GEOTECHNICAL STUDY

NEW SPORT FIELDS
WEED, CALIFORNIA

Prepared For:

College of the Siskiyous




CGi Technical Services, Inc.

College of the Siskiyous

CGI Technical Services, Inc.

October 12, 2018
CGI: 17-2333.01

Mr. Eric C. Rulofson
Director of Maintenance & Operations
College of the Siskiyous
800 College Ave. Weed CA 96094

Subject: **Geotechnical Report**
College of the Siskiyous
New Sport Fields & Associated Improvements
Weed, California

Dear Mr. Rulofson,

CGI Technical Services, Inc. (CGI), is pleased to submit this geotechnical report for the proposed sport fields and associated improvements at the College of the Siskiyous Weed Campus located in Siskiyou County, California. This report presents our findings, conclusions, and recommendations for design of the proposed development.

We appreciate the opportunity to perform this study and look forward to continued participation during the design and construction phases of this project. If you have any questions pertaining to this report, or if we may be of further service, please contact us at (530) 244-6277 at your earliest convenience.

Regards,

CGI TECHNICAL SERVICES, INC.

REGISTERED PROFESSIONAL ENGINEER AZEDDINE BAHLOUL GE 2847 GEOTECHNICAL STATE OF CALIFORNIA

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COLLEGE OF THE SISKIYOUS
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1 GENERAL

This report presents the results of CGI Technical Services, Inc. (CGI), geotechnical study for a proposed sports field and associated improvements located at the College of the Siskiyous (COS), Weed Campus in Weed, California. CGI has prepared this report at the request of COS. The project location is shown on Plate 1 – Site Location Map. The following sections present our understanding of the project, the purpose of our study, and the findings, conclusions, and recommendations of this study.

1.1 PROJECT LOCATION

The proposed project site is located at the southern portion of the College of the Siskiyous campus at 800 College Avenue in Weed, California, as shown on Plate 1. Latitude and longitude for the approximate center of the proposed project site are as follows:

- Latitude: 41° 24' 38.51" (41.410696 °)
- Longitude: -122° 23' 23.60" (-122.389912°)

1.2 PROJECT UNDERSTANDING

We understand that the project consists of the design and construction of a new sport fields located near the current grass sports field at the COS Weed Campus. We understand that with proposed fields will be the construction of improvements consisting of a new scoreboard, a new black vinyl chain fence, a new concrete walkway, and bleacher pad.

No structures are proposed for this phase of the project but if in the future work should include structures it is anticipated that the structure will be supported on shallow foundation systems (spread foundations).

1.3 STUDY PURPOSE

The purpose of our geotechnical study was to explore and evaluate selected site surface and subsurface conditions in order to provide geotechnical engineering recommendations related to the design and construction of the proposed project. Exploration locations for the project are shown on Plate 3 – Geotechnical Map.

1.4 PREVIOUS WORK PERFORMED & REFERENCES REVIEWED

CGI knows of no prior geotechnical study that have been performed at the project site. A geotechnical study was performed for a proposed surface replacement for existing sport fields.

1.5 SCOPE OF SERVICES

Services performed for this study are in general conformance with California Building Code. Our scope of services included:

- Reconnaissance of the site surface conditions, topography, and existing drainage features;
- Attempted acquisition of existing, available geotechnical data relevant to the project site;
- Performance of reconnaissance-level geologic mapping of the project site.
- Excavation of seven test pits and previously five exploratory drill holes at selected locations on the project property, as shown on Plate 3. Exploration procedures and logs of drill holes are presented in Appendix A;
- Performance of laboratory testing on selected samples obtained during our field investigation. Laboratory test procedures and results of those tests are presented in Appendix B – Laboratory Testing;
- Preparation of this report, which includes:
    - A description of the proposed project;
    - A summary of our field exploration and laboratory testing programs;
    - A description of site surface and subsurface conditions encountered during our field investigation;
    - California Building Code (CBC) seismic design criteria;
    - A geotechnical map showing approximate field exploration locations, presented as Plate 3;
    - Geotechnical recommendations for:
        - Site preparation, engineered fill, site drainage, and subgrades;
        - Suitability of on-site materials for use as engineered fill;
        - Total and differential settlement;
        - Foundation and slab-on-grade design;
        - Temporary excavations, shoring, and trench backfill;
        - Trench backfill and compaction recommendations; and
        - Lateral earth pressures for retaining wall design and construction.
    - Appendices that present a summary of our field investigation procedures and laboratory testing programs.

2 FINDINGS

2.1 SURFACE CONDITIONS

The project site is relatively flat with gentle slopes to perimeter of the site. The site is covered with grass. Several score boards, field goals and track and field associated improvements exist across the site.

Drainage at the site occurs as sheetflow toward the perimeter of the fields site. The elevation at the site is about 3,581 feet above mean sea level (MSL).

2.2 SUBSURFACE CONDITIONS

The project site is underlain by top soil and pinkish brown, silty sand with gravel to a depth of about 8 feet. At two test pits (TP-3 & TP-5) dark brown material with debris overlaying the previous material was encountered. This material appears to be an artificial fill. The native pinkish brown, silty sand materials were not fully penetrated in any excavations advanced for this study.

2.3 SOILS & GEOLOGIC CONDITIONS

2.3.1 Regional Geology

The project site is located in the Cascade Range geologic/geomorphic province of California. The Cascade Range province extends from the northern end of the Sierra Nevada north to the Canadian border. In the project vicinity the Cascade Range province is bounded to the west by the Klamath Mountain province, to the east by the Modoc Plateau province, to the south by the Sierra Nevada province, and to the north by the Cascade Range extending through Oregon and Washington.

The Cascade Range province consists of a north-northwest-trending, relatively linear belt of active and dormant strata and shield volcanoes. The regional geologic conditions are dominated by andesitic, rhyolitic and basaltic volcanic rocks mantled with surficial deposits consisting of pyroclastic rocks, lahar deposits, alluvium, and local lacustrine sediments (Hinds, 1952).

2.3.2 Local Geologic Setting

The project site is located on the Shastina Pyroclastic Flow (Qv^{ps}) area of Siskiyou County (Wagner & Saucedo, 1987). Pyroclastic flows consist predominately of granular soils with abundant sand and gravel. Changes in grain size, color and distribution of larger grained material occur often throughout the soil depth.

Artificial fill associated with original athletic field construction activities are present on site. Those fill materials consist of a mixture of organic soil, and silty sand with a trace of gravel. The artificial fill exists as a layer at the surface above the native soils.

2.3.3 Groundwater

Groundwater was not encountered during the excavation and exploration of the site. The depth to groundwater beneath the project site is expected to be at least 11.5 feet below ground surface (12/7/17). Groundwater elevations will fluctuate over time. The depth to groundwater can vary throughout the year and from year to year. Intense and long duration precipitation, modification of topography, and cultural land use changes at the reservoir and at surrounding properties, such as irrigation, water well usage, on site waste disposal systems, utility leakage, and water diversions can contribute to fluctuations in groundwater levels. Localized saturated conditions or perched groundwater conditions near the ground surface could be present during and following periods of heavy precipitation or if on-site sources contribute water. If groundwater is encountered during construction, it is the Contractor's responsibility to install mitigation measures for adverse impacts caused by groundwater encountered in excavations.

3 GEOLOGICAL HAZARDS

The following sections address geologic hazards that could influence the project and provide a discussion and opinion regarding the potential impact of each of those hazards to the project.

It should be noted that the project site does not lie within any established Geologic Hazards Zones, either within the City of Weed or the County of Siskiyou.

3.1 FAULTING & SEISMICITY

3.1.1 Seismic Setting

The State of California designates faults as active, potentially active, and inactive depending on the recency of movement that can be substantiated for a fault. Fault activity is rated as follows:

FAULT ACTIVITY RATINGS		
Fault Activity Rating	**Geologic Period of Last Rupture**	**Time Interval (Years)**
Active	Holocene	Within last 11,000 Years
Potentially Active	Quaternary	>11,000 to 1.6 Million Years
Inactive	Pre-Quaternary	Greater than 1.6 Million Years

The California Geologic Survey (CGS) evaluates the activity rating of a fault in fault evaluation reports (FER). FERs compile available geologic and seismologic data and evaluate if a fault should be zoned as active, potentially active, or inactive. If an FER evaluates a fault as active, then it is typically incorporated into a Special Studies Zone in accordance with the Alquist-Priolo Earthquake Hazards Act (AP). AP Special Studies Zones require site-specific evaluation of fault location and require a structure setback if the fault is found traversing a project site.

The site is not located within an Alquist-Priolo Earthquake Fault Zone and no active faults are known to pass through the project site (Jennings, 1994; Hart & Bryant, 1997). However, a number of regional and local faults traverse the project region. The closest mapped fault is the potentially active Yellow Butte fault, located about 10 miles northeast of the site (Jennings, 1994). The closest active fault, as zoned by the State, is the Cedar Mountain fault, located about 26 miles east of the site.

3.1.2 CBC Design Recommendations

At a minimum, structures should be designed in accordance with the current CBC seismic design criteria as follows:

CBC SEISMIC DESIGN PARAMETERS		
California Building Code	**Parameter**	**CBC Designation**
Site Coordinates	Latitude	41.410696°
	Longitude	-122.389912°
Section 1613.3.3 Table 1613.3.3(1)	Site Coefficient, F_a	1.212
Section 1613.3.3 Table 1613.3.3(2)	Site Coefficient, F_v	1.745
Section 1613.3.1 Figure 1613.3.1	Site Class Designation	D
	Seismic Factor, Site Class B at 0.2 Seconds, S_s	0.735g
	Seismic Factor, Site Class B at 1.0 Seconds, S_1	0.328g
Section 1613.3.3	Site Specific Response Parameter for Site Class D at 0.2 Seconds, S_{MS}	0.891g
	Site Specific Response Parameter for Site Class D at 1.0 Seconds, S_{M1}	0.572g
Section 1613.3.4	$S_{DS}=2/3S_{MS}$	0.594g
	$S_{D1}=2/3S_{M1}$	0.381g

3.2 LANDSLIDES

The site is relatively flat and no signs of active or incipient slope failures were observed during this study. It is our opinion that natural landslides pose a low risk to the proposed project.

3.3 LIQUEFACTION AND LATERAL SPREADING

Liquefaction is described as the sudden loss of soil shear strength due to a rapid increase of soil pore water pressures caused by cyclic loading from a seismic event. In simple terms, it means that a liquefied soil acts more like a fluid than a solid when shaken during an earthquake. In order for liquefaction to occur, the following are needed:

- Granular soils (sand, silty sand, sandy silt, and some gravels);
- A high groundwater table; and
- A low density in the granular soils underlying the site.

If those criteria are present, then there is a potential that the soils could liquefy during a seismic event.The adverse effects of liquefaction include local and regional ground settlement, ground cracking and expulsion of water and sand, the partial or complete loss of bearing and confining forces used to support loads, amplification of seismic shaking, and lateral spreading. In general, the effects of liquefaction on the proposed project could include:

- Lateral spreading;
- Vertical settlement; and/or
- The soils surrounding lifelines can lose their strength and those lifelines can become damaged or severed.

Lateral spreading is defined as lateral earth movement of liquefied soils, or soil riding on a liquefied soil layer, down slope toward an unsupported slope face, such as a creek bank, or an inclined slope face. In general, lateral spreading has been observed on low to moderate gradient slopes, but has been noted on slopes inclined as flat as one degree.

Another potentially adverse secondary seismic effect is co-seismic compaction of moderately consolidated, sandy, relatively cohesionless soils above or below groundwater. Co-seismic compaction is soil densification resulting from dynamic loading of relatively loose, non-cohesive soil materials. That is, shaking or vibration can densify loose to moderately consolidated granular soils, resulting in settlement of the ground surface.

The project site is underlain by sediments derived from volcanic rock sources. Because of the sediment consistency/density, it is our opinion that liquefaction poses a low risk to the proposed project.

3.4 EXPANSION POTENTIAL

There is a direct relationship between plasticity of a soil and the potential for expansive behavior, with expansive soil generally having a high plasticity. Thus, granular soils typically have a low potential to be expansive, whereas, clay-rich soils can have a low to high potential to be expansive. The majority of soils encountered during this study were granular and, therefore, nonexpansive.

3.5 SOIL CHEMISTRY

One selected sample of near-surface soils encountered at the site was subjected to chemical analysis for the purpose of assessment of corrosion and reactivity with concrete. The samples were tested for soluble sulfates and chlorides. Testing was conducted by HDR of Claremont and results are presented below, as well as included in the appendix of laboratory results.

SOIL CHEMISTRY RESULTS					
Sample	**Sample Depth**	**Sulfates (ppm)**	**Chlorides (ppm)**	**pH**	**Resistivity (ohms-cm)**
DH-4	0-5’	12	3.4	6.4	20,400

According to the ACI-318, a sulfate concentration below 0.10 percent by weight (1,000 ppm) is negligible. A chloride content of less than 500 ppm is generally considered non-corrosive to reinforced concrete.

Minimum resistivity testing performed on the soil sample indicated the soils are considered to be mildly corrosive to buried metal objects. A commonly accepted correlation between soil resistivity and corrosivity towards ferrous metals (NACE Corrosion Basics, 1984) is provided below:

RESISTIVITY & CORROSION CORRELATION	
Minimum Resistivity (ohm-cm)	**Corrosion Potential**
0 to 1000	Severely Corrosive
1,000 to 2,000	Corrosive
2,000 to 10,000	Moderately Corrosive
Over 10,000	Mildly Corrosive

If engineered fill materials will be placed to establish grades and backfill adjacent to concrete structures, we recommend that verification samples be tested to confirm that soils in contact with concrete and steel have similar corrosion potential characteristics as the sample tested for this study.

