APPENDIX E: GEOTECHNICAL REPORT

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Geotechnical Engineering Investigation
Covina Bowl 1060 West San Bernardino Road
City of Covina, California

for

Watt Communities, LLC

May 19, 2017 W.O. 7043

MDN 19198

GeoSoils Consultants Inc.

GeoSoils Consultants Inc.
GEOTECHNICAL GEOLOGIC ENVIRONMENTAL

May 19, 2017
W.O. 7043

WATT COMMUNITIES, LLC
2716 Ocean Park Boulevard, Suite 2025
Santa Monica, California 90405

Attention: Mr. Efrem Joelson

Subject: Geotechnical Engineering Investigation, Covina Bowl, 1060 West San Bernardino Road, City of Covina, California

Dear Mr. Joelson:

At your request, GeoSoils Consultants, Inc. (GSC) has prepared this geotechnical engineering report for the subject property. The purpose of this investigation was to evaluate the geotechnical engineering characteristics of the underlying earth materials in order to evaluate their suitability to receive the planned improvements.

The site was explored by excavating four borings drilled with an 8-inch diameter hollow-stem drill rig. The field exploration procedures and boring logs are attached in appendix A. Laboratory test procedures and results are enclosed in Appendix B. Grading guidelines are presented in Appendix C. The boring locations are shown on the Boring Location Map, Plate 1. This report has been prepared in accordance with generally accepted geotechnical engineering practices.

SITE LOCATION AND DESCRIPTION

The subject site is located at 1060 West San Bernardino Road in Covina, California. Rectangular in shape, the site occupies approximately 4.36 acres of flat terrain. The

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property is bounded by streets on three sides: West San Bernardino Road to the north, Rimsdale Avenue to the east and Badillo Street to the south. Commercial and residential developments exist along the west side of the property. Numerous trees and the Covina Bowling Alley currently occupy the site.

Geologic Setting

The subject property is located within the Transverse Ranges Geomorphic province of California. The Transverse Ranges consist of generally east-west trending mountains and valleys, which are in contrast to the north-northwest regional trend elsewhere in the state. The structure of the Transverse Ranges is controlled by the effects of north-south compressive deformation (crustal shortening), which is attributed to convergence between the big bend of the San Andreas fault north of the San Gabriel Mountains and the motion of the Pacific Plate. The valleys and mountains of the Transverse Ranges are typically bounded by a series of east west trending, generally north dipping reverse faults with left-lateral oblique movement.

The Transverse Ranges are characterized by a very thick, nearly continuous sequence of Upper Cretaceous through Quaternary sedimentary rocks that has been deformed into a series of east-west trending folds associated with thrust and reverse faults. This deformation has created intrabasin highlands and intervening lowlands.

Earth Units

Alluvium underlies the property. A brief description of the earth materials are as follows:

Alluvium (Qal): The alluvium on site consists of interbedded silty very fine to fine sand, fine to medium sand and fine to very coarse sand with gravel/rock. All loose/soft soils will require removal and recompaction. Grading guidelines are presented in Appendix C.

Surface and Subsurface Water Conditions

Surface water on the site is limited to precipitation falling directly on the site and irrigation. Springs or seeps were not observed on the site.

No groundwater was encountered in any of the borings drilled on site. The groundwater maps from the Seismic Hazard Zone Report for the Baldwin Park 7.5 Minute Quadrangle published by the Department of Conservation Division of Mines and Geology indicate that the historic high groundwater more than 150 feet below existing ground surface.

FAULTING AND SEISMICITY

The proposed site is not within an Alquist-Priolo Earthquake Fault Zone; therefore, there are no known active faults on the property. This site has experienced earthquake-induced ground shaking in the past and can be expected to experience further shaking in the future. There are some faults in close enough proximity to the site to cause moderate to intense ground shaking during the lifetime of the proposed development.

Secondary Earthquake Effects

Secondary earthquake effects include ground rupture, landsliding, seiches and tsunamis, and liquefaction.

Ground Rupture

Ground rupture occurs when movement on a fault breaks through to the surface. Surface rupture usually occurs along pre-existing fault traces where zones of weakness already exist. The State has established Earthquake Fault Zones for the purpose of mitigating the hazard of fault rupture by prohibiting the location of most human occupancy structures across the traces of active faults. Earthquake fault zones are regulatory zones that encompass surface traces of active faults with a potential for future surface fault rupture. Since the site is not located within a State established Earthquake Fault Zone, the ground rupture hazard for the site is considered to be low.

Landsliding

Earthquake-induced landsliding often occurs in areas where previous landslides have moved and in areas where the topographic, geologic, geotechnical and subsurface groundwater conditions are conducive to permanent ground displacements. No significant slopes are present on or near the site; thus, the site is not located in an area defined by the State for earthquake-induced landslides. The potential for earthquake-induced landsliding is considered low.

Seiches and Tsunamis

A seiche is the resonant oscillation of a body of water, typically a lake or swimming pool caused by earthquake shaking (waves). The hazard exists where water can be splashed out of the body of water and impact nearby structures. No bodies of constant water are near the site, therefore, the hazards associated with seiches are considered low.

Tsunamis are seismic sea waves generated by undersea earthquakes or landslides. When the ocean floor is offset or tilted during an earthquake, a set of waves are generated similar to the concentric waves caused by an object dropped in water.

Tsunamis can have wavelengths of up to 120 miles and travel as fast as 500 miles per hour across hundreds of miles of deep ocean. Upon reaching shallow coastal waters, the once two-foot high wave can become up to 50 feet in height causing great devastation to structures within reach. Tsunamis can generate seiches as well. Since the site is not located near the shoreline or within 50 feet of sea level, the tsunami hazard is considered low.

Liquefaction

Liquefaction describes a phenomenon where cyclic stresses, which are produced by earthquake-induced ground motions, creates excess pore pressures in cohesionless soils. As a result, the soils may acquire a high degree of mobility, which can lead to lateral spreading, consolidation and settlement of loose sediments, ground oscillation, flow failure, loss of bearing strength, ground fissuring, and sand boils, and other damaging deformations. This phenomenon occurs only below the water table, but after liquefaction has developed, it can propagate upward into overlying, non-saturated soil as excess pore water escapes. Descriptions of each of the phenomena associated with liquefaction are described below:

Lateral Spreading: Lateral spreading is the lateral movement of stiff, surficial blocks of sediments as a result of a subsurface layer liquefying. The lateral movements can cause ground fissures or extensional, open cracks at the surface as the blocks move toward a slope face, such as a stream bank or in the direction of a gentle slope. When the shaking stops, these isolated blocks of sediments come to rest in a place different from their original location and may be tilted.

Ground Oscillation: Ground oscillation occurs when liquefaction occurs at depth but the slopes are too gentle to permit lateral displacement. In this case, individual blocks may separate and oscillate on a liquefied layer. Sand boils and fissures are often associated with this phenomenon.

Flow Failure: A more catastrophic mode of ground failure than either lateral spreading or ground oscillation, involves large masses of liquefied sediment or blocks of intact material riding on a liquefied layer moving at high speeds over large distances. Generally flow failures are associated with ground slopes steeper than those associated with either lateral spreading or ground oscillation.

Bearing Strength Loss: Bearing strength decreases with a decrease in effective stress. Loss of bearing strength occurs when the effective stresses are reduced due to the cyclic loading caused by an earthquake. Even if the soil does not liquefy, the bearing of the soil may be reduced below its value either prior to or after the earthquake. If the bearing strength is sufficiently reduced, structures supported on the sediments can settle, tilt, or even float upward in the case of lightly loaded structures such as gas pipelines.

Ground Fissuring and Sand Boils: Ground fissuring and sand boils are surface manifestations associated with liquefaction and lateral spreading, ground oscillation, and flow failure. As apparent from the above descriptions, the likelihood of ground fissures developing is high when lateral spreading, ground oscillations, and flow failure occur. Sand boils occur when the high pore water pressures are relieved by drainage to the surface along weak spots that may have been created by fissuring. As the water flows to the surface, it can carry sediments, and if the pore water pressures are high enough create a gusher (sand boils) at the point of exit.

- Sediments must be relatively young in age and must not have developed large amounts of cementation;
- Sediments must consist mainly of cohesionless sands and silts;
- The sediment must not have a high relative density;
- Free groundwater must exist in the sediment; and
- The site must be exposed to seismic events of a magnitude large enough to induce straining of soil particles.

At the time of exploration, groundwater was not encountered in the borings. In addition, according to the Division of Mines and Geology Seismic Hazard Evaluation of the Baldwin Park 7.5 minute Quadrangle, Seismic Hazard Zone Report, the historical high groundwater table is more than 150 feet below grade. Therefore, liquefaction is not considered a hazard to the site.

Total and Differential Settlement

Based upon the consolidation test results and high blow counts, static settlement is expected to be less than 1.0-inch, while differential settlement is expected to be less than 0.5-inch.

CONCLUSIONS

The proposed development is feasible from a geotechnical engineering viewpoint, provided that the following recommendations are incorporated into the final design and construction phase of the proposed development.

Preliminary Infiltration testing was performed in a ten foot deep boring (B-4) with perforated screen from 5 to 10 feet. The results are provided in Appendix D. Please note these results are considered preliminary only. Once the final location and depth of an infiltration system have been determined, additional infiltration testing may be required.

RECOMMENDATIONS

Site Grading

Standard grading recommendations are enclosed in Appendix C. These recommendations should be incorporated into the development plans, where applicable.

Removals

The subsurface exploration revealed that on the northern portion of the site the upper 5 to 6 feet of material on the site consists of loose alluvium. On the southern portion of the site the loose alluvium extended to depths of 10 to 12 feet (see Plate 1). All loose alluvium should be overexcavated and replaced as compacted fill. Locally, removals may be deeper depending on field conditions exposed during grading. The alluvium is suitable for replacement as engineered fill, provided that the materials do not contain debris or large rocks. All building pads must be underlain by a minimum of 5 feet of compacted fill.

Removals should extend a minimum of 5 feet beyond the proposed building footprint or a distance equal to the depth of fill placement, which is greater.

Seismic Design Criteria

Based upon the 2016 CBC (California Building Code), the following table provides design parameters for the subject site.

2016 CBC Section 1613, Earthquake Loads	
Site Class Definition (Table 1613.5.2)	D
Mapped Spectral Response Acceleration Parameter, Ss (Figure 1613.5(3) for 0.2 second)	2.159
Mapped Spectral Response Acceleration Parameter, S1 (Figure 1613.5(4) for 1.0 second)	0.759
Site Coefficient, Fa (Table 1613.5.3(1) short period)	1.0
Site Coefficient, Fv (Table 1613.5.3(2) 1-second period)	1.5
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter SMS (Eq. 16-37)	2.159
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter SM1 (Eq. 16-38)	1.138
Design Spectral Response Acceleration Parameter, SDS (Eq. 16-39)	1.439
Design Spectral Response Acceleration Parameter, SD1 (Eq. 16-40)	0.759
Notes:34.0877, -117.9117 1. Site Class Designation: Class D is recommended based on subsurface condition. 2. Ss, SMs, and SDs are spectral response accelerations for the period of 0.2 second. 3. S1, SM1, and SD1 are spectral response accelerations for the period of 1.0 second.	

