GEOTECHNICAL SUBSURFACE EXPLORATION REPORT
VELO PARK APARTMENTS
LOT 1C – AIRPORT SOUTH REPLAT C
3289 AIRPORT ROAD, BOULDER, COLORADO
SOILOGIC # 16-1002
August 31, 2018

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Thistle Velo, LLC
6000 Spine Road #101
Boulder, Colorado 80301

Attn: Ms. Mary Duvall

Re: Geotechnical Subsurface Exploration Report
Velo Park Apartments
Lot 1C, Airport South Replat C
3289 Airport Road, Boulder, Colorado
Soilogic Project # 16-1002

Ms. Duvall:

Soilogic, Inc. (Soilogic) personnel have completed the geotechnical subsurface exploration you requested for the proposed Velo Park Apartments to be constructed on an approximate 2.6-acre parcel of land identified as Lot 1C – Airport South Replat C in Boulder, Colorado. The results of our exploration and pertinent geotechnical engineering recommendations are included with this report.

Approximately 4 inches of vegetation and topsoil was encountered at the surface at the boring locations. At the location of borings B-4 and B-7, the vegetative soil layer was underlain by apparent fill soils consisting of brown/rust sand and gravel extending to depths ranging from approximately 3 to 3½ feet below present site grade. Light brown to brown/rust lean clay with varying amounts of silt, sand and scattered gravel was encountered underlying the apparent sand and gravel fill at the location of borings B-4 and B-7 and underlying the vegetative soil layer at the remainder of the completed site borings. The lean clay varied from very stiff to hard in terms of consistency, exhibited low to high swell potential at in-situ moisture and density conditions and extended to depths between about 1 to 9½ feet below ground surface. At the location of boring B-2, the lean clay extended to a depth of approximately 5 feet below ground surface and was underlain by rust/brown silty sand and gravel. The sand and gravel would be expected to be non-expansive or possess low swell based on the physical properties and engineering characteristics of the material and extended to a depth of approximately 7 feet below present site grade. The lean clay encountered in borings B-1 and B-3 through B-8 and silty sand and gravel encountered at the location of boring B-2 were underlain by rust/ olive/grey claystone bedrock. The claystone varied from weathered to very hard in terms of relative hardness, exhibited low to very high swell potential at in-situ moisture and density conditions and extended to the bottom of all borings at a depths ranging from approximately 15 to 30 feet below present site grade. Groundwater was not encountered in any of the completed site borings at the time of drilling.

Due to the presence of variably expansive clay soils and highly to very highly expansive claystone bedrock encountered relatively near-surface at this site, we recommend the proposed buildings be supported on drilled pier and grade beam foundation systems. Similar drilled pier foundations should be considered for support of site retaining walls. This type of system extends the foundation elements through expansive materials which are subjected to wetting and swelling and can place them in materials not as likely to experience significant moisture changes and resulting volume change. Similarly, swell-consolidation tests indicate that a majority of the lean clay soils and claystone bedrock likely to influence slab-on-grade construction exhibit high to very high swell potential, such that we recommend the building living area and garage floors be constructed as structural floors, supported independent of the subgrade materials. Overexcavation/ backfill procedures are recommended to redevelop low volume change potential exterior flatwork and pavement subgrade support, reducing the potential for total and differential heaving of those supported elements subsequent to construction. As a lower cost/higher risk alternative, similar overexcavation/backfill procedures could be considered to develop conventional spread footing foundation support for site retaining walls and conventional garage floor slab support, provided movement can be tolerated. The risk of some movement cannot be eliminated and some movement of improvements supported on a zone of reconditioned soil should be expected. Recommendations concerning drilled pier foundation design criteria for the buildings and site retaining walls and structural floor systems for the buildings, as well as overexcavation/backfill procedures for retaining walls, conventional garage floor slabs, exterior flatwork and site pavements are included with this report.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning the enclosed information or if we can provide any further assistance, please do not hesitate to contact us.

Very Truly Yours,
Soilogic, Inc.

Wolf von Carlowitz, P.E.
Principal Engineer

Reviewed by:

Darrel DiCarlo, P.E.
Senior Project Engineer

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INTRODUCTION

This report contains the results of the completed geotechnical subsurface exploration for the proposed Velo Park Apartments to be constructed on Lot 1C in the Airport South Replat C development in Boulder, Colorado. The purpose of our exploration was to describe the subsurface conditions encountered in the completed site borings and develop the test data necessary to provide recommendations concerning design and construction of the proposed building foundations and support of floor slabs, exterior flatwork and site pavements. Pavement section design recommendations are also provided. The conclusions and recommendations outlined in this report are based on the results of the completed field and laboratory testing and our experience with subsurface conditions in this area.

PROPOSED CONSTRUCTION

Based on the provided site plan and our discussion with the client, we understand this project involves the construction of four (4) apartment buildings and one (1) live/work building and associated exterior flatwork and site drive and parking area pavements on an approximate 2.6-acre tract of land located at 3289 Airport Road in Boulder, Colorado. We understand the apartment buildings will be two-story wood-frame structures constructed over conventional and reverse walkout basements. The live/work building will be a two-story wood-frame structure constructed at-grade. Foundation loads for the structures are expected to be relatively light, with continuous wall loads less than 3.5 kips per lineal foot and individual column loads less than 75 kips. Site drive and parking area pavements are also anticipated as part of the proposed site improvements. Traffic loading on site pavements is expected to consist of low volumes of light passenger vehicles with occasional heavier trash and delivery truck traffic. We anticipate grade changes on the order of 8 feet may be required to develop finish site grades in the building and pavement areas.

SITE DESCRIPTION

The development parcel includes approximately 2.6 acres of vacant/undeveloped land identified as Lot 1C in the Airport South Replat C development, located at 3289 Airport Road in Boulder, Colorado. At the time of our site exploration, the ground surface at the site was vegetated with native weeds and grasses, with ground surface sloping moderately downward to the south/southwest overall. The maximum difference in ground surface elevation across the site was estimated to be approximately 20 feet based on review of available USGS topographic maps of the area. Evidence of prior building construction was not observed on the subject parcel by Soilogic personnel at the time of our site exploration.

SITE EXPLORATION

Field Exploration

To develop subsurface information for the proposed site improvements, a total of eight (8) soil borings were completed. Three (3) of these borings were completed as part of a preliminary subsurface exploration performed in 2016 for this site and outlined in a report dated February 5, 2016. The borings were advanced within or near the approximate building footprints to depths between approximately 15 to 30 feet below present site grade. The boring locations were established in the field by Soilogic, Inc. (Soilogic) personnel based on a provided site plan and by pacing and estimating angles from identifiable site references. A diagram indicating the approximate boring locations is included with this report. The boring locations outlined on the attached diagram should be considered accurate only to the degree implied by the methods used to make the field measurements. Graphic logs of each of the auger borings are also included.

The test holes were advanced using 4-inch diameter continuous-flight auger, powered by truck-mounted CME-45 and 55 drill rigs. Samples of the subsurface materials were obtained at regular intervals using California barrel sampling procedures in general accordance with ASTM specification D-1586. As part of the D-1586 sampling procedure, standard sampling barrels are driven into the substrata using a 140-pound hammer falling a distance of 30 inches. The number of blows required to advance the sampler a distance of 12 inches is recorded and helpful in estimating the consistency, relative density or hardness of the soils or bedrock encountered. In the California barrel sampling procedure, lesser disturbed samples are obtained in removable brass liners. Samples of the subsurface materials obtained in the field were sealed and returned to the laboratory for further evaluation, classification and testing.

Laboratory Testing

The samples collected were tested in the laboratory to measure natural moisture content and visually or manually classified in accordance with the Unified Soil Classification System (USCS). The USCS group symbols are indicated on the attached boring logs. An outline of the USCS classification system is included with this report. Classification of bedrock was completed through visual and tactual observation of disturbed samples. Other bedrock types could be revealed through petrographic analysis.

As part of the laboratory testing, a calibrated hand penetrometer (CHP) was used to estimate the unconfined compressive strength of essentially cohesive specimens. The CHP also provides a more reliable estimate of soil consistency than tactual observation alone. Dry density, Atterberg limits, -200 wash and swell/consolidation tests were completed on selected samples to help establish specific soil characteristics. Atterberg limits tests are used to determine soil and bedrock plasticity. The percent passing the #200 size sieve (-200 wash test) is used to determine the percentage of fine grained materials (clay and silt) in a sample. Swell/consolidation tests are performed to evaluate soil/bedrock volume change potential with variation in moisture content. The results of the completed laboratory tests are outlined on the attached boring logs and swell/consolidation test summaries. As part of the laboratory testing, water soluble sulfate (WSS) concentration tests are currently being completed on four (4) selected soil/bedrock samples to help evaluate corrosive soil characteristics with respect to buried concrete and results will be provided under separate cover once they become available.

SUBSURFACE CONDITIONS

Approximately 4 inches of vegetation and topsoil was encountered at the surface at the boring locations. At the location of borings B-4 and B-7, the vegetative soil layer was underlain by apparent fill soils consisting of brown/rust sand and gravel extending to depths ranging from approximately 3 to 3½ feet below present site grade. Light brown to brown/rust lean clay with varying amounts of silt, sand and scattered gravel was encountered underlying the apparent sand and gravel fill at the location of borings B-4 and B-7 and underlying the vegetative soil layer at the remainder of the completed site borings. The lean clay varied from very stiff to hard in terms of consistency, exhibited low to high swell potential at in-situ moisture and density conditions and extended to depths between about 1 to 9½ feet below ground surface. At the location of boring B-2, the lean clay extended to a depth of approximately 5 feet below ground surface and was underlain by rust/brown silty sand and gravel. The sand and gravel would be expected to be non-expansive or possess low swell based on the physical properties and engineering characteristics of the material and extended to a depth of approximately 7 feet below present site grade. The lean clay encountered in borings B-1 and B-3 through B-8 and silty sand and gravel encountered at the location of boring B-2 were underlain by rust/ olive/grey claystone bedrock. The claystone varied from weathered to very hard in terms of relative hardness, exhibited low to very high swell potential at in-situ moisture and density conditions and extended to the bottom of all borings at a depths ranging from approximately 15 to 30 feet below present site grade.

The stratigraphy indicated on the included boring logs represents the approximate location of changes in soil and bedrock types. Actual changes may be more gradual than those indicated.

Groundwater was not encountered in any of the completed site borings at the time of drilling. Groundwater levels will vary seasonally and over time based on weather conditions, site development, irrigation practices and other hydrologic conditions. Perched and/or trapped groundwater conditions may also be encountered at times throughout the year. Perched water is commonly encountered in soils overlying less permeable soil layers and/or bedrock. Trapped water is typically encountered within more permeable zones of layered soil and bedrock systems. The location and amount of perched/trapped water can also vary over time.

ANALYSIS AND RECOMMENDATIONS

General

The lean clay overburden soils and underlying claystone bedrock encountered relatively near-surface at this site exhibited low to very high swell potential at in-situ moisture and density conditions. Total and differential heaving of site improvements placed directly on or immediately above the expansive lean clay and claystone bedrock would be expected as the moisture content of those materials increases subsequent to construction.

In order to reduce the potential for movement of the proposed buildings in the expansive soil/bedrock environment, we recommend the structures be supported by drilled pier and grade beam foundation systems. Similar drilled pier foundations should be considered for support of site retaining walls. This type of system extends the foundation elements through expansive materials which are subjected to wetting and swelling and can place them in materials not as likely to experience significant moisture changes and resulting volume change. At the same time, drilled piers better concentrate building dead-loads, aiding in the resistance to uplift forces caused by expansive materials. We note, however, that there will remain some risk associated with building in areas of expansive bedrock. The risk of some movement and associated distress cannot be eliminated.

Swell-consolidation tests indicate that the lean clay and claystone bedrock likely to influence slab-on-grade construction exhibit variable swell potential, ranging from low to very high. For this site, we estimate total slab heave of 12 inches or more could be realized over time if deep wetting of the site occurs. Therefore, we recommend all garage and living area floors be constructed as structural floors, supported independent of the subgrade materials. Recommendations concerning drilled pier foundation design criteria and structural floor systems are outlined below.

In addition to the presence of expansive clay and claystone bedrock, approximately 3 to 3½ feet of apparent undocumented fill soils were encountered near surface at the location of borings B-4 and B-7. Undocumented fill soils would not be considered suitable for support of any overlying improvements and should be completely removed beneath all building, exterior flatwork, pavement and any proposed fill areas as part of site development.

Lean clay soils and claystone bedrock with high to very high swell potential will support exterior flatwork and pavement improvements at this site. We recommend all exterior flatwork and site pavements be supported on a minimum of five (5) feet of properly moisture conditioned and compacted overexcavation/backfill developed as outlined below in the 'Site Development' section of this report. Greater overexcavation depths could be considered to further reduce potential movement. Overexcavation areas should extend a minimum of 8 inches laterally past the exterior edges of exterior flatwork and site pavements for every 12 inches of overexcavation depth. The reconditioned mat will provide a zone of material immediately beneath exterior flatwork and site pavements which will have low potential for volume change subsequent to construction. The low volume-change (LVC) mat and surcharge loads placed on the underlying soils by the reconditioned mat would reduce the potential for total and differential movement of the supported improvements subsequent to construction. The reconditioned zone would also assist in distributing movement in the event that some swelling of the materials underlying the reconditioned zone occurs. It may be prudent from a constructability standpoint to complete all overexcavation/backfill procedures in these areas as part of site development and in advance of building construction.

As a lower-cost, higher-risk alternative, similar overexcavation/backfill procedures to those outlined above could be considered to develop LVC support conditions for conventional garage floor slab and retaining wall footing foundation construction, provided movement can be tolerated. Higher risk would include anticipated post-construction heaving of garage slabs and retaining walls and resultant cracking and faulting of garage floor slab and retaining wall concrete. There would be a higher likelihood of slab and retaining wall movement, and the need for post-construction remedial repairs. With a minimum of five (5) feet of overexcavation/backfill completed beneath garage floor slabs and retaining wall footing foundations, and assuming a 20-foot wetting depth, total slab and retaining wall footing foundation heave on the order of 10 inches is theoretically possible. If deeper wetting of the site than generally anticipated occurs, greater heave movements could result. If the amount of movement and associated types of distress outlined above cannot be tolerated, retaining walls should be supported on drilled pier foundations and structural garage floors utilized.

Site Development

All existing topsoil and vegetation should be removed from the building, pavement, exterior flatwork and any proposed fill areas. In addition, all undocumented fill soils should be completely removed at this time. After stripping and completing all cuts and exterior flatwork/pavement overexcavation, prior to placement of any fill, overexcavation/backfill or overlying improvements, we recommend the exposed subgrade soils be scarified to a depth of 9 inches, adjusted in moisture content and compacted to at least 95% of the materials standard Proctor maximum dry density. The moisture content of scarified lean clay soils should be adjusted to within -1 to +3% of standard Proctor optimum moisture content at the time of compaction. Scarified claystone bedrock subgrades should be adjusted to within the range of 0 to +4% of standard Proctor optimum moisture content at the time of compaction.

Fill and overexcavation/backfill soils required to develop the site should consist of approved LVC soils free from organic matter, debris and other objectionable materials. Based on the results of the completed laboratory testing, it is our opinion the site lean clay could be used as fill and overexcavation/backfill provided care is taken to develop the proper moisture content in those materials at the time of placement and compaction. Claystone bedrock should <u>not</u> be used as fill in any structural areas of the site. If it is necessary to import fill material to the site, those materials should have low potential for volume change, be relatively impervious and approved prior to use. Typically soils with a liquid limit less than 40 and plasticity index less than 18 and containing at least 25% fines (material passing the #200 size sieve) could be used as LVC fill. Essentially-granular structural fill and the cleaner site sand and gravel soils should also <u>not</u> be used as fill due to the high permeability and the ability of those materials to pond and transmit water. The site sand and gravel could be blended with overburden lean clay soils to reduce permeability making these soils suitable for use as fill and overexcavation/

backfill. We recommend the site lean clay and/or similar soils be placed in loose lifts not to exceed 9 inches thick, adjusted in moisture content and compacted as recommended for the scarified materials above. LVC import soils should be adjusted to within ±2% of the material's standard Proctor optimum moisture content and also compacted as outlined above.

Care should be taken to avoid disturbing the prepared subgrade soils and placed fill and overexcavation/backfill materials prior to construction of any overlying improvements. In addition, care should be taken to maintain the proper moisture content in the bearing/subgrade soils prior to concrete placement and/or paving. The prepared fill and overexcavation/backfill soils should not be left exposed for extended periods of time, or it may be necessary to add water to the surface of the developed subgrade soils intermittently until surfacing can be completed. Subgrade soils which are disturbed by construction activities or allowed to become wet and softened or dry and desiccated should be removed and replaced or reworked in place prior to placement of any fill or overlying improvements.