3.6 VOLCANIC HAZARDS

The project site is located in the Cascade Range, which contains numerous active but dormant volcanoes. Volcanic hazards can occur from a variety of causes but are typically associated with the following:

- Ground deformation
- Lava flows;
- Pyroclastic flows;
- Volcanoclastic debris flows;
- Tephra; and/or
- Volcanic Gasses

The closest significant eruptive center to the project site is Mount Shasta, located about 10 miles (14 kilometers) east of the project site. Other volcanic sources in the region include Medicine Lake, Goosenest Mountain, Mount McLaughlin. It is likely that Mount Shasta poses the greatest risk to the project site due to its proximity, activity, and size. Mount Shasta has an eruption recurrence interval of about 600 years on average and last erupted about 200 years ago (Miller, 1980). Thus, while an eruption could occur any time, it is unlikely to occur soon based on its past history.

Ground deformation consists of the tilting, doming, or collapse of the ground surface in the vicinity of a volcanic center. Significant ground deformation can be experienced due to the rise of magma leading up to and following a volcanic eruption or due to migration of subsurface magma that does not lead to eruption. Typically, this deformation occurs on or immediately adjacent to the volcanic source. Because the proposed project is located over 9 miles from the closest volcanic cone, there is a low risk of ground deformation that will adversely affect the project site.

Typically, lava flows pose a risk to life and property when people or improvements are located within about 5 miles of the source (Miller, 1980). The project site is well outside of that range and thus, has a low risk of being affected by lava flows.

Pyroclastic flows are masses of volcanic rocks mixed with hot gasses that can travel very rapidly down volcanic slopes and extend onto adjacent ground for some distance. They generally follow valleys and other depressions but can build up sufficient momentum to carry them over ridges and low hills. It is anticipated that pyroclastic flows originating high on Mount Shasta could extend as far as about 10 or 11 miles from origin (Miller, 1980). Thus, with this site is in within this range, but at the furthest edge of the affected range, the risk to the site from pyroclastic flows is low to moderate.

The site is located in the southern portion of volcaniclastic deposits associated with a gigantic debris avalanche that occurred about 300,000 to 380,000 years ago (Crandell, 1988). That debris avalanche extended north-northwest of Mt Shasta through Weed, Grenada and north of Montague. It was derived from an ancestral and much larger Mt Shasta whose remnants are no longer apparent (Crandell, 1988).

The result of the ancestral debris avalanche was to mobilize large blocks of andesite derived from Mt Shasta across the Shasta Valley. The large blocks are, in turn, surrounded and locally covered with a matrix of unsorted and unstratified debris consisting of volcanic ash, pebbles, cobbles and boulders in a silty sand (Crandell, 1988). This has created the morphology in the Shasta Valley where there are isolated hills surrounded by relatively flat or slightly undulating valleys. Such occurrences of large volcaniclastic debris flows or immense landslides capable of traveling tens of miles from their source are rare and occur on a limited basis in geologic time. The potential of such a failure impacting the propose project during the anticipated life span of the project is improbable and in our opinion poses little risk.

Tephra includes ash, rock, and pumice which are erupted into the atmosphere above a volcano. Large tephra particles typically fall to earth in areas relatively close to the source; however, ash can be carried long distances from the source and poses health and structure damage, specifically when thick accumulation of wet ash occur on a structure. As noted in (Crandell, 1988) the project site is located within about 25 kilometers of Mount Shasta, which places it within an area that feasibly could receive a significant thickness (4 to 39-inchs) of tephra deposits. However, tephra is typically deposited in lobate shaped areas that follow prevalent winds in the region. According to Miller (1980), the prevalent winds in the region occur to the northeast and southeast about 82 percent of the time. The risk for ashfall is present at the site, though it is likely to not result in thick accumulations of ash that could impact the project.

Volcanoes can discharge hot and toxic gasses that pose a threat to life and property. These fumarolic gasses are influenced by the wind and dissipate relatively quickly, thus, are typically a risk confined to areas on or immediately near the source. The project site is sufficiently removed to make discharge of volcanic gasses a low risk for the project.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our investigation, it is our opinion that the site is suitable for the proposed improvements provided recommendations presented, herein, are utilized during design and construction of the project. Specific comments and recommendations regarding the geotechnical aspects of project design and construction are presented in the following sections of this report and are intended to be refined, where needed, as the project moves into final design and construction.

Recommendations presented, herein, are based upon the preliminary site plans provided by Client along with stated assumptions. Changes in the configuration from those studied during this investigation may require supplemental recommendations.

4.2 SITE PREPARATION AND GRADING

4.2.1 Stripping

Prior to general site grading and/or construction of planned improvements, debris and deleterious materials, where present, should be stripped and disposed of off-site or outside the construction limits. Stripping depths of about 2 to 5 inches should be anticipated for the project except in those areas discussed in Section 4.2.3, which will extend deeper. In areas where trees and dense shrubs might have been present prior to the site development or are removed for the project, root balls and concentrations of organic materials could be encountered. In areas where concrete and foundations (if any) are encountered, those materials should be removed. If those materials are exposed, we recommend that they be stripped and removed from the project site prior to engineered fill placement or construction of project improvements. Any voids created by removal of roots, debris, and/or deleterious materials should be filled using engineered fill materials described in Section 4.2.11 and/or 4.2.12, and placed according to recommendation made in Section 4.2.14 unless those areas are within proposed cut slopes and will be removed in their entirety during grading.

4.2.2 Existing Utilities, Wells, and/or Foundations

If subsurface utilities are encountered during construction, they should be removed and/or rerouted beyond construction limits. Buried tanks or wells, if present, should be removed/destroyed in compliance with applicable regulatory agency requirements. Existing, below-grade utility pipelines that extend beyond the limits of the proposed construction and that will be abandoned in-place should be plugged with lean concrete or grout to prevent migration of soil and/or water. All excavations resulting from removal and demolition activities should be cleaned of loose or disturbed material prior to placing any fill or backfill.

4.2.3 Overexcavation

Artificial fill materials cover portion of the project site (TP-3 & TP-5) and along utility lines. Depending on project specifications artificial fill materials may be removed to a depth at which native soil is contacted, this may require soil to be overexcavated and replaced with engineered fill.

We recommend that a CGI engineer or geologist observe and approve any overexcavated areas prior to placement of engineered fill materials per recommendations made Section 4.2.14 of this report.

4.2.4 Keying and Benching

It is not anticipated that engineered fill materials will be placed on slopes having inclinations of 5:1 (horizontal to vertical) or steeper, except within areas of overexcavation, which are confined. Therefore, keying is not anticipated to be necessary during this project.

4.2.5 Scarification and Compaction

Following site stripping and overexcavation, areas to receive engineered fill should be scarified to a minimum depth of 8 inches, uniformly moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent of the maximum dry density as determined using standard test method ASTM D1557[1].

4.2.6 Wet/Unstable Soil Conditions

On-site soils encountered during grading may be significantly over optimum moisture content, depending on when construction is performed. These conditions could hinder equipment access as well as efforts to compact site soils to a specified level of compaction. If over optimum soil moisture content conditions are encountered during construction, disking to aerate, replacement with imported material, chemical treatment, stabilization with a geotextile fabric or grid, and/or other methods will likely be required to facilitate earthwork operations. The applicable method of stabilization is the Contractor's responsibility and will depend on the contractor's capabilities and experience, as well as other project-related factors beyond the scope of this investigation. Therefore, if over-optimum moisture within the soil is encountered during construction, CGI should review these conditions (as well as the contractor's capabilities) and, if requested, provide recommendations for their treatment.

4.2.7 Site Drainage

Grading should be performed in such a manner that provides positive surface gradient away from all structures for a minimum distance of at least 10 feet. The ponding of water should not be allowed adjacent to structures or retaining walls. Surface runoff should be directed

[1] This test procedure applies wherever relative compaction, maximum dry density, or optimum moisture content is referenced within this report.

toward engineered collection systems or suitable discharge areas and not allowed to flow over slopes. Discharge from structures should also be collected in solid (unperforated) pipelines, conveyed, and discharged away into engineered systems, such as storm drains. Landscape plantings around structures should be avoided or be dry climate tolerant and require minimal irrigation.

Based on testing performed for the proposed surface replacement of the existing sport fields, the upper two to three feet of soil/material at the project site have a lower hydraulic conductivity (permeability) than the material below. For any sports field, drainage should be designed by the civil project engineer.

4.2.8 Temporary Slopes

This section explicitly excludes trench slopes for buried utilities. Temporary trench excavations are discussed in Section 4.5.1 of this report.

Construction of temporary slopes to facilitate construction of the proposed project is not anticipated, except in the area of overexcavation noted in Section 4.2.3. Temporary excavations must comply with applicable local, state, and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards. Construction site safety is the responsibility of the Contractor, who should be solely responsible for the means, methods, and sequencing of construction operations so that a safe working environment is maintained.

Temporary construction slopes can be constructed at inclinations of up to 45 degrees. If possible, we recommend that temporary slopes in excess of 15 feet in height be exposed only during seasonal dry times of year and not be allowed to remain exposed between November and March.

Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a 1:1 (horizontal to vertical) projection from the toe of the excavation to the ground surface, unless shoring is being used and has specifically been designed for those surcharge loads. Where the stability of adjoining improvements, walls, or other structures is endangered by excavation operations, support systems such as shoring, bracing, or underpinning may be required to provide structural stability and to protect personnel working within the excavation.

During wet weather, earthen berms or other methods should be used to prevent runoff water from entering excavations. All runoff water entering the excavation(s) should be collected and disposed of outside the construction limits.

4.2.9 Permanent Slopes & Setbacks

If permanent slopes are required for the project, we recommend that those slopes be inclined no steeper than 2:1 (horizontal to vertical). If steeper slopes are required then CGI should be contacted to help engineer those slopes or retaining walls should be utilized in the site design.

In order to comply with CBC regulations, minimum setbacks for proposed structures should be equivalent to the height of the slope divided by 3, but need not exceed 40 feet. If the desired setbacks are less than these requirements, then the foundations of the structures should be deepened or opt for alternate setbacks in accordance with requirements of section 1805.3.5 of 2016 CBC.

4.2.10 On-Site Soil Materials

It is our opinion that most of the near-surface soils encountered at the site can be used for general engineered fill provided they are free of organics, debris, oversized particles (>3") and deleterious materials. Gravel and aggregate base materials free of debris, organics, and deleterious materials are also acceptable for use within general engineered fill. If highly plastic clayey materials (materials having a plasticity index exceeding 30 and a liquid limit in excess of 50) are encountered during grading, those materials should be segregated and excluded from engineered fill, where possible. If potentially unsuitable soil is considered for use as engineered fill, CGI should observe, test, and provide recommendations as to the suitability of the material prior to placement as engineered fill.

4.2.11 Imported Fill Materials - General

If imported fill materials are used for this project, they should consist of soil and/or soil-aggregate mixtures generally less than 3 inches in maximum dimension, nearly free of organic or other deleterious debris, and essentially non-plastic. Typically, well-graded mixtures of gravel, sand, non-plastic silt, and small quantities of clay are acceptable for use as imported engineered fill within foundation areas. Imported fill materials should be sampled and tested prior to importation to the project site to verify that those materials meet recommended material criteria noted below. Specific requirements for imported fill materials, as well as applicable test procedures to verify material suitability are as follows:

IMPORTED FILL RECOMMENDATIONS				
GRADATION				
Sieve Size	**General Fill**	**Granular Fill**	**Test Procedures**	
	Percent Passing		**ASTM**	**AASHTO**
3-inch	100	100	D422	T88
¾-inch	70 – 100	70 – 100	D422	T88
No. 200	0 - 30	<5	D422	T88
PLASTICITY				
Liquid Limit	<30	NA	D4318	T89
Plasticity Index	<12	Nonplastic	D4318	T90
ORGANIC CONTENT	<3%	<3%	D2974	NA
Soil chemistry tests are recommended on imported soils to evaluate corrosivity to buried improvements.				

4.2.12 Materials - Granular

All granular fill should consist of imported soil mixtures generally less than 3 inches in maximum dimension, nearly free of organic or other deleterious debris, and essentially non-plastic. Specific requirements for granular fill, as well as applicable test procedures to verify material suitability are presented in Section 4.2.11 of this report.

4.2.13 Controlled Low Strength Material

Controlled low strength material (CLSM) can be used to backfill excavated areas or as engineered fill materials. CLSM consists of a fluid, workable mixture of aggregate, cement, and water that is of limited strength as to allow future excavation and maintenance of buried improvements yet capable of supporting the proposed improvements. If CLSM is used as engineered fill materials, we recommend that those materials conform and be placed according to specifications presented in Section 19-3 of the 2010 Caltrans Standard Specifications. Use of CLSM in sports fields should be approved by the civil project engineer.

It should be noted that CLSM exhibits lower hydraulic conductivity (permeability) than onsite soils.

4.2.14 Placement & Compaction

Soil and/or soil-aggregate mixtures used for engineered fill should be uniformly moisture-conditioned to within 3 percent of optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent relative compaction.

It is recommended that fill materials be placed and compacted uniformly in elevation around the buried structures and that the vertical elevation differential of contiguous lifts diverge no more than three feet around the structure during compaction. Testing should be performed to verify that the relative compactions are being obtained as recommended herein.

Compaction testing, at a minimum, should consist of one test per every 500 cubic yards of soil being placed or at every 1.5-foot vertical fill interval, whichever comes first.

In general, a "sheep's foot" or "wedge foot" compactor should be used to compact fine-grained fill materials. A vibrating smooth drum roller could be used to compact granular fill materials and final fill surfaces.