Conformance to the above criteria for seismic excitation does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid all damage, since such design may be economically prohibitive. Following a major earthquake, a properly designed building may be damaged beyond repair, yet not collapse.

CONVENTIONAL FOUNDATION CRITERIA

The on-site materials have a low expansion index. Sulfate testing has been performed and the results are presented in Appendix B. Complete chemical series testing and additional expansion index testing will be performed at the completion of grading. The following engineering criteria are recommended, should conventional foundations be used.

1. An allowable soil bearing pressure of 1,500 pounds per square foot can be used for design of conventional spread foundations founded in compacted fill. A one-third increase in the above bearing value may be used for transient live loadings such as

wind and seismic forces. Footings should be continuous and be founded a minimum of 18 inches into compacted fill with a minimum width of 12 inches for both one and two story structures. Footings should be reinforced with a minimum two, No. 4 rebar, both top and bottom.

2. A friction coefficient for concrete on compacted soil of 0.4, and a lateral bearing value of 250 pounds per square foot of depth may be employed to resist lateral loads. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third. For design of isolated poles, the allowable passive pressure may be increased by 100 percent.

3. Standard International Building Code structural setback guidelines per Section 1808.7 should be followed.

4. Subgrade soil beneath footings should be pre-moistened prior to placement of concrete.

General Recommendations

a. The above parameters are applicable provided the structures have gutters and downspouts and positive drainage is maintained away from the structure. All slab foundation areas should be moisture conditioned to at least optimum moisture

b. The above recommendations assume and GeoSoils Consultants, Inc. strongly recommends that surface water will be kept from infiltrating into the subgrade adjacent to the structures foundation system. This may include, but not be limited to rain water, roof water, landscape water and/or leaky plumbing.

Slabs-on-Grade

Should conventional slabs on grade be used, the following recommendations apply:

Floor slabs-on-grade should be designed for a nominal thickness of 4 inches, reinforced with No. 4 rebar at 16 inches on-center in both directions, placed at mid-height in the slab.

A 10-mil Visqueen vapor barrier should be placed underneath all slabs. This barrier should be placed between two, one-inch thick sand layers. This vapor barrier shall be lapped and sealed adequately (especially around the utility perforations) to provide a continuous waterproof barrier under the entire slab. Subgrade soils beneath slabs should be pre-moistened prior to the placement of concrete.

POST-TENSIONED SLAB FOUNDATION

The following may be considered as an alternative to conventional foundations. These post-tensioned slabs should be designed in accordance with the recommendations of either the California Foundation Slab Method or Post-Tensioning Institute. Based on review of laboratory data for the on-site materials, the average soil modulus of subgrade reaction, K, to be used for design is 100 pounds per cubic inch. Specific recommendations for the design of *California Foundation Slab* and *Post Tension Institute* methods are presented below.

A surface bearing value of 1,000 pounds per square foot can also be used in design.

1. **California Foundation Slab (Spanability) Method**

   It is recommended that slabs be designed for a free span of 15 feet. From a soil expansion/shrinkage standpoint, a common contributing factor to distress of structures using post-tensioned slabs is fluctuation of moisture in soils underlying the perimeter of the slab, compared to the center, causing a "dishing" or "arching" of the slabs. To mitigate this possibility, a combination of soil presaturation and construction of a perimeter "cut off" wall should be employed.

   All slab foundation areas should be moisture conditioned to at least optimum moisture, but no more than 5 percent above optimum moisture for a depth of at least 12 inches for low EI soil. A continuous perimeter curtain wall should extend to a depth of at least 12 inches for low EI soil to preserve this moisture. The cut-off walls may be integrated into the slab design or independent of the slab and should be a minimum of 6 (six) inches wide.

2. **Post-Tensioning Institute Method**

Post-tensioned slabs should have sufficient stiffness to resist excessive bending due to non-uniform swell and shrinkage of subgrade soils. The differential movement can occur at the corner, edge, or center of slab. The potential for differential uplift can be evaluated using design specifications of the Post-Tensioning Institute. The following table presents suggested minimum coefficients to be used in the Post-Tensioning Institute design method.

Suggested Coefficients	
Thornthwaite Moisture Index	-20 in/yr
Depth to Constant Soil Suction	9 (feet)
Constant Soil Suction: (pf)	3.8

The coefficients are considered minimums and may not be adequate to represent worst case conditions such as adverse drainage, excess watering, and/or improper landscaping and maintenance. The above parameters are applicable provided structures have gutters and downspouts, yard drains, and positive drainage is maintained away from structure perimeters. Also, the values may not be adequate if the soils below the foundation become saturated or dry such that shrinkage occurs. The parameters are provided with the expectation that subgrade soils below the foundations are maintained in a relatively uniform moisture condition. Responsible irrigation of landscaping adjacent to the foundation must be practiced since over-irrigation of landscaping can cause problems. Therefore, it is important that information regarding drainage, site maintenance, and settlements be passed on to future homeowners.

Based on the above parameters, the following values were obtained from the Post Tensioning Institute Design manual. If a stiffer slab is desired, higher values of y_m may be warranted.

Expansion Index of Soil Subgrade	**Low EI**
e_m center lift	9.0 feet
e_m edge lift	4.7 feet
Y_m center lift	0.34 inch
Y_m edge lift	0.48 inch

Deepened footings/edges around the slab perimeter must be used as indicated above to minimize non-uniform surface moisture migration (from an outside source) beneath the slab. An edge depth of at least 12 inches for low EI soil is recommended. The bottom of the deepened footing/edge should be designed to resist tension, using cable or reinforcement per the Structural Engineer.

Retaining Walls

If retaining walls are planned, the footings should have a minimum embedment depth of 18 inches into compacted fill and be designed in accordance to the recommendations presented herein. The near surface on site soils has a low expansion index.

The equivalent fluid pressures recommended are based on the assumption of a uniform backfill and no build-up of hydrostatic pressure behind the wall. To prevent the build-up of lateral soil pressures in excess of the recommended design pressures, over compaction of the fill behind the wall should be avoided. This can be accomplished by placement of the backfill above a 45-degree plane projected upward from the base of the wall, in lifts not exceeding eight inches in loose depth, and compacting with a hand-operated or small, self-propelled vibrating plates. (Note: Placement of free-draining material in this zone could also prevent the build-up of lateral soils pressures.)

1. Conventional (Yielding) Retaining Walls

All recommendations for active lateral earth pressures contained herein assume that the anticipated retaining structures are in tight contact with the fill soil (or alluvium) that they are supposed to support. The earth support system must be sufficiently stiff to hold horizontal movements in the soil to less than one percent of the height of the vertical face, but should be free-standing to the point that they yield at the top at least 0.1 percent of the height of the wall.

2. Earth Pressures on Conventional (Yielding) Retaining Walls

The earth pressures on walls retaining permeable material, compacted fill, or natural soil shall be assumed equal to that exerted by an equivalent fluid having a density not less than that shown in the following table:

Backfill Slope (Horizontal to Vertical)	Equivalent Fill Fluid Density
Level	30 pcf
2:1	43 pcf

3. Restrained (Non-Yielding) Walls

Earth pressures will be greater on walls where yielding at the top of the wall is limited to less than 1/1000 the height of the wall either by stiffness (i.e., return walls, etc.) or structural floor network prior to backfilling. Utilizing the recommended backfill compaction of 90 percent Modified Proctor Density per ASTM D-1557-12, we recommend the following equivalent fluid density for non-yielding walls:

Backfill Slope (Horizontal to Vertical)	Equivalent Fluid Density
Level	45 pcf
2:1	65 pcf

General

Any anticipated superimposed loading (i.e., upper retaining walls, other structures etc.) within a 45 degree projection upward from the wall bottom, except retained earth, shall be considered as surcharge and provided in the design.

A vertical component equal to one-third of the horizontal force so obtained, may be assumed at the application of force.

The depth of the retained earth shall be the vertical distance below the ground surface, measured at the wall face for stem design or measured at the heel of the footing for overturning and sliding.

The walls should be constructed with weep holes near the bottom, on five-foot centers or with perforated drainpipe in a gravel envelope at the bottom and behind the wall. A one-foot thick zone of clean granular, free-draining material should be placed behind the wall to within three feet of the surface. On-site soil may be used for the remainder of the backfill and should be compacted to 90 percent relative compaction as determined by ASTM Test Designation D-1557-12.

A concrete-lined swale is recommended behind retaining walls that can intercept surface runoff from upslope areas. The surface runoff shall be transferred to an approved drainage channel via non-erosive drainage devices.

Property Line Walls

Property line walls may be located in areas of unsuitable materials as removals adjacent to property boundaries cannot extend off site. We recommend deepened foundations or different wall design to accommodate an unsuitable foundation soil situation.

Temporary Excavations

Temporary cuts may be made vertical up to five feet in height, thereafter; cuts should be laid back to a 1:1 or less.

The recommended temporary excavation slopes do not preclude local ravelling or sloughing. All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act, and the Construction Safety Act should be met.

Where sloped embankments are used, the top of the slope should be barricaded to prevent equipment and heavy storage loads within five feet of the top of the slope. If the temporary construction embankments are to be maintained for long periods, berms should be constructed along the top of the slope to prevent runoff water from eroding the slope faces.

The soils exposed in the temporary backcut slopes during excavation should be observed by our personnel so that modifications of the slopes can be made if variations in the soil conditions occur.

Removals at the southeast corner of the property may extend down to 10 to 12 feet. In order to protect adjacent offsite improvements and roadways, it is recommended removals in these areas be performed in ABC slot cuts. Slot cut width should not exceed 8 feet. All "A" slots should be removed and the material replaced prior to excavation of "B" slots and so on. As an alternative, shoring may be utilized. Recommendations can be provided upon request.

Shrinkage

Based upon our field and laboratory test data, the on-site materials are expected to shrink between 9 to 14 percent.

Preliminary Pavement Design

Assuming a traffic index of 6 and an R value of 50, a pavement section of 3 inches of AC overlying 4.5 inches of base material (minimum R value of 78) may be utilized. R-value testing will be performed at the completion of grading and finalized pavement sections provided at that time.

Drainage/Landscape Maintenance

Water should not be allowed to pond or seep into the ground, or flow over slopes in a concentrated manner. Roof gutters and yard drains should be provided. Pad drainage should be directed toward the street or any approved watercourse area swale via non-erosive channel, pipe and/or dispersion devices.

Control of moisture is important in regard to control of mold within the future living environment. Molds can deteriorate building materials and lead to health problems such as asthma episodes and allergic reactions in susceptible individuals. Mold spores waft through both indoor and outdoor continually. When mold spores land on damp areas, they begin growing and digesting the host material in order to survive. Some molds propagate much more quickly than others. Molds can grow when moisture is present on and within wood, paper, carpet, and foods. Mold growth will often occur when excessive moisture accumulates in buildings or on building materials, particularly if moisture problems remain undiscovered, or are not addressed.

Obviously, the key to mold control is moisture control. Generally speaking, in the semi-arid climate of Southern California, we would not have mold problems if we did not have excessive landscape watering and the occasional leaking water, storm drain, or sewer pipe.

The average annual rainfall in Southern California is less than 15 inches per year; however, studies have shown that the average Southern California homeowner applies at least 200 inches of equivalent rainfall to their yard each year. It is important than in addition to control of landscape watering, that pad drainage slopes away from structures. Placement of planters next to houses can also lead to increased moisture under pad areas.

