		Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation		Percent Passing No. 200 Sieve	Atterberg Limits		Swell Pressure (psf)	Swell w/1or2 ksf Surcharge (%)	Unconfined Compressive Strength (psf)	R-Value at 300 psi Exudation Pressure	AASHTO Class (Group Index)	Soil or Bedrock Description
Boring No.	Depth (feet)			Gravel (%)	Sand (%)		Liquid Limit (%)	Plasticity Index (%)						
1	1-10			46	42	12	21	6				60	A-1-a	Poorly graded gravel with silty clay and sand, fill
1	24	20.7	107						0	0.1				Sandy lean clay, fill
1	34	20.2	101											Clayey sand with gravel, fill
1	49			4	92	4	NV	NP					A-1-b	Well-graded sand
1	54	16.2	116								6,980			Claystone bedrock
1	59	17.3	112						1,030	0.8				Claystone bedrock
1	64	16.3	115								17,860			Claystone bedrock
2	4	20.4	106	1	49	50	37	21					A-6(7)	Sandy lean clay, fill
2	34	15.0	119	15	44	41	37	22					A-6(4)	Clayey sand with gravel, fill
2	54	15.9	105								6,530			Claystone bedrock
3	4	22.6	100	2	35	63	44	26			1,750		A-7-6(14)	Sandy lean clay, fill
3	9	15.9	109											Silty, sandy clay with gravel, fill
3	34	32.9	92	3	23	74	57	34	0	0.1			A-7-6(25)	Fat clay with sand, fill
3	44			24	62	14	NV	NP					A-1-b	Silty sand with gravel
3	59	17.3	111								12,000			Claystone bedrock
4	9	21.0	103								3,000			Sandy lean clay, fill
4	24	17.4	110						0	0.0				Clayey sand with gravel, fill
4	39			16	75	9	NV	NP					A-1-b	Well-graded sand with silt and gravel
4	49	18.5	110								8,750			Claystone bedrock
4	54	16.3	112						0	0.1				Sandstone bedrock
5	1-5			4	44	52	40	24				<5	A-6(9)	Sandy lean clay, fill
5	44			17	78	5	NV	NP					A-1-b	Well-graded sand with silt and gravel
5	54	16.0	110						0	0.0				Claystone bedrock
6	1-5			26	53	21	26	10				62	A-2-4(0)	Clayey sand with gravel, fill
6	14	22.2	104	0	39	61	41	25					A-7-6(12)	Sandy lean clay, fill
6	34	50.5	68	0	28	72	52	21					A-7-5(16)	Elastic silt with sand
6	49	15.6	110	0	9	91	43	20	0	0.5				Claystone bedrock
6	54	17.5	110			91	48	24			3,910			Claystone bedrock

Project #: G10.1354.002

TABLE 2
SUMMARY OF LABORATORY CHEMICAL TEST RESULTS

Client: **Wilson & Company**
Project Name: **6th Avenue over BNSF**

Sample Location		Natural Moisture Content	Natural Dry Density	Water Soluble Sulfates	Laboratory Resistivity	pH	Chloride Water Soluble	Sulfide	AASHTO Class. (Group Index)	Soil or Bedrock Description
Boring No.	Depth (feet)	(%)	(pcf)	(%)	(ohm-cm)		(%)			
1	1-10			0.07	1,300	6.9	0.0256		A-1-a	Poorly graded gravel with silty clay and sand, fill
1	34	20.2	101	0.06	1,200	6.8	0.0180			Clayey sand with gravel, fill
1	49			0.07	4,500	7.0	0.0019		A-1-b	Well-graded sand
2	4	20.4	106	0.07	420	6.3	0.0696	Positive	A-6(7)	Sandy lean clay, fill
2	34	15.0	119	0.07	1,600	6.8	0.0021	Positive	A-6(4)	Clayey sand with gravel, fill
3	9	15.9	109	0.07	790	7.6	0.0196	Trace		Silty, sandy clay with gravel, fill
3	44			0.05	3,700	7.4	0.0030	Negative	A-1-b	Silty sand with gravel
4	9	21.0	103	0.18	490	6.9	0.0139	Positive		Sandy lean clay
5	54	16.0	110	0.06	680	6.7	0.0015	Positive		Claystone bedrock
6	14	22.2	104	0.13	460	6.7	0.0349	Trace	A-7-6(12)	Sandy lean clay
6	34	50.5	68	0.08	1,400	6.8	0.0079	Positive	A-7-5(16)	Elastic silt with sand