4.3 FOUNDATIONS & SLABS

4.3.1 General

Transition lots, where structures span across both native cut materials and engineered fills, can lead to differential settlement issues. Foundations should not span both cuts and fills.

4.3.2 Shallow Foundations

4.3.2.1 General

Foundations must be sized, embedded, and reinforced in accordance with recommendation made by the project structural engineer. All foundation excavations should be made level, with the exception of vertical steps. The allowable bearing pressures provided below are based on a recommended minimum embedment depth of 12 inches below the graded engineered fill surface and a minimum width of 12 inches. Footing size should be determined by the Structural Engineer.

4.3.2.2 Allowable Bearing Pressures

It is assumed that all foundations for the proposed structures, with the exception of isolated foundations for items such as light standards, will rest entirely on undisturbed natural soils or rock materials as discussed above. In general, soils at the site conform to Class of Materials Type 4 in accordance with Table 1806.2 of the 2016 CBC.

Isolated and continuous footing elements should be proportioned for dead loads plus probable maximum live load, and an allowable bearing pressure of 2,000 pounds per square foot (psf). The allowable bearing capacity can be increased by 150 psf for every additional foot of embedment beneath the minimum specified CBC foundation depth, up to a maximum allowable bearing capacity of 1.5 times the allowable bearing capacity.

If large structures should be included in the project at a later time, CGI should be notified and allowed to evaluate the impacts of those loads on underlying soils.

The allowable bearing pressures provided are net values. Therefore, the weight of the foundation (which extends below finished subgrade) may be neglected when computing dead loads. The allowable bearing pressure applies to dead plus live loads and includes a calculated factor of safety of at least 3. An increase of allowable bearing pressure by one-third for short-term loading due to wind or seismic forces should NOT be incorporated unless an alternative load combination, as described in Section 1605.3.2 of the 2016 CBC, is applied. The allowable bearing value is for vertical loads only; eccentric loads may require adjustment to the values recommended above. We recommend that CGI be allowed to observe foundation excavations to confirm projected site conditions.

4.3.2.3 Estimated Settlements

The anticipated total settlement for structure foundations, if construction occurs as recommended within this report, should be less than one inch. Differential settlement for the structure foundations is anticipated to be less than ½-inch in 20 feet.

4.3.3 Slab-on-Grade Design

All ground-supported slabs should be designed by a Civil Engineer to support the anticipated loading conditions. Reinforcement for slabs should be designed by a Civil Engineer to maintain structural integrity, and should not be less than that required to meet pertinent code, shrinkage, and temperature requirements. Reinforcement should be placed at mid-thickness in the slab with provisions to ensure it stays in that position during construction and concrete placement.

The mat can be designed using a flat slab on an elastic half-space analog. A modulus of subgrade reaction (k_{s1}) of 150 kcf is recommended for design of mat-type foundations. That modulus of subgrade reaction value represents a presumptive value based on soil classification. No plate-load tests were performed as part of this study. The modulus value is for a 1-foot-square plate and must be corrected for mat size and shape, assuming a cohesionless subgrade.

Subgrade soils supporting interior concrete floor slabs should be scarified to a minimum depth of 8 inches, uniformly moisture-conditioned to near the optimum moisture content, and compacted to at least 90 percent relative compaction.

4.3.4 Rock Capillary Break/Vapor Barrier

Interior concrete floor slabs supported-on-grade should be underlain by a capillary break consisting of a blanket of compacted, free-draining, durable rock at least 4 inches thick, graded such that 100 percent passes the 1-inch sieve and less than 5 percent passes the No. 4 sieve.[2] Furthermore, a vapor barrier should be placed beneath all interior concrete floor slabs supported-on-grade that will be covered with moisture-sensitive equipment or floor coverings. This barrier may consist of a plastic or vinyl membrane placed directly over the rock capillary break. The vapor barrier should be sealed around all penetrations, including utilities. If a vapor barrier is not installed, there is a risk of moisture vapors and salts penetrating the slab-on-grade. For this project, equipment and flooring materials on slabs-

[2] In general, Caltrans Class 2 aggregate base (or similar material) does not meet the requirements provided above for a capillary break. Therefore, we recommend this material <u>not</u> be used for a capillary break beneath interior concrete slabs supported-on-grade.

on-grade are unknown. It is our recommendation that American Concrete Institute (ACI) guidelines ACI 302 and ACI 360 be referred to regarding installation of vapor barriers based on the anticipated flooring materials to be installed.

A capillary break and/or vapor barrier may not be required for some types of construction (such as equipment buildings, warehouses, garages, and other uninhabited structures insensitive to water intrusion and/or vapor transmission through the slab). For these types of structures, the gravel capillary break and/or vapor barrier recommended above may be omitted and the slab placed directly on the prepared subgrade or other approved surface if it is determined by the civil engineer and architect that water vapors will not adversely affect improvements resting on the slab-on-grade. In the event a capillary break and/or vapor barrier is not to be used, CGI should review the planned structure in order to assess the applicability of the approach and provide (if necessary) additional recommendations regarding subgrade preparation and/or support.

4.3.5 **Lateral Earth Pressures**

It is our understanding that buried structures and retaining walls (heretofore referred to as retaining walls) are likely not to be utilized in this project. However, in the event that such improvements are needed, we have provided the following recommendations.

Retaining walls, including buried concrete tank walls, should be designed to resist earth pressures exerted by the retained, compacted backfill plus any additional lateral force that will be applied to the wall due to surface loads placed at or near the wall. The recommended equivalent fluid weights presented below are for static (non-earthquake) conditions.

LATERAL EARTH PRESSURES UNDER STATIC CONDITIONS

Lateral Earth Pressure Condition	Slope Inclination Above Retaining Structure	Equivalent Fluid Weight (pcf)(*)
		Drained
At-Rest	Flat	55
Active	Flat	45
At-Rest	2:1	80
Active	2:1	65

Lower pressures can be provided if granular material (sandy gravel or gravelly sand) are used.

The resultant force of the static lateral force prism should be applied at a distance of 33 percent of the wall height above the soil elevation on the toe side of the wall.

The tabulated values are based on a non-plastic soil unit weight of 125 pounds per cubic foot (pcf), and do not provide for surcharge conditions resulting from construction materials, equipment, or vehicle traffic. Loads not considered as surcharges should bear behind a 1:1 (horizontal to vertical) line projected upward from the base of the shoring. If surcharges are expected, CGI should be advised so that we can provide additional recommendations as needed. Surcharge loads induce additional pressures on earth retaining structures. An additional lateral load on non-yielding walls equal to 0.5 times the applied surcharge pressure should be included in the design for uniform area surcharge pressures. Lateral pressures for other surcharge loading conditions can be provided, if required.

Ultimate sliding resistance, passive pressures, and safety factors are discussed below in Section 4.3.6 through 4.3.8, respectively.

4.3.6 Sliding Resistance

Sliding resistance generated through a compacted soil/concrete interface can be computed by:

- Multiplying the soil/concrete adhesion (130 psf for artificial fill) by the footing contact area for cohesive soils. In no case should the lateral sliding resistance exceed one-half the dead load; or
- Multiplying the total dead weight structural loads by the friction coefficient of 0.35 for imported and native granular engineered fill.

4.3.7 Passive Resistance

Passive resistance developed from lateral bearing of shallow foundation elements bearing against compacted soil surfaces for that portion of the foundation element extending below a depth of 1 foot below the lowest adjacent grade can be estimated using an equivalent fluid weight of 150 pcf.

4.3.8 Safety Factors

Sliding resistance and passive pressure may be used together without reduction in conjunction with recommended safety factors outlined below. A minimum factor of safety of 1.5 is recommended for foundation sliding.

4.3.9 Frost Penetration

The project site is subject to soil frost penetration during winter months. It is estimated that the project area has a frost penetration depth of less than 12 inches. In accordance with Section 1805.2.1 of the CBC, foundations should extend to a depth beneath estimated frost penetration.

4.3.10 Construction Considerations

In general, soils having a tendency to run, flow or cave were observed during our study across much of the proposed development area. There is a potential that shallow un-shored excavation could locally cave.

Prior to placing steel or concrete, foundation excavations should be cleaned of all debris, loose or disturbed soil, and any water. A representative of CGI should observe all foundation excavations prior to concrete placement.

4.4 RETAINING WALLS

It is our understanding that no retaining walls will be utilized in the construction of the project site. If retaining walls are utilized in the project, CGI should be advised so that we can provide additional recommendations as needed in the design and inspection of retaining walls.

4.5 PIPELINES & TRENCH BACKFILL

4.5.1 **Trenches and Dewatering**

Utility trenches greater than 5 feet deep should be braced or shored in accordance with good construction practices and all applicable safety ordinances. In general, soils having a tendency to run, flow or cave were observed during our study across much of the proposed development area. However, there is a potential that shallow un-shored trenches excavated with sidewalls steeper than 1:1 could locally cave. The actual construction of the trench walls and worker safety is the sole responsibility of the contractor.

Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a 1:1 (horizontal to vertical) projection from the toe of the trench excavation to the ground surface. Where the stability of adjoining buildings, walls, buried utilities within the trench sidewalls, or other structures is endangered by excavation operations, support systems such as shoring, bracing, or underpinning may be required to provide structural stability and to protect personnel working within the excavation.

Groundwater might be encountered within the depths of typical trench excavations and could enter utility trenches excavated for this project. If groundwater is encountered during construction, it is recommended that the contractor install measures to capture and/or divert groundwater from entering the excavation. If this is not possible, then the contractor should channel groundwater to flow towards collection points to be removed from the trench and disposed of at an approved area.

4.5.2 **Materials**

Pipe zone and trench zone nomenclature used within this study are illustrated on Plate 6 – Trench Nomenclature. Pipe zone backfill (i.e., material placed from the trench bottom to a minimum of 6 inches over the pipeline crown) should consist of imported soil having a Sand Equivalent (SE) of no less than 30 and having a particle size no greater than ½-inch in maximum dimension. On site soils will likely not meet this recommendation. Trench zone backfill (i.e., material placed between the pipe zone backfill and finished subgrade) may consist of on-site soil that meets the material requirements previously provided for engineered fill with 100 percent passing the ¾-inch sieve.

Recommendations provided above for pipe zone backfill are minimum requirements only. More stringent material specifications may be required to fulfill local codes and/or bedding requirements for specific types of pipe. We recommend the project Civil Engineer develop these material specifications based on planned pipe types, bedding conditions, and other factors beyond the scope of this investigation.

If poorly graded gravelly and cobbly materials are present around the pipe zone, there is a risk of migration of fines from the pipe zone, hence, creating voids. It is our recommendation to install geofabric around the pipe zone where it comes in contact with such material to avoid migration and piping.

4.5.3 Placement and Compaction

Backfill in temporary excavations should be placed and compacted in accordance with recommendations previously provided for engineered fill. Mechanical compaction is strongly recommended; ponding or jetting should not be allowed. Special care should be given to ensuring that adequate compaction is made beneath the haunches of utility pipes (that area from the pipe springline to the pipe invert) and that no voids remain in this space.

4.5.4 Trench Subgrade Stabilization

Soft and yielding trench subgrade could be encountered along the bottom of trench excavations, especially in colluvial soils. It is recommended that the bottom of trenches be stabilized prior to placement of the pipeline bedding so that, in the judgment of the geotechnical engineer, the trench subgrade is firm and unyielding. The Contractor should have the sole responsibility for design and implementation of trench subgrade stabilization techniques. Some methods that we have observed used to stabilize trench subgrades include the following:

- Use of ¾–inch to 1½-inch floatrock worked into the trench bottom and covered with a geotextile fabric such as Mirafi 500X;
- Placement of a geotextile fabric, such as Mirafi 500X, on the trench bottom and covered with at least one foot of compacted processed miscellaneous base (PMB) conforming to the requirements of Section 200-2.5 of the Greenbook, latest edition;
- Overexcavation of trench subgrade and placement of two-sack sand-cement slurry; and
- In extreme conditions, injection grouting along the trench alignment.

If floatrock is used, typically sand with an SE of 50 or more should be used to fill the voids in the rock prior to placement of pipe bedding materials.

4.6 SHORING CONSIDERATIONS

If shoring systems are utilized in this project, they should be designed to resist earth pressures exerted by the retained soils plus any additional lateral force that will be applied to the shoring due to surface loads placed at or near the excavation. Retaining systems that are free to rotate or translate laterally (for example, cantilevered retaining walls) through a horizontal distance to shoring height ratio of no less than 0.004 are referred to as unrestrained or yielding retaining structures. Such shoring systems can generally move enough to develop active conditions. Retaining systems that are unable to rotate or deflect laterally (for example, restrained basement walls) are referred to as restrained or non-yielding. If such shoring systems cannot move or translate very much, then at-rest conditions develop.

Recommended equivalent fluid weights for active and at-rest conditions are presented in Section 4.3.5.

4.7 PRELIMINARY PAVEMENT DESIGN

4.7.1 *R-Values*

An estimated R-value of 25 was used for this preliminary design. Because the actual subgrade materials that will be present at finish subgrade are unknown at this time, we recommend that confirmatory R-value tests be obtained during construction. If construction R-values are significantly different than the R-value reported above, then we can modify the pavement design at that time to reflect the constructed conditions.