Review and Inspection

The site foundation and grading plans, including foundation-loading details, should be forwarded to the Geotechnical Engineer for review and approval prior to finalizing design.

All foundation and bottom excavations shall be observed by an engineering geologist or a geotechnical engineer prior to the placement of any steel to verify that the proper foundation material has been encountered. The local governing agency, Department of Building and Safety Inspector should also observe the excavation.

LIMITATIONS

The findings and recommendations of this report were prepared in compliance with the current Grading and Building Code and in accordance with generally accepted professional geotechnical engineering principles and practices. We make no other warranty, either express or implied.

We appreciate this opportunity to be of service to you. If you have any questions regarding the content of this report or any other aspects of the project, please do not hesitate to contact us.

Very truly yours,

GEOSOILS CONSULTANTS, INC.

KAREN L. MILLER
GE 2257

REGISTERED PROFESSIONAL ENGINEER
KAREN L. MILLER
No. 2257
Exp. 3/31/18
GEOTECHNICAL
STATE OF CALIFORNIA

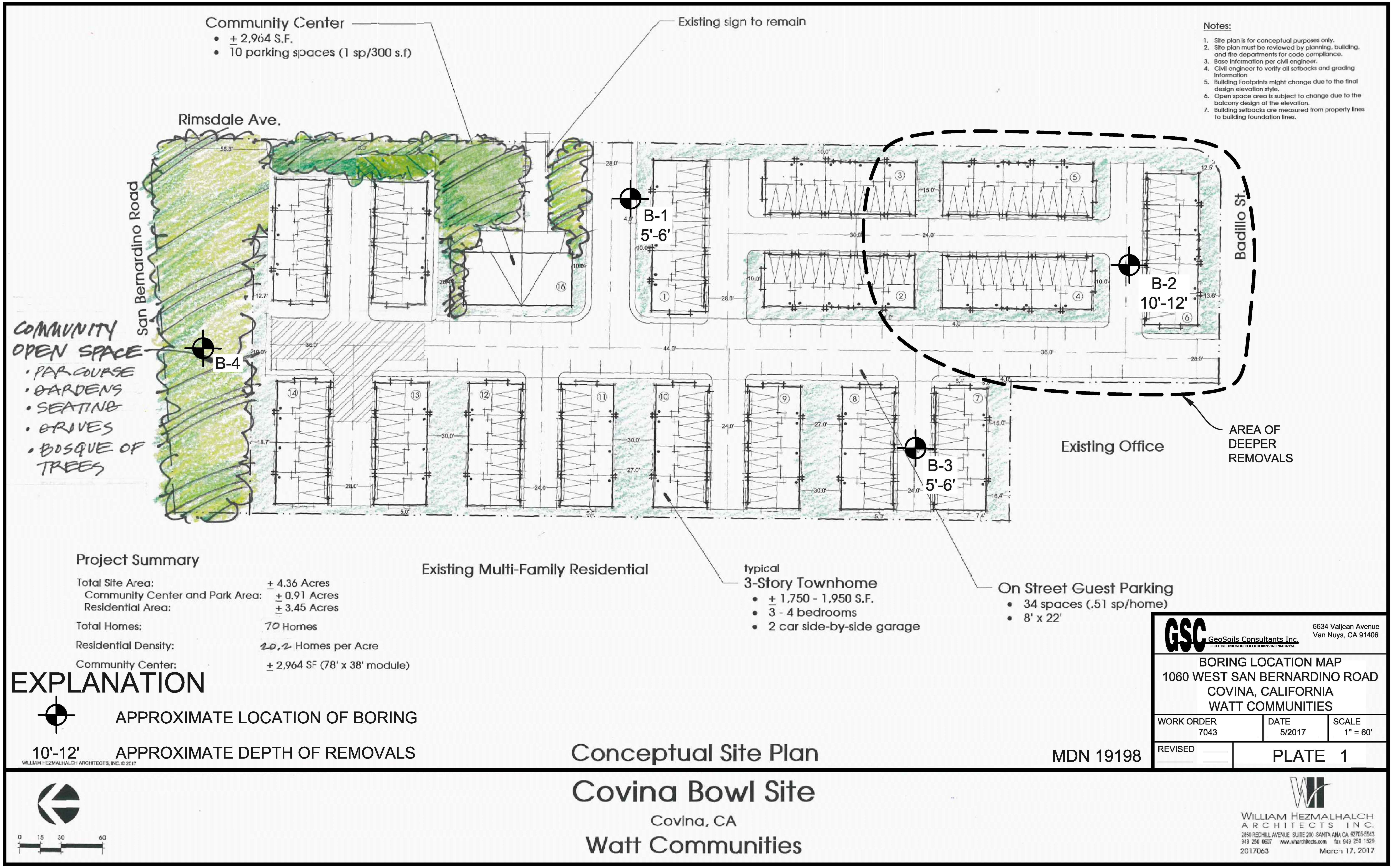
KLM.W:Geol & Geot Eng Inv.

Encl: Plate 1, Boring Location Map
Appendix A, Field Exploration Procedures
Plates A-1 through A-5, Boring Logs
Appendix B, Laboratory Testing Procedures and Test Results
Plates C-1 through C-7, Consolidation Diagrams
Plates SH-1 and SH-2, Shear Test Diagrams
Plate AA-1, Sulfate Test Results
Appendix C, Grading Guidelines
Appendix D, Infiltration Testing

cc: (5) Addressee

APPENDIX A

FIELD EXPLORATION PROCEDURES



X:\2001 w.o\gsc-cad\7043\CURRENT\PLATE 1 BORING TEST LOCATION MAP 5-18-17.dwg, 5/23/2017 2:04:14 PM, Adobe PDF.pc3

APPENDIX A

FIELD EXPLORATION PROCEDURES

Our exploratory borings were drilled with a truck-mounted drill rig operated by an independent drilling company working under subcontract to GSC. Drilling programs utilized an eight-inch diameter hollow-stem auger. Samples were obtained via the California ring sampler and Standard Penetration Test (SPT) sampler.

A representative from our firm continuously observed the boring, logged the subsurface conditions, and collected representative soil samples. All samples were stored in watertight containers and later transported to our laboratory for further visual examination and testing, as deemed necessary. After the boring was completed, the borehole was backfilled with soil cuttings.

The California ring samples were obtained at by means of the latest ASTM standard. The California ring sampling procedure consists of driving a standard 3-inch-diameter steel sampler with 18, 1-inch wide rings, 18 inches into the soil with a 140-pound hammer free-falling 30 inches. The number of blows required to drive the sampler through each 6-inch interval is counted, and the total number of blows struck is recorded.

The enclosed Boring Logs (Plates A-1 through A-5) describes the vertical sequence of soils and materials encountered in each boring, based primarily on our field classifications and supported by our subsequent laboratory examination and testing. Where a soil contact was observed to be gradational, our log indicates the average contact depth. Where a soil type changed between sample intervals, we inferred the contact depth. Our log also graphically indicates the blow count, sample type, sample number, and approximate depth of each soil sample obtained from the borings, as well as any laboratory tests performed on these soil samples. If any groundwater was encountered in a borehole, the approximate groundwater depth is depicted on the boring log. Groundwater depth estimates are typically based on the

Appendix A

moisture content of soil samples, the wetted height on the drilling rods, and the water level measured in the borehole after the auger has been extracted.

GEOTECHNICAL BORING LOG

PROJECT NAME: Watt			W.O. NO. 7043
DRILLING COMPANY: Gregg Drilling		DATE STARTED: 5-12-17	BORING NO. B-1
TYPE OF DRILL RIG: Truck		LOGGED BY: Jame Van Meter	SHEET 1 OF 2
DRILLING METHOD: Hollow Stem		HAMMER WEIGHT (LBS)	GROUND ELEVATION (FT)
DIAMETER OF HOLE: 8		DROP (IN)	GW ELEVATION
BORING LOCATION:			

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
	Bulk Sample		4" Asphalt 4"-50' Alluvium (Qal)			
5	California Ring	3/2	@ 5' Medium brown silty fine SAND, slightly to moderately moist, slightly dense.	12.1	107.8	
	California Ring	3/4	@7.5' Medium brown very fine to fine SAND, slightly moist, slightly to moderately dense.	8.8	103.5	
10	California Ring	3/4	@10' Medium fine silty very fine to fine SAND, slightly to moderately moist, slightly to moderately dense.	21.7	97.4	
15	California Ring	3/4	@15' Medium brown clayey silty very fine SAND, moist, slightly to moderately dense.	26.4	100.3	
20	California Ring	8/11	20' Light brown fine to very coarse SAND with gravel size rock fragments, slightly moist, dense.	8.0	111.5	
25	California Ring	19/44	25' Light grayish brown fine to very coarse SAND with abundant pea sized grevel, dry to slightly moist.	2.0	126.4	
30	California Ring	28/31		13.4	104.7	

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-1

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME Watt
W.O. NO. 7043
DRILLING COMPANY Gregg Drilling
DATE STARTED: 5-12-17
BORING NO. B-1
TYPE OF DRILL RIG Truck
LOGGED BY Jame Van Meter
SHEET 2 OF 2
DRILLING METHOD Hollow Stem
HAMMER WEIGHT (LBS)
GROUND ELEVATION (FT)
DIAMETER OF HOLE 8
DROP (IN)
GW ELEVATION
BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
	California Ring	21/18	35' Lightl gray fine to very coarse SAND with rock fragments, dry dense medium brown clayey silty CLAY fine SAND, moderately dense.	10.7	108.7	
40	California Ring	10/13	40' Medium brown silty very fine to fine SAND, slightly moist dense.	9.3	97.1	
45	California Ring	9/12	45' Medium brown silty very fine to fine SAND, slightly moist, dense.	4.9	107.5	
			50' Light to medium brown very fine to fine SAND, slightly moist, dense.			
50	California Ring	12/15	Total Depth 50' No Ground Water Hole Backfilled.			
55						
60						
65						

LEGEND

Standard Penetration Test
California Ring
Rock Core
Bulk Sample
Shelby Tube
Water Seepage
Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-2

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME Watt			W.O. NO. 7043
DRILLING COMPANY Gregg Drilling	DATE STARTED: 5-12-17		BORING NO. B-2
TYPE OF DRILL RIG Truck	LOGGED BY Jame Van Meter		SHEET 1 OF 1
DRILLING METHOD Hollow Stem	HAMMER WEIGHT (LBS)		GROUND ELEVATION (FT)
DIAMETER OF HOLE 8	DROP (IN)		GW ELEVATION
BORING LOCATION:			

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
			5" Asphalt 5"-30' Alluvium (Qal)			
5	Bulk Sample; California Ring	3/3	@5' Medium brown silty fine sand, slightly moist, slightly dense	6.2	99.4	
	California Ring	4/5	7.5 Light to medium brown very fine to fine SAND, slightly moist, slightly to moderately dense.	4.1	100.0	
10	California Ring	7/10	@10' Light to medium brown very fine to fine SAND, medium brown silty very fine SAND, slightly moist, moderately dense.	8.2	100.5	
15	California Ring	7/12	@15' Light brown silty very fine SAND, slightly moist, dense.	7.3	100.5	
20	California Ring	15/37	@20' Light brown to light gray brown fine to very coarse SAND with gravel, dry, dense.	1.5	125.6	
25	California Ring	15/32	@25' Light gray fine to medium SAND, dry, dense.	1.9	102.4	
			@30' Light gray fine to course SAND with gravel and medium brown silty fine SAND, moist, dense.			
30	California Ring	9/10		17.1	150.0	
			Total Depth 30' No Ground Water Hole Backfilled.			