Inherent risks exist when building in areas of expansive soils/bedrock. The overexcavation/backfill procedures outlined above will reduce, but not eliminate, the potential for movement of exterior flatwork and site pavements subsequent to construction. The in-place materials below the moisture conditioned zone can increase in moisture content causing movement of the overlying improvements and some movement of exterior flatwork and site pavements should be expected. Care should be taken to ensure that when exterior flatwork and site pavements move, positive drainage will be maintained away from the structures.

Drilled Pier Foundations

We recommend drilled pier foundations extend a minimum of 15 feet into competent bedrock, with minimum shaft lengths of 33 feet and be designed using a maximum allowable end bearing pressure of 30 kips per square foot (ksf). An allowable skin friction value of 3,000 psf could be used for that portion of the pier extended into competent bedrock. Credit for skin friction should be neglected for the top 3 feet of bedrock penetration.

We recommend the drilled piers be designed to maintain a minimum dead-load pressure of 16 ksf based on the cross-sectional area of the piers. If the minimum recommended dead-load pressure cannot be achieved, increasing the minimum length and bedrock penetration requirements outlined above could be considered to compensate for the deficiency. An uplift skin friction resistance value of 2,000 psf could be used to calculate additional uplift resistance for the increase in pier length only.

Piers should be designed with full length steel reinforcement to help transmit any axial tension loads that may develop in the pier shaft. Uplift forces for use in reinforcing steel design can be calculated using the formula (F_{up} (kips) = 132 x D) where D is pier diameter in feet. A minimum 12-inch continuous void space should be constructed beneath the foundation grade beams to concentrate dead-load on the piers and allow for some movement of the subgrade soils to occur without transmitting stresses to the overlying structures. Voids should be formed using approved methods to prevent soil and debris from entering the void space. Void form material should be collapsible enough such that sufficient loads cannot be transmitted through the void form to mobilize the grade beams. Some type of backfill retainer should be employed on the exterior of the foundation wall in order to reduce the potential for foundation wall backfill soils entering the grade-beam void space.

For design of drilled piers to resist lateral loading, the horizontal modulus for varying pier diameters are outlined below in Table I. The values provided do not include a factor of safety.

TABLE I – HORIZONTAL MODULUS OF SUBGRADE REACTION (tons/ft^3)		
Pier Diameter (in)	Overburden Sandy Lean Clay Soils	Claystone Bedrock
12	100	250
18	67	167
24	50	125

When the lateral capacity of drilled piers/driven piles is evaluated by the L-Pile/Com64 design program, we recommend that internally generated load-deformation (p-y) curves be used. Piers or piles may be designed using the following lateral load criteria.

Parameters	Overburden Sandy Lean Clay Soils	Claystone Bedrock
In-Situ Unit Weight (pcf)	115 *	130
Angle of Internal Friction	15	20
Cohesion (psf)	1000	5000
Strain at 50%	.007	.005

* Reduce values by 62.4 pcf below groundwater table

Group reductions would apply if piers are spaced within three (3) pier diameters of each other. Piers in line with the direction of lateral load should be spaced a minimum of six (6) pier diameters center-to-center based on the largest diameter pier in the series. The horizontal modulus for initial and trailing piers should be reduced if spacing less than six (6) pier diameters is required.

Based on the materials encountered in the completed site borings, we expect pier excavations could be completed using conventional auguring techniques. If layers/zones of well-cemented bedrock are encountered at the time of caisson construction, specialized rock augers or core barrels may be required to fully penetrate these materials in order to achieve the minimum design lengths. Pier excavations would be expected to remain stable for short periods during construction such that we do not expect temporary casing of the drilled shafts would be required across a majority of the site. Casing or mudded excavations may be required in the area of boring B-2 where relatively clean sand and gravel soils were encountered overlying the bedrock unit. Groundwater was not encountered in the completed site borings at the time of drilling. If rapid/excessive groundwater infiltration is encountered at the time of caisson construction, a tremmie would be required to place pier concrete. A maximum three (3) inch water depth is acceptable at the bottom of pier excavations immediately prior to concrete placement. Free-fall concrete placement is only acceptable if provisions are taken to avoid striking the concrete on the sides of the caisson excavation and reinforcing steel.

Care will be needed to minimize the amount of sloughing/caving of the pier excavation side walls. Sloughed soils will need to be removed from the bottom of the pier excavations immediately prior to placement of reinforcing steel and pier concrete.

Pier concrete should have a slump in the range of 5 to 7 inches and be placed in the pier excavations immediately after the completion of drilling, cleaning and placement of reinforcing steel. Care should be taken in forming the upper edges of the pier excavation to avoid "mushrooming" at the top of the drilled pier excavations. Care will also be needed in the area of boring B-2 to avoid development of a concrete bulge in the layer of cleaner sand and gravel soils. The mushroom/bulge shapes would provide additional area for expansive soil uplift forces. Cylindrical cardboard forms or other approved means may be necessary to maintain a consistent upper shaft diameter.

We estimate long term settlement of the drilled caisson foundations designed and constructed as outlined above and resulting from the assumed structural loads would be less than ¾ of an inch.

Seismicity

Based on the results of our exploration and our review of the International Building Code (2003), a soil profile type C could be used for the site strata. Based on our review of United States Geologic Survey (USGS) mapped information, design spectral response acceleration values of S_{DS} = .178 (17.8%) and S_{D1} = .068 (6.8%) could be used.

Building Floors

In order to help reduce the potential for movement of both living area and garage floors, we recommend these floors be constructed as a structurally-supported floor over a void space. Building codes should be followed for clear space requirements below structurally-supported floors with crawl space areas and will depend, in part, upon the type of materials used to construct the floor, as well as the expansion potential of the underlying soils/bedrock. Clear spaces for these types of floors normally range from 18 to 24 inches. A larger crawl space area has the advantage of allowing maintenance of grade beam void spaces and sub floor utilities. Where other floor support systems and materials are used, we recommend a minimum clear space/void of 14 inches be maintained between the underside of the structural floor system and the surface of the subgrade/ exposed earth. It is prudent to maintain the minimum clear space/void below all plumbing lines. This can be accomplished by hanging plumbing on the underside of the structural floor between joists, or by trenching below the lines.

We recommend the subgrades in the voided/crawl space areas be sloped to drain to a perimeter drain system in case of water infiltration into the crawl space/void areas. A vapor barrier should be employed in the voided/crawl space areas in order to help maintain in-situ subgrade moisture conditions and reduce the potential for migration of soil moisture into the sub floor areas. It may be prudent to consult with a specialist in regard to mold prevention during design of the voided/crawl space area of the buildings.

Below Grade Construction

We recommend a perimeter drain system be installed around all below and at-grade voided/crawl space areas to help reduce the potential for development of hydrostatic pressures behind the foundation walls and water infiltration into the voided/crawl space areas. A perimeter drain system should consist of a four (4) inch diameter perforated drain pipe surrounded by a minimum of six (6) inches of free-draining gravel. A filter fabric should be installed around the free-draining gravel or perforated pipe to reduce the potential for an influx of fine-grained soils into the system. The drain pipe should be placed at approximate void space subgrade level around the exterior perimeters of the structures with a minimum slope of ⅛-inch per foot to facilitate efficient water removal and should be designed to discharge to a sump pump and pit system or free outfall. If free outfalls will be constructed, flapgates or other approved methods should be employed to reduce the potential for reverse flow and animal access into the systems.

Backfill placed adjacent to the foundation walls should consist of relatively impervious soils free from organic matter, debris and other objectionable materials. Based on results of the completed field and laboratory testing, it is our opinion the site lean clay could be used as foundation wall backfill provided care is taken to develop the proper moisture content in those materials at the time of placement and compaction. Claystone bedrock should not be used as foundation wall backfill. Cleaner sand and gravel soils should also not be used as foundation wall backfill due to the ability of those materials to pond and transmit water. As previously outlined, the cleaner sand and gravel soils could be blended with the overburden lean clay soils to reduce permeability making these materials suitable for use as backfill. Foundation wall backfill should contain a minimum of 25% fines in order to reduce permeability. We recommend the site lean clay and/or similar backfill soils be placed in loose lifts not to exceed 9 inches thick, adjusted in moisture content and compacted as outlined in the 'Site Development' section of this report.

Excessive lateral stresses can be imposed on the foundation walls when using heavier mechanical compaction equipment. We recommend compaction of unbalanced foundation wall backfill be completed using light mechanical or hand compaction equipment.

Lateral Earth Pressures

For design of unbraced and unilaterally-loaded foundation walls where preventative measures have been taken to reduce the potential for development of hydrostatic loads on the walls, we recommend using an active equivalent fluid pressure value of 50 pounds per cubic foot. Some rotation of the foundation walls must occur to develop the active earth pressure state. That rotation can result in cracking of the walls typically in between corners and other restrained points. The amount of deflection of the top of the wall can be estimated at 0.5% times the height of the wall. An equivalent fluid pressure value of 70 pounds per cubic foot could be used for restrained conditions.

Variables that affect lateral earth pressures include but are not limited to the shrink/swell potential of the backfill soils, backfill compaction and geometry, wetting of the backfill soils, surcharge loads and point loads developed in the backfill materials. The recommended equivalent fluid pressure values do not include a factor of safety or an allowance for hydrostatic loads. Use of expansive soil backfill, excessive compaction of the wall backfill or surcharge loads placed adjacent to the foundation walls can add to the lateral earth pressures causing the equivalent fluid pressure values used in design to be exceeded.

Conventional Garage Floor Slab Construction

Provided the owner is willing to accept the increased risk associated with overexcavation/ backfill procedures as outlined above, the garage floor slabs could be supported directly on suitable overexcavation/backfill soils placed and compacted as previously outlined in the "Site Development" section of this addendum report.

Floor slabs should designed and constructed as floating slabs, separated from foundation walls, columns and plumbing and mechanical penetrations by the use of block outs or appropriate isolation material. As a precaution, we recommend omitting partition walls supported above garage slabs. If included in the design, partition walls should be constructed as floating walls to help reduce the potential for differential slab to foundation movement causing distress in upper sections of the building. A minimum six (6) inch void space should be developed beneath all partition walls. Frequent monitoring of these void spaces should be completed to ensure that a sufficient space is maintained throughout the life of the structures. Special attention to door and stair framing, drywall installation, trim carpentry and garage door tracks should be taken to isolate those elements from the floor slabs, allowing for some differential floor slab-to-foundation movement to occur without transmitting stresses to the overlying structure.

Depending on the type of floor covering and floor covering adhesive used in finished slab-on-grade areas (if any), a vapor barrier may be required immediately beneath the floor slabs in order to maintain flooring product manufacturer warranties. A vapor barrier would help reduce the transmission of moisture through the floor slab. However, the unilateral moisture release caused by placing concrete on an impermeable surface can increase slab curl. The amount of slab curl can be reduced by careful selection of an appropriate concrete mix, however, slab curl cannot be eliminated. We recommend the owner, architect and flooring contractor consider the performance of the slab, in conjunction with the proposed flooring products to help determine if a vapor barrier will be required and where best to position the vapor barrier in relation to the floor slab. Additional guidance and recommendations concerning slab-on-grade design can be found in American Concrete Institute (ACI) section 302.

Pavements

Site pavements could be supported directly on suitable overexcavation/backfill soils placed and compacted as outlined above. The site lean clay soils would be subject to low remolded shear strength. A resistance value (R-value) of 5 was estimated for the pavement subgrade soils and used in pavement section design. Traffic loading on site pavements is expected to consist of areas of low volumes of automobiles and light trucks, as well as areas of higher light vehicle traffic volumes and occasional heavier trash, delivery and emergency vehicle traffic. Equivalent 18-kip single axle loads (ESAL's) were estimated for the quantity of site traffic anticipated. Two (2) general design classifications are outlined below in Table I. Standard duty pavements could be considered in automobile drive and parking areas. Heavy duty pavements should be considered for access drives and other areas of the site expected to receive higher traffic volumes or heavier trash, delivery and emergency truck traffic.

After completing the overexcavation/backfill procedures as outlined above, the developed subgrade soils may need to be stabilized prior to asphaltic concrete paving. At the high end of the recommended moisture content range, some instability of the subgrade soils would be expected. Proofrolling of the pavement subgrades should be completed to help identify unstable areas. Areas which pump or deform excessively should be mended prior to aggregate base course and asphaltic concrete placement. Isolated areas of subgrade instability can be mended on a case-by-case basis. If more widespread areas of subgrade instability are observed we recommend consideration be given to stabilization of the pavement subgrades with Class C fly ash. With the increase in support strength developed by the fly ash stabilization procedures, it is our opinion some credit for the stabilized zone could be included in the pavement section design, reducing the required thickness of overlying asphaltic concrete and aggregate base course. Fly ash stabilization can also eliminate some of the uncertainty associated with attempting to pave during periods of inclement weather. Pavement section design options incorporating some structural credit for the fly ash-stabilized subgrade soils are outlined below in Table I.

TABLE I – PAVEMENT SECTION DESIGN		
	Standard Duty	**Heavy Duty**
Option A – Composite		
Asphaltic Concrete (Grading S or SX)	4”	5”
Aggregate Base (Class 5 or 6)	6”	8”
Option B – Composite on Stabilized Subgrade		
Asphaltic Concrete (Grading S or SX)	3”	4”
Aggregate Base (Class 5 or 6)	4”	6”
Fly Ash Stabilized Subgrade	12”	12”
Option C - Portland Cement Concrete Pavement		
PCCP	5”	6”

Asphaltic concrete should consist of a bituminous plant mix composed of a mixture of aggregate, filler, binders and additives (if required) meeting the design requirements of the City of Boulder. Aggregate used in the asphaltic concrete should meet specific gradation requirements such as Colorado Department of Transportation (CDOT) grading S (¾-inch minus) or SX (½-inch minus) specifications. Hot mix asphalt designed using “Superpave” criteria should be compacted to within 92 to 96% of the materials Maximum Theoretical Density. Aggregate base should be consistent with CDOT requirements for Class 5 or Class 6 aggregate base, placed in loose lifts not to exceed 9 inches thick and compacted to at least 95% of the materials standard Proctor maximum dry density.

If fly ash stabilization procedures will be completed, we recommend the addition of 12% Class ‘C’ fly ash based on component dry unit weights. A 12-inch thick stabilized zone should be constructed by thoroughly blending the fly ash with the in-place subgrade soils. Some “fluffing” of the finish subgrade level should be expected with the stabilization procedures. The blended materials should be adjusted in moisture content to within the range of ±2% of standard Proctor optimum moisture content and compacted to at least 95% of the material’s standard Proctor maximum dry density within two (2) hours of fly ash addition.

For areas subjected to truck turning movements and/or concentrated and repetitive loading such as dumpster or truck parking and loading areas, we recommend consideration be given to the use of Portland cement concrete pavement with a minimum thickness of 6 inches. The concrete used for site pavements should be entrained and have a minimum 28-day compressive strength of 4,000 psi. Woven wire mesh or fiber entrained concrete should be considered to help in the control of shrinkage cracking.

The proposed pavement section designs do not include an allowance for excessive loading conditions imposed by heavy construction vehicles or equipment. Heavily loaded concrete or other building material trucks and construction equipment can cause some localized distress to site pavements. The recommended pavement sections are minimums and periodic maintenance efforts should be expected. A preventative maintenance program can help increase the service life of site pavements.

Site Retaining Walls

Retaining walls are anticipated on the site to facilitate development of finish grades. For design of retaining wall footing foundations bearing on a minimum of five (5) feet of suitable overexcavation/backfill developed as outlined above, we recommend using a maximum net allowable soil bearing pressure of 1,500 psf. The retaining wall footing foundation should bear a minimum of 30 inches below grade at the front of the wall to provide frost protection. Assuming a 20-foot wetting depth, total and differential heaving of retaining wall footing foundations on the order of ten (10) inches is theoretically possible even if five (5) feet of overexcavation/backfill is developed beneath the walls. Greater movements are possible if deeper wetting of retaining wall areas occurs. If this amount of retaining wall movement cannot be tolerated, retaining walls should be supported on drilled pier foundations designed utilizing the parameters outlined in the 'Drilled Pier Foundations' section of this report.

Care should be taken to prevent the development of unbalanced hydrostatic loads on all retaining walls. A drainage blanket consisting of 12 inches of free-draining rock placed behind the wall and extending the full height of the wall from approximate grade at the front of the wall to approximately 12 to 18 inches below finish grade on the retained soil side of the wall should be constructed. We recommend ¾-inch or larger washed rock be used to construct the drainage blanket. The top 12 to 18 inches of retaining wall backfill should consist of an essentially cohesive soil to reduce the potential for immediate surface water infiltration into the wall backfill. A filter fabric should be employed

between all free-draining aggregate and adjacent soil interfaces to reduce the potential for the migration of finer-grained soils into the gap-graded rock.
Weep holes or other approved methods should be employed to help transfer any collected water to the front of the wall. A water collection system, similar to a perimeter drain system could also be considered. A typical collection drain system would consist of 4-inch diameter rigid perforated pipe surrounded by a minimum of 6 inches of the free-draining aggregate and placed at the base of base of soil retention on the retained soil side of the wall. The invert of the drain pipe at the high point of the system should be placed at approximate front-of-wall grade and sloped a minimum of ⅛-inch per foot to facilitate efficient water removal to an appropriate outfall. Flap gates or other approved methods should be employed at all free outfalls to reduce the potential for animal access and reverse flow in the system.