4.7.2 *Subgrade Preparation*

All subgrade soils should be scarified to a minimum depth of 1-foot, moisture conditioned as necessary to near optimum moisture conditions and compacted to a minimum of 95 percent of the maximum dry density as determined by AASHTO (American Association of State Highway and Transportation Officials) Test Method T-180. The subgrade should be smooth and unyielding prior to the placement of aggregate base rock. Density testing and proof rolling of the subgrade using a loaded water truck should be performed with satisfactory results prior to placement of the aggregate base rock. Concrete curbs and landscape planters that border pavement sections should be embedded into the subgrade soils a minimum of 2 inches to reduce the migration of meteoric and irrigation water into the pavement section.

Because of the size of the project site and its previous use, soft and yielding areas may exist. In the event of the presence of such areas during construction, CGI should review these conditions (as well as the contractor's capabilities) and, if requested, provide recommendations for their treatment.

4.7.3 *Aggregate Base*

The aggregate baserock (AB) should be of such quality as to meet or exceed Caltrans specifications for Class 2 AB and should have a minimum R-value of 78. The AB should be spread in thin lifts restricted to 8 inches in loose thickness or less, moisture conditioned as necessary to near optimum moisture content and compacted to a minimum of 95 percent of the maximum dry density as determined by AASHTO T-180. Density testing and/or proof rolling should be performed prior to placement of the asphalt paving.

If poorly graded gravel and/or cobble materials are present beneath the AB, there is a risk of migration of fines from the AB layer, hence, creating voids. It is our recommendation to install geofabric beneath the AB where it comes in contact with such material to avoid migration and piping.

4.7.4 *Asphalt Concrete Paving*

An estimated R-value of 25 was used for this preliminary design. To provide recommendations for structural pavement sections, we evaluated design criteria for TIs ranging from 4.0 through 10.0. Using those criteria, we have prepared AC structural pavement section recommendations. Recommendations for full depth AC, and AC and AB sections are provided in the following table:

MINIMUM RECOMMENDED STRUCTURAL PAVEMENT SECTIONS[1]

Section	Traffic Index	Type B AC Thickness (in)	Class 2 AB Thickness (in)
Full Depth AC	4.0	5.0	- - -
	6.0	8.0	- - -
	8.0	10.0	- - -
	10.0	12.5	- - -
AC and AB	4.0	2.0	6.0
	6.0	3.0	9.5
	8.0	5.0	12
	10.0	6.0	18

1 –Caltrans Standards

Asphalt paving materials and equipment should meet or exceed current Caltrans specifications.

5 ADDITIONAL SERVICES

This report and its associated recommendations were intended to assist the project team during design stages of the project. We recommend that as the project becomes better defined that CGI be given the opportunity to collaborate on the project refinements so that: 1) we can confirm that project design conforms with recommendations made, herein; and 2) recommendations made within this report can be refined, where necessary, based on the design elements of the project.

It should be noted that CGI provides materials testing and special inspection services that can be applied during construction of the project. Those services include:

- Soil and aggregate materials
- Masonry block, mortar, grout, brick, and prisms
- Structural and reinforcing steel
- Concrete, gunite, shotcrete, and reinforced concrete
- Asphalt concrete design and testing
- Materials source development
- Welding: prequalification, field, and shop
- Non-destructive testing
- Fireproofing density, thickness, and moisture content
- Building component testing
- Anchor bolt yield strengths and pullout forces;
- Structural steel and welding inspection
- Post-tensioning, pile driving, drilled piers, caissons

In addition, we provide a host of asphaltic concrete mix design, inspection, and testing services.

We would be pleased to prepare a proposal to provide these services during construction of the project.

6 GEOTECHNICAL OBSERVATION AND TESTING

This report was based, in part, upon review of data obtained from a limited number of observations, site visits, soil excavations, samples, and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soils or geologic conditions can be experienced within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report are only valid if CGI has the opportunity to observe subsurface conditions during grading in order to confirm that our collected data are representative for the site.

Geotechnical observation and testing should be conducted at the following stages:

- Upon completion of clearing and grubbing;
- During and upon completion of overexcavation of deleterious materials;
- During all phases of rough grading, including removals, benching and fill operations;
- During installation of subdrains and filter materials (if necessary);
- During excavation of footings for foundations and retaining walls;
- During trench and retaining wall backfill operations;
- During roadway and parking lot subgrade and aggregate base placement and compaction; and
- When any conditions are encountered during grading that vary from the conditions described in this report.

7 LIMITATIONS

This report has been prepared in substantial accordance with the generally accepted geotechnical engineering practice, as it existed in the site area at the time our services were rendered. No other warranty, either express or implied, is made. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations, as described in Section 6.0, will be conducted by CGI during the construction phase in order to evaluate compliance with our recommendations.

Conclusions and recommendations contained in this report were based on the conditions encountered during our evaluation of geologic hazards at the site and our field investigation and are applicable only to those project features described herein (see Section 1.2 – Project Understanding). Soil and rock deposits can vary in type, strength, and other geotechnical properties between points of observation and exploration. Additionally, groundwater and soil moisture conditions can also vary seasonally and for other reasons. Therefore, we do not and cannot have a complete knowledge of the subsurface conditions underlying the project site. The conclusions and recommendations presented in this report are based upon the findings at the points of exploration, and interpolation and extrapolation of information between and beyond the points of observation, and are subject to confirmation based on the conditions revealed by construction.

The scope of services provided by CGI for this project did not include the investigation and/or evaluation of toxic substances, or soil or groundwater contamination of any type. If such conditions are encountered during site development, additional studies may be required. Further, services provided by CGI for this project did not include the evaluation of the presence of critical environmental habitats or culturally sensitive areas.

This report may be used only by our client and their agents and only for the purposes stated herein, within a reasonable time from its issuance. Land use, site conditions, and other factors may change over time that may require additional studies. In the event significant time elapses between the issuance date of this report and construction, CGI shall be notified of such occurrence in order to review current conditions. Depending on that review, CGI may require that additional studies be conducted and that an updated or revised report is issued.

Any party other than our client who wishes to use all or any portion of this report shall notify CGI of such intended use. Based on the intended use as well as other site-related factors, CGI may require that additional studies be conducted and that an updated or revised report be issued. Failure to comply with any of the requirements outlined above by the client or any other party shall release CGI from any liability arising from the unauthorized use of this report.

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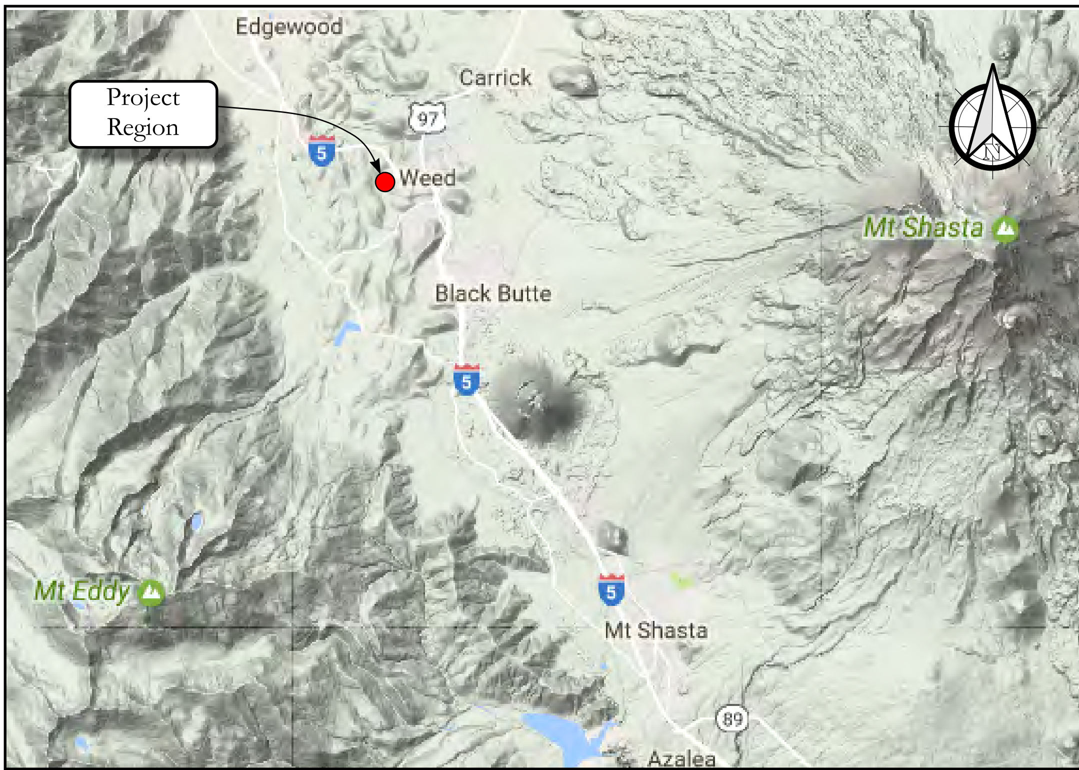
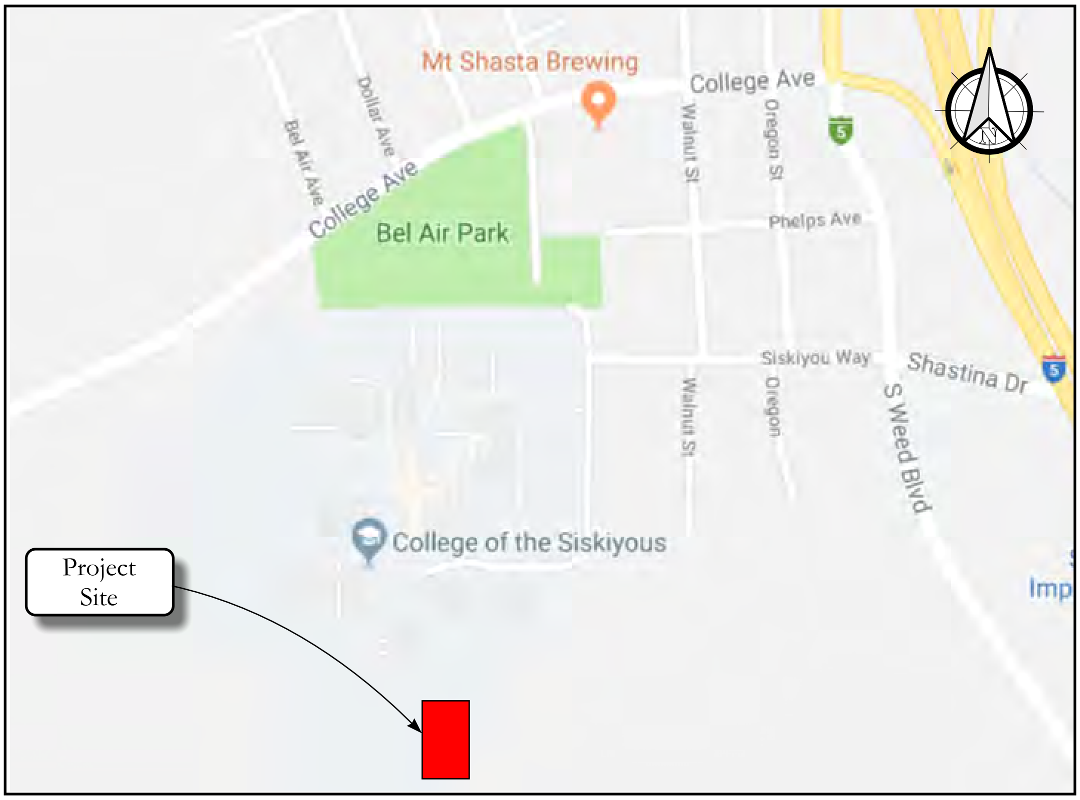




CGi Technical Services, Inc.

Project No.: 17-2333.01

SITE LOCATION MAP
COS NEW SPORT FIELDS
COLLEGE OF THE SISKIYOUS
WEED, CALIFORNIA

Plate

1

Base map from Google Earth (2018)

Scale not Determined

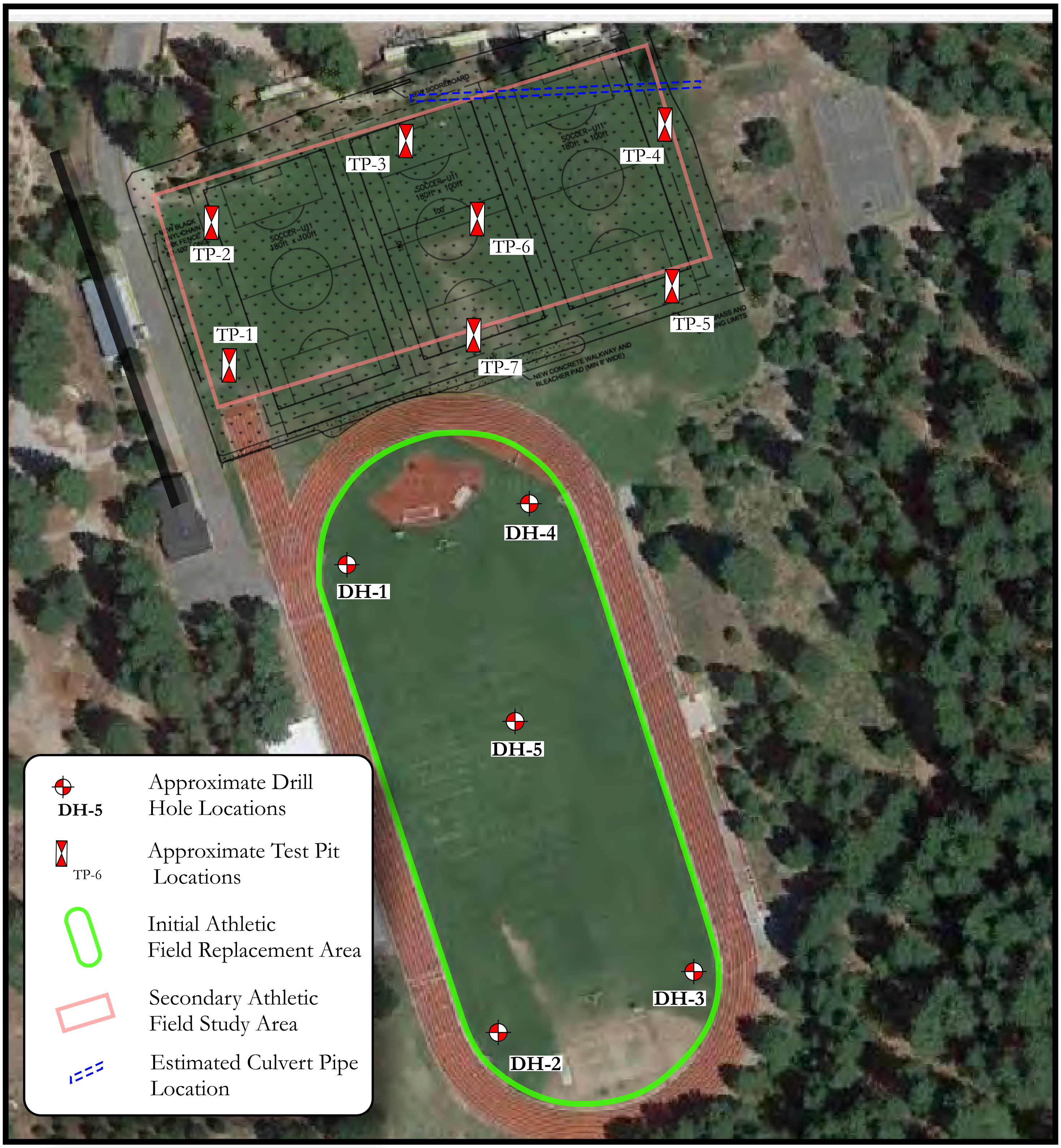
CGi Technical Services, Inc.