APPENDIX A

PAVEMENT DESIGN DATA AND ANALYSIS
30% DESIGN

Annual Average Daily Traffic (AADT) Volumes for Highway 006G

Route	Ref Pt	End Ref Pt	Length (Miles)	Annual Average Daily Traffic	AADT Year	AADT Single Trucks	AADT Comb Trucks	Percent Trucks	Design Hour Volume (% of AADT)	Daily Vehicle Miles Traveled	Segment Description
006G	282.333	283.469	1.106	115,000	2010	2750	1250	3.50	10	127,190	SHERIDAN BLVD INTERCHANGE STR (F-16-FL) - JCT SH 095A N AND S - RD N AND S (SHERIDAN BLVD) OVERPASS SEPARATION - LEAVE JEFFERSON COUNTY - LEAVE LAKEWOOD CITY LIMITS
006G	284.187	284.748	0.560	141,000	2010	2400	1700	2.90	8	78,960	MAJOR STR (F-16-EN) - RD N AND S (BRYANT ST) OVERPASS SEPARATION

ESAL Calculations - CDOT Pavement Design Manual 2012

Design: GAB   Checked:
Date:12/13/2011   Date:

Asphalt Pavement

Assume 20-year design for Asphalt Pavement for approaches to 6th Avenue Bridge over BNSF

Assume 3% Growth Factor

6 lane, Design Lane Factor = 0.3, Percentage of Trucks in Design Lane: 60%

Traffic Volumes

2011 ADT	115,000	
2031 ADT	207,701	
2041 ADT	254,052	
3-Bin Vehicle Classification Percentages		
Cars	96.5%	
Single Unit Trucks	2.4%	
Combination Trucks	1.1%	
2021 Projected Traffic Counts (Midpoint Design Life)		
Cars	155,703	(Car % * 2021 ADT)
Single Unit Trucks	2,323	(Single Unit Truck % * 2021 ADT)
Combination Trucks	1,065	(Combination Truck % * 2021 ADT)

Table 1.2 Colorado Equivalency Factors

3-Bin Vehicle Classification	Flexible Pavement	Rigid Pavement
Passenger cars & pickup trucks	0.003	0.003
Single Unit Trucks	0.249	0.285
Combination Trucks	1.087	1.692

Daily ESALs

Daily ESALs for Cars	467.1	(2026 Traffic Count * 0.003)
Daily ESALs for Single Unit Trucks	578.5	(2026 Traffic Count * 0.249)
Daily ESALs for Combination Trucks	1,157.6	(2026 Traffic Count * 1.087)
Total Daily ESALs	2,203.2	(Sum of Daily ESALs)
Design Life	20.0	years
Total Design Period ESALs	**4,825,027**	(Total Daily ESALs * 365 day/year * 20 years* 0.3)

ESAL Calculations - CDOT Pavement Design Manual 2012

Design: GAB	Checked:
Date:12/13/2011	Date:

Concrete Pavement

Assume 30-year design for Concrete Pavement for approaches to 6th Avenue Bridge over BNSF

Assume 3% Growth Factor

6 lane, Design Lane Factor = 0.3, Percentage of Trucks in Design Lane: 60%

Traffic Volumes

2011 ADT	115,000	
2031 ADT	207,701	
2041 ADT	254,052	
3-Bin Vehicle Classification Percentages		
Cars	96.5%	
Single Unit Trucks	2.4%	
Combination Trucks	1.1%	
2026 Projected Traffic Counts (Midpoint design life)		
Cars	178,068	(Car % * 2026 ADT)
Single Unit Trucks	2,657	(Single Unit Truck % * 2026 ADT)
Combination Trucks	1,218	(Combination Truck % * 2026 ADT)