Retaining wall backfill should consist of approved low-volume-change (LVC) and essentially granular materials free from organic matter and debris. Essentially-granular soils offer better stacking characteristics and are less prone to movements associated with freezing through the face of the walls than finer-grained materials. Materials consistent with Colorado Department of Transportation (CDOT) Class 7 aggregate base course or Class I structure backfill could be used as retaining wall backfill. Retaining wall backfill should be placed in loose lifts not to exceed 9 inches thick, adjusted in moisture content and compacted to at least 95% of the materials standard Proctor maximum dry density. The moisture content of the backfill soils should be adjusted to within ±2% of standard Proctor optimum moisture content at the time of compaction.

Excessive lateral stresses can be imposed on retaining walls during backfilling when using heavier mechanical compaction equipment. We recommend compaction of retaining wall backfill be completed using light mechanical or hand compaction equipment.

For design of retaining walls protected from hydrostatic loading and backfilled with select granular fill, we recommend using an angle of internal friction of Φ=30° and active equivalent fluid pressure value of 40 pounds per cubic foot in addition to any surcharge loads. The equivalent fluid pressure value outlined above is based on an active stress distribution analysis in which some rotation of the retaining wall is assumed. The angle of internal friction and equivalent fluid pressure values outlined above do not include a factor of safety. Sloped backfill geometry, surcharge loads on the retained soil side of the walls or point loads developed in the wall backfill can add to the lateral forces on retaining walls.

The lateral driving forces on the walls will be resisted through a combination of the sliding friction of the footing foundations and passive earth pressure against the embedded portion of the wall below frost depth. A passive equivalent fluid pressure value of 250 pcf could be used for that portion of the wall extended below frost depth, considered to be 30 inches in this area. A coefficient of friction of 0.35 could be used between foundation concrete and the bearing soils to resist sliding. The recommended passive equivalent fluid pressure value and coefficient of friction do not include a factor of safety.

Drainage

Positive drainage is imperative for satisfactory long-term performance of the proposed buildings and associated site improvements. With the generally high to very high swell potential of the subsurface soils and bedrock determined in laboratory testing, extra care will be needed at this site in the planning and design of surface drainage. We recommend positive drainage be developed away from the structures during construction and maintained throughout the life of the site improvements, with twelve (12) inches of fall in the first 10 feet away from the buildings. Shallower slopes could be considered in hardscape areas. In the event that poor or negative drainage develops adjacent to the structures over time, the original grade and associated positive drainage outlined above should be immediately restored.

Extra care should be taken at this site in the planning, design, control and maintenance of landscape watering systems to avoid features which could result in the fluctuation of the moisture content of all site soils underlying building, flatwork and pavement elements. Use of xeriscaping and/or landscape features which do not require irrigation water should be considered. We recommend watering systems be placed a minimum of 5 and preferably 10 feet away from the perimeter of the site structures and be designed to discharge away from all site improvements. Gutter systems should be considered to help reduce the potential for water ponding adjacent to the structures, exterior flatwork and pavements, with the gutter downspouts, roof drains or scuppers extended to discharge a minimum of 5 and preferably 10 feet away from structural, flatwork and pavement elements. Water which is allowed to pond adjacent to the site improvements can result in unsatisfactory performance of those improvements over time.

LIMITATIONS

This report was prepared based upon the data obtained from the completed site explorations, laboratory testing, engineering analysis and any other information discussed. The completed borings provide an indication of subsurface conditions at the boring locations only. Variations in subsurface conditions can occur in relatively short distances away from the borings. This report does not reflect any variations which may occur across the site or away from the borings. If variations in the subsurface conditions anticipated become evident, the geotechnical engineer should be notified immediately so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any biological or environmental assessment of the site or identification or prevention of pollutants or hazardous materials or conditions. Other studies should be completed if concerns over the potential of such contamination or pollution exist.

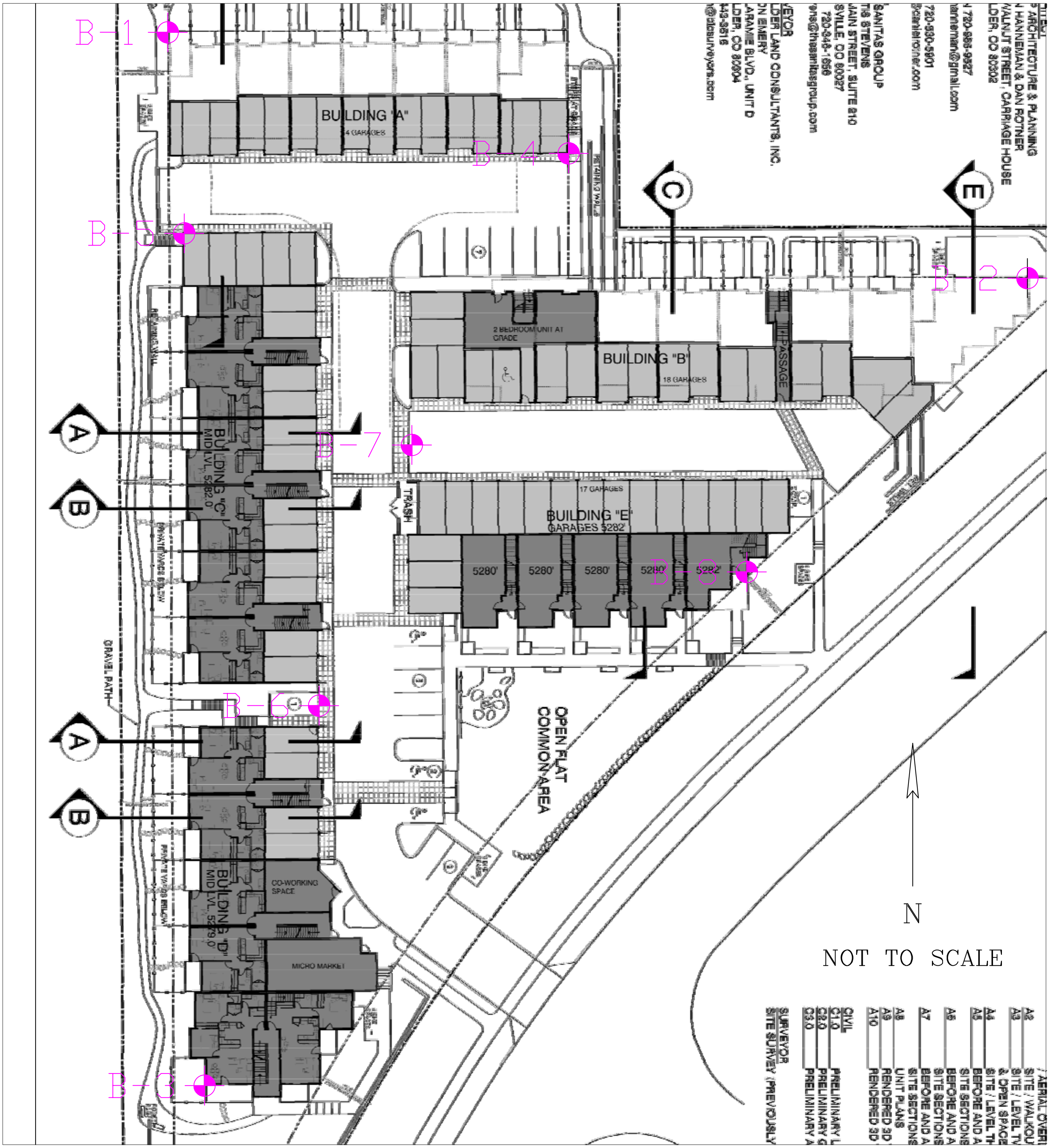
The geotechnical engineer should be retained to review the plans and specifications so that comments can be made regarding the interpretation and implementation of our geotechnical recommendations in the design and specifications. The geotechnical engineer should also be retained to provide testing and observation services during construction to help determine that the design requirements are fulfilled.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with the generally accepted standard of care for the profession. No warranties express or implied, are made. The conclusions and recommendations contained in this report should not be considered valid in the event that any changes in the nature, design or location of the project as outlined in this report are planned, unless those changes are reviewed and the conclusions of this report modified and verified in writing by the geotechnical engineer.

AUGUST 2018
PROJECT #16-1002

BORING LOCATION DIAGRAM

SOILOGIC



VELO PARK APARTMENTS

3289 AIRPORT ROAD, BOULDER, COLORADO

VELO PARK APARTMENTS

3289 AIRPORT ROAD, BOULDER, COLORADO

Project # 16-1002

February 2016

LOG OF BORING B-1

Sheet	1/1	Drilling Rig:	CME 55	Water Depth Information	
Start Date	1/21/2016	Auger Type:	4" CFA	During Drilling	None
Finish Date	1/21/2016	Hammer Type:	Manual	After Drilling	None
Surface Elev.	-	Field Personnel:	BMc	2 Weeks After Drilling	None

USCS	SOIL DESCRIPTION	Depth (ft)	Sampler	"N"	MC (%)	DD (pcf)	Estimated q_u (psf)	% Swell @ 500 psf	Swell Pressure (psf)	Atterberg Limits LL	Atterberg Limits PI	% Passing # 200 Sieve (%)
	0 - 4" VEGETATION & TOPSOIL	-										
CL	SANDY LEAN CLAY	1										
	light brown/rust	-										
	hard	2										
		-										
	With Scattered Gravel	3	CS	38	11.9	106.9	9000+	2.3%	3000	-	-	-
		-										
		4										
		-										
		5	CS	50/5	12.2	116.7	9000+	1.5%	2250	-	-	-
		-										
		6										
		-										
		7										
		-										
		8										
		-										
	CLAYSTONE	9										
	grey/rust/olive/brown	-										
	very hard	10	CS	50/4	16.4	116.4	9000+	2.1%	4000	-	-	-
		-										
	With Interbedded Siltstone/Sandstone	11										
		-										
		12										
		-										
		13										
		-										
		14										
		-										
		15	CS	50/4	13.0	111.8	9000+	-	-	-	-	-
		-										
		16										
		-										
		17										
		-										
		18										
		-										
		19										
		-										
		20	CS	50/2	13.6	109.0	9000+	-	-	-	-	-
		-										
		21										
		-										
		22										
		-										
		23										
		-										
		24										
		-										
		25										
		-										
		26										
		-										
		27										
		-										
		28										
		-										
		29										
		-										
	BOTTOM OF BORING @ 30.0'	30	CS	50/2	12.3	101.4	9000+	-	-	-	-	-

VELO PARK APARTMENTS

3289 AIRPORT ROAD, BOULDER, COLORADO

Project # 16-1002

February 2016

LOG OF BORING B-2

Sheet	1/1	Drilling Rig:	CME 55	Water Depth Information	
Start Date	1/21/2016	Auger Type:	4" CFA	During Drilling	None
Finish Date	1/21/2016	Hammer Type:	Manual	After Drilling	None
Surface Elev.	-	Field Personnel:	BMc	2 Weeks After Drilling	None

USCS	SOIL DESCRIPTION	Depth (ft)	Sampler	"N"	MC (%)	DD (pcf)	Estimated q_u (psf)	% Swell @ 500 psf	Swell Pressure (psf)	Atterberg Limits LL	Atterberg Limits PI	% Passing # 200 Sieve (%)
	0 - 4" VEGETATION & TOPSOIL	-										
		1										
		-										
CL	SANDY LEAN CLAY	2										
	light brown/rust	-										
	very stiff	3										
		-										
	With Scattered Gravel	4										
		-										
		5	**CS**	**26**	**4.1**	**-**	**9000+**	**-**	**-**	**-**	**-**	**-**
		-										
SP-SM	SILTY SAND AND GRAVEL	6										
GP-GM	rust/brown	-										
		7										
		-										
		8										
		-										
	CLAYSTONE	9										
	grey/rust/olive/brown	-										
	very hard	**10**	**CS**	**50/6**	**19.2**	**112.8**	**9000+**	**1.8%**	**3000**	**-**	**-**	**-**
		-										
	With Interbedded Siltstone/Sandstone	11										
		-										
		12										
		-										
		13										
		-										
		14										
		-										
		15	**CS**	**50/4**	**16.0**	**100.6**	**9000+**	**-**	**-**	**-**	**-**	**-**
		-										
		16										
		-										
		17										
		-										
		18										
		-										
		19										
		-										
		20	**CS**	**50/3**	**16.0**	**109.7**	**9000+**	**-**	**-**	**-**	**-**	**-**
		-										
		21										
		-										
		22										
		-										
		23										
		-										
		24										
		-										
		25										
		-										
		26										
		-										
		27										
		-										
		28										
		-										
		29										
		-										
	BOTTOM OF BORING @ 30.0'	**30**	**CS**	**50/3**	**12.5**	**105.4**	**9000+**	**-**	**-**	**-**	**-**	**-**

VELO PARK APARTMENTS

3289 AIRPORT ROAD, BOULDER, COLORADO

Project # 16-1002

February 2016

LOG OF BORING B-3

Sheet	1/1	Drilling Rig:	CME 55	Water Depth Information	
Start Date	1/21/2016	Auger Type:	4" CFA	During Drilling	None
Finish Date	1/21/2016	Hammer Type:	Manual	After Drilling	None
Surface Elev.	-	Field Personnel:	BMc	2 Weeks After Drilling	None

USCS	SOIL DESCRIPTION	Depth (ft)	Sampler	"N"	MC (%)	DD (pcf)	Estimated q_u (psf)	% Swell @ 500 psf	Swell Pressure (psf)	Atterberg Limits LL	Atterberg Limits PI	% Passing # 200 Sieve (%)
	0 - 4" VEGETATION & TOPSOIL	-										
		1										
		-										
CL	SANDY LEAN CLAY	2										
	light brown/rust	-										
	very stiff to hard	3	CS	46	10.7	106.9	9000+	0.2%	800	-	-	-
		-										
	With Scattered Gravel	4										
		-										
		5	CS	35	9.3	123.1	9000+	1.1%	2750	31	17	51.3%
		-										
		6										
		-										
		7										
		-										
		8										
		-										
	CLAYSTONE	9										
	grey/rust/olive/brown	-										
	very hard	10	CS	50/5	15.6	115.2	9000+	1.8%	3250	-	-	-
		-										
	With Interbedded Siltstone/Sandstone	11										
		-										
		12										
		-										
		13										
		-										
		14										
		-										
		15	CS	50/3	11.6	114.6	9000+	-	-	-	-	-
		-										
		16										
		-										
		17										
		-										
		18										
		-										
		19										
		-										
		20	CS	50/3	11.7	111.9	9000+	-	-	-	-	-
		-										
		21										
		-										
		22										
		-										
		23										
		-										
		24										
		-										
		25										
		-										
		26										
		-										
		27										
		-										
		28										
		-										
		29										
		-										
	BOTTOM OF BORING @ 30.0'	30	CS	50/2	14.1	105.8	9000+	-	-	-	-	-

VELO PARK APARTMENTS

3289 AIRPORT ROAD, BOULDER, COLORADO
Project # 16-1002
August 2018

LOG OF BORING B-4

SOILOGIC

Sheet	1/1	Drilling Rig:	CME 45	Water Depth Information	
Start Date	8/9/2018	Auger Type:	4" CFA	During Drilling	None
Finish Date	8/9/2018	Hammer Type:	Automatic	After Drilling	None
Surface Elev.	-	Field Personnel:	ZG/BM	24 Hours After Drilling	-

USCS	SOIL DESCRIPTION	Depth (ft)	Sampler	"N"	MC (%)	DD (pcf)	Estimated q_u (psf)	% Swell @ 500 psf	Swell Pressure (psf)	Atterberg Limits LL	Atterberg Limits PI	% Passing # 200 Sieve (%)
	0-4" VEGETATION AND TOPSOIL	-										
		1										
SP-GP	FILL - SAND AND GRAVEL	-										
	brown/rust	2										
	medium dense	-										
		3	CS	42	3.4	105.6	9000+	-	-	-	-	-
		-										
		4										
		-										
		5	CS	26	10.6	99.5	9000+	4.0%	3400	-	-	-
CL	SANDY LEAN CLAY	-										
	brown/grey	6										
	very stiff	-										
	With Gravel	7										
		-										
		8										
		-										
		9										
		-										
		10	CS	22	12.4	116.6	9000+	9.6%	16000	-	-	-
		-										
	CLAYSTONE	11										
	rust/olive/grey	-										
	weathered to medium hard	12										
		-										
		13										
		-										
		14										
		-										
		15	CS	50/10	16.1	117.4	9000+	-	-	-	-	-
	BOTTOM OF BORING @ 15.0'	-										
		16										
		-										
		17										
		-										
		18										
		-										
		19										
		-										
		20										
		-										
		21										
		-										
		22										
		-										
		23										
		-										
		24										
		-										
		25										