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-3

GeoSoils Consultants, Inc.
GEOTECHNICAL ★ GEOLOGIC ★ ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME Watt
W.O. NO. 7043
DRILLING COMPANY Gregg Drilling
DATE STARTED: 5-12-17
BORING NO. B-3
TYPE OF DRILL RIG Truck
LOGGED BY Jame Van Meter
SHEET 1 OF 1
DRILLING METHOD Hollow Stem
HAMMER WEIGHT (LBS)
GROUND ELEVATION (FT)
DIAMETER OF HOLE 8
DROP (IN)
GW ELEVATION
BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
			3" Asphalt 3"-30' Alluvium (Qal)			
5	Bulk Sample; California Ring	2/2	@5' Medium brown silty fine SAND, moderately moist, slightly dense.	9.3	104.2	
	California Ring	3/5	@7.5' Medium brown silty fine SAND, moderately moist slightly to moderately dense.	7.6	103.7	
10	California Ring	5/6	@10' Mediium brown fine SAND, slightly moist, moderately dense.	6.2	102.9	
15	California Ring	6/7	15' Medium brown silty fine to medium SAND, moderately moist, moderately dense.	11.0	115.1	
20	California Ring	27/44	20' Light brown fine to very coarse SAND with rock fragments, slightly moist, dense.	2.4	128.2	
25	California Ring	44/50 for 4"	25' Partial sample, light gray fine to very coarse SAND with rock fragment, dry, dense.			
			30' Medium brown silty fine SAND and fine to medium SAND, moderately moist, moderately dense.			
30	California Ring	6/18		10.4	97.7	
			Total Depth 30' No Groung Water Hole Backfilled.			

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-4

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME Watt			W.O. NO. 7043
DRILLING COMPANY Gregg Drilling		DATE STARTED: 5-12-17	BORING NO. B-4
TYPE OF DRILL RIG Truck		LOGGED BY Jame Van Meter	SHEET 1 OF 1
DRILLING METHOD Hollow Stem		HAMMER WEIGHT (LBS)	GROUND ELEVATION (FT)
DIAMETER OF HOLE 8		DROP (IN)	GW ELEVATION
BORING LOCATION:			

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
			0-6" Asphalt **6"-10' Alluvium (Qal)**			
5	California Ring	3/4	@5' Medium brown silty fine sand, moderately moist, slightly dense.	8.7	106.5	
	California Ring	4/5	@7.5' Medium brown silty fine SAND, moderatley moist, lightly to moderateley dense.	7.2	103.1	
10	California Ring	4/7	@10' Medium brown silty fine SAND, moderatly moist, moderateley dense.	9.5	96.6	
15			Total Dept 10' Pipe/gravel installed No Ground Water.			
20						
25						
30						

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE:	GRAIN SIZE ANALYSIS
MAX:	MAXIMUM DRY DENSITY
DS:	DIRECT SHEAR
CONS:	CONSOLIDATION
HYDR:	HYDROMETER ANALYSIS
EXPAN:	EXPANSION INDEX
CHEM:	CHEMICAL TESTS

PLATE A-5

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

May 19, 2017
W.O. 7043

APPENDIX B

LABORATORY TEST PROCEDURES AND TEST RESULTS

May 19, 2017
W.O. 7043

APPENDIX B

LABORATORY TEST PROCEDURES AND TEST RESULTS

Moisture-Density

The in-situ moisture content and dry unit weights were determined for each of the undisturbed ring samples. The data obtained are shown on the boring logs.

Compaction Tests

One compaction test was performed to determine to moisture density relationships of the typical surficial soils encountered on the site. The laboratory standard used was in accordance with ASTM Test Designation D-1557-12. Summaries of the compaction test results are shown in Table B-2.

TABLE B-2
COMPACTION TEST RESULTS

Borings No. and Sample Depth	Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B-3 @ 3.5-6'	Dark brown, silty SAND	128.5	9.5

Consolidation Tests

Seven consolidation tests were performed on selected ring samples. The samples were inundated at an approximate load of one ton per square foot to monitor the hydroconsolidation. Loads were applied to the samples in several increments in geometric progression and resulting deformations were recorded at selected time intervals. Results of the consolidation tests are presented on Plates C-1 through C-7.

Appendix B

Direct Shear Tests

Natural and remolded (90 percent of the material's maximum density) samples were sheared in a strain-control type Direct Shear Machine. The sample was sheared under varying confining loads in order to determine the Coulomb shear strength parameters: cohesion (c), and angle of internal friction (ϕ) for peak and residual strength conditions. The samples were tested in an artificially-saturated condition. The results are plotted and a linear approximation is drawn of the failure curve. Results are shown on the Shear Test Diagrams, Plates SH-1 and SH-2 included in this appendix.

Expansion Index Test

To determine the expansion potential of the on-site soils, one expansion index test was conducted in accordance with the ASTD D-4829 on a sample from B-3 @ 3.5-6 feet. The ranges for expansion index potential are as follows:

0–20	Very Low
21–50	Low
51–90	Medium
91–130	High
>130	Very High

Table B-3 below presents the results.

TABLE B-3 EXPANSION INDEX TEST RESULTS		
Sample	**Expansion Index**	**Expansion Potential**
B-3@3.5-6'	Very low	3

Appendix B

Sulfates

Soluble sulfates react chemically with the hydrated lime and calcium aluminate of hardened cement to form calcium aluminate and calcium sulfo-aliminate. The effect is disintegration of the concrete. In addition to the potential detrimental effects of high concentrations of sulfate to certain admixtures of concrete, sulfates may catalyze reaction of certain clay minerals in soil columns which then undergo large, isolated volume changes which prove detrimental to some structures. Type V cement is normally used where sulfates are present.

Testing for soluble sulfates was performed on one representative sample of the material concentrated within the subject site by American Analytics (see Plate AA-1 this appendix). The results indicate that the soluble sulfate content is 39 ppm within the soil sample; therefore, the soils will have a negligible impact on the cement used at the site.

SULFATE EXPOSURE	RECOMMENDATIONS FOR CONCRETE IN SULFATE ENVIRONMENTS (AFTER TABLE 19-A-4)				
	SOLUBLE SULFATES IN SOIL, %	SULFATES IN WATER, PPM	CEMENT TYPE	MAXIMUM WATER/CEMENT RATIO	MINIMUM CEMENT CONTENT, LBS
Negligible	0-0.10	0-150			
Moderate	0.0.10-0.20	150-1,500	II	0.55	470
Severe	0.20-2.0	1,500-10,000	V	0.45	660
Very Severe	Over 2.0	Over 10,000	V + Pozzolan	0.45	660

Watt - Covina
W.O.: 7043

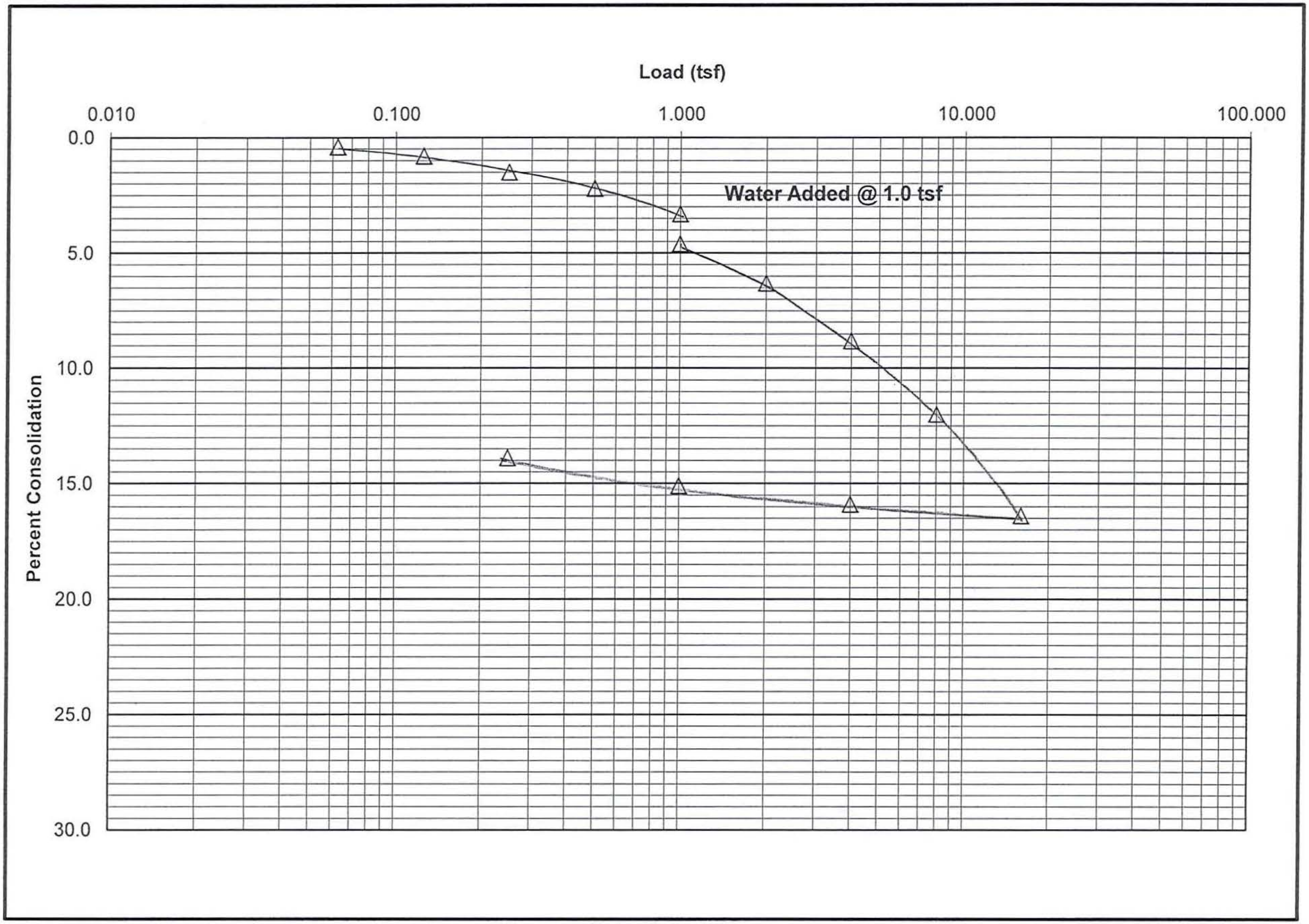
Date of Test: 5/17

GeoSoils Consultants, Inc.

Geotechnical Engineering * Engineering Geology

Moisture(%)
Before: 8.8 After: 20.7

Sample(in.)
Height: 1.00 Diameter: 2.36



B-1 @ 7.5'
Brown, slightly sandy SILT.