Project No:17-2333.01

PROJECT ELEMENTS
COS NEW SPORT FIELDS
COLLEGE OF THE SISKIYOUS
WEED, CALIFORNIA

Plate

2



Base map from Google Earth (2018)

Scale not Determined

CGi Technical Services, Inc.

Project No:17-2333.01

GEOTECHNICAL MAP
COS NEW SPORT FIELD
COLLEGE OF THE SISKIYOUS
WEED, CALIFORNIA

Plate

3

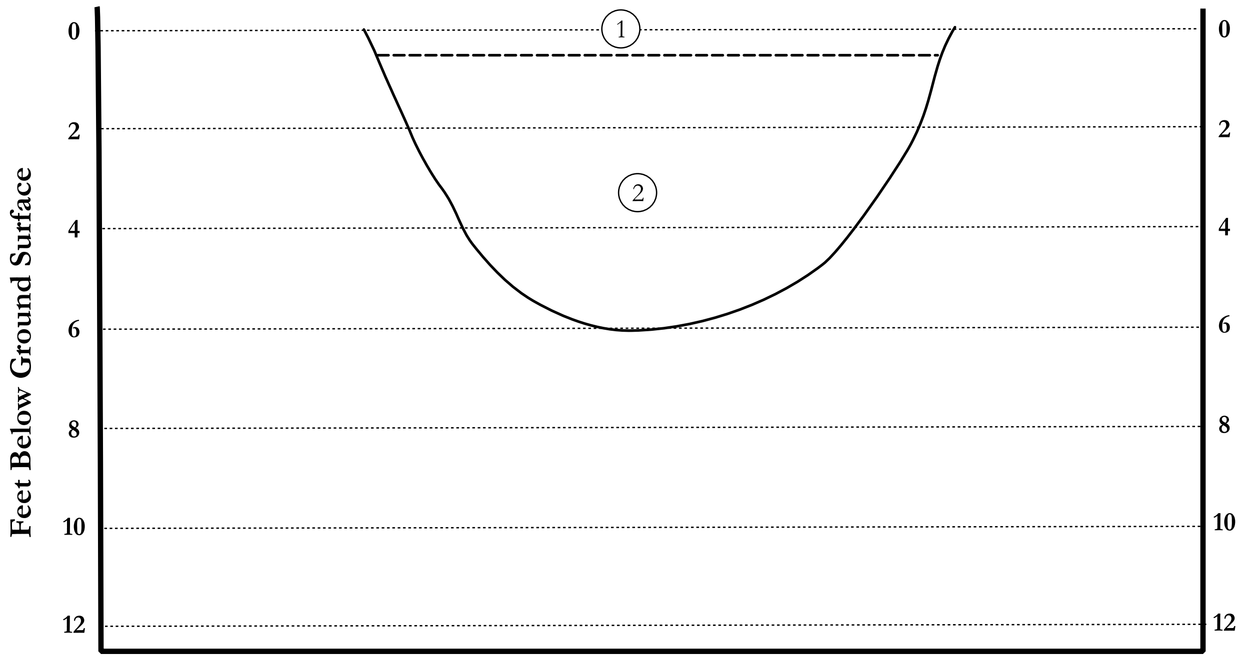
APPENDIX A
SUBSURFACE EXPLORATION

The subsurface exploration program for the proposed project consisted of excavating and logging of seven (7) test pits and five (5) exploratory drill holes. Test pits and drill hole locations are shown on Plate 3.

The test pits were advanced on August 23 and the drill holes on December 7, 2017 using a Mobile B59 truck mounted drill rig using an 8-inch hollow stem auger. The test pits were advanced to depths up to about 8 feet below ground surface. The drill holes were excavated to a depth of approximately 11.5 feet below the existing ground surface. Select soil samples were collected for laboratory classification and testing. The results of the testing procedures are attached within Appendix B.

The exploration drill logs describe the earth materials encountered. The logs also show the location, exploration number, date of exploration, and the names of the logger and equipment used. A CGI geologist or geotechnical engineer, using ASTM 2488 for visual soil classification, logged the explorations. The boundaries between soil types shown on the log are approximate because the transition between different soil layers may be gradual and may change with time. Test pits and drill hole logs for this study are presented as Plates A-1.1 through A-1.7 and A-2.1 through A-2.5 respectively. A legend to the test pit logs and drill holes is presented in the of this appendix.

LOG OF TEST PIT



Soil Descriptions	
1	Top Soil
2	Silty Sand with Gravel (SM), pinkish brown, dry, moderately dense, sand fine to coarse, gravel sub-angular to angular less than ¾ -inch diameter. 1.25-inch diameter plastic pipe at depth of 14-inches

Date Logged:	August 23rd, 2018	Excavated With:	John Deere 310D (24" Bucket)
Logged by:	Joshua Smith	Backfilled With:	Excavated Cuttings
Excavator:	COS Staff	Depth to Water (ft):	Not Encountered

CGi Technical Services, Inc.

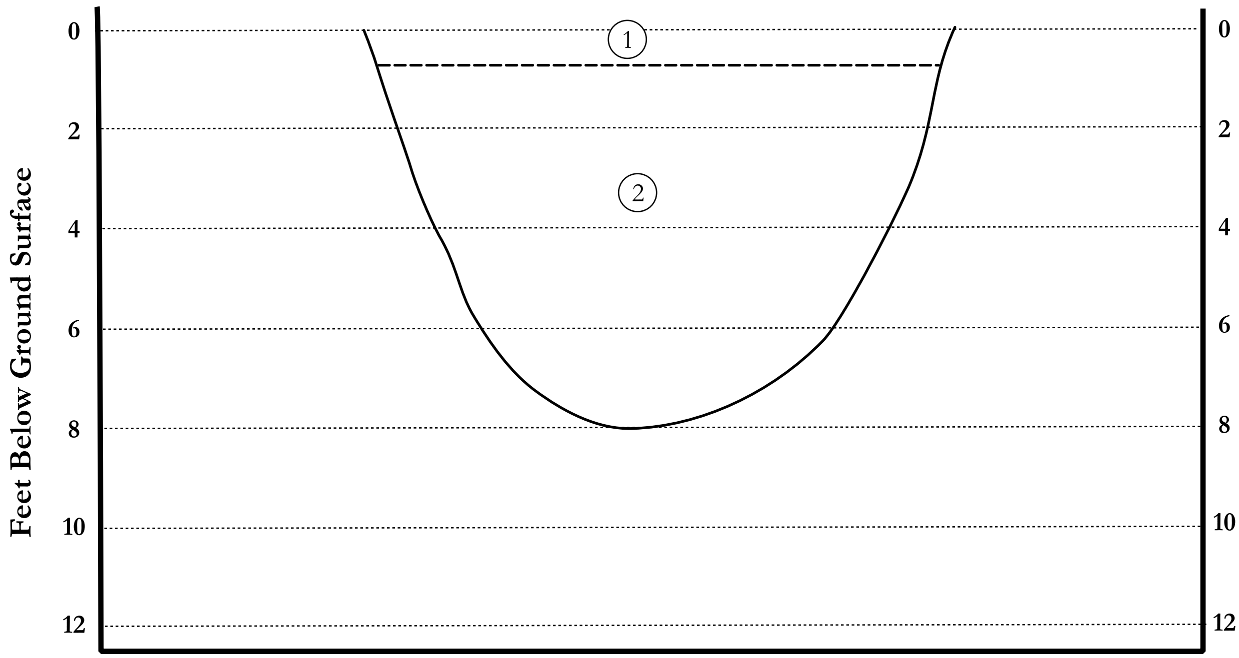
Project No.: 17-2333.02

TEST PIT LOG TP-1
COS ATHLETIC FIELD
COLLEGE OF THE SISKIYOUS
WEED, CALIFORNIA

Plate No.

A-1.1

LOG OF TEST PIT



Soil Descriptions	
1	Top Soil
2	Silty Sand with Gravel (SM), pinkish brown, dry, moderately dense, sand fine to coarse, gravel sub-angular to angular less than ¾ -inch diameter. 1.25-inch diameter plastic pipe at depth of 18-inches

Date Logged:	August 23rd, 2018	Excavated With:	John Deere 310D (24" Bucket)
Logged by:	Joshua Smith	Backfilled With:	Excavated Cuttings
Excavator:	COS Staff	Depth to Water (ft):	Not Encountered

CGi Technical Services, Inc.

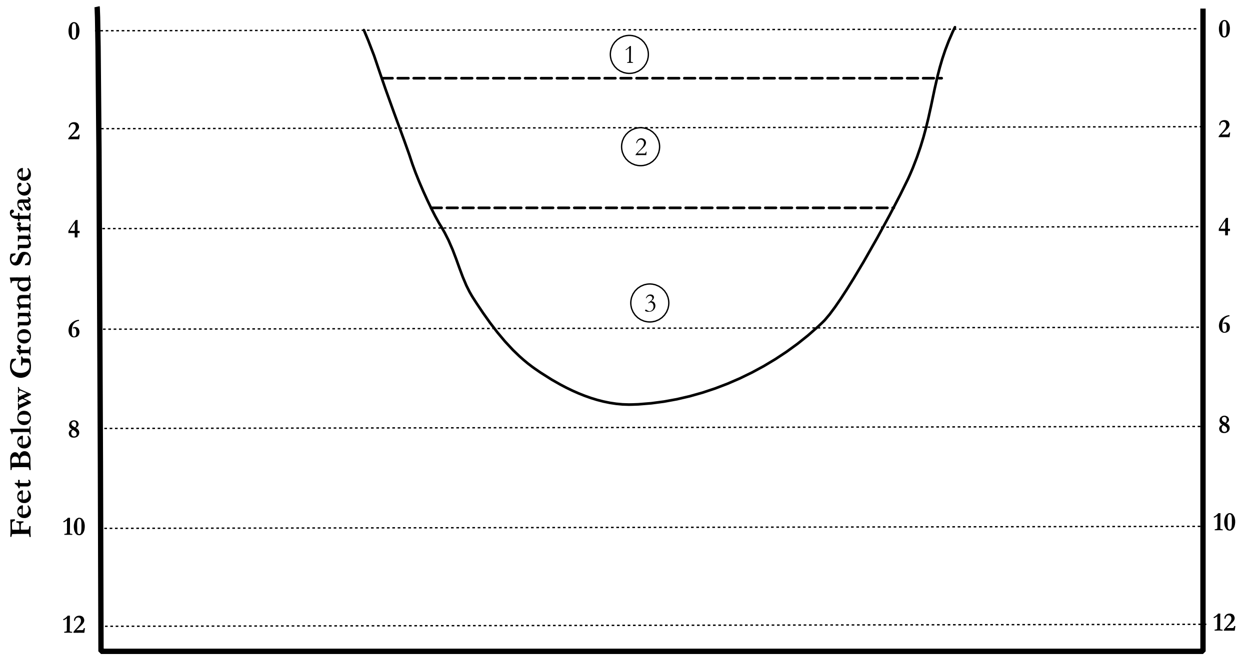
Project No.: 17-2333.02

TEST PIT LOG TP-2
COS ATHLETIC FIELD
COLLEGE OF THE SISKIYOUS
WEED, CALIFORNIA

Plate No.