VELO PARK APARTMENTS

3289 AIRPORT ROAD, BOULDER, COLORADO
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August 2018

LOG OF BORING B-5

Sheet	1/1	Drilling Rig:	CME 45	Water Depth Information	
Start Date	8/9/2018	Auger Type:	4" CFA	During Drilling	None
Finish Date	8/9/2018	Hammer Type:	Automatic	After Drilling	None
Surface Elev.	-	Field Personnel:	ZG/BM	24 Hours After Drilling	-

USCS	SOIL DESCRIPTION	Depth (ft)	Sampler	"N"	MC (%)	DD (pcf)	Estimated q_u (psf)	% Swell @ 500 psf	Swell Pressure (psf)	Atterberg Limits LL	Atterberg Limits PI	% Passing # 200 Sieve (%)
	0-4" VEGETATION AND TOPSOIL	-										
		1										
		-										
CL	SANDY LEAN CLAY	2										
	brown/olive	-										
	very stiff	3										
		-										
		4										
		-										
		5	**CS**	**26**	**15.0**	**101.9**	**9000+**	**7.6%**	**5700**	**-**	**-**	**-**
		-										
		6										
		-										
		7										
		-										
		8										
		-										
		9										
		-										
	CLAYSTONE	**10**	**CS**	**50/11**	**14.2**	**115.4**	**9000+**	**5.5%**	**5700**	**-**	**-**	**-**
	rust/olive/grey	-										
	medium hard	11										
		-										
		12										
		-										
		13										
		-										
		14										
		-										
		15	**CS**	**50**	**15.2**	**118.3**	**9000+**	**-**	**-**	**-**	**-**	**-**
	BOTTOM OF BORING @ 15.0'	-										
		16										
		-										
		17										
		-										
		18										
		-										
		19										
		-										
		20										
		-										
		21										
		-										
		22										
		-										
		23										
		-										
		24										
		-										
		25										

VELO PARK APARTMENTS

3289 AIRPORT ROAD, BOULDER, COLORADO
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August 2018

LOG OF BORING B-6

Sheet	1/1	Drilling Rig:	CME 45	Water Depth Information	
Start Date	8/9/2018	Auger Type:	4" CFA	During Drilling	None
Finish Date	8/9/2018	Hammer Type:	Automatic	After Drilling	None
Surface Elev.	-	Field Personnel:	ZG/BM	24 Hours After Drilling	-

USCS	SOIL DESCRIPTION	Depth (ft)	Sampler	"N"	MC (%)	DD (pcf)	Estimated q_u (psf)	% Swell @ 500 psf	Swell Pressure (psf)	Atterberg Limits LL	Atterberg Limits PI	% Passing # 200 Sieve (%)
	0-4" VEGETATION AND TOPSOIL	-										
CL	SANDY LEAN CLAY brown very stiff With Gravel	1										
		2										
		3	CS	25	7.4	117.1	9000+	3.5%	3800	-	-	-
		4										
		5	CS	29	6.7	106.9	9000+	2.4%	-	37	18	32.2%
		6										
		7										
	CLAYSTONE rust/olive/grey hard	8										
		9										
		10	CS	50/9	10.5	124.8	9000+	11.1%	>16,000	-	-	-
		11										
		12										
		13										
		14										
		15	CS	50/8	14.3	120.1	9000+	-	-	-	-	-
	BOTTOM OF BORING @ 15.0'	16										
		17										
		18										
		19										
		20										
		21										
		22										
		23										
		24										
		25										

VELO PARK APARTMENTS

3289 AIRPORT ROAD, BOULDER, COLORADO
Project # 16-1002
August 2018

LOG OF BORING B-7

SOILOGIC

Sheet	1/1	Drilling Rig:	CME 45	Water Depth Information	
Start Date	8/9/2018	Auger Type:	4" CFA	During Drilling	None
Finish Date	8/9/2018	Hammer Type:	Automatic	After Drilling	None
Surface Elev.	-	Field Personnel:	ZG/BM	24 Hours After Drilling	-

USCS	SOIL DESCRIPTION	Depth (ft)	Sampler	"N"	MC (%)	DD (pcf)	Estimated q_u (psf)	% Swell @ 500 psf	Swell Pressure (psf)	Atterberg Limits LL	Atterberg Limits PI	% Passing # 200 Sieve (%)
	0-4" VEGETATION AND TOPSOIL	-										
SP-GP	FILL - SAND AND GRAVEL brown/rust	1 - 2 - 3										
CL	SANDY LEAN CLAY tan hard	- 4 -										
		5	CS	37	11.1	96.2	9000+	2.3%	1800	-	-	-
		- 6										
	CLAYSTONE rust/olive/grey medium hard to very hard	- 7 - 8 - 9 -										
		10	CS	50	15.3	118.6	9000+	-	-	-	-	-
		- 11 - 12 - 13 - 14 -										
		15	CS	50/8	14.5	118.8	9000+	4.1% @ 1000	9400	-	-	-
		- 16 - 17 - 18 - 19 -										
		20	CS	50/7	14.6	118.8		-	-	-	-	-
		- 21 - 22 - 23 - 24 - 25 - 26 - 27 - 28 - 29 -										
	BOTTOM OF BORING @ 30.0'	30	CS	50/5	14.2	116.9		-	-	-	-	-

VELO PARK APARTMENTS

3289 AIRPORT ROAD, BOULDER, COLORADO
Project # 16-1002
August 2018

LOG OF BORING B-8

Sheet	1/1	Drilling Rig:	CME 45	Water Depth Information	
Start Date	8/9/2018	Auger Type:	4" CFA	During Drilling	None
Finish Date	8/9/2018	Hammer Type:	Automatic	After Drilling	None
Surface Elev.	-	Field Personnel:	ZG/BM	24 Hours After Drilling	-

USCS	SOIL DESCRIPTION	Depth (ft)	Sampler	"N"	MC (%)	DD (pcf)	Estimated q_u (psf)	% Swell @ 500 psf	Swell Pressure (psf)	Atterberg Limits LL	Atterberg Limits PI	% Passing # 200 Sieve (%)
	0-4" VEGETATION AND TOPSOIL	-										
CL	SANDY LEAN CLAY WITH GRAVEL	1										
		-										
		2										
	CLAYSTONE	-										
	rust/olive/grey	3	CS	26	11.5	119.9	9000+	8.4%	14400	-	-	-
	firm to very hard	-										
		4										
		-										
		5	CS	34	11.8	116.6	9000+	-	-	44	23	93.5%
		-										
		6										
		-										
		7										
		-										
		8										
		-										
		9										
		-										
		10	CS	50/9	13.7	116.8	9000+	5.5%	-	-	-	-
		-										
		11										
		-										
		12										
		-										
		13										
		-										
		14										
		-										
		15	CS	50/5	13.2	119.2	9000+	-	-	-	-	-
	BOTTOM OF BORING @ 15.0'	-										
		16										
		-										
		17										
		-										
		18										
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		25										

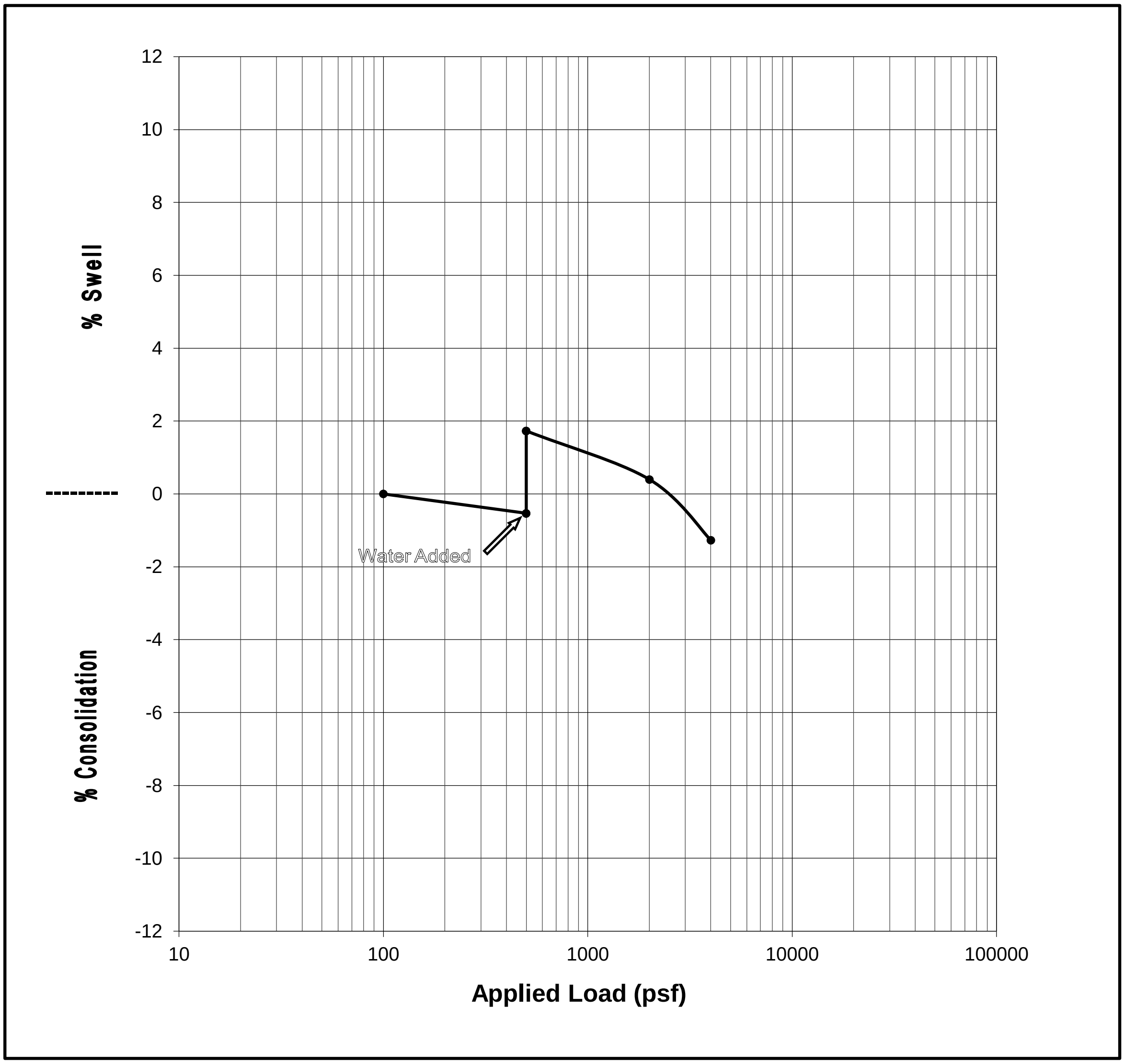
VELO PARK APARTMENTS

3289 AIRPORT ROAD, BOULDER, COLORADO

Project # 16-1002

February 2016

SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-1 @ 2'

Sample Description: Light Brown/Rust Sandy Lean Clay with Scattered Gravel (CL)

Initial Moisture	11.9%	Liquid Limit	-
Final Moisture	23.4%	Plasticity Index	-
% Swell @ 500 psf	2.3%	% Passing #200	-
Swell Pressure (psf)	3,000	Dry Density (pcf)	106.9

VELO PARK APARTMENTS

3289 AIRPORT ROAD, BOULDER, COLORADO

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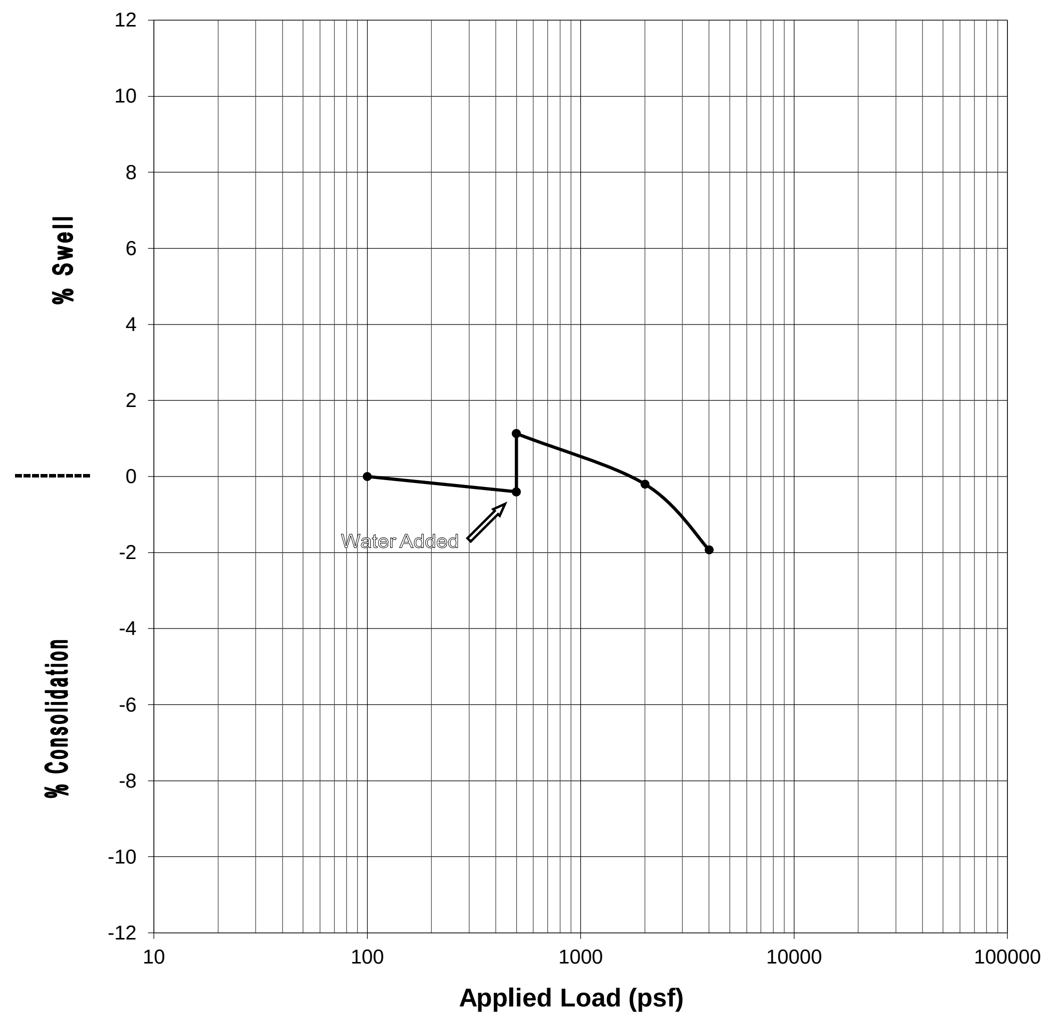
February 2016

SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-1 @ 4'

Sample Description: Light Brown/Grey Claystone with Interbedded Siltstone/Sandstone

Initial Moisture	12.2%	Liquid Limit	-
Final Moisture	20.2%	Plasticity Index	-
% Swell @ 500 psf	1.5%	% Passing #200	-
Swell Pressure (psf)	2,250	Dry Density (pcf)	116.7

VELO PARK APARTMENTS

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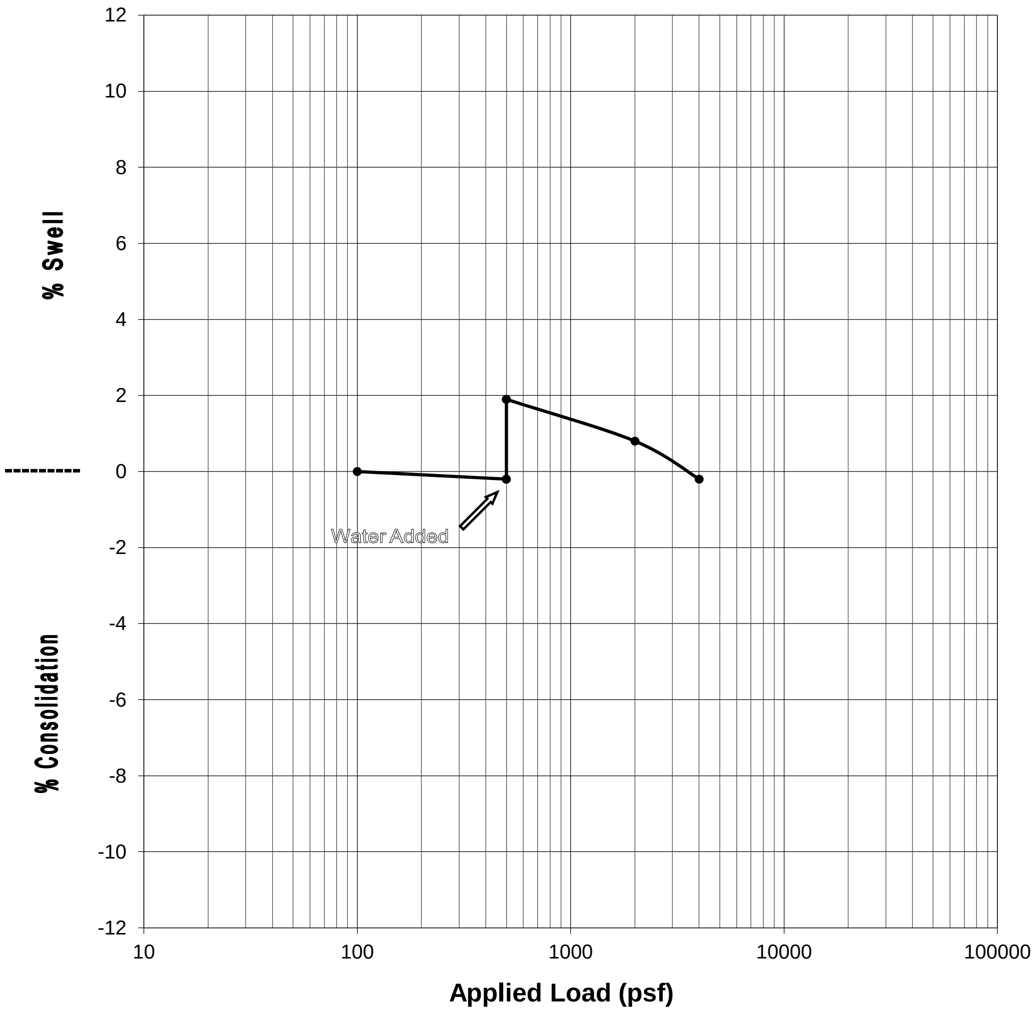
SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-1 @ 9'