Table 1.2 Colorado Equivalency Factors		
3-Bin Vehicle Classification	**Flexible Pavement**	**Rigid Pavement**
Passenger cars & pickup trucks	0.003	0.003
Single Unit Trucks	0.249	0.285
Combination Trucks	1.087	1.692

Daily ESALs

Daily ESALs for Cars	534.2	(2026 Traffic Count * 0.003)
Daily ESALs for Single Unit Trucks	757.3	(2026 Traffic Count * 0.249)
Daily ESALs for Combination Trucks	2,060.6	(2026 Traffic Count * 1.087)
Total Daily ESALs	3,352.1	(Sum of Daily ESALs)
Design Life	30.0	years
Total Design Period ESALs	**11,011,767**	(Total Daily ESALs * 365 day/year * 30 years * 0.3)

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Matt Eckhart

Flexible Structural Design Module

6th Avenue over BNSF Bridge Replacement
Approach Pavements
Design Life: 20 Years (Asphalt)
Assume R-50 Subgrade

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	4,825,027
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	95 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	13,168 psi
Stage Construction	1
Calculated Design Structural Number	3.70 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	HMA	0.44	1	7	-	3.08
2	ABC	0.12	1	6	-	0.72
Total	-	-	-	13.00	-	3.80

Layered Thickness Design

Thickness precision Actual

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Spec Thickness (Di)(in)	Min Thickness (Di)(in)	Elastic Modulus (psi)	Width (ft)	Calculated Thickness (in)	Calculated SN (in)
1	HMA	0.44	1	-	-	-	-	8.41	3.70
Total	-	-	-	-	-	-	-	8.41	3.70

Rigid Pavement Design - Based on AASHTO Supplemental Guide

Reference: *LTPP DATA ANALYSIS - Phase I: Validation of Guidelines for k-Value Selection and Concrete Pavement Performance Prediction*

Results

Project #
Description: Design of approach pavements to Bridge Structure

Location: Region 6

Slab Thickness Design

Pavement Type	JPCP	
18-kip ESALs Over Initial Performance Period (million)	11.00	million
Initial Serviceability	4.5	
Terminal Serviceability	2.5	
28-day Mean PCC Modulus of Rupture	650	psi
Elastic Modulus of Slab	3,400,000	psi
Elastic Modulus of Base	15,000	psi
Base Thickness	6.0	in.
Mean Effective k-Value	175	psi/in
Reliability Level	95	%
Overall Standard Deviation	0.34	
Calculated Design Thickness	**10.43**	in

Temperature Differential

Mean Annual Wind Speed	8.8	mph
Mean Annual Air Temperature	50.3	^{o}F
Mean Annual Precipitation	15.3	in
Maximum Positive Temperature Differential	**8.09**	^{o}F

Modulus of Subgrade Reaction

Period	Description	Subgrade k-Value, psi

Seasonally Adjusted Modulus of Subgrade Reaction	**165**	psi/in
Modulus of Subgrade Reaction Adjusted for Rigid Layer and Fill Section		psi/in

Traffic

Performance Period		years
Two-Way ADT		
Number of Lanes in Design Direction		
Percent of All Trucks in Design Lane		
Percent Trucks in Design Direction		

Vehicle Class	Percent of ADT	Annual Growth	Initial Truck Factor	Annual Growth in Truck Factor	Accumulated 18-kip ESALs (millions)

Total Calculated Cumulative ESALs		million

Faulting

Doweled

Dowel Diameter	1.5	in
Drainage Coefficient	1.00	

Average Fault for Design Years with Design Inputs	**0.05**	in
Criteria Check **PASS**		

Nondoweled

Drainage Coefficient

Average Fault for Design Years with Design Inputs		in
Criteria Check		

Rigid Pavement Design - Based on AASHTO Supplemental Guide

Reference: *LTPP DATA ANALYSIS - Phase I: Validation of Guidelines for k-Value Selection and Concrete Pavement Performance Prediction*

I. General

Agency: CDOT
Street Address: 6th Avenue over BNSF
City: Denver
State: Colorado

Project Number:

ID: 6th Avenue over BNSF

Description: Design of approach pavements to Bridge Structure

Location: Region 6

II. Design

Serviceability

Initial Serviceability, P1:	4.5	
Terminal Serviceability, P2:	2.5	

PCC Properties

28-day Mean Modulus of Rupture, $(S'_c)'$:	650	psi
Elastic Modulus of Slab, E_c:	3,400,000	psi
Poisson's Ratio for Concrete, m:	0.15	

Base Properties

Elastic Modulus of Base, E_b:	15,000	psi
Design Thickness of Base, H_b:	6.0	in
Slab-Base Friction Factor, f:	1.4	

Reliability and Standard Deviation

Reliability Level (R):	95.0	%
Overall Standard Deviation, S_0:	0.34	

Climatic Properties

Mean Annual Wind Speed, WIND:	8.8	mph
Mean Annual Air Temperature, TEMP:	50.3	°F
Mean Annual Precipitation, PRECIP:	15.3	in

Subgrade k-Value

175 psi/in

Design ESALs

11.0 million

Pavement Type, Joint Spacing (L)

- (●) JPCP
- ( ) JRCP
- ( ) CRCP

Joint Spacing: 15.0 ft

JPCP

Effective Joint Spacing: 180 in

Edge Support

- (●) Conventional 12-ft wide traffic lane
- ( ) Conventional 12-ft wide traffic lane + tied PCC
- ( ) 2-ft widened slab w/conventional 12-ft traffic lane

Edge Support Factor: 1.00

Sensitivity Analysis

Slab Thickness used for Sensitivity Analysis: 10.43 in

- ( ) Modulus of Rupture
- ( ) Elastic Modulus (Slab)
- ( ) Elastic Modulus (Base)
- ( ) Base Thickness
- (●) k-Value
- ( ) Joint Spacing
- ( ) Reliability
- ( ) Standard Deviation

Calculated Slab Thickness for Above Inputs: 10.43 in

Faulting

DOWELED PAVEMENT

Dowel Diameter: 1.50 in
K_d: 1,500,000 psi/in
E_s: 29,000,000 psi

Base/Slab Frictional Restraint

- ○ Stabilized Base
- ◉ Aggregate Base or LCB w/ bond breaker

ALPHA: 0.000006 /°F
TRANGE: 120.0 °F
e: 0.00015 strain
D: 10.43 in
P: 9,000 lbf
T: 0.45

Base Type

- ○ Stabilized Base
- ◉ Unstabilized Base

FI: 660 °F-days
CESAL: 11.00 million
Age: 22.0 years
C_d: 1.00

Faulting (doweled)

0.05 in

Faulting Check - **PASS**

NONDOWELED PAVEMENT

Days90: days

D: 10.43 in

Base Type

- ○ Stabilized Base
- ◉ Unstabilized Base

FI: °F-days
CESAL: 11.00 million
Age: years
C_d:

Faulting (nondoweled)

in

Faulting Check -

Recommended critical mean joint faulting levels for design (Table 28)

Joint Spacing	Critical Mean Joint Faulting
< 25 ft	0.06 in
> 25 ft	0.13 in

Note: Joint load position stress checks need to be performed only for nondoweled pavements

Only two numbers need to be entered in this sheet:

Temperature gradient

Tensile stress at top of slab

Step 1:

Total Negative Temperature Differential

Slab Thickness: 10.43 in

Total Negative Temperature Differential: -6.1 °F

Construction Curling and Moisture Gradient Temperature Differential

Enter temperature gradient: 2.0 °F/in (enter positive value from below)

For temperature gradient use:

Wet Climate:	0 to 2 °F/in	(Annual Precipitation >= 30 in or Thornthwaite Moisture Index > 0)
Dry Climate:	1 to 3 °F/in	(Annual Precipitation < 30 in or Thornthwaite Moisture Index < 0)

Total Effective Negative Temp. Differential: -27.0 °F

Step 2:

Use one or more of the following charts to estimate the tensile stress at top of slab. Note that the charts show the variation of tensile stress with negative temperature differential for slab thicknesses ranging from 7 to 13 in. These are plotted for a base course thickness of 6 in. The six charts represent three k-values (100, 250 and 500 psi/in) and two values for the elastic modulus of the base (25,000 psi and 1,000,000 psi). Use judgment to extrapolate the value of the tensile stress at the top of the slab from these charts.