Consolidation Diagram

C7043.1.xls

Plate C-1

Watt - Covina
W.O.: 7043

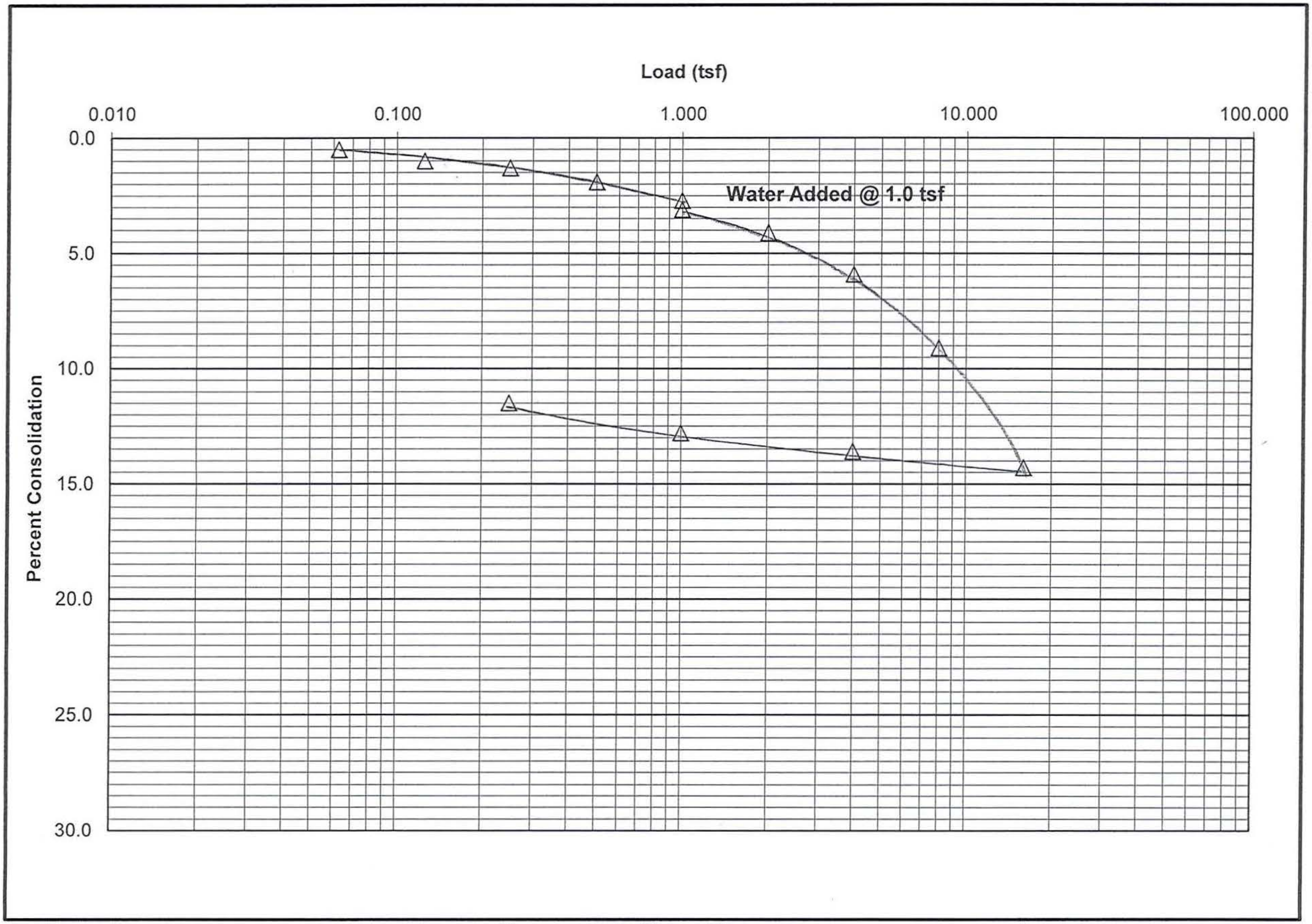
Date of Test: 5/17

GeoSoils Consultants, Inc.

Geotechnical Engineering * Engineering Geology

Moisture(%)
Before: 21.7 After: 19.9

Sample(in.)
Height: 1.00 Diameter: 2.36



B-1 @ 10.0'
Brown, clayey SILT.

Consolidation Diagram

C7043.2.xls

Plate C-2

Watt - Covina
W.O.: 7043

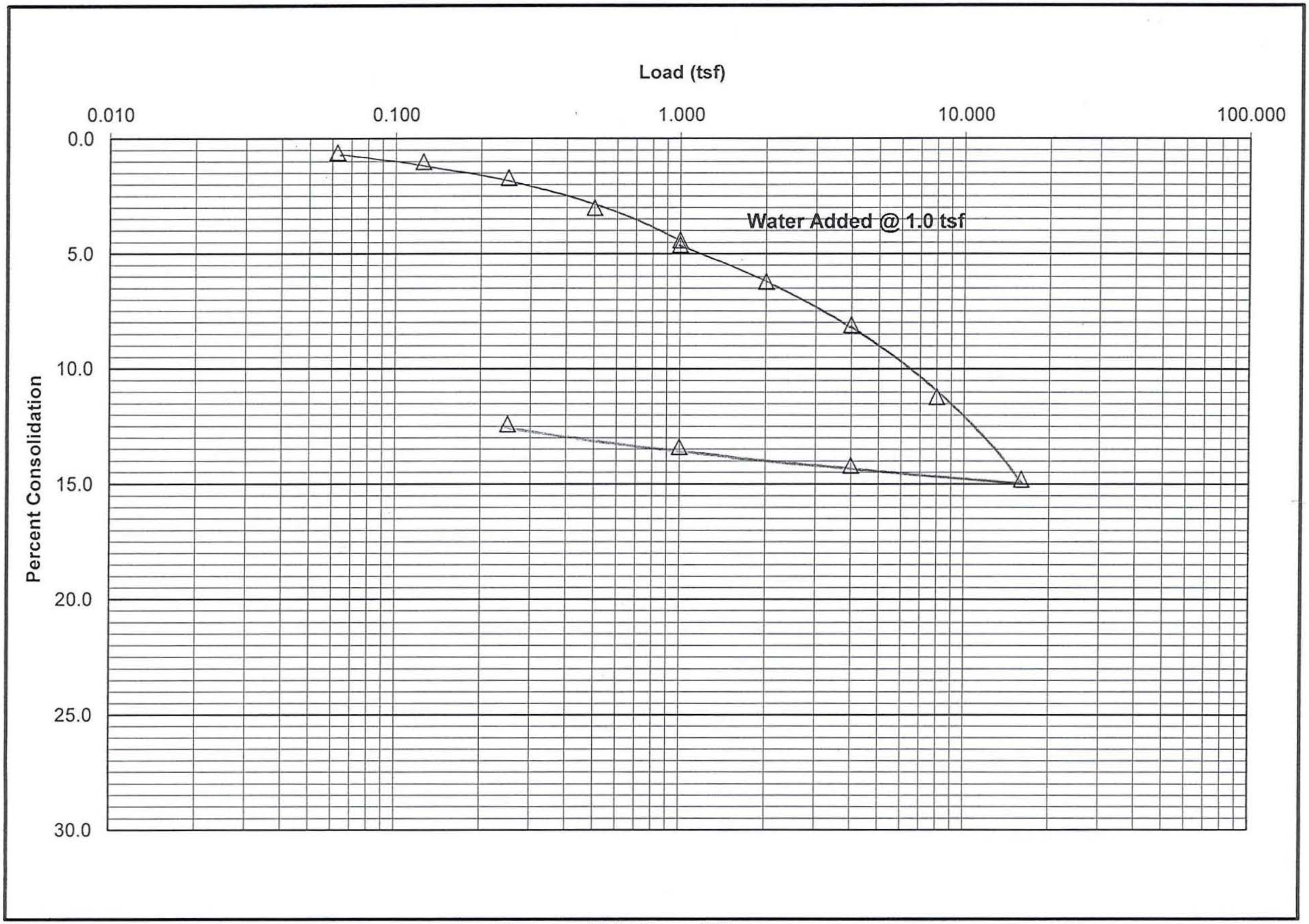
Date of Test: 5/17

GeoSoils Consultants, Inc.

Geotechnical Engineering * Engineering Geology

Moisture(%)
Before: 26.4 After: 14.5

Sample(in.)
Height: 1.00 Diameter: 2.36



B-1 @ 15.0'
Brown, clayey SILT.

Consolidation Diagram

C7043.3.xls

Plate C-3

Watt - Covina
W.O.: 7043

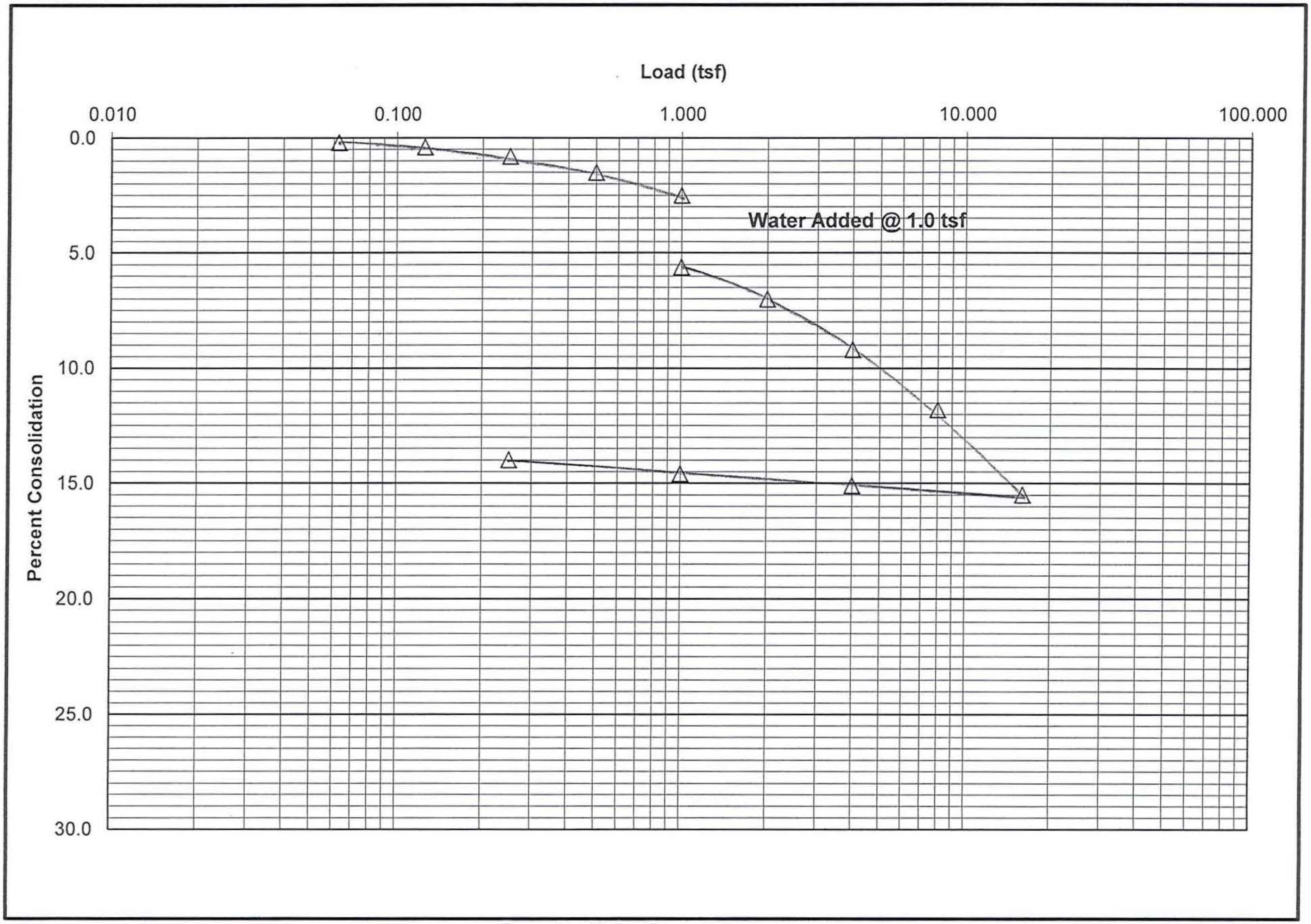
Date of Test: 5/17

GeoSoils Consultants, Inc.