Sample Description: Light Brown/Grey Claystone with Interbedded Siltstone/Sandstone

Initial Moisture	16.4%	Liquid Limit	-
Final Moisture	18.2%	Plasticity Index	-
% Swell @ 500 psf	2.1%	% Passing #200	-
Swell Pressure (psf)	4,000	Dry Density (pcf)	116.4

VELO PARK APARTMENTS

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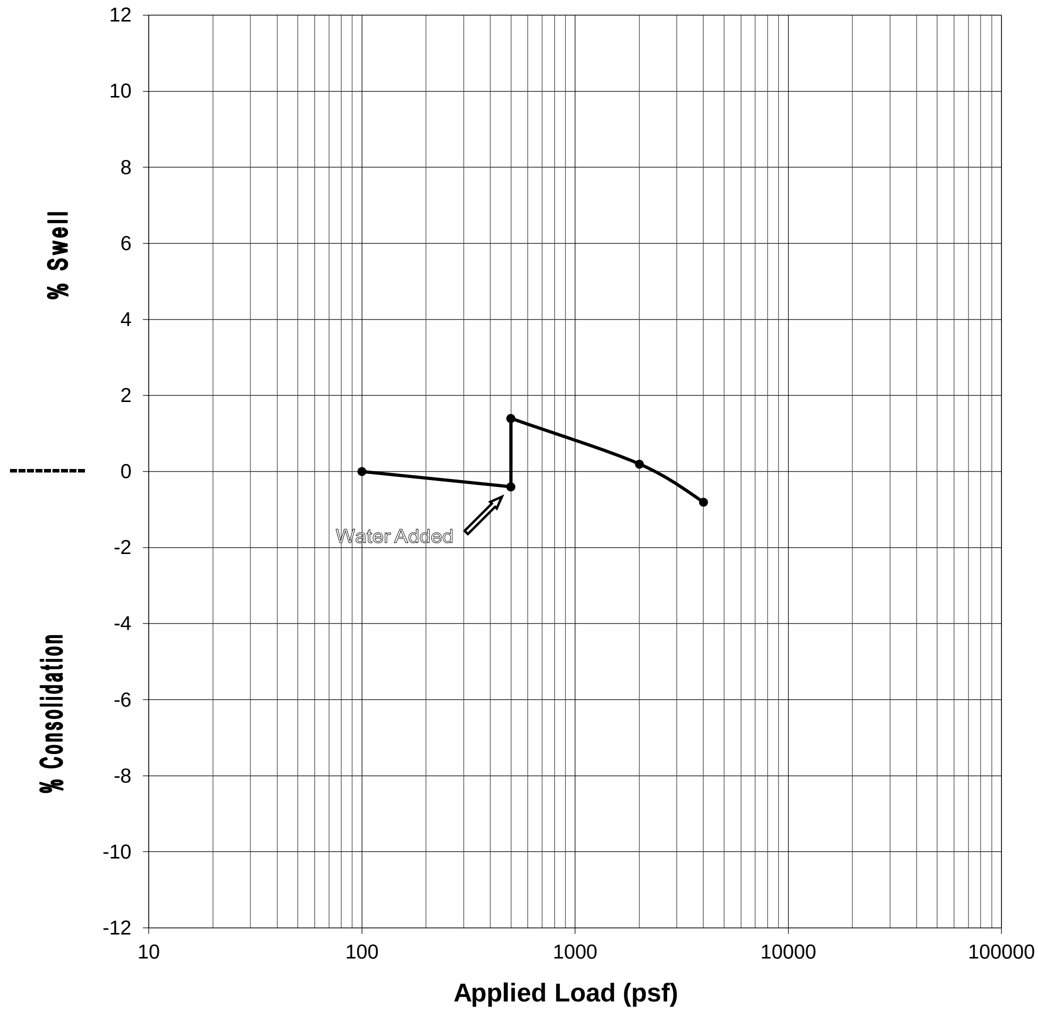
SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-2 @ 4'

Sample Description: Grey/Rust/Olive/Brown Claystone with Interbedded Siltstone/Sandstone

Initial Moisture	19.2%	Liquid Limit	-
Final Moisture	19.3%	Plasticity Index	-
% Swell @ 500 psf	1.8%	% Passing #200	-
Swell Pressure (psf)	3,000	Dry Density (pcf)	112.8

VELO PARK APARTMENTS

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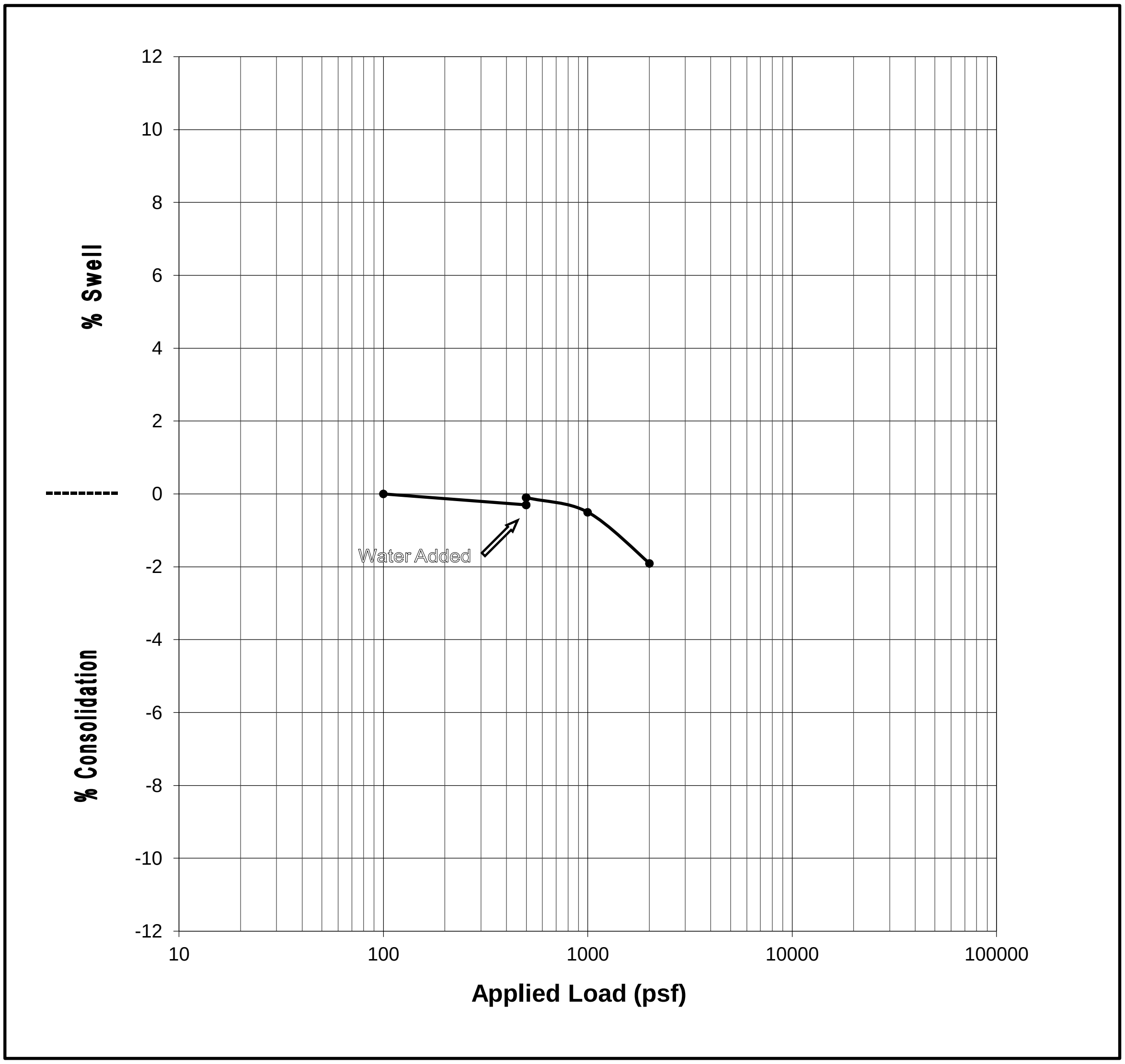
SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-2 @ 9'

Sample Description: Light Brown/Rust Sandy Lean Clay with Scattered Gravel (CL)

Initial Moisture	10.7%	Liquid Limit	-
Final Moisture	18.3%	Plasticity Index	-
% Swell @ 500 psf	0.2%	% Passing #200	-
Swell Pressure (psf)	800	Dry Density (pcf)	106.9

VELO PARK APARTMENTS

3289 AIRPORT ROAD, BOULDER, COLORADO

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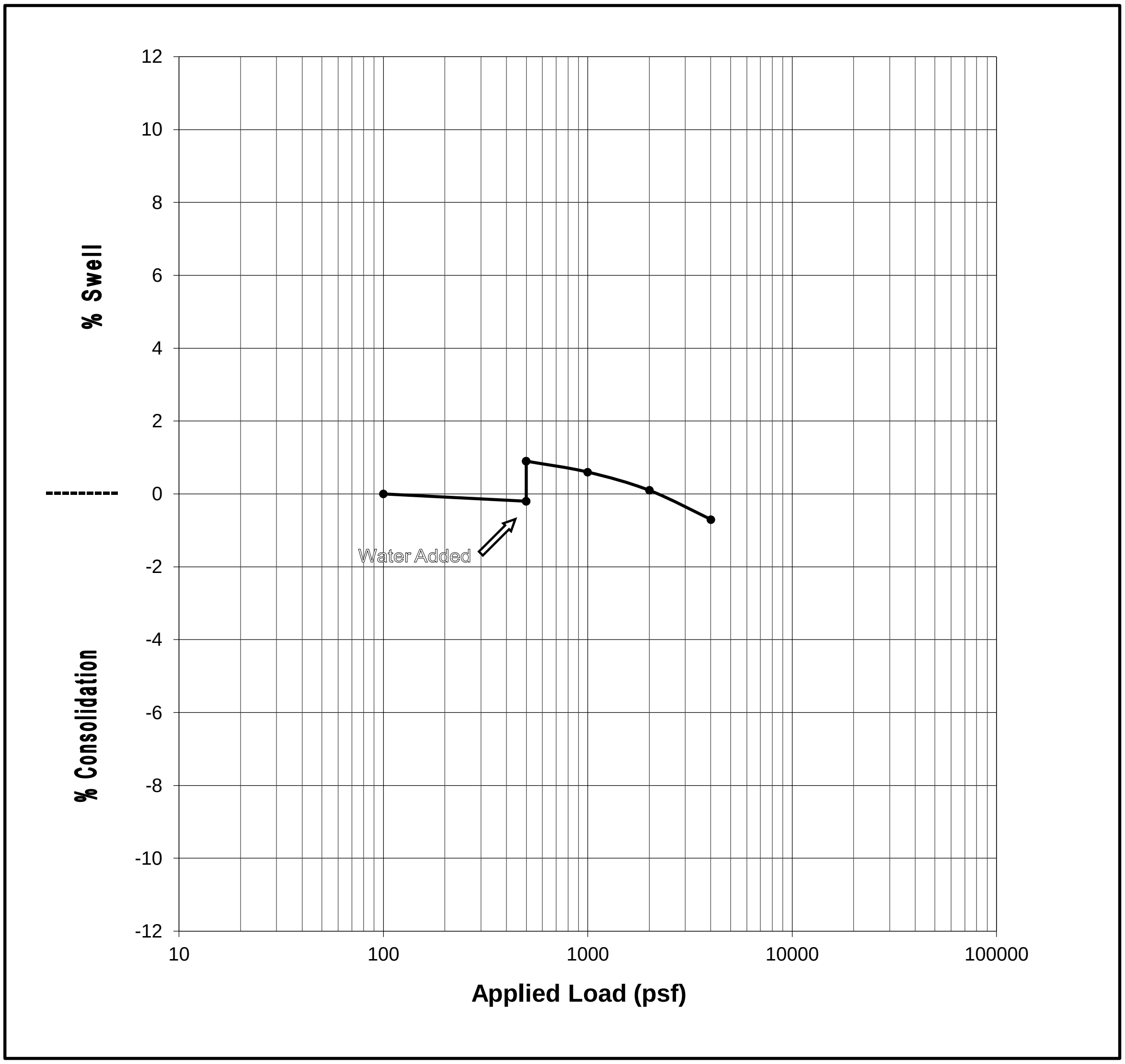
February 2016

SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-3 @ 4'

Sample Description: Light Brown/Rust Sandy Lean Clay with Scattered Gravel (CL)

Initial Moisture	9.3%	Liquid Limit	31
Final Moisture	13.2%	Plasticity Index	17
% Swell @ 500 psf	1.1%	% Passing #200	51.3%
Swell Pressure (psf)	2,750	Dry Density (pcf)	123.1

VELO PARK APARTMENTS

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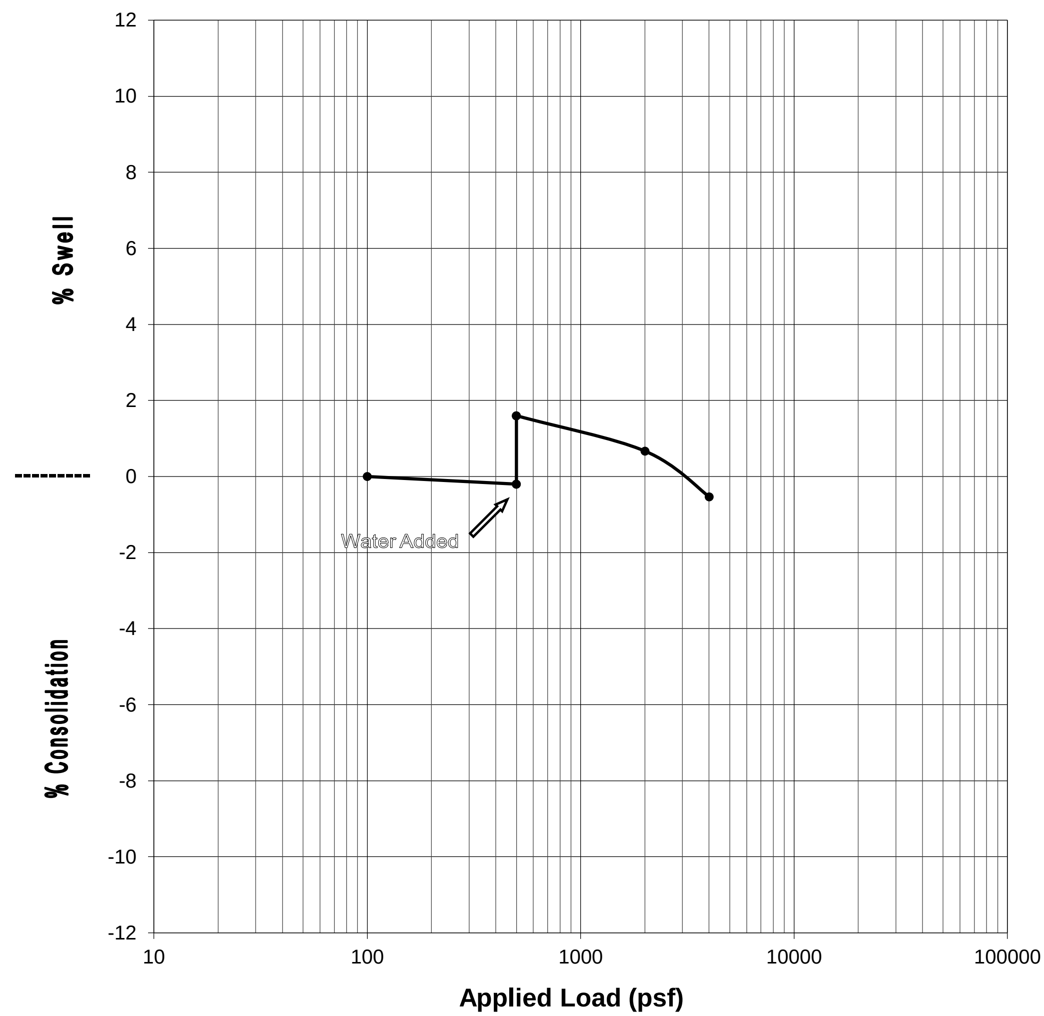
SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-3 @ 9'