Enter Tensile Stress at Top of Slab: 200 psi (use charts below)

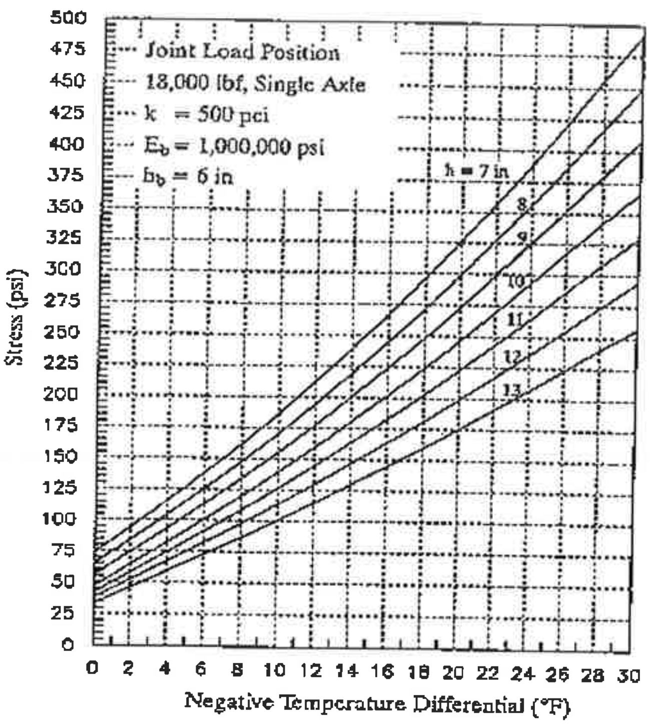
Step 3:

Compare the above tensile stress with the maximum tensile stress at the bottom of the slab for which the slab is designed. For the given inputs and the above thickness, this value is

207	psi

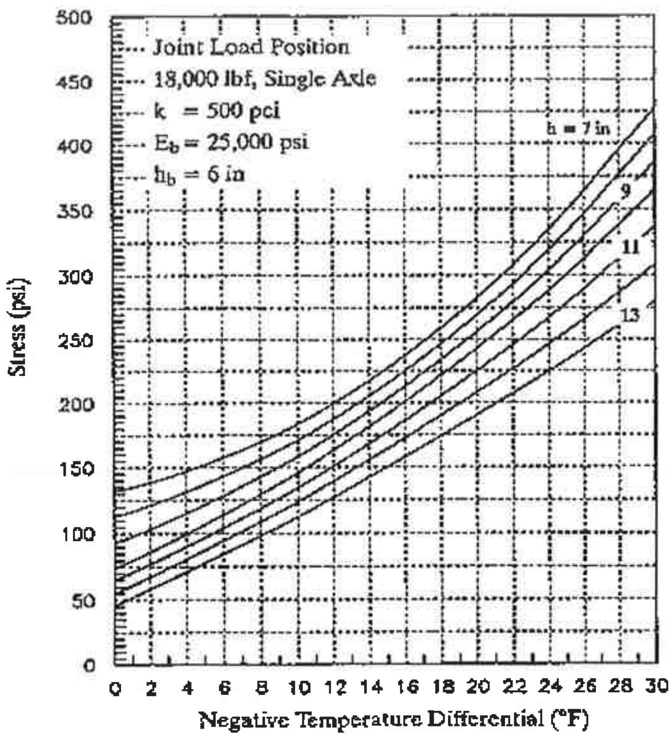
The slab is designed for a tensile stress of 207 psi.
If the tensile stress at the top of the slab (obtained from the charts below and entered above) is less than the design stress, the design is acceptable. If the check fails, new inputs have to be provided.