Geotechnical Engineering * Engineering Geology

Moisture(%)
Before: 4.1 After: 19.3

Sample(in.)
Height: 1.00 Diameter: 2.36



B-2 @ 7.5'
Brown, silty SAND.

Consolidation Diagram

C7043.4.xls

Plate C-4

Watt - Covina
W.O.: 7043

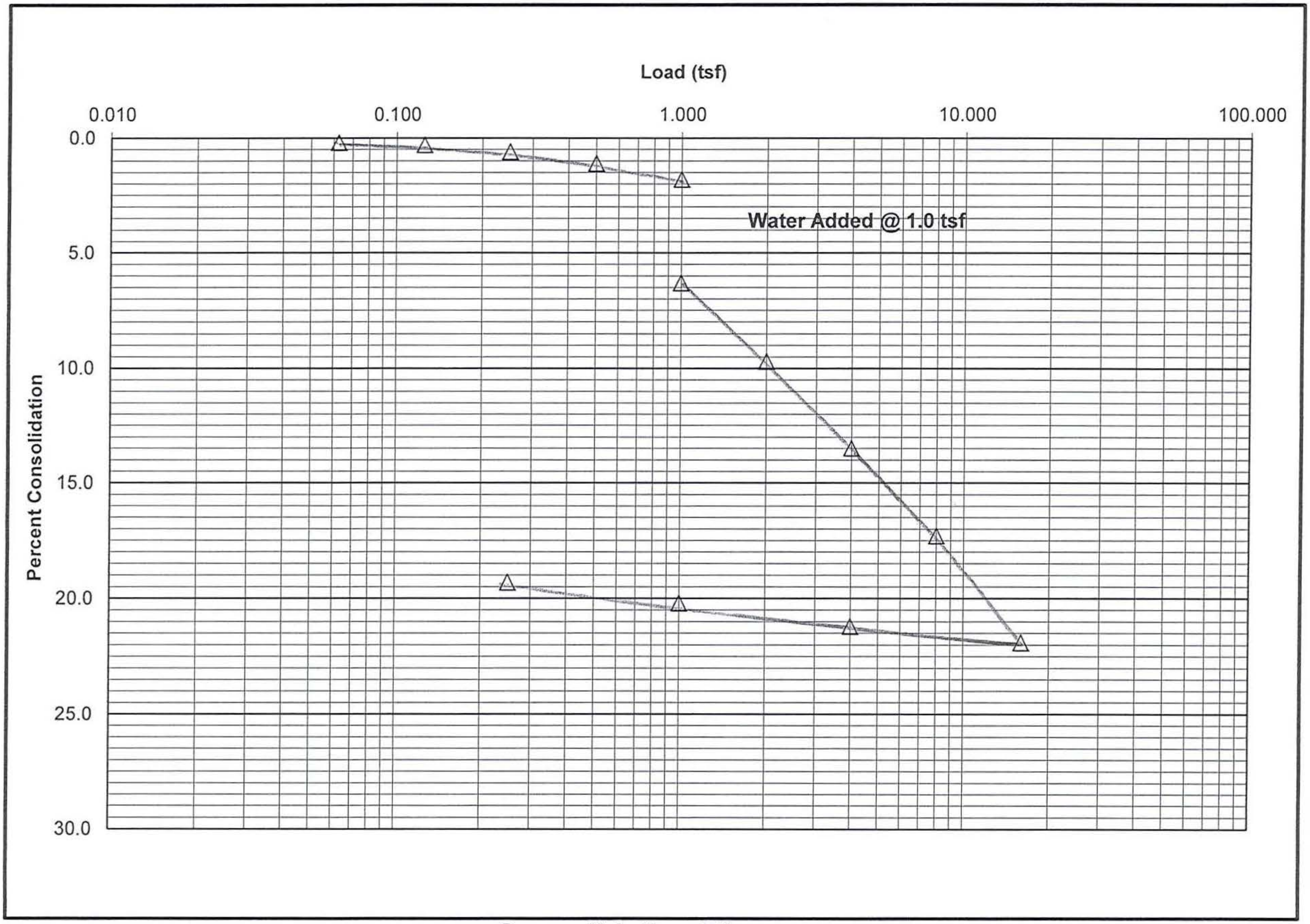
Date of Test: 5/17

GeoSoils Consultants, Inc.

Geotechnical Engineering * Engineering Geology

Moisture(%)
Before: 8.2 After: 16.7

Sample(in.)
Height: 1.00 Diameter: 2.36



B-2 @ 10.0'
Orange-brown, silty, slightly clayey SAND.

Consolidation Diagram

C7043.5.xls

Plate C-5

Watt - Covina
W.O.: 7043

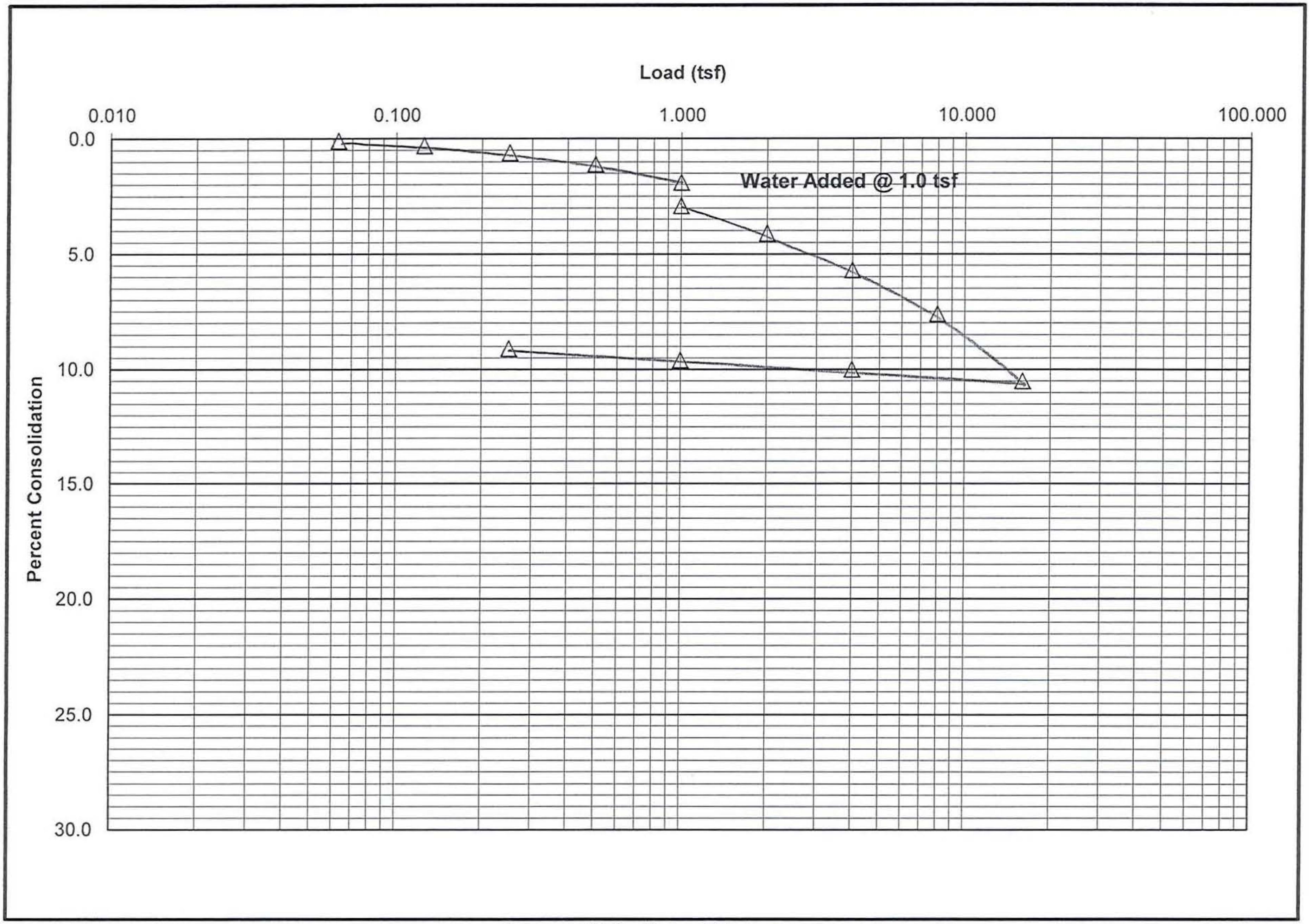
Date of Test: 5/17

GeoSoils Consultants, Inc.

Geotechnical Engineering * Engineering Geology

Moisture(%)
Before: 6.2 After: 20.4

Sample(in.)
Height: 1.00 Diameter: 2.36



B-3 @ 10.0'
Brown, silty SAND.