Sample Description: Grey/Rust/Olive/Brown Claystone with Interbedded Siltstone/Sandstone

Initial Moisture	15.6%	Liquid Limit	-
Final Moisture	18.3%	Plasticity Index	-
% Swell @ 500 psf	1.8%	% Passing #200	-
Swell Pressure (psf)	3,250	Dry Density (pcf)	115.2

VELO PARK APARTMENTS

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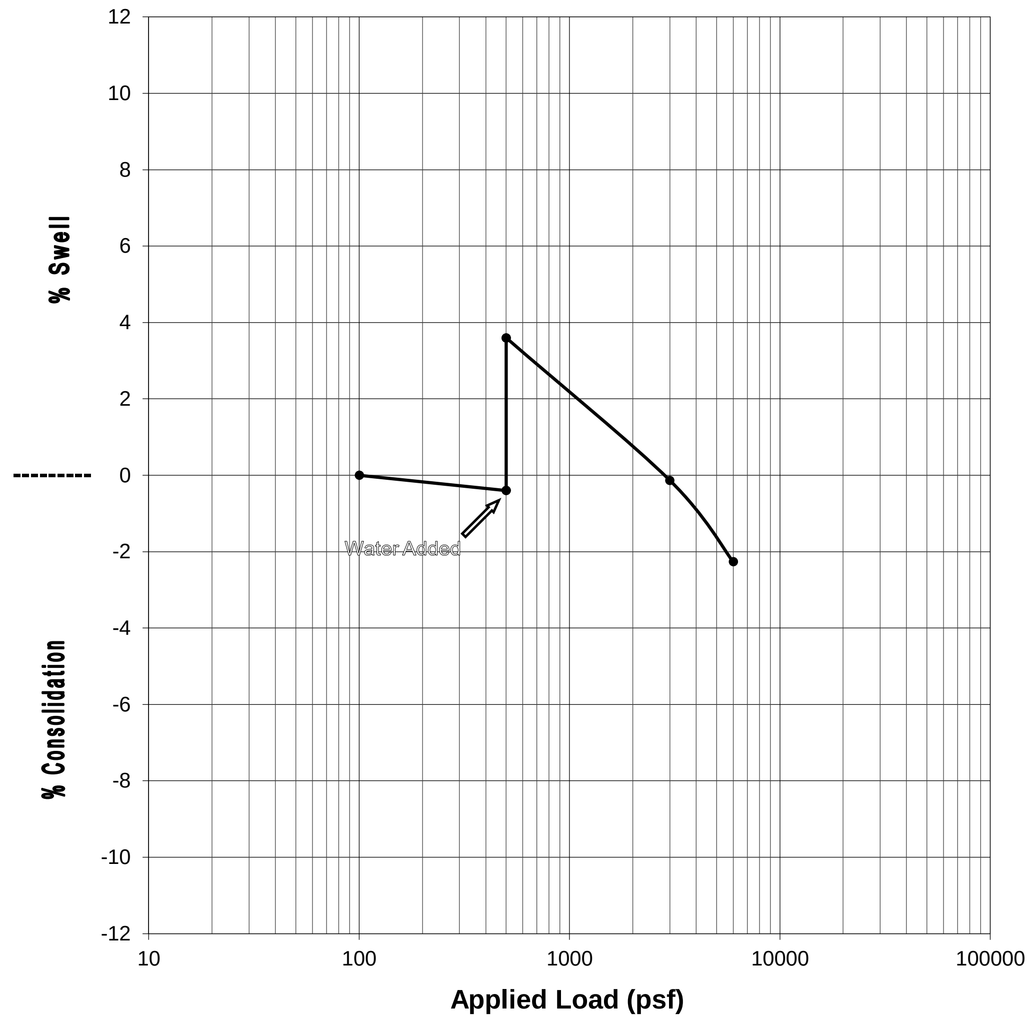
SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-4 @ 4

Sample Description: Brown/Grey Sandy Lean Clay (CL)

Initial Moisture	10.6%	Liquid Limit	-
Final Moisture	28.4%	Plasticity Index	-
% Swell @ 500 psf	4.0%	% Passing #200	-
Swell Pressure (psf)	3,400	Dry Density (pcf)	99.5

VELO PARK APARTMENTS

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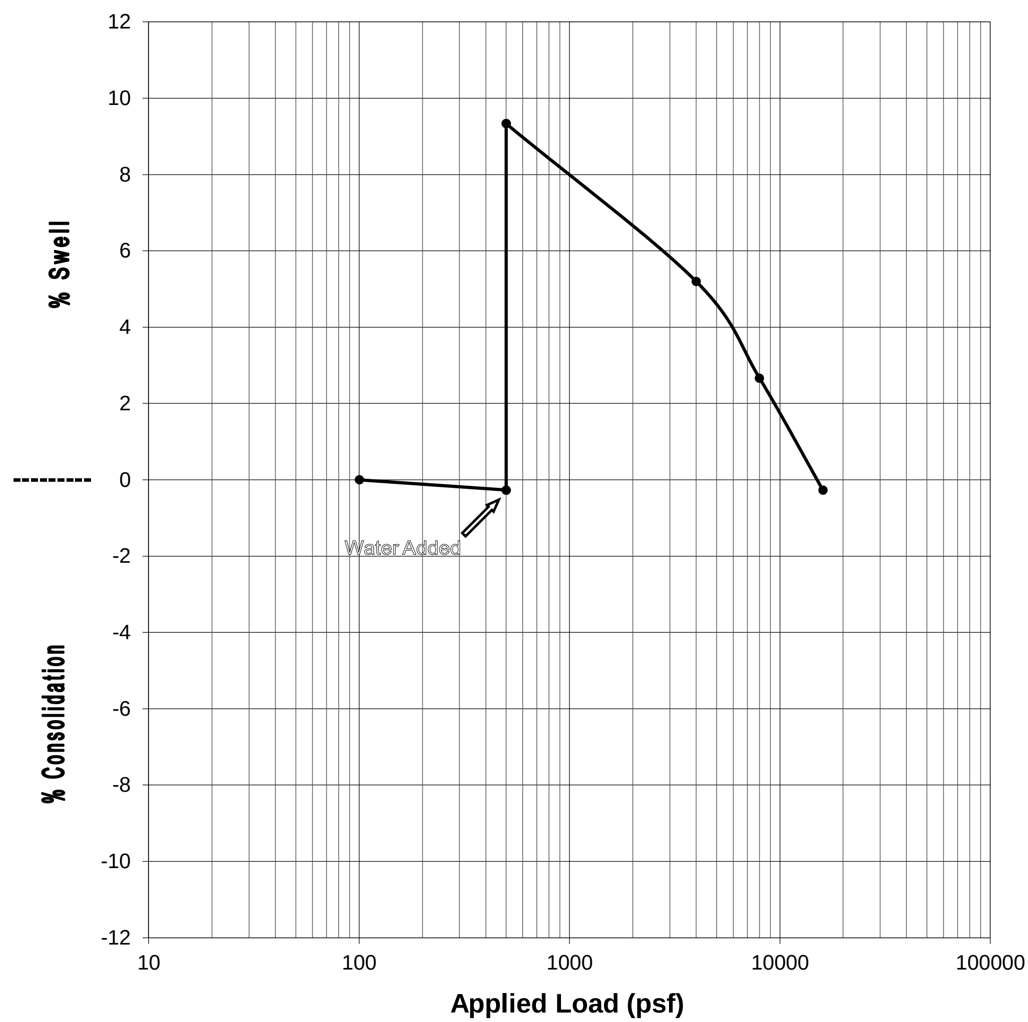
SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-4 @ 9

Sample Description: Rust/Olive/Grey Claystone

Initial Moisture	12.4%	Liquid Limit	-
Final Moisture	20.5%	Plasticity Index	-
% Swell @ 500 psf	9.6%	% Passing #200	-
Swell Pressure (psf)	16,000	Dry Density (pcf)	116.6

VELO PARK APARTMENTS

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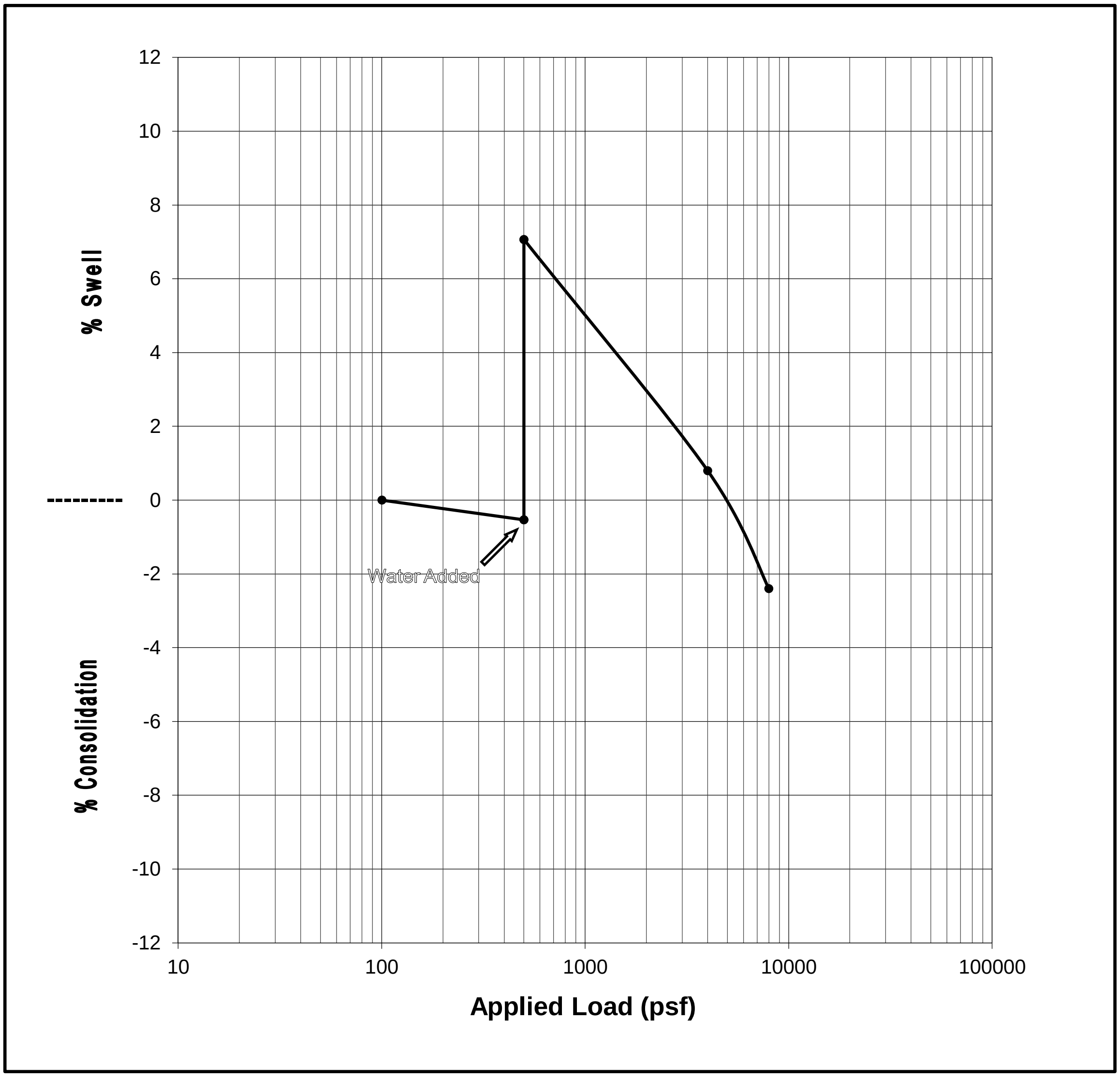
August 2018

SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-5 @ 4

Sample Description: Brown/Olive Sandy Lean Clay (CL)

Initial Moisture	15.0%	Liquid Limit	-
Final Moisture	27.0%	Plasticity Index	-
% Swell @ 500 psf	7.6%	% Passing #200	-
Swell Pressure (psf)	5,700	Dry Density (pcf)	101.9

VELO PARK APARTMENTS
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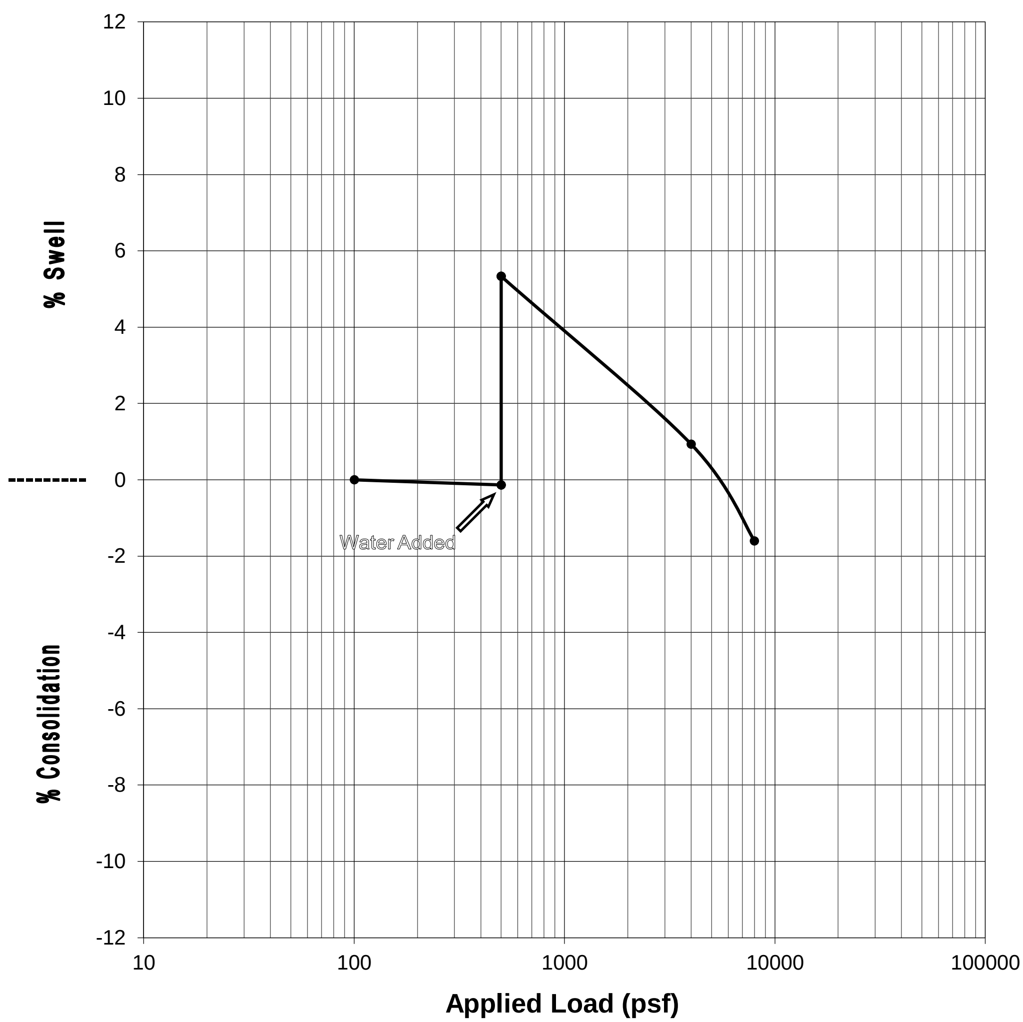
SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-5 @ 9
Sample Description: Rust/Olive/Grey Claystone

Initial Moisture	14.2%	Liquid Limit	-
Final Moisture	20.7%	Plasticity Index	-
% Swell @ 500 psf	5.5%	% Passing #200	-
Swell Pressure (psf)	5,700	Dry Density (pcf)	115.4