Corner Break Check: **PASS**



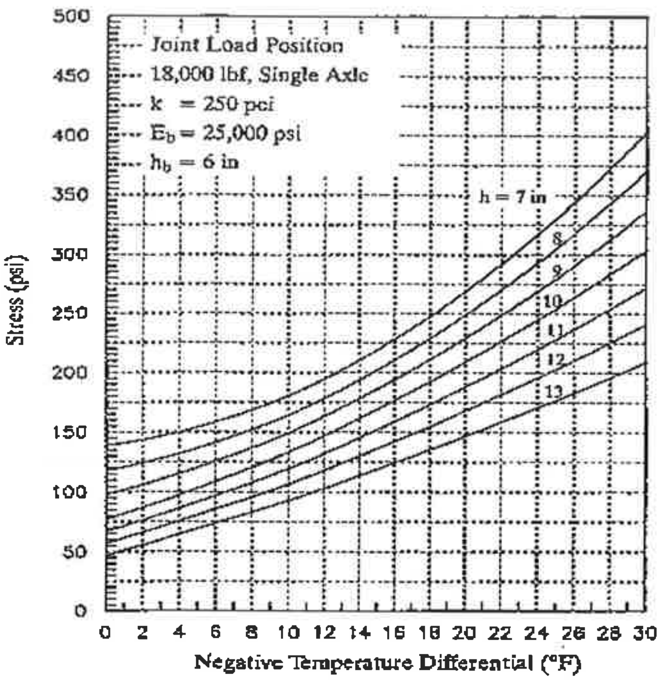
1 lbf = 4.45 N, 1 pci = 0.271 kPa/mm, 1 psi = 6.89 kPa, 1 in = 25.4 mm, °C = (°F - 32)/1.8

Figure 59. Tensile stress at top of slab for joint loading position, negative temperature differential, and full friction, for high-strength base and stiff subgrade.



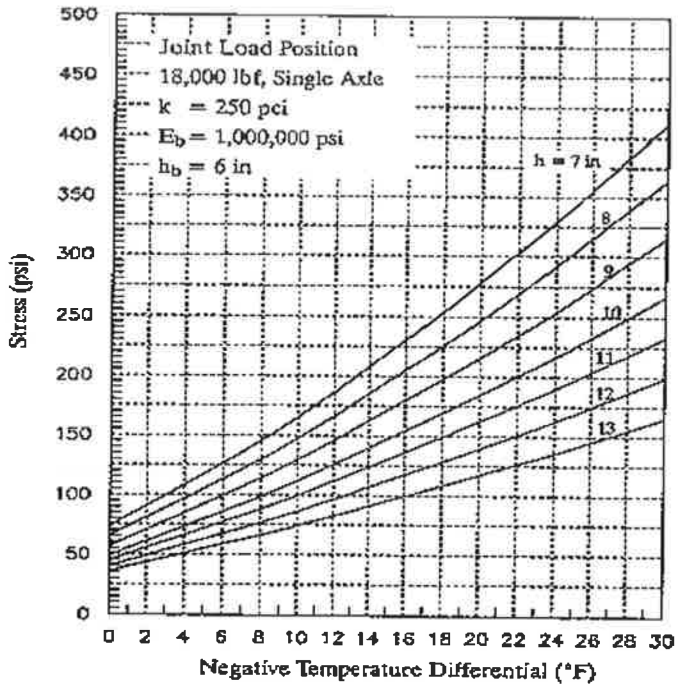
1 lbf = 4.45 N, 1 pci = 0.271 kPa/mm, 1 psi = 6.89 kPa, 1 in = 25.4 mm, °C = (°F - 32)/1.8

Figure 58. Tensile stress at top of slab for joint loading position, negative temperature differential, and full friction, for aggregate base and stiff subgrade.