Consolidation Diagram

C7043.6.xls

Plate C-6

Watt - Covina
W.O.: 7043

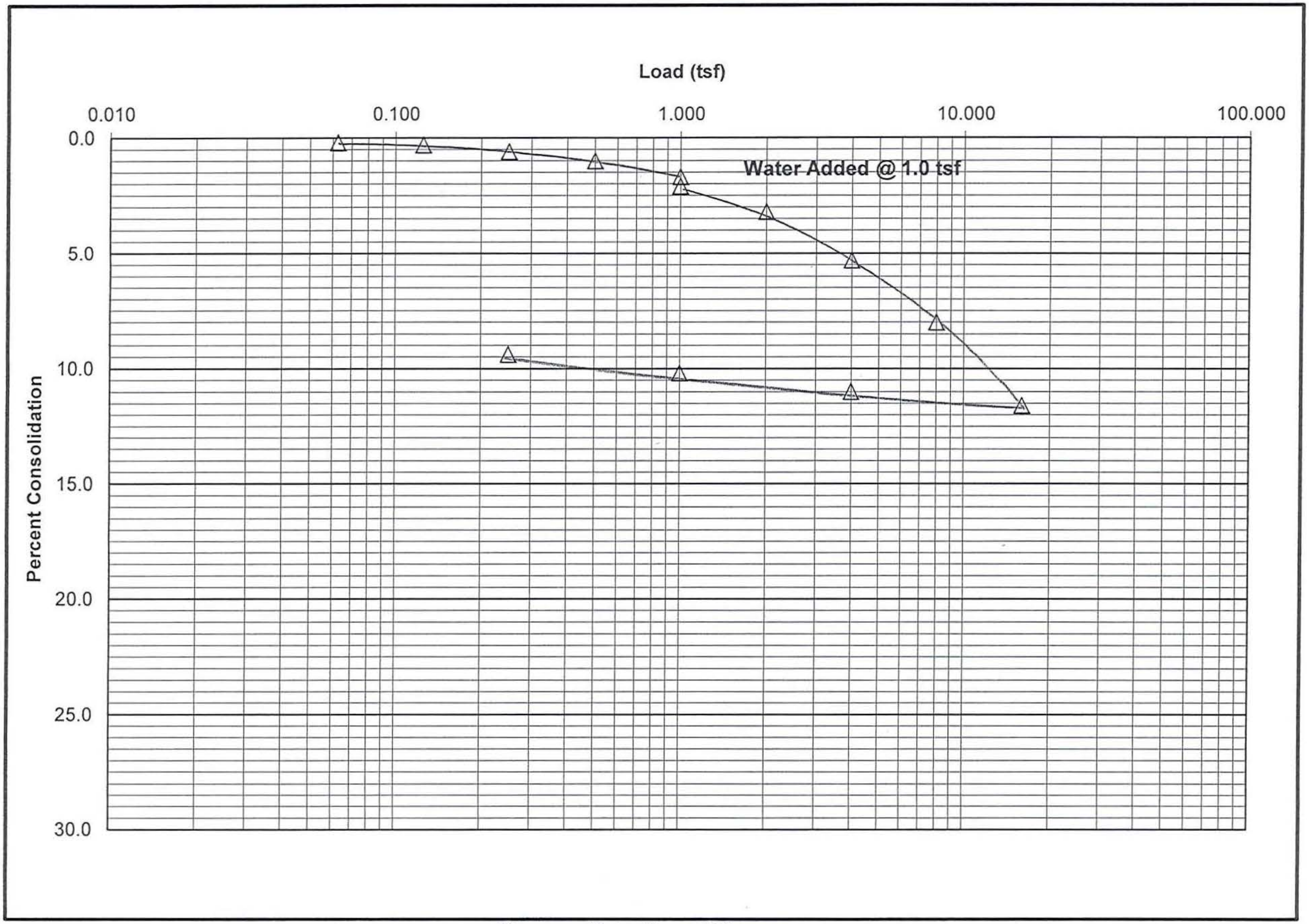
Date of Test: 5/17

GeoSoils Consultants, Inc.

Geotechnical Engineering * Engineering Geology

Moisture(%)
Before: 11.0 After: 14.1

Sample(in.)
Height: 1.00 Diameter: 2.36



B-2 @ 15.0'
Brown, silty SAND.

Consolidation Diagram

C7043.7.xls

Plate C-7

Watt - Covina
W.O.: 7043

Date of Test: 5/17

Sample: **B-2 @ 5.0'**

GeoSoils Consultants, Inc.

Geotechnical Engineering * Engineering Geology

PLATE SH-1

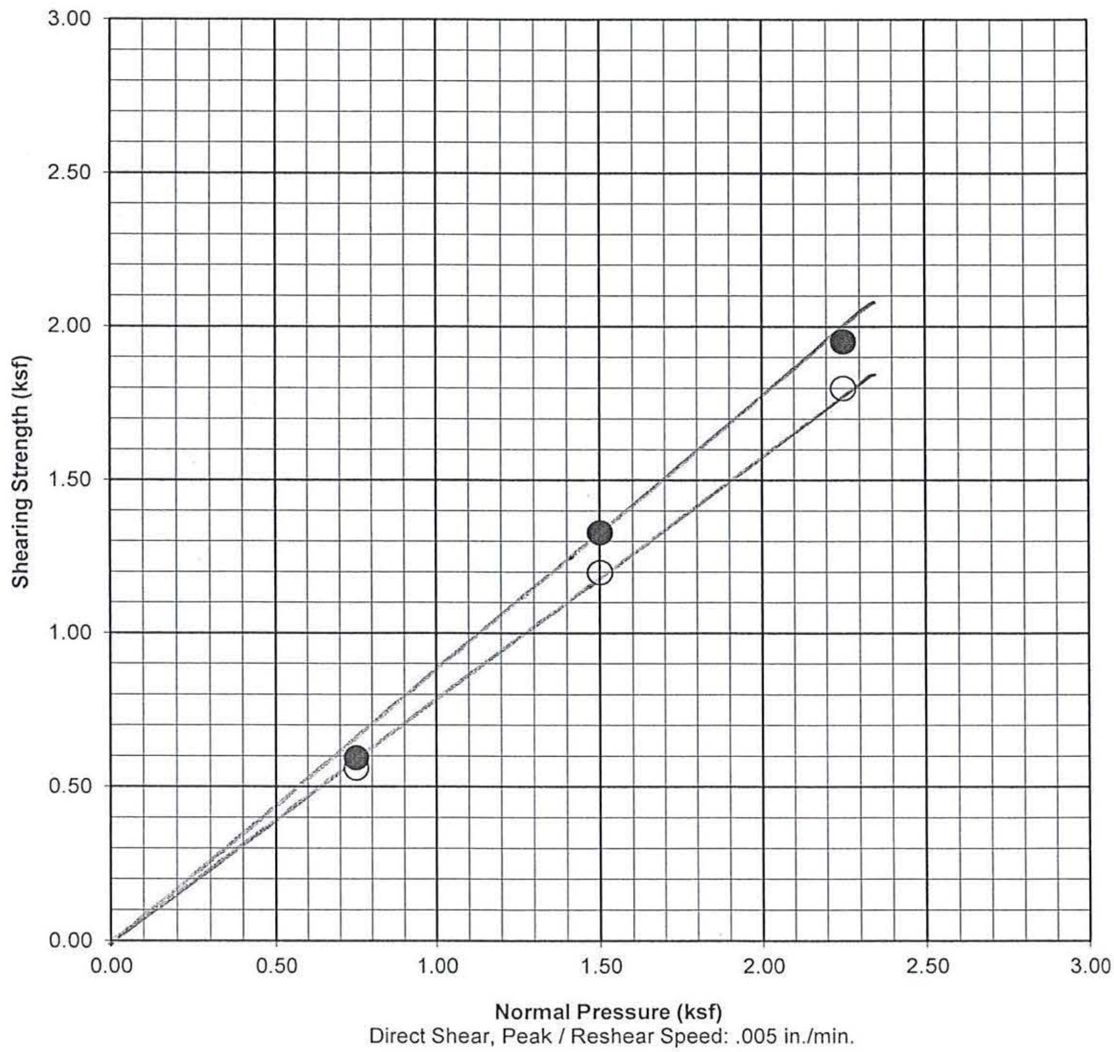
Shear Test Diagram

Peak

C(psf): 0 Phi (degrees): 40.0

Reshear

C(psf): 0 Phi (degrees): 37.0



Undisturbed **Natural** Shear-Saturated

Brown, silty SAND.

23.5% Saturated Moisture Content

7043.1.xls

Watt - Covina
W.O.: 7043

Date of Test: 5/17

Sample: **B-3 @ 3.5'-6.0'**

PLATE SH-1

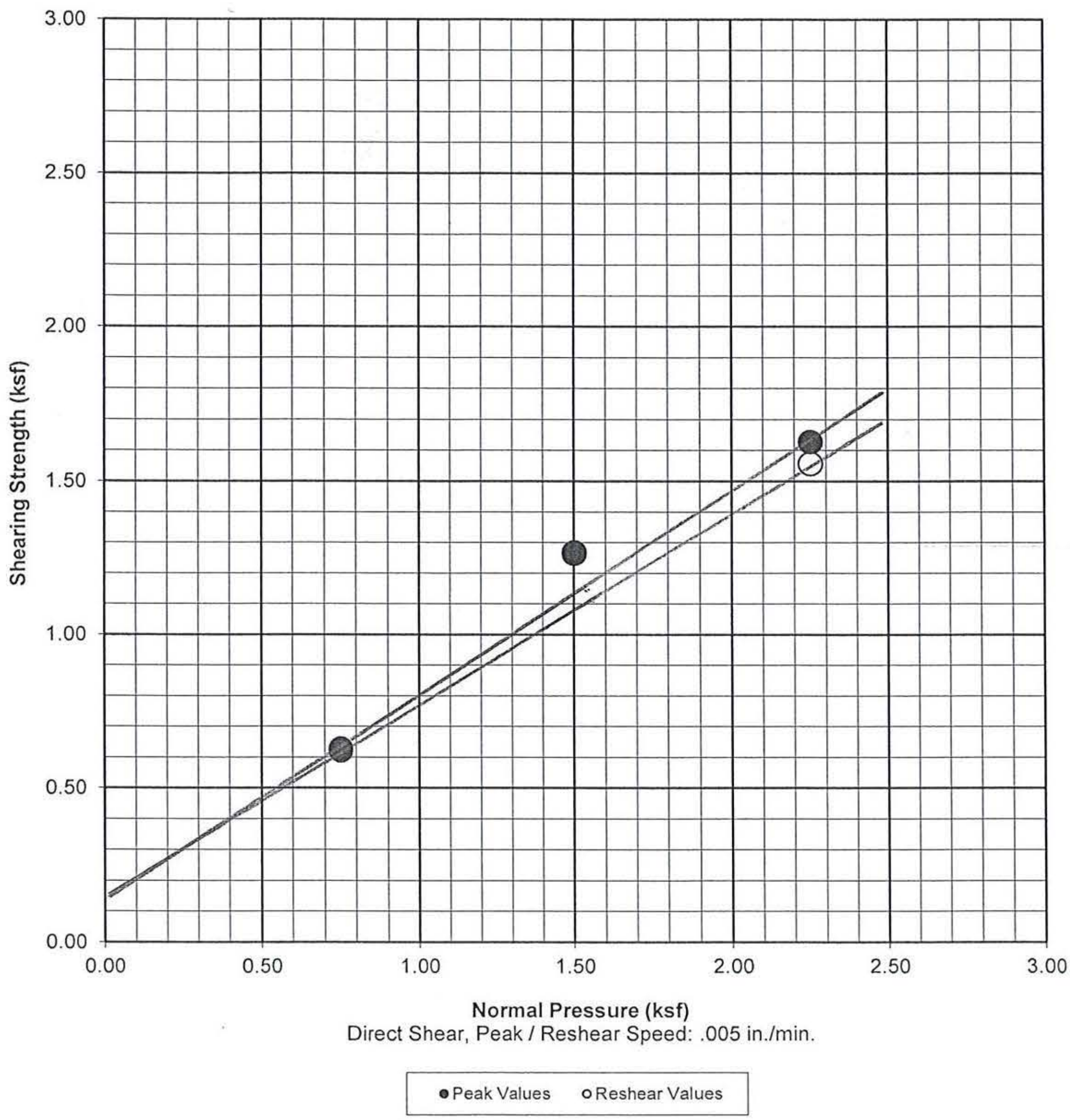
GeoSoils Consultants, Inc.

Geotechnical Engineering * Engineering Geology

Shear Test Diagram

Peak
C(psf): **150** Phi (degrees): **33.0**

Reshear
C(psf): **150** Phi (degrees): **32.0**



Sample **Remolded** to 90% Relative Density, saturated.
Remolded Dry Density = 115.7 PCF

Dark brown silty sand.