VELO PARK APARTMENTS

3289 AIRPORT ROAD, BOULDER, COLORADO

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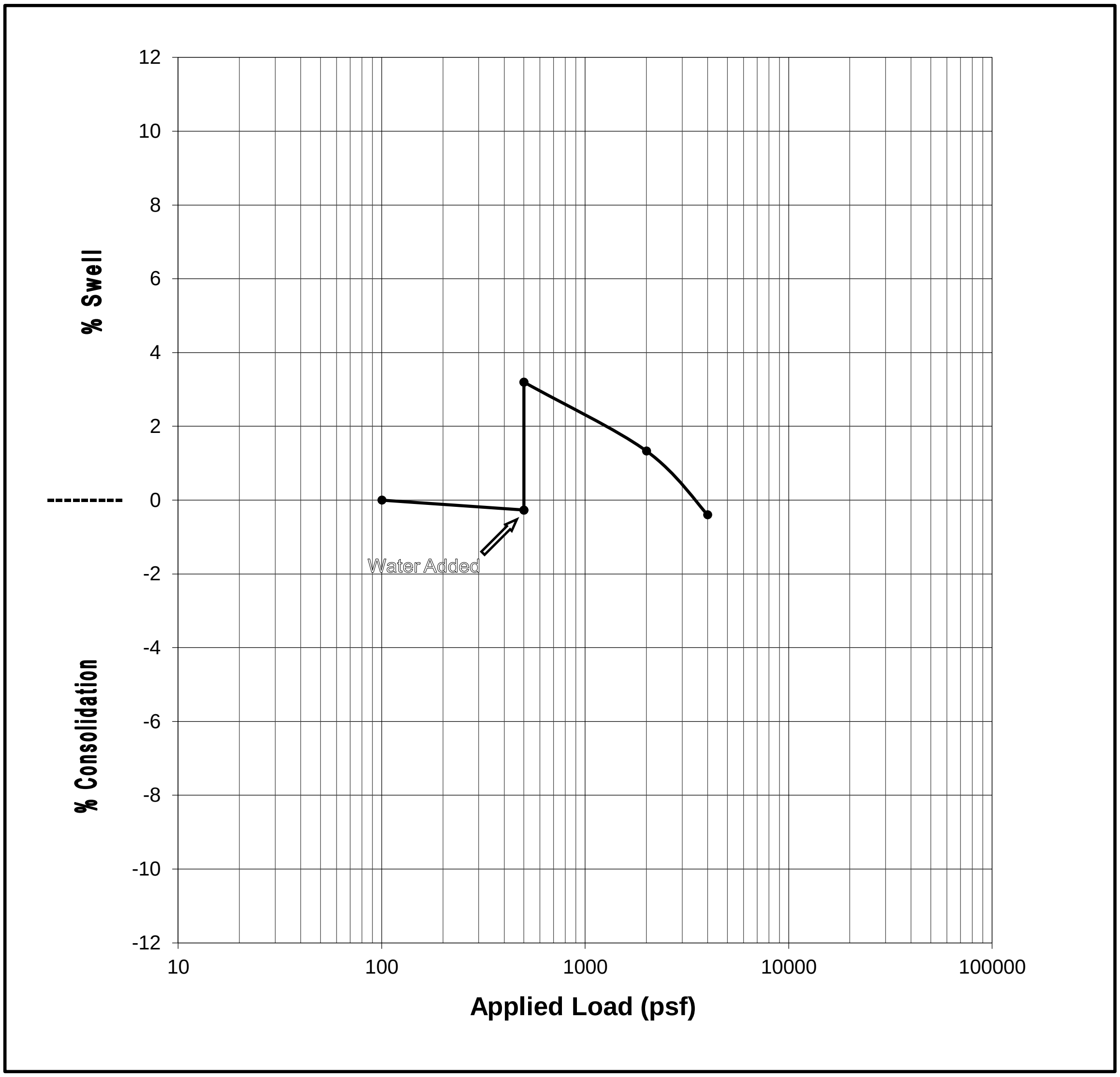
SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-6 @ 2

Sample Description: Brown Sandy Lean Clay (CL)

Initial Moisture	7.4%	Liquid Limit	-
Final Moisture	20.0%	Plasticity Index	-
% Swell @ 500 psf	3.5%	% Passing #200	-
Swell Pressure (psf)	3,800	Dry Density (pcf)	117.1

VELO PARK APARTMENTS

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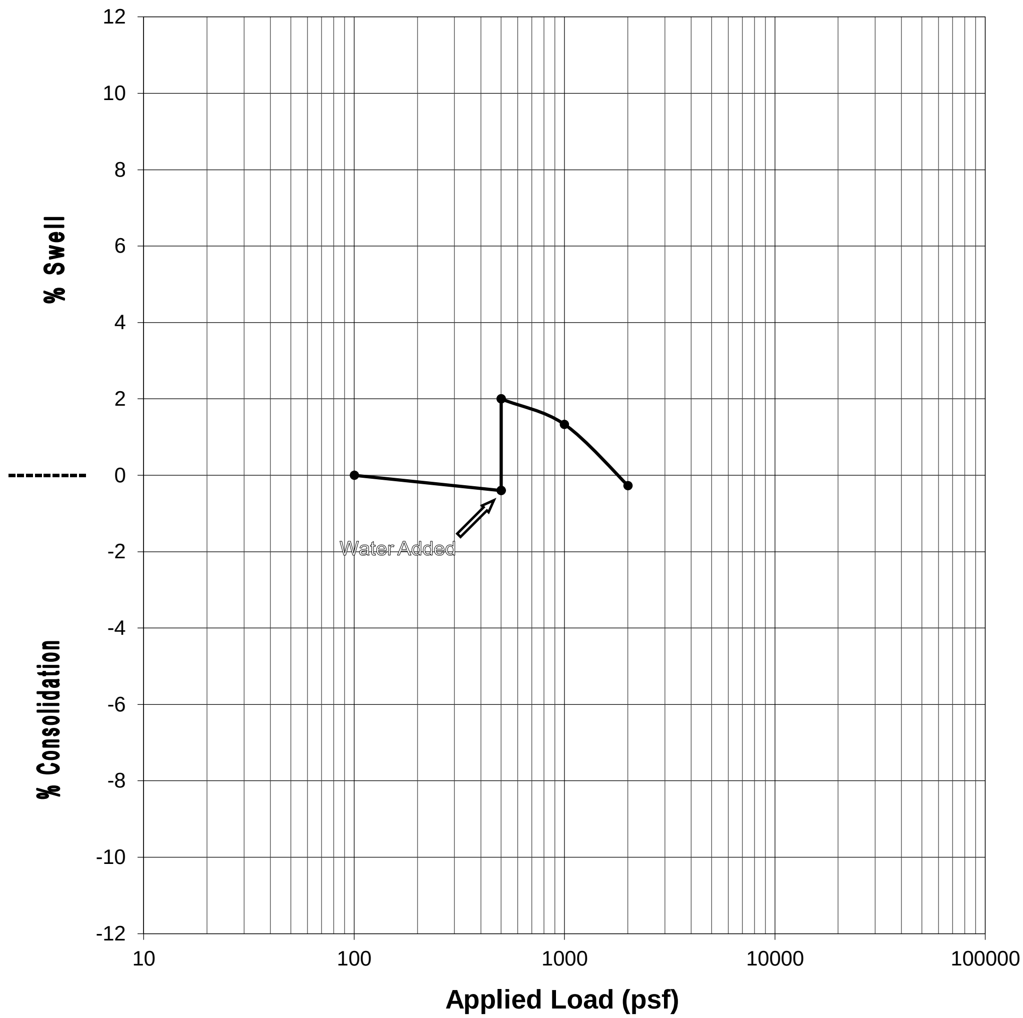
SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-6 @ 4

Sample Description: Brown Sandy Lean Clay (CL)

Initial Moisture	6.7%	Liquid Limit	37
Final Moisture	22.6%	Plasticity Index	20
% Swell @ 500 psf	2.4%	% Passing #200	32.2%
Swell Pressure (psf)	-	Dry Density (pcf)	106.9

VELO PARK APARTMENTS

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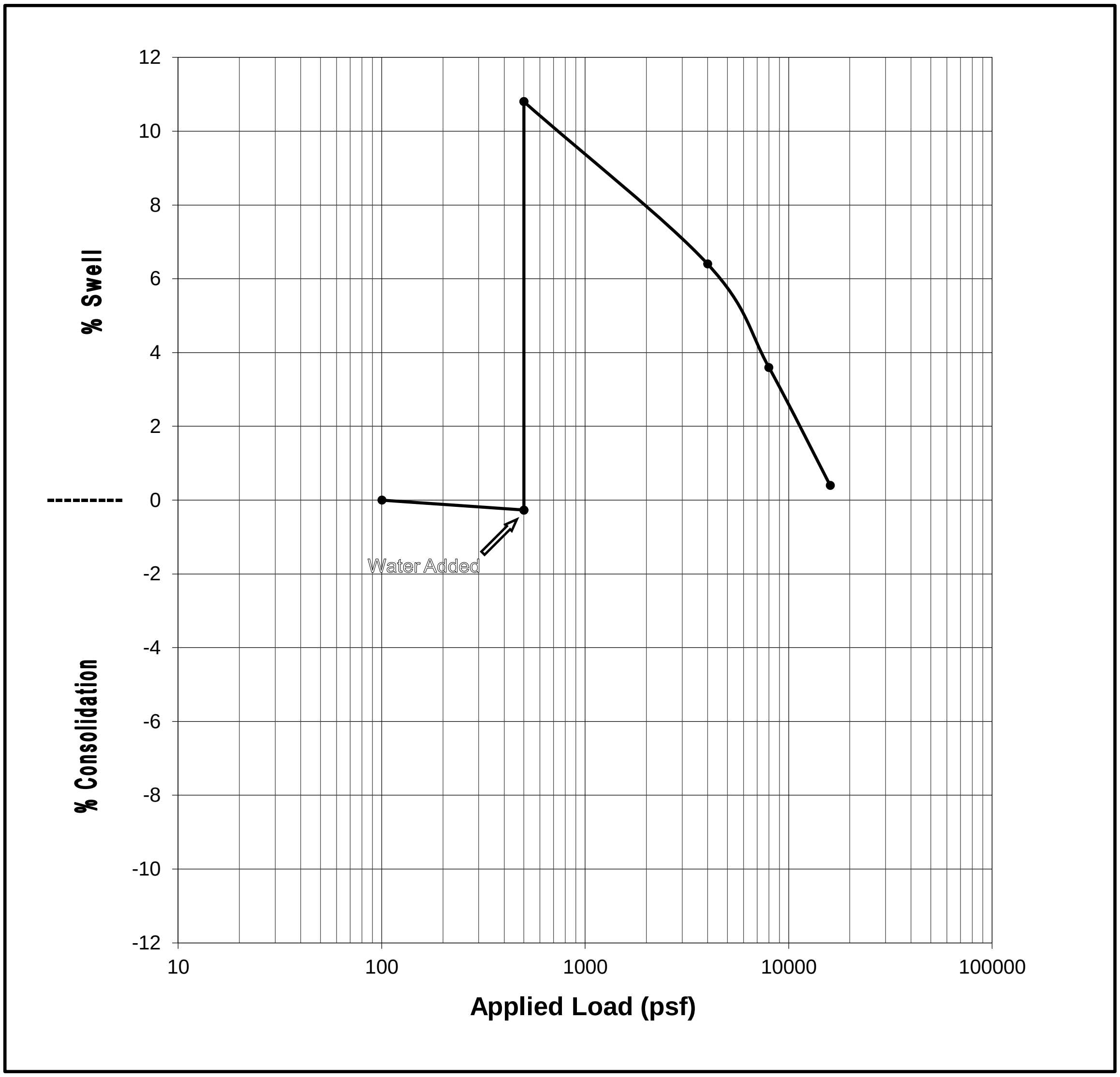
SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-6 @ 9

Sample Description: Rust/Olive/Grey Claystone

Initial Moisture	10.5%	Liquid Limit	-
Final Moisture	18.6%	Plasticity Index	-
% Swell @ 500 psf	11.1%	% Passing #200	-
Swell Pressure (psf)	>16,000	Dry Density (pcf)	124.8

VELO PARK APARTMENTS

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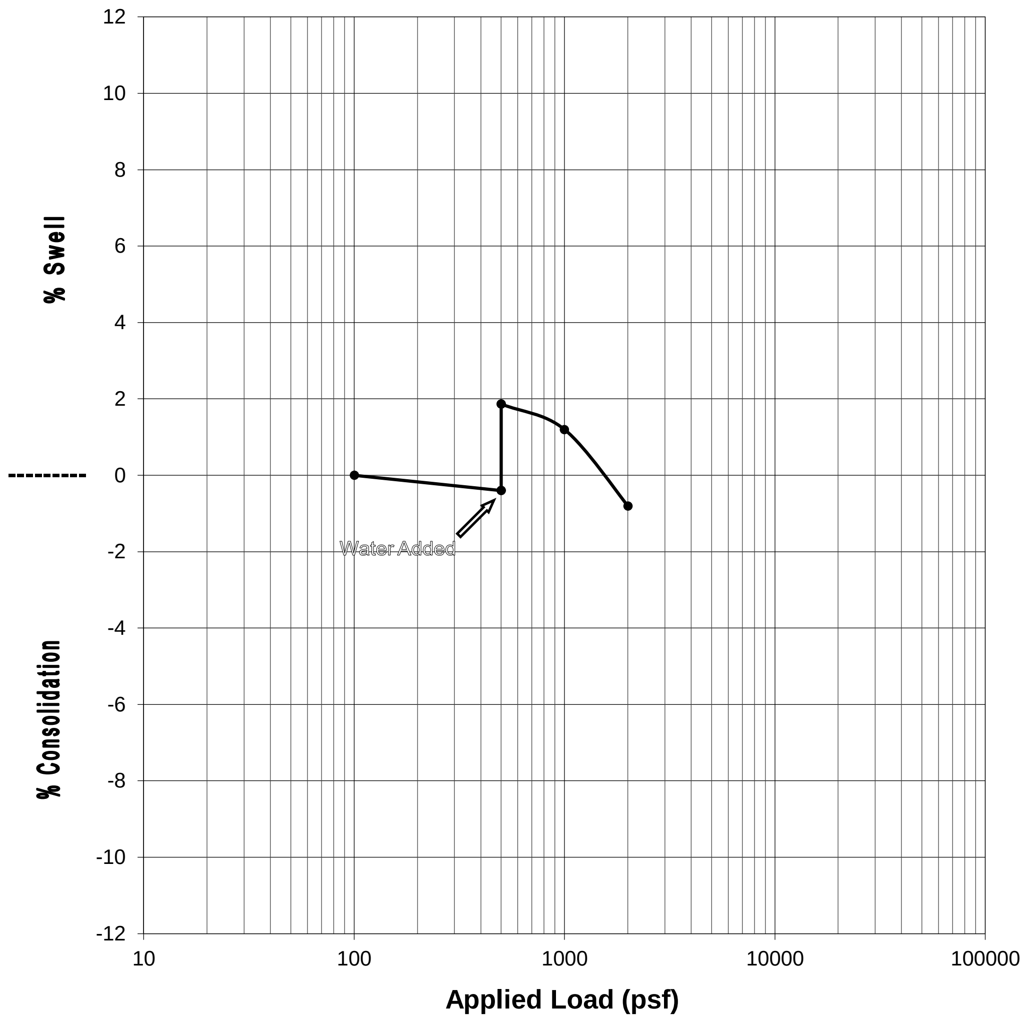
SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-7 @ 4

Sample Description: Tan Sandy Lean Clay (CL)

Initial Moisture	11.1%	Liquid Limit	-
Final Moisture	31.5%	Plasticity Index	-
% Swell @ 500 psf	2.3%	% Passing #200	-
Swell Pressure (psf)	1,800	Dry Density (pcf)	96.2

VELO PARK APARTMENTS

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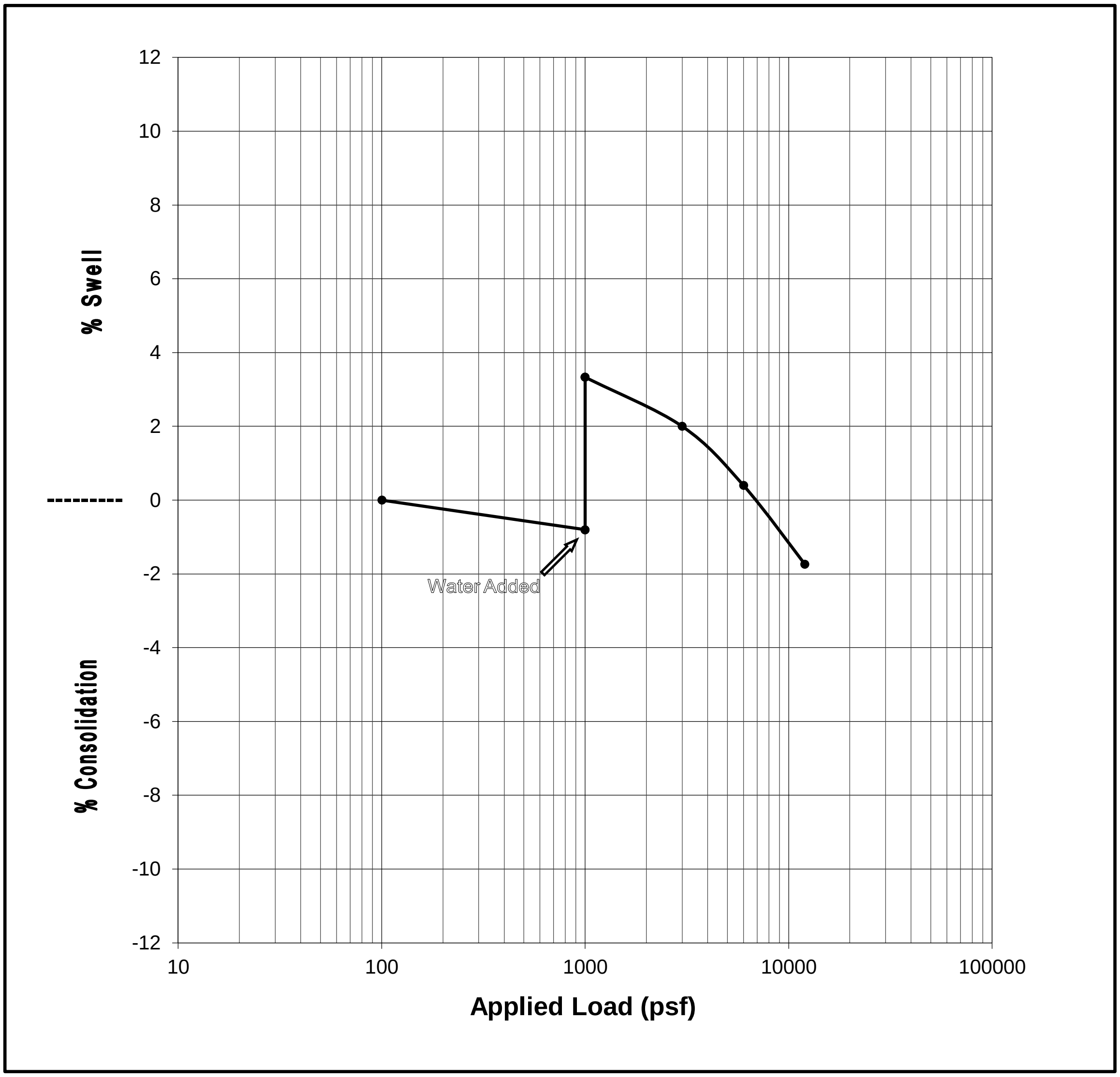
SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-7 @ 14

Sample Description: Rust/Olive/Grey Claystone

Initial Moisture	14.5%	Liquid Limit	-
Final Moisture	19.0%	Plasticity Index	-
% Swell @ 1,000 psf	4.1%	% Passing #200	-
Swell Pressure (psf)	9,400	Dry Density (pcf)	118.8

VELO PARK APARTMENTS

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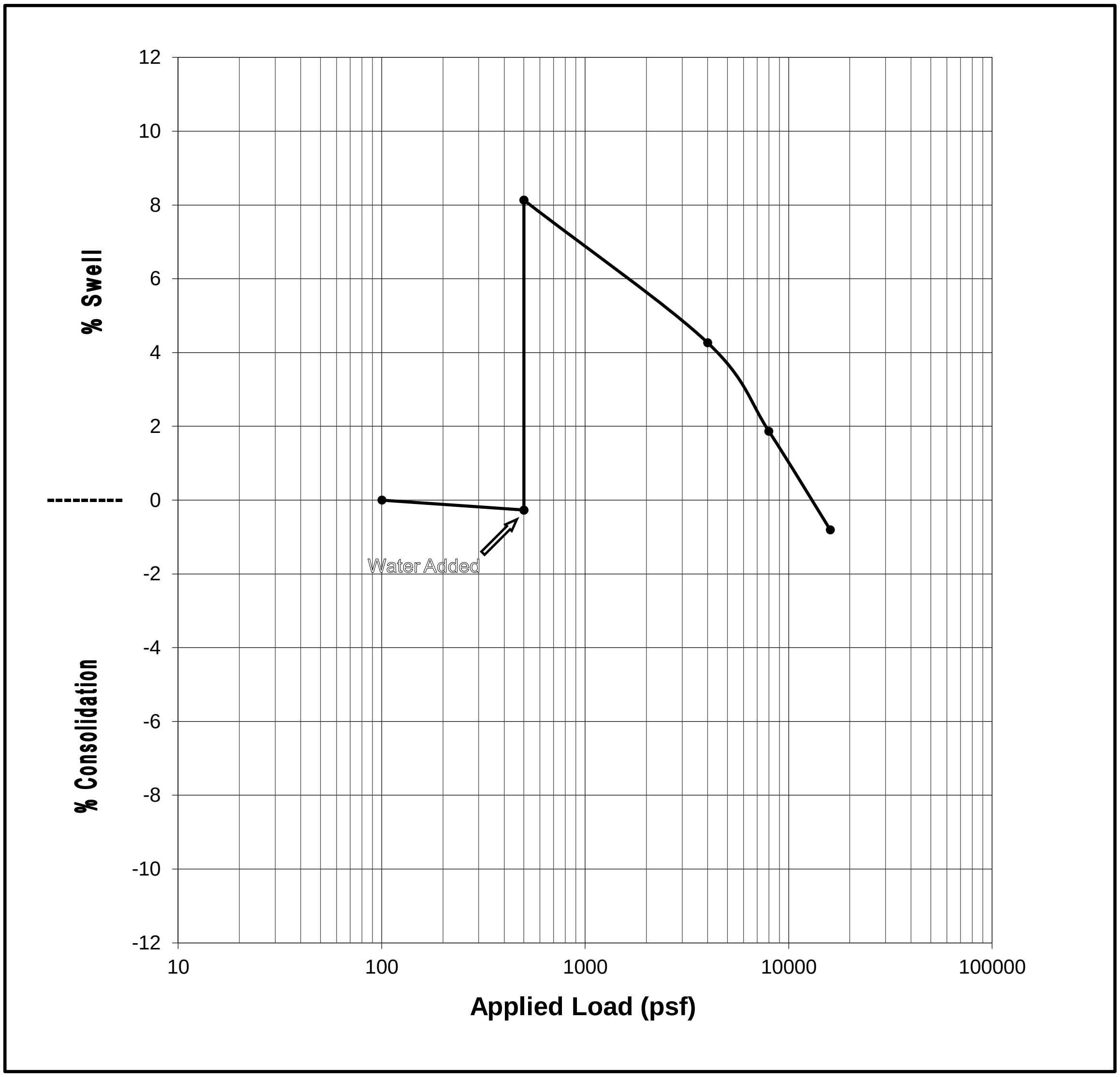
SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-8 @ 2

Sample Description: Rust/Olive/Grey Claystone

Initial Moisture	11.5%	Liquid Limit	-
Final Moisture	22.6%	Plasticity Index	-
% Swell @ 500 psf	8.4%	% Passing #200	-
Swell Pressure (psf)	14,400	Dry Density (pcf)	119.9

VELO PARK APARTMENTS

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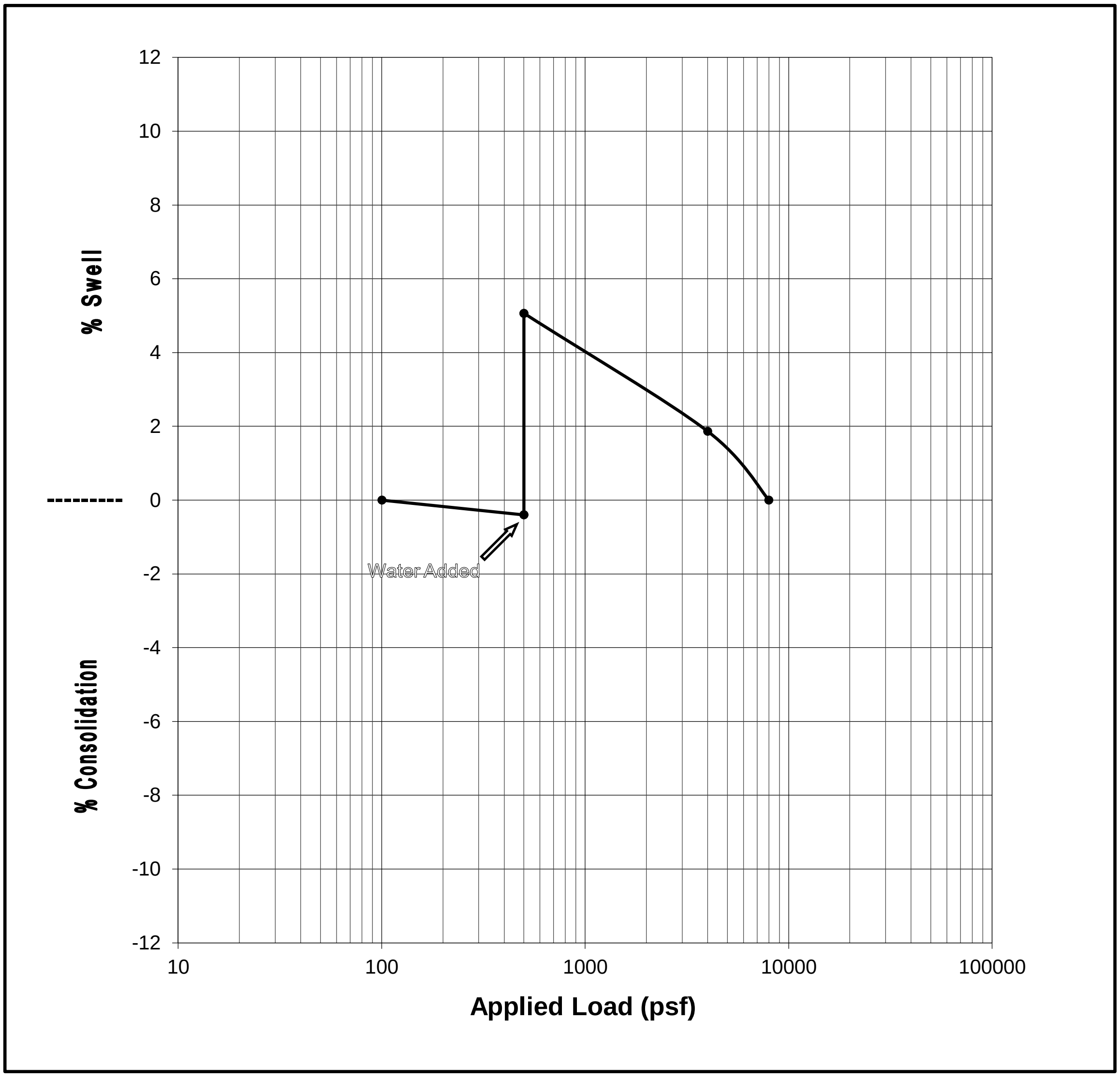
SWELL/CONSOLIDATION TEST SUMMARY



Sample ID: B-8 @ 9

Sample Description: Rust/Olive/Grey Claystone

Initial Moisture	13.7%	Liquid Limit	-
Final Moisture	18.4%	Plasticity Index	-
% Swell @ 500 psf	5.5%	% Passing #200	-
Swell Pressure (psf)	-	Dry Density (pcf)	116.8

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification	
				Group Symbol	Group Name[B]
Coarse Grained Soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[C]	Cu ! 4 and 1 ≤ Cc ≤ 3[E]	GW	Well graded gravel[F]
			Cu < 4 and/or 1 > Cc > 3[E]	GP	Poorly graded gravel[F]
		Gravels with Fines More than 12% fines[C]	Fines classify as ML or MH	GM	Silty gravel[F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel[F,G,H]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[D]	Cu ! 6 and 1 ≤ Cc ≤ 3[E]	SW	Well graded sand[I]
			Cu < 6 and/or 1 > Cc > 3[E]	SP	Poorly graded sand[I]
		Sands with Fines More than 12% fines[D]	Fines classify as ML or MH	SM	Silty sand[G,H,I]
			Fines classify as CL or CH	SC	Clayey sand[G,H,I]
Fine-Grained Soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI > 7 and plots on or above "A" line[J]	CL	Lean clay[K,L,M]
			PI < 4 or plots below "A" line[J]	ML	Silt[K,L,M]
		Organic	**Liquid limit - oven dried** / **Liquid limit - not dried** < 0.75	OL	Organic clay[K,L,M,N]
					Organic silt[K,L,M,O]
	Silts and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay[K,L,M]
			PI plots below "A" line	MH	Elastic silt[K,L,M]
		Organic	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OH	Organic clay[K,L,M,P]
					Organic silt[K,L,M,Q]
Highly organic soils	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-in. (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well graded gravel with silt, GW-GC well graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well graded sand with silt, SW-SC well graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ! 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains ! 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains ! 30% plus No. 200 predominantly sand, add "sandy" to group name.

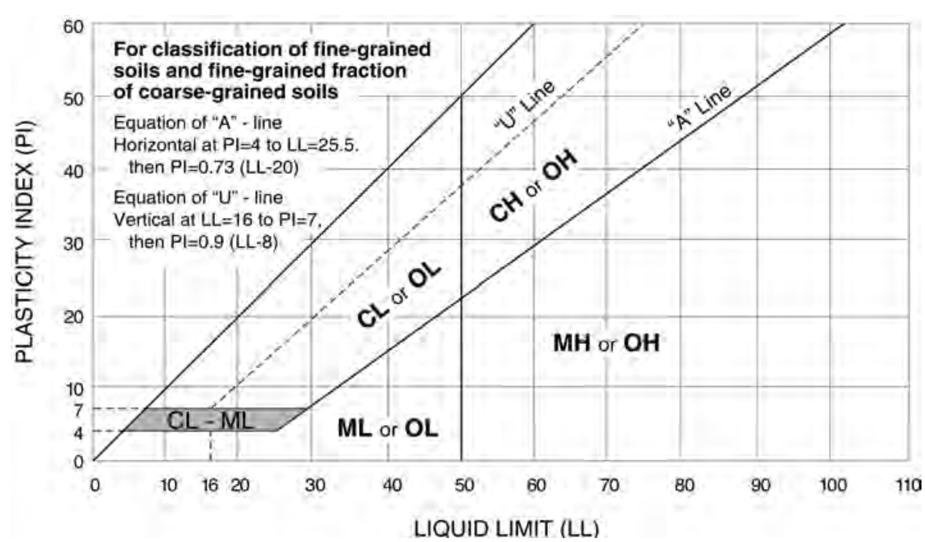
[M] If soil contains ! 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI ! 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.



GENERAL NOTES

DRILLING & SAMPLING SYMBOLS:

SS:	Split Spoon - 1⅜" I.D., 2" O.D., unless otherwise noted	HS:	Hollow Stem Auger
ST:	Thin-Walled Tube – 2.5" O.D., unless otherwise noted	PA:	Power Auger
RS:	Ring Sampler - 2.42" I.D., 3" O.D., unless otherwise noted	HA:	Hand Auger
CS:	California Barrel - 1.92" I.D., 2.5" O.D., unless otherwise noted	RB:	Rock Bit
BS:	Bulk Sample or Auger Sample	WB:	Wash Boring or Mud Rotary

The number of blows required to advance a standard 2-inch O.D. split-spoon sampler (SS) the last 12 inches of the total 18-inch penetration with a 140-pound hammer falling 30 inches is considered the "Standard Penetration" or "N-value". For 2.5" O.D. California Barrel samplers (CB) the penetration value is reported as the number of blows required to advance the sampler 12 inches using a 140-pound hammer falling 30 inches, reported as "blows per inch," and is not considered equivalent to the "Standard Penetration" or "N-value".

WATER LEVEL MEASUREMENT SYMBOLS:

WL:	Water Level	WS:	While Sampling
WCI:	Wet Cave in	WD:	While Drilling
DCI:	Dry Cave in	BCR:	Before Casing Removal
AB:	After Boring	ACR:	After Casing Removal

Water levels indicated on the boring logs are the levels measured in the borings at the times indicated. Groundwater levels at other times and other locations across the site could vary. In pervious soils, the indicated levels may reflect the location of groundwater. In low permeability soils, the accurate determination of groundwater levels may not be possible with only short-term observations.

DESCRIPTIVE SOIL CLASSIFICATION: Soil classification is based on the Unified Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

FINE-GRAINED SOILS

(CB) Blows/Ft.	(SS) Blows/Ft.	Consistency
< 3	0-2	Very Soft
3-5	3-4	Soft
6-10	5-8	Medium Stiff
11-18	9-15	Stiff
19-36	16-30	Very Stiff
> 36	> 30	Hard

COARSE-GRAINED SOILS

(CB) Blows/Ft.	(SS) Blows/Ft.	Relative Density
0-5	< 3	Very Loose
6-14	4-9	Loose
15-46	10-29	Medium Dense
47-79	30-50	Dense
> 79	> 50	Very Dense

BEDROCK

(CB) Blows/Ft.	(SS) Blows/Ft.	Consistency
< 24	< 20	Weathered
24-35	20-29	Firm
36-60	30-49	Medium Hard
61-96	50-79	Hard
> 96	> 79	Very Hard

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Terms of Other Constituents	Percent of Dry Weight
Trace	< 15
With	15 – 29
Modifier	> 30

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300mm)
Cobbles	12 in. to 3 in. (300mm to 75 mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm)
Silt or Clay	Passing #200 Sieve (0.075mm)

RELATIVE PROPORTIONS OF FINES

Descriptive Terms of Other Constituents	Percent of Dry Weight
Trace	< 5
With	5 – 12
Modifiers	> 12

PLASTICITY DESCRIPTION

Term	Plasticity Index
Non-plastic	0
Low	1-10
Medium	11-30
High	30+