A-1.2

LOG OF TEST PIT

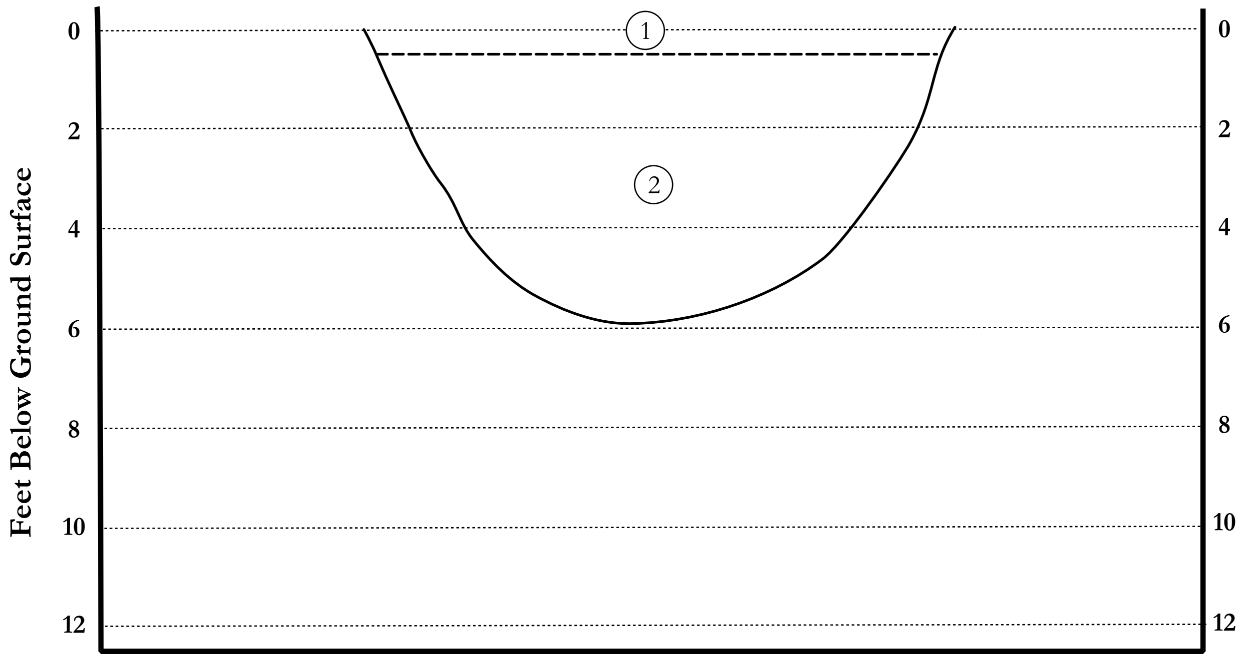


Soil Descriptions	
1	Top Soil
2	Silty Sand with Gravel (SM), brown, dry, moderately dense, sand fine to coarse, gravel sub-angular to angular less than ¾ -inch diameter.
3	Silty Sand with Gravel (SM), pinkish brown, dry, moderately dense, sand fine to coarse, gravel sub-angular to angular less than ¾ -inch diameter.

Date Logged:	August 23rd, 2018	Excavated With:	John Deere 310D (24" Bucket)
Logged by:	Joshua Smith	Backfilled With:	Excavated Cuttings
Excavator:	COS Staff	Depth to Water (ft):	Not Encountered

CGi Technical Services, Inc. Project No.: 17-2333.02	TEST PIT LOG TP-3 COS ATHLETIC FIELD COLLEGE OF THE SISKIYOUS WEED, CALIFORNIA	Plate No. A-1.3

LOG OF TEST PIT

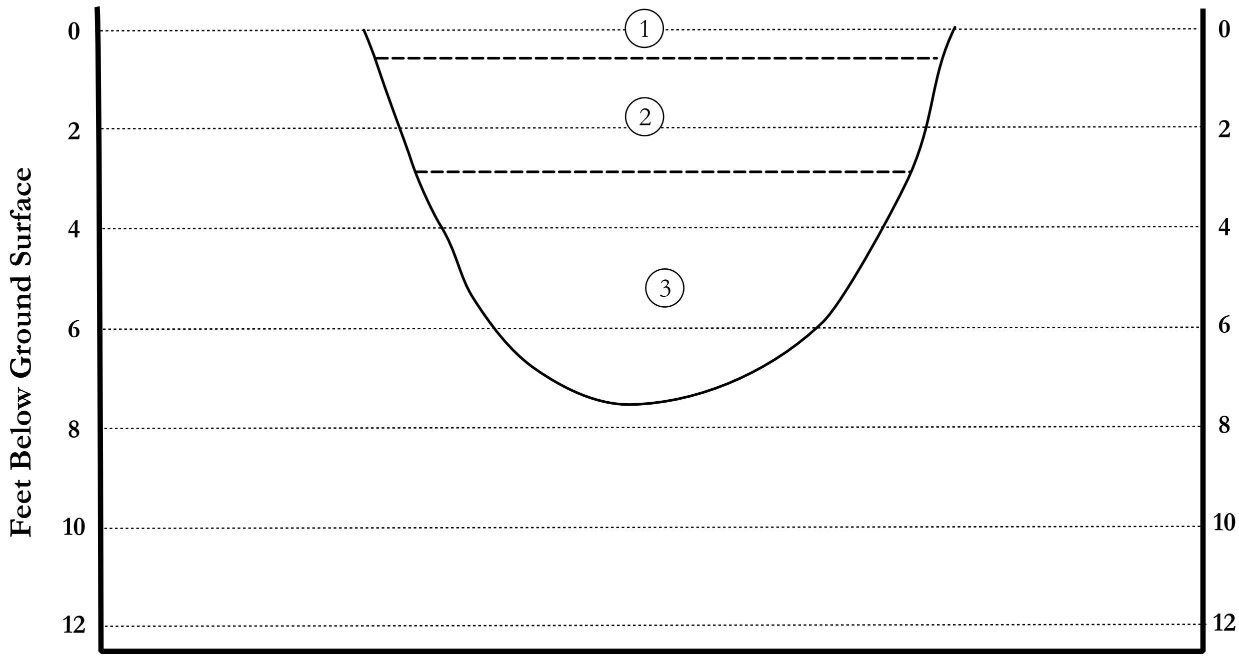


Soil Descriptions	
1	Top Soil
2	Silty Sand with Gravel (SM), pinkish brown, dry, moderately dense, sand fine to coarse, gravel sub-angular to angular less than ¾ -inch diameter. 1.25-inch diameter plastic pipe at depth of 18-inches

Date Logged:	August 23rd, 2018	Excavated With:	John Deere 310D (24" Bucket)
Logged by:	Joshua Smith	Backfilled With:	Excavated Cuttings
Excavator:	COS Staff	Depth to Water (ft):	Not Encountered

CGi Technical Services, Inc. Project No.: 17-2333.02	TEST PIT LOG TP-4 COS ATHLETIC FIELD COLLEGE OF THE SISKIYOUS WEED, CALIFORNIA	Plate No. A-1.4

LOG OF TEST PIT



Soil Descriptions	
1	Top Soil
2	Silty Sand with Gravel (SM), brown, dry, moderately dense, sand fine to coarse, gravel sub-angular to angular less than ¾ -inch diameter. Trace wood debris throughout.
3	Silty Sand with Gravel (SM), pinkish brown, dry, moderately dense, sand fine to coarse, gravel sub-angular to angular less than ¾ -inch diameter. 1.25-inch diameter plastic pipe at depth of 18-inches

Date Logged:	August 23rd, 2018	Excavated With:	John Deere 310D (24" Bucket)
Logged by:	Joshua Smith	Backfilled With:	Excavated Cuttings
Excavator:	COS Staff	Depth to Water (ft):	Not Encountered

CGi Technical Services, Inc.

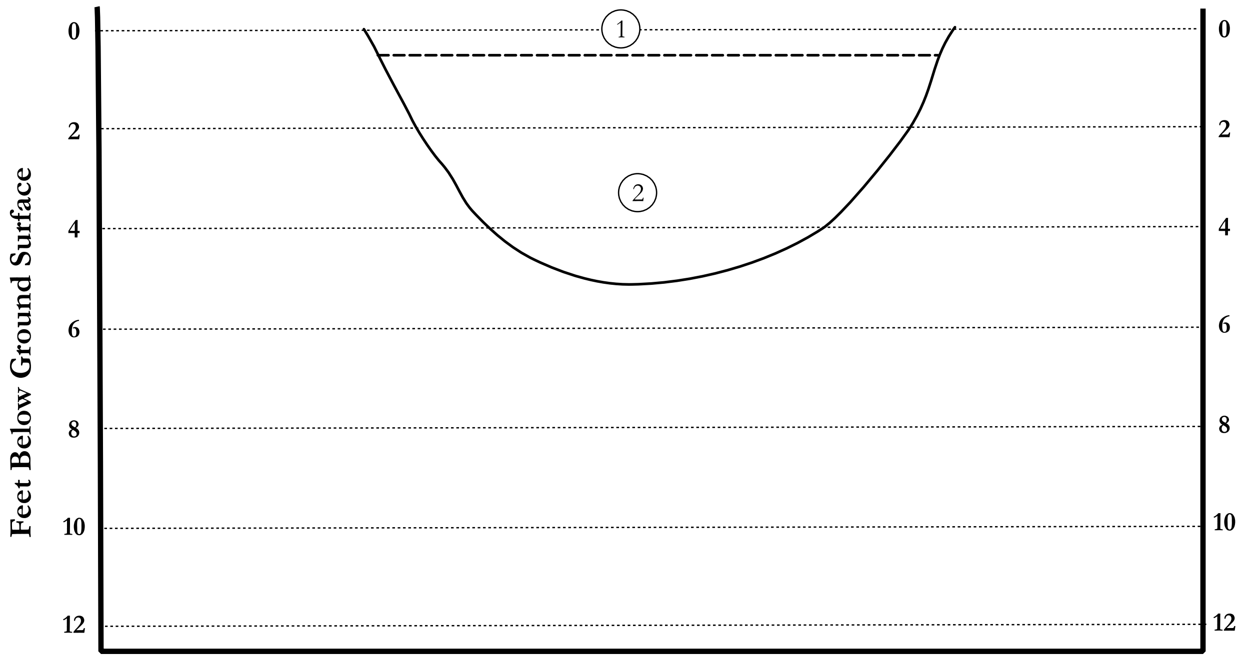
Project No.: 17-2333.02

TEST PIT LOG TP-5
COS ATHLETIC FIELD
COLLEGE OF THE SISKIYOUS
WEED, CALIFORNIA

Plate No.

A-1.5

LOG OF TEST PIT



Soil Descriptions	
1	Top Soil
2	Silty Sand with Gravel (SM), pinkish brown, dry, moderately dense, sand fine to coarse, gravel sub-angular to angular less than ¾ -inch diameter. 1.25-inch diameter plastic pipe at depth of 24-inches

Date Logged:	August 23rd, 2018	Excavated With:	John Deere 310D (24" Bucket)
Logged by:	Joshua Smith	Backfilled With:	Excavated Cuttings
Excavator:	COS Staff	Depth to Water (ft):	Not Encountered

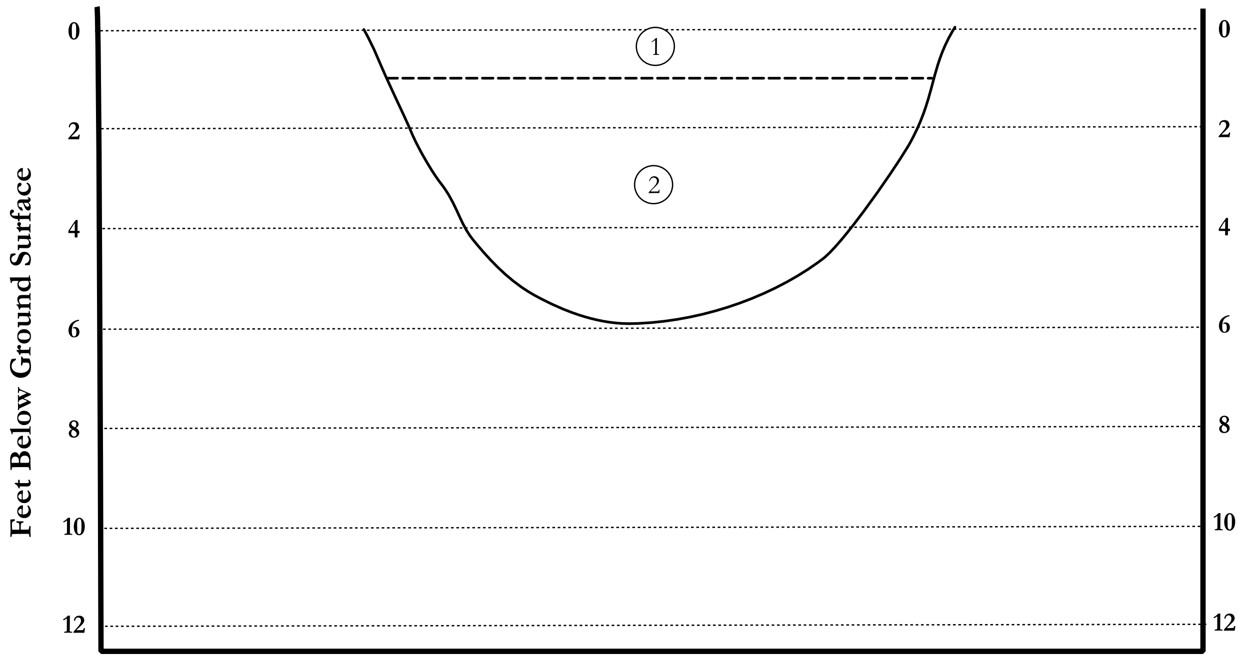
CGi Technical Services, Inc.