MAX: 128.5 PCF: 9.5%
18.8% Saturated Moisture Content
7043.1.xls

AMERICAN ANALYTICS

LABORATORY ANALYSIS RESULTS

Client: Geosoils Consultants, Inc.
Project No: NA
Project Name: 7043

AA Project No: A61017/8
Date Received: 05/15/17
Date Reported: 05/22/17

ANALYTICAL DATA SUMMARY

Analyte	Sample Name	Result	MRL	Units	Dilution	Prepared	Analyzed	Method
Sulfate by Ion Chromatography								
Sulfate	7043 B-2@5-7.5	**39**	5.0	mg/kg	1	05/19/17	05/19/17	EPA 300.0

Allen A

Allen Aminian
QA/QC Manager

Plate AA-1

APPENDIX C

GRADING GUIDELINES

APPENDIX C

GRADING GUIDLINES

These specifications present the minimum requirements for grading operations performed under the control of GeoSoils Consultants, Inc.

No deviation from these specifications would be allowed, except where specifically superseded in the preliminary geotechnical report, or in other written communication signed by the Geotechnical Engineer or Engineering Geologist.

1. **General**

   A. The Geotechnical Engineer and Engineering Geologist are the Owner's or Builder's representative on the project. For the purpose of these specifications, supervision by the Geotechnical Engineer or Engineering Geologist includes that inspection performed by any person or persons employed by, and responsible to, the licensed Geotechnical Engineer or Engineering Geologist signing the Geotechnical report.

   B. All clearing, site preparation or earthwork performed on the project should be conducted by the Contractor under the observation of the Geotechnical Engineer or Engineering Geologist.

   C. It is the Contractor's responsibility to prepare the ground surface to receive the fills to the satisfaction of the Geotechnical Engineer or Engineering Geologist and to place, spread, mix, water, and compact the fill in accordance with the specifications of the Geotechnical Engineer or Engineering Geologist. The Contractor should also remove all material considered unsatisfactory by the Geotechnical Engineer or Engineering Geologist

Appendix C

D. It is also the Contractor's responsibility to have suitable and sufficient compaction equipment on the jobsite to handle the amount of fill being placed. If necessary, excavation equipment would be shut down to permit completion of compaction. Sufficient watering apparatus would also be provided by the Contractor, with due consideration for the fill material, rate of placement and time of year.

E. A final report should be issued by the Geotechnical Engineer and Engineering Geologist attesting to the Contractor's conformance with these specifications.

F. At all times, safety would have precedence over production work. If an unsafe job condition is noted by a GeoSoils Consultants, Inc. representative, it would be brought to the attention of the Grading Contractor's foreman, the on-site developer's representative or both. Once this condition is noted, it should be corrected as soon as possible, or work related to the unsafe condition may be terminated.

2. **Site Preparation**

A. All vegetation and deleterious material, such as rubbish, should be disposed of off-site. This removal must be concluded prior to placing fill.

B. The Contractor should locate all houses, sheds, sewage disposal systems, large trees or structures on the site, or on the grading plan, to the best of his knowledge prior to preparing the ground surface.

Appendix C

C. Soils, alluvium or rock materials determined by the Geotechnical Engineer as being unsuitable for placement in compacted fills should be removed and wasted from the site. Any material incorporated as a part of a compacted fill must be approved by the Geotechnical Engineer.

D. After the ground surface to receive fill has been cleared, it should be scarified, disced or bladed by the Contractor until it is uniform and free from ruts, hollows, hummocks or other uneven features, which may prevent uniform compaction.

The scarified ground surface should then be brought to approximately 120 percent of optimum moisture, mixed as required, and compacted as specified. If the scarified zone is greater than 12 inches in depth, the excess should be removed and placed in lifts restricted to 6 inches.

Prior to placing fill, the ground surface to receive fill should be inspected, tested and approved by the Geotechnical Engineer.

E. Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines or other not located prior to grading are to be removed or treated in a manner prescribed by the Geotechnical Engineer.

3. **Compacted Fills**

A. Material imported or excavated on the property may be utilized in the fill, provided such material has been determined to be suitable by the Geotechnical Engineer. Roots, tree branches and other deleterious matter missed during clearing should be removed from the fill as directed by the Geotechnical Engineer.

Appendix C

B. Rock fragments less than six inches in diameter may be utilized in the fill, provided:

1. they are not placed in concentrated pockets;
2. there is a sufficient percentage of fine-grained material to surround the rocks.
3. the distribution of the rocks is supervised by the Geotechnical Engineer.

C. Rocks greater than six inches in diameter should be taken off-site. Fills on-site are not deep enough below pad grade to provide for rock disposal.

D. Material that is spongy, subject to decay, or otherwise considered unsuitable should not be used in the compacted fill.

E. Representative samples of materials to be utilized as compacted fill should be analyzed in the laboratory by the Geotechnical Engineer to determine their physical properties. If any material other than that previously tested is encountered during grading, the appropriate analysis of this material should be conducted by the Geotechnical Engineer as soon as possible.

F. Material used in the compacting process should be evenly spread in thin lifts not to exceed six inches in thickness, watered, processed and compacted to obtain a uniformly dense layer. The fill should be placed and compacted on a <u>horizontal</u> plane, unless otherwise approved by the Geotechnical Engineer. This includes material placed for slope repairs, and utility trench backfills on slope areas.

Appendix C

G. Each layer should be compacted to at least a minimum of 90 percent of the maximum density in compliance with the testing method specified by the controlling governmental agency (in general, ASTM D-1557-12 would be used).

If compaction to a lesser percentage is authorized by the controlling governmental agency because of a specific land use or expansive geotechnical conditions, the area to receive fill compacted to less than 90 percent should either be delineated on the grading plan or appropriate reference made to the area in the geotechnical report.

H. All fills must be placed at approximately 120 percent of optimum moisture. If excessive moisture in the fill results in failing tests or an unacceptable "pumping" condition, then the fill should be allowed to dry until the moisture content is within the necessary range to meet above compaction requirements, or should be removed or reworked until acceptable conditions are obtained.

I. If the moisture content or relative density varies from that required by the Geotechnical Engineer, the Contractor should rework the fill until it is in accordance with the requirements of the Geotechnical Engineer. If a compaction test indicates that the fill meets or exceeds the minimum required relative compaction but is below 120 percent of optimum, then the fill should be reworked until it meets the moisture content requirements.

5. **Grading Control**

A. Inspection of the fill placement should be provided by the Geotechnical Engineer during the progress of grading.

Appendix C

B. In general, density tests should be made at intervals not exceeding two feet of fill height or every 500 cubic yards of fill placed. These criteria would vary depending on soil conditions and the size of the job. In any event, an adequate number of field density tests should be made to verify that the required compaction is being achieved.

C. Density tests should also be made on the surface material to receive fill as required by the Geotechnical Engineer.

D. All cleanout, processed ground to receive fill, key excavations, subdrains and rock disposal should be inspected and approved by the Geotechnical Engineer prior to placing any fill. It should be the Contractor's responsibility to notify the Geotechnical Engineer when such areas are ready for inspection. In most jurisdictions, these items must also be inspected by a representative of the controlling governmental agency prior to fill placement.

6. Construction Considerations

A. Erosion control measures, when necessary, should be provided by the Contractor during grading and prior to the completion and construction of permanent drainage controls.

B. Upon completion of grading and termination of inspections by the Geotechnical Engineer, no further filling or excavating, including that necessary for footings, foundations, large tree wells, retaining walls, or other features should be performed without the approval and observation of the Geotechnical Engineer or Engineering Geologist.

Appendix C

C. Care should be taken by the Contractor during final grading to preserve any berms, drainage terraces, interceptor swales, or other devices of a permanent nature on or adjacent to the property.

May 19, 2017
W.O. 7043

APPENDIX D

INFILTRATION TESTING

APPENDIX D

INFILTRATION TESTING

As requested, GeoSoils Consultants, Inc. (GSC) performed infiltration testing on the subject site. It is our understanding the infiltration test results will be used for design of the underground infiltration BMP necessary to satisfy SUSMP/LID requirements.

One boring, Boring B-4 (Plate A-5) were drilled for the infiltration testing. The boring was drilled to 10 feet and a 10 foot pipe with five feet of slotted screen was installed.

Geologic Conditions

Alluvium underlies the area of the proposed development.

Groundwater Evaluation

No groundwater was encountered. As previously stated, the depth to groundwater is more than 50 feet below grade.

PERCOLATION BORINGS

One hollow stem boring (B-4) was excavated to a depth of 10 feet below existing grade. This boring was drilled for the installation of an infiltration well by the procedure explained below. The materials exposed in the excavation consisted of alluvium to the total depth of the boring. The perforated zone of the 10 foot boring was from 5 to 10 feet

A 2-inch diameter pipe was installed in the boring. A cap was placed on the base of the pipe. A perforated pipe was installed above the base cap for the intended percolation zone to be tested and the annular space filled with gravel. A minimum 2-foot bentonite seal was placed in the annulus of the boring to seal the perforated/ solid pipe connection. The remaining area above the bentonite seal was backfilled with on-site material.

PERCOLATION TESTING FOR STORM WATER INFILTRATION

Pre-soak

Pre-soaking was performed for four hours prior to the test.

Infiltration Testing

The infiltration test was performed on May 16, 2017. The field measurements are presented herein as Plate P-1, (Boring Infiltration Testing Field Logs).

The infiltration test readings were performed until a stabilized rate (highest and lowest readings are within 10 percent of each other for three consecutive tests) were obtained. The average drop of the stabilized rate over the last three consecutive readings is the pre-adjusted infiltration rate for the test location, expressed in inches per hour. The flow rate of the water drained faster than on infiltration rate of 14 inches per hour. The high flow rate percolation test procedure per Los Angeles County Guidelines. The following table represents the pre-adjusted infiltration rates for the test locations.

Percolation Boring	Test Depth (feet)	Infiltration Rate
B-4	5-10	126.7 inches/hr

Boring/Excavation Percolation Testing Field Log

Date 5-16-2017
W.O. 7043

Project Location	1060 West San Bernadino Road	Boring/Test Number	B-4
Earth Description	Medium brown silty fine send	Diameter of Boring	8" Diameter of Casing 2"
Tested by	JLV	Depth of Boring	10'
Liquid Description	H2O	Depth to Invert of BMP	
Measurement Method		Depth to Water Table	100'+
		Depth to initial Water Depth ($d_{1)}$	4'

Time Interval Standard
Start Time for Pre-Soak 7:15 to 7:45 7:47 to 8:17
Start Time for Standard 11:15

Water Remaining In Boring (Y/N) N/Y
Standard Time Interval Between Readings

Reading Number	Time Start/End (hh:mm)	Elapsed Time Δtime (mins)	Water Drop During Standard Time Interval Δd (inches)	Percolation Rate for Reading (in/hr)	Soil Description/Notes/Comments
1	1115 1125	10	4.0'-7.25'=3.25'	234"	
2	1127 1137	10	4.0'-7.35=3.35'	241.2"	
3	1138 1148	10	4.0'-7.26=3.26'	234.72	
4	1149 1159	10	4.0'-7.25=3.25'	234"	
5	1200 1210	10	4.0'-7.3=3.3	237.6"	
6	1211 1221	10	4.0'-7.26=3.26'	234.72'	
7	1222 1232	10	4.0'-1.23=3.23'	232.56"	
8	1233.5 1243.5	10	4.0'-7.25'=3.25'	234"	

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