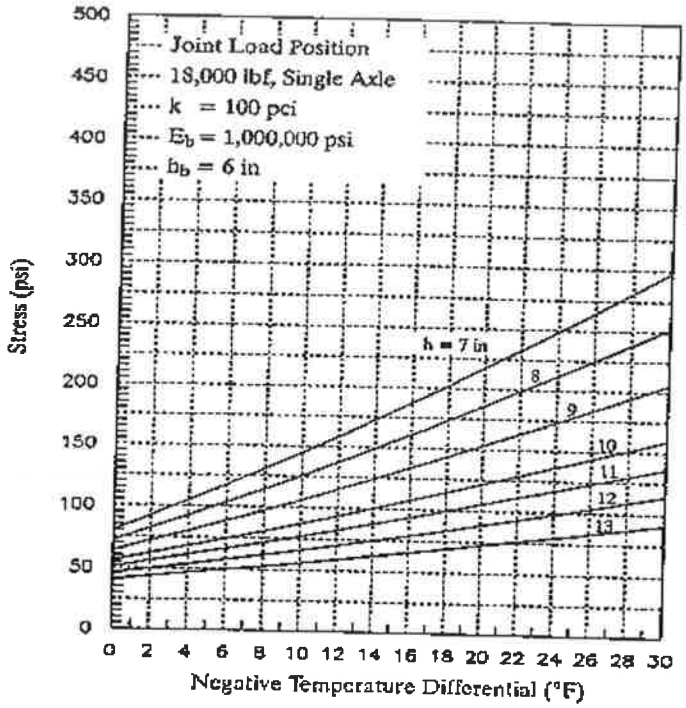
1 lbf = 4.45 N, 1 pci = 0.271 kPa/mm, 1 psi = 6.89 kPa, 1 in = 25.4 mm, °C = (°F - 32)/1.8

Figure 56. Tensile stress at top of slab for joint loading position, negative temperature differential, and full friction, for aggregate base and medium subgrade.



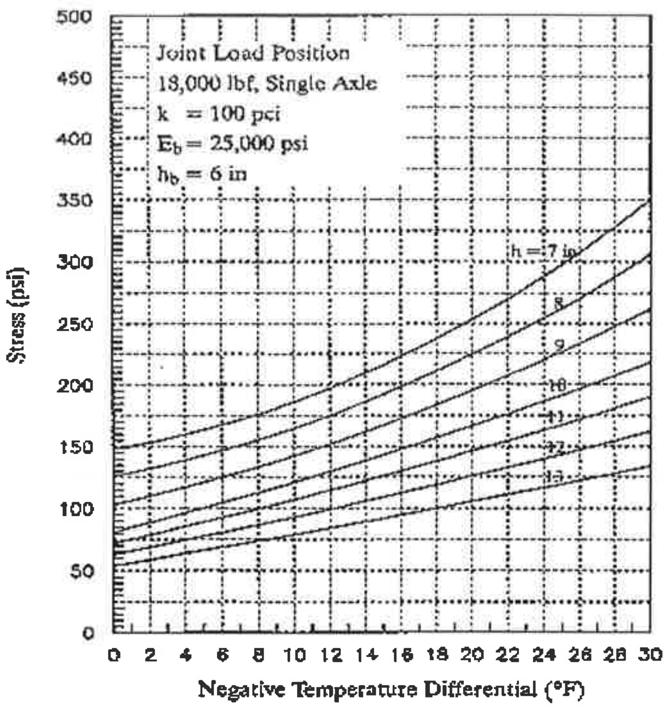
1 lbf = 4.45 N, 1 pci = 0.271 kPa/mm, 1 psi = 6.89 kPa, 1 in = 25.4 mm, °C = (°F - 32)/1.8

Figure 57. Tensile stress at top of slab for joint loading position, negative temperature differential, and full friction, for high-strength base and medium subgrade.



1 lbf = 4.45 N, 1 pci = 0.271 kPa/mm, 1 psi = 6.89 kPa, 1 in = 25.4 mm, °C = (°F - 32)/1.8

Figure 55. Tensile stress at top of slab for joint loading position, negative temperature differential, and full friction, for high-strength base and soft subgrade.



1 lbf = 4.45 N, 1 pci = 0.271 kPa/mm, 1 psi = 6.89 kPa, 1 in = 25.4 mm, °C = (°F - 32)/1.8

Figure 54. Tensile stress at top of slab for joint loading position, negative temperature differential, and full friction, for aggregate base and soft subgrade.