Project No.: 17-2333.02

TEST PIT LOG TP-6
COS ATHLETIC FIELD
COLLEGE OF THE SISKIYOUS
WEED, CALIFORNIA

Plate No.
A-1.6

LOG OF TEST PIT



Soil Descriptions	
1	Top Soil
2	Silty Sand with Gravel (SM), pinkish brown, dry, moderately dense, sand fine to coarse, gravel sub-angular to angular less than ¾ -inch diameter. 1.25-inch diameter plastic pipe at depth of 18-inches

Date Logged:	August 23rd, 2018	Excavated With:	John Deere 310D (24” Bucket)
Logged by:	Joshua Smith	Backfilled With:	Excavated Cuttings
Excavator:	COS Staff	Depth to Water (ft):	Not Encountered

CGi Technical Services, Inc.

Project No.: 17-2333.02

TEST PIT LOG TP-7
COS ATHLETIC FIELD
COLLEGE OF THE SISKIYOUS
WEED, CALIFORNIA

Plate No.

A-1.7

Major Divisions			USCS Symbol	Description
COARSE-GRAINED SOILS More than 50% of sample or material is larger than the No. 200 Sieve (0.0029 inches)	GRAVELS More than 50% of the coarse fraction is retained on No. 4 sieve (0.187 inches)	GRAVELS Clean Gravels, few fines	GW	Well graded gravels and sand mixtures with little to no fines
			GP	Poorly graded gravels & gravel/sand mixtures with little to no fines
		GRAVELS With appreciable fines	GM	Silty gravels and poorly graded gravel/sand/silt mixtures
			GC	Clayey gravels and poorly graded gravel/sand/clay mixtures
	SANDS More than 50% of the coarse fraction passes the No. 4 sieve (0.187 inches)	SANDS Clean Sands, few fines	SW	Well graded sands and gravelly sands with little to no fines
			SP	Poorly graded sands and gravelly sands with little to no fines
		SANDS With appreciable fines	SM	Silty sands and poorly graded sand/gravel/silt mixtures
			SC	Clayey sands and poorly graded sand/gravel/clay mixtures
FINE-GRAINED SOILS More than 50% of sample or material is smaller than the No. 200 Sieve (0.0029 inches)	SILTS & CLAYS Liquid limit less than 50		ML	Inorganic silts with very fine sands, silty and/or clayey fine sands, clayey silts with slight plasticity
			CL	Inorganic clays with low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and clays with low plasticity
	SILTS & CLAYS Liquid limit greater than 50		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts
			CH	Inorganic clays with high plasticity, fat clays
			OH	Orgainic silts and clays with high plasticity
HIGHLY ORGANIC SOIL			PT	Peat, humus, swamp soil with high organic content

Samples

Bulk or disturbed sample

Relatively undisturbed sample

Symbols

Groundwater

Caving

Contact Between Soil/Rock Layers

GENERAL NOTES

Dual symbols (such as ML/CL or SM/SC) are used to indicate borderline classifications.
In general, USCS designations shown on the logs were evaluated using visual methods. Actual designations (based on laboratory tests) may vary.
Logs represent general soil conditions observed on the date and locations indicated. No warranty is provided regarding soil continuity between locations.
Lines separating soil strata on logs are approximate. Actual transitions may be gradual and vary with depth.

CGi Technical Services, Inc.

Project No.: ##-####.##

LEGEND TO TEST PIT LOGS
PROJECT NAME
CLIENT
PROJECT LOCATION

Plate No.

A-2.1

LOG OF EXPLORATION: DH-1

PROJECT:	COS Athletic Field	**EXPL. VENDOR:**	Diamond Core	**SURFACE ELEVATION:**	3,581 feet
PROJECT NO.:	17-2333.01	**EXPL. METHOD:**	Hollowstem Auger 8"	**DEPTH OF HOLE:**	11.5 feet
LOCATION:	Weed, CA	**LOGGED BY:**	J. Smith	**DEPTH TO WATER:**	Not Encountered
START DATE:	12/7/17	**CHECKED BY:**	A. Bahloul	**BACKFILLED WITH:**	Bentonite & Cuttings
END DATE:	12/7/17	**HAMMER TYPE:**	140-Lb.		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0					OL	Top Soil							
			1.1		SM	Silty Sand (SM), dark brown, moist moderately dense, sand fine to coarse				13.7			
			1.2	(13)				83..4	15.8				
			1.3		SM	Silty Sand with Gravel (SM), pinkish brown, moist moderately dense, sand fine to coarse, gravel subrounded to angular less than 3/4 inch in diameter							
5			1.4	(32)				105.6	10.8				
10			1.5	(32)									
						Bottom of Drill Hole at 11.5 feet							
15													

CGi Technical Services, Inc.

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

PLATE NO.: A-2.1

LOG OF EXPLORATION: DH-2

PROJECT:	COS Athletic Field	**EXPL. VENDOR:**	Diamond Core	**SURFACE ELEVATION:**	3,581 feet
PROJECT NO.:	17-2333.01	**EXPL. METHOD:**	Hollowstem Auger 8"	**DEPTH OF HOLE:**	11.5 feet
LOCATION:	Weed, CA	**LOGGED BY:**	J. Smith	**DEPTH TO WATER:**	Not Encountered
START DATE:	12/7/17	**CHECKED BY:**	A. Bahloul	**BACKFILLED WITH:**	Bentonite & Cuttings
END DATE:	12/7/17	**HAMMER TYPE:**	140-Lb.		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0					OL	Top Soil							
			2.1		SM	Silty Sand with trace Cobbles(SM), dark brown, moist moderately dense, sand fine to coarse, cobbles subangular less than 6-inches in diameter							
			2.2	(11)									
					SM	Silty Sand with Gravel (SM), pinkish brown, moist moderately dense, sand fine to coarse, gravel subrounded to angular less than 3/4 inch in diameter							
5			2.3	(15)									Perm.
10			2.4	(28)									
						Bottom of Drill Hole at 11.5 feet							
15													

LOG OF EXPLORATION: DH-3

PROJECT: COS Athletic Field
PROJECT NO.: 17-2333.01
LOCATION: Weed, CA
START DATE: 12/7/17
END DATE: 12/7/17

EXPL. VENDOR: Diamond Core
EXPL. METHOD: Hollowstem Auger 8"
LOGGED BY: J. Smith
CHECKED BY: A. Bahloul
HAMMER TYPE: 140-Lb.

SURFACE ELEVATION: 3,581 feet
DEPTH OF HOLE: 11.5 feet
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Bentonite & Cuttings

-

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0					OL	Top Soil							
					SM	Silty Sand (SM), dark brown, moist moderately dense, sand fine to coarse,							
					SM	Silty Sand with Gravel and trace Cobbles (SM), pinkish brown, moist moderately dense, sand fine to coarse, gravel subrounded to angular less than 3/4 inch in diameter, cobbles subangular less than 3-inches in diameter							
5			3.1	(39)				94.1	11.6				
			3.2							21.1			
10			3.3	(35)									
						Bottom of Drill Hole at 11.5 feet							
15													

CGi Technical Services, Inc.

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

PLATE NO.: A-2.3

LOG OF EXPLORATION: DH-4

PROJECT: COS Athletic Field
PROJECT NO.: 17-2333.01
LOCATION: Weed, CA
START DATE: 12/7/17
END DATE: 12/7/17

EXPL. VENDOR: Diamond Core
EXPL. METHOD: Hollowstem Auger 8"
LOGGED BY: J. Smith
CHECKED BY: A. Bahloul
HAMMER TYPE: 140-Lb.

SURFACE ELEVATION: 3,581 feet
DEPTH OF HOLE: 11.5 feet
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Bentonite & Cuttings

-

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0					OL	Top Soil							
					SM	Silty Sand (SM), dark brown, moist moderately dense, sand fine to coarse							
			4.1		SM	Silty Sand with Gravel (SM), pinkish brown, moist moderately dense, sand fine to coarse, gravel subrounded to angular less than 3/4 inch in diameter							
5			4.2	(13)									
10			4.3	(35)				99.8	12.0				
						Bottom of Drill Hole at 11.5 feet							
15													

LOG OF EXPLORATION: DH-5

PROJECT: COS Athletic Field
PROJECT NO.: 17-2333.01
LOCATION: Weed, CA
START DATE: 12/7/17
END DATE: 12/7/17

EXPL. VENDOR: Diamond Core
EXPL. METHOD: Hollowstem Auger 8"
LOGGED BY: J. Smith
CHECKED BY: A. Bahloul
HAMMER TYPE: 140-Lb.

SURFACE ELEVATION: 3,582 feet
DEPTH OF HOLE: 11.5 feet
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Bentonite & Cuttings

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0					OL	Top Soil							
			5.1			Silty Sand (SM), dark brown, moist moderately dense, sand fine to coarse, at 2-3.5 feet trace wood debris/sawdust and black organic material				13.7			
			5.2	(15)	SM								Perm.
					SM	Silty Sand with Gravel (SM), pinkish brown, moist moderately dense, sand fine to coarse, gravel subrounded to angular less than 3/4 inch in diameter							
5			5.3	(9)									
10			5.4	(26)		Bottom of Drill Hole at 11.5 feet							
15													

CGi Technical Services, Inc.

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

PLATE NO.: A-2.5

LOG OF EXPLORATION: Expl. No.

PROJECT:	CGI's Project Name	**EXPL. VENDOR:**	Expl. Subcontractor	**SURFACE ELEVATION:**	Expl. Elevation
PROJECT NO.:	CGI's Project No.	**EXPL. METHOD:**	Method of Expl.	**TOTAL DEPTH OF HOLE:**	Total Depth of Expl.
LOCATION:	General Location	**LOGGED BY:**	CGI's Logger	**DEPTH TO WATER:**	Depth to Water
START DATE:	Date Started	**CHECKED BY:**	CGI's Reviewer	**BACKFILLED WITH:**	Backfill Materials
END DATE:	Date Finished	**HAMMER TYPE:**	Type of Sample Hammer		

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0			1			SAMPLES/BLOW COUNT SYMBOLS KEY Bulk Soils Sample							
			2	(24)		California modified split spoon sampler (CMSS) Brackets on blow counts indicates CMSS sample							CMSS: 2-3/8" ID, 3" OD, Driven
5			3	50:5"		Standard penetration test (SPT) sample and blow count							SPT: 1-3/8" ID, 2" OD, Driven
						No sample recovery							Blow counts are recorded as the number of blows required for one foot of sampler penetration using a 140-lb hammer falling 30 inches. Typically, sampler is driven 18" and the initial 6" discarded.
						LITHOLOGIC GRAPHICS DESCRIPTIONS FOR SOILS MATERIALS (per ASTM D2487 & D2488)							
10					GW	well graded GRAVEL							
					GP	poorly graded GRAVEL							
					GM	silty GRAVEL							
					GC	clayey GRAVEL							
15					SW	well graded SAND							Initial water level measurement
					SP	poorly graded SAND							Water level after initial measurement (may not represent stabilized water levels)
					SM	silty SAND							
					SC	clayey SAND							
20					ML	low plasticity SILT							
					MH	high plasticity SILT							
					CL	lean CLAY							Lab Abbreviations DS-direct shear; C-consolidation; GS-sieve; EI-Expansion Index; PI-Plasticity; UC-Unconfined; SC-soil chem.; SE-sand equiv.; R-R value; P-curve; PP-pocket penetrometer.
25					CH	fat CLAY							
					PT	organic soils or peat							
					OL	organic SILTS or CLAYS with low plasticity							
					OH	organic SILTS or CLAYS with high plasticity							
30					RX	ROCK							

CGI Technical Services Inc.

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

PLATE NO.: A-1.1

APPENDIX B
LABORATORY TESTING

Laboratory Analyses

Laboratory tests were performed on selected bulk soil samples to estimate engineering characteristics of the various earth materials encountered. Testing was performed under procedures described in one of the following references:

- ASTM Standards for Soil Testing, latest revision;
- Lambe, T. William, Soil Testing for Engineers, Wiley, New York, 1951;
- Laboratory Soils Testing, U.S. Army, Office of the Chief of Engineers, Engineering Manual No. 1110-2-1906, November 30, 1970.

Grain Size Distribution

Grain size distribution was determined for two select soil samples in accordance with standard test method ASTM D422. The grain size distribution data are shown on the attached plates labeled Laboratory Sieve Analysis.

Soil-Chemistry

One test was performed on selected soil samples to evaluate pH, resistivity, chloride and sulfate contents, along with other cations and anions. The results of the tests are presented on the attached *Soil Chemistry* sheets.

Permeability

Two permeability tests were performed on selected samples using standard test method ASTM D5084. The results of the tests are presented on attached plate labeled *Hydraulic Conductivity*.

In Situ Moisture Density Relations

Dry density estimates and/or moisture content evaluations were performed on selected soil samples collected during this study. Tests were performed using standard test methods ASTM D2216 for moisture content or ASTM D2937 for dry unit weights. The results are presented on the Log of Drill Hole.

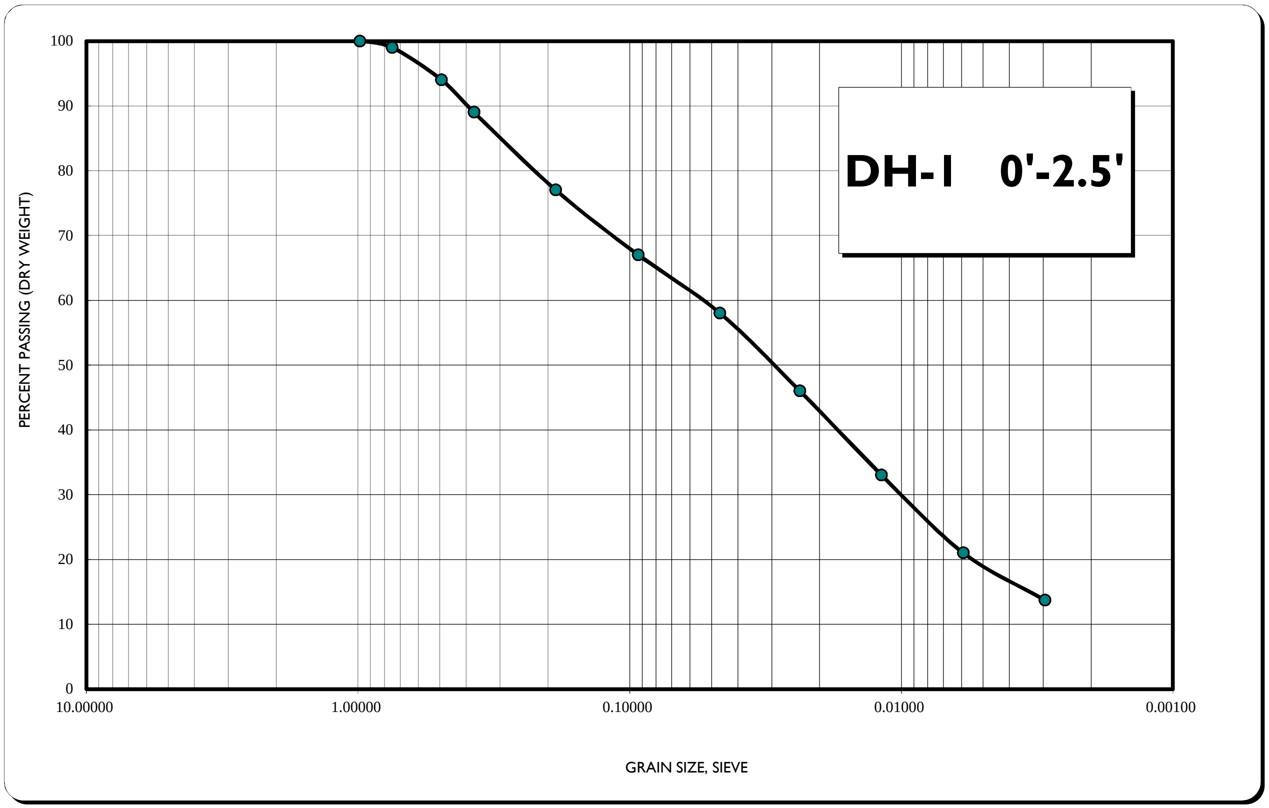
CGi Technical Services, Inc.

LABORATORY TEST RESULTS

Client: COS
Project: Turf Field
Material Type: Native
Test Procedures: AASHTO T-11,T-27

Material Supplier: N/A
Sampled By: JDS
Date Sampled: 12/7/17
Tested By: T.Kinsey

Job No.: 17-2333.01
Lab No.: 9837
Date Received: 12/7/17
Date Tested: 12/19/17
Date Reviewed: 12/20/17



Sieve Analysis			
Sieve Size Standard	Grain Size (mm)	Percent Passing	Operating Range*
5"	127.00		
4"	101.60		
2"	50.00		
1"	25.00	100	
3/4"	19.00	99	
1/2"	12.50	94	
3/8"	9.50	89	
#4	4.75	77	
#8	2.36	67	
#16	1.18	58	
#30	600um	46	
#50	300um	33	
#100	150um	21	
#200	75um	13.7	

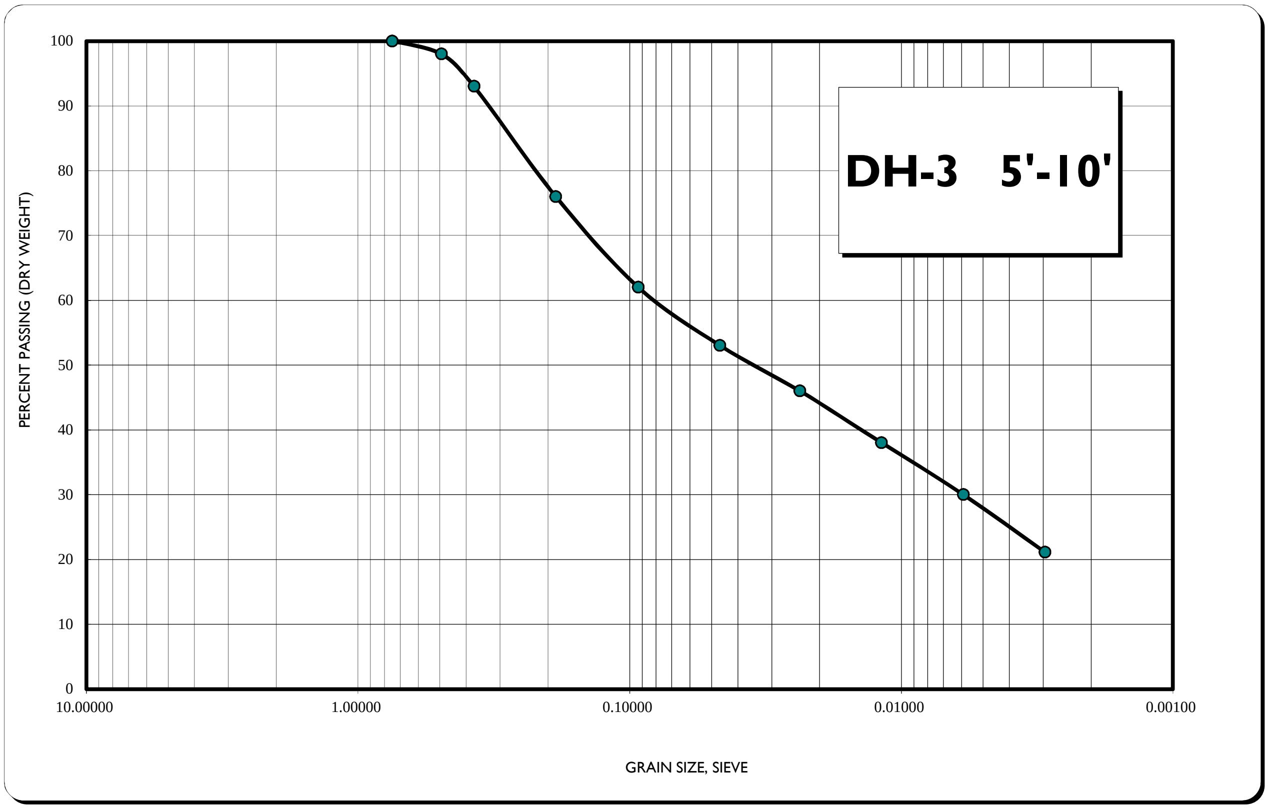
CGi Technical Services, Inc.

LABORATORY TEST RESULTS

Client: COS
Project: Turf Field
Material Type: Native
Test Procedures: AASHTO T-11,T-27

Material Supplier: N/A
Sampled By: JDS
Date Sampled: 12/7/17
Tested By: T.Kinsey

Job No.: 17-2333.01
Lab No.: 9837
Date Received: 12/7/17
Date Tested: 12/19/17
Date Reviewed: 12/20/17



Sieve Analysis			
Sieve Size Standard	Grain Size (mm)	Percent Passing	Operating Range*
5"	127.00		
4"	101.60		
2"	50.00		
1"	25.00		
3/4"	19.00	100	
1/2"	12.50	98	
3/8"	9.50	93	
#4	4.75	76	
#8	2.36	62	
#16	1.18	53	
#30	600um	46	
#50	300um	38	
#100	150um	30	
#200	75um	21.1	

Table 1 - Laboratory Tests on Soil Samples

CGI Technical Services
COS Turf Field
Your #17-2333.01, HDR Lab #17-0888LAB
26-Dec-17

Sample ID

			DH-4 @ 0-5'
Resistivity		**Units**	
as-received		ohm-cm	36,800
saturated		ohm-cm	20,400
pH			6.4
Electrical Conductivity		mS/cm	0.06
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	24
magnesium	Mg^{2+}	mg/kg	15
sodium	Na^{1+}	mg/kg	16
potassium	K^{1+}	mg/kg	13
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	61
fluoride	F^{1-}	mg/kg	ND
chloride	Cl^{1-}	mg/kg	3.4
sulfate	SO_4^{2-}	mg/kg	12
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
ammonium	NH_4^{1+}	mg/kg	19
nitrate	NO_3^{1-}	mg/kg	17
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.
Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

COOPER TESTING LABORATORY

Hydraulic Conductivity
ASTM D 5084
Method C: Falling Head Rising Tailwater

Job No:	591-103	**Boring:**	DH-5	**Date:**	01/04/18
Client:	CGI Technical Services	**Sample:**	5.2	**By:**	MD/PJ
Project:	17-2333.01	**Depth, ft.:**	2.0	**Remolded:**	

Visual Classification: Brown Clayey SAND w/ Gravel/ Sandy CLAY w/ Gravel

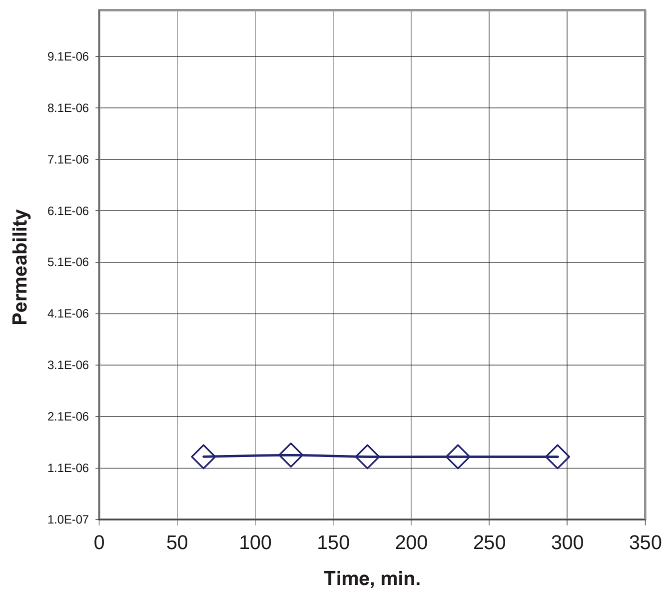
Max Sample Pressures, psi:

Cell:	Bottom	Top	Avg. Sigma3
54	49.5	48.5	5

B: = >0.95 ("B" is an indication of saturation)

Max Hydraulic Gradient: = 14

Date	Minutes	Head, (in)	K,cm/sec
1/3/2018	0.00	42.69	Start of Test
1/3/2018	67.00	40.29	1.3E-06
1/3/2018	123.00	38.29	1.3E-06
1/3/2018	172.00	36.79	1.3E-06
1/3/2018	230.00	34.99	1.3E-06
1/3/2018	294.00	33.09	1.3E-06



Average Hydraulic Conductivity: 1.E-06 cm/sec

Sample Data:	Initial (As-Received)		Final (At-Test)
Height, in	3.00		2.98
Diameter, in	2.36		2.36
Area, in2	4.37		4.37
Volume in3	13.10		13.00
Total Volume, cc	214.6		213.0
Volume Solids, cc	119.4		119.4
Volume Voids, cc	95.2		93.6
Void Ratio	0.8		0.8
Total Porosity, %	44.4		44.0
Air-Filled Porosity (θa),%	7.0		2.1
Water-Filled Porosity (θw),%	37.3		41.9
Saturation, %	84.2		95.3
Specific Gravity	2.70	Assumed	2.70
Wet Weight, gm	402.5		411.6
Dry Weight, gm	322.3		322.3
Tare, gm	0.00		0.00
Moisture, %	24.9		27.7
Wet Bulk Density, pcf	117.0		120.6
Dry Bulk Density, pcf	93.7		94.4
Wet Bulk Dens.ρb, (g/cm^3)	1.87		1.93
Dry Bulk Dens.ρb, (g/cm^3)	1.50		1.51

Remarks:

COOPER TESTING LABORATORY

Constant Head Permeability Test
ASTM D2434

CTL Job No:	591-103	**Boring:**	DH-2	**Date:**	1/8/2018
Client:	CGI Technical Services	**Sample:**	2.3	**By:**	PJ
Project Name:	COS Turf Field	**Depth, ft:**	5		
Project No.:	17-2333.01				

Soil Description: Dusky Red GRAVEL w/ Silt & Sand

Remolding Data: Undisturbed

		Constant Head Calculation, K=QL/thA				
Test #	**Elapsed Time t, (sec)**	**Volume Q, (cc)**	**Head Loss h (cm)**	**Water Temp (^{o}C)**	**Hydraulic Gradient**	**Coef. Of Permeability K, (cm/sec)**
1	3094	20	0.3	20.3	0.05	0.0045
2	4067	25	0.3	20.3	0.05	0.0044
3	4010	24	0.3	20.3	0.05	0.0043
4	3780	23	0.3	20.3	0.05	0.0044
		Average Permeability (cm/sec):				**0.0044**
		Average Permeability (in/hr):				**6.2**

Sample Data:		**Initial**		**Final**	
Height, (L)	**in.:**	5.30		5.30	
Diameter,	**in.:**	2.37		2.37	
Area, (A)	**in^2:**	4.41		4.41	
Volume,	**in^3:**	23.38		23.38	
Total Volume.	**cc:**	383		383	
Vol. of Solids,	**cc:**	238		238	
Vol. of Voids,	**cc:**	145		145	
Void Ratio	**e:**	0.61		0.61	
Porosity,	**%:**	37.9		37.9	
Saturation,	**%:**	72.1		97.3	
Sp. Gravity:		2.65	assumed	2.65	assumed
Wet Weight,	**gm:**	735.4		772.0	
Dry Weight	**gm:**	630.8		630.8	
Moisture,	**%:**	16.6		22.4	
Density,	**pcf:**	102.8		102.8	

Remarks: