

**Additional Subsurface Exploration Program and  
Geotechnical Recommendations  
University of Colorado at Boulder  
Williams Village Phase IIA – Extended Scope  
Boulder, Colorado**

**Prepared for:**

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**GROUND**

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## PURPOSE AND SCOPE OF STUDY

GROUND recently completed a subsurface exploration program to provide geotechnical and pavement section recommendations for design and construction of the proposed Pod "C" Residence Hall in Williams Village located on the campus of the University of Colorado at Boulder, in Boulder Colorado. The results were summarized in GROUND's report, titled, *Subsurface Exploration Program, Geotechnical Recommendations, University of Colorado at Boulder, Williams Village Phase II Pod "C" Residence Hall, Boulder, Colorado, Job No. 09-3011*, prepared for University of Colorado, dated April 22, 2009. Reference is made to our April 22, 2009 report for our general geotechnical findings and recommendations. We consider all recommendations in that report not specifically superseded herein to remain valid.

We were informed that the original proposed project's scope was expanded beyond the residence hall building site and additional subsurface exploration and geotechnical recommendations are needed to complete the design of the expanded scope items.

This report presents the results of an additional subsurface exploration program and laboratory testing performed by GROUND Engineering Consultants, Inc. (GROUND) to provide additional geotechnical recommendations for design and construction of the expanded scope items for the proposed University of Colorado at Boulder Williams Village Phase IIA project, in Boulder, Colorado. Our services were performed in general accordance with GROUND's proposal dated December 14, 2009 (Job No. 09-3011).

A field exploration program was conducted to obtain information on subsurface conditions. Material samples obtained during the subsurface exploration were tested in the laboratory to provide data on the classification and engineering characteristics of the on-site soils. The results of the field exploration and laboratory testing are presented herein.

This report has been prepared to summarize the data obtained and to present our conclusions and recommendations based on the proposed construction and the subsurface conditions encountered. Design parameters and a discussion of geotechnical engineering considerations related to the construction of the proposed project are included.

## **PROPOSED CONSTRUCTION**

The Williams Village Phase IIA project now includes:

- Exploring the feasibility of construction of two dry wells on the west and east sides of the proposed residence hall to assist with storm water drainage.
- Construction of two sign walls located on both sides of the proposed 35<sup>th</sup> Street alignment, south of Baseline Road.
- Construction of additional parking lots, and rehabilitation and/or reconstruction of the existing parking lots located to the north and northeast of the proposed residence hall.
- Reconstruction of the existing 30<sup>th</sup> Street Bus Stop.

If the proposed additional construction differs significantly from that described above, GROUND should be notified to re-evaluate the recommendations contained herein.

## **SITE CONDITIONS**

At the time of our exploration, construction activities were commencing for the proposed residence hall. The general contractor for the construction project is Whiting-Turner and, during our fieldwork, subcontractor EZ Excavating was demolishing the site and preparing to install the underground utilities to serve the proposed residence hall. The remainder of the project site existed as asphalt and aggregate surfaced parking lots, private driveways, and a bus stop. The general topography of the site was flat to gently sloping.

## **SUBSURFACE EXPLORATION**

The subsurface exploration for the project was conducted in January 2010. A total of eleven (11) test holes were drilled. Of the eleven test holes, two (2) test holes (Test Holes 1 and 2) were drilled near the proposed dry well locations to depths of 15 to 25 feet below the existing grades. Test Holes 1 and 2 were also constructed as temporary groundwater observation points and used to perform slug testing at each location. One (1) test hole was drilled within the proposed footprint of the 35<sup>th</sup> Street sign walls, two (2) test holes were drilled within the 30<sup>th</sup> Street Bus Stop, and six (6) test holes were

advanced to depths of 5 to 10 feet within the existing and proposed paved parking areas.

The test holes were drilled with truck-mounted, continuous flight, power auger rigs to evaluate subsurface conditions, including depths to groundwater and bedrock (where encountered), as well as to retrieve samples for laboratory testing and analysis. GROUND engineers directed the subsurface exploration program, logged the test holes in the field, and prepared the samples for transport to our laboratory.

Relatively undisturbed samples of the subsurface materials were taken with a 2-inch I.D. "California" -type liner sampler. The sampler was driven into the substrata with blows from a 140-pound hammer falling 30 inches. This procedure is similar to the Standard Penetration Test described by ASTM Method D1586. Penetration resistance values (blows per distance driven, typically 12 inches), when properly evaluated, indicate the relative density or consistency of soils and bedrock. A composite disturbed (bulk) sample of the shallow soils in the pavement areas was collected from the auger returns. Depths at which the samples were taken and associated penetration resistance values are shown on the test hole logs.

The approximate locations of the test holes are shown in Figures 1 and 2. Logs of the exploratory test holes are presented in Figures 3 through 5. Explanatory notes and a legend are provided in Figure 6.

## **LABORATORY TESTING**

Samples retrieved from our test holes were examined and visually classified in the laboratory by the project engineer. Laboratory testing of soil samples obtained from the subject site included standard property tests, such as natural moisture contents, dry unit weights, grain size analyses, liquid and plastic limits. Water-soluble sulfate content and swell-consolidation tests were performed on selected samples, as well. Compaction and resilient modulus tests were performed on a composite bulk sample obtained from the pavement test hole auger cuttings. Laboratory tests were performed in general accordance with applicable ASTM and AASHTO protocols. Data from the laboratory-testing program are summarized on Table 1. Compaction test results are presented in Figure 8. Resilient modulus test results are presented under the *Pavement Sections* section of this report.

## **SUBSURFACE CONDITIONS**

### ***Dry Well Locations:***

Test Holes 1 and 2 were advanced through a layer of fill, approximately 1 foot thick, associated with the construction activities for the proposed residence hall. Beneath the fill materials, the subsurface conditions encountered consisted of overburden sands and clays with gravels that extended to depths of approximately 8 to 14.5 feet below the existing grades. These overburden materials were underlain by weathered claystone that extended to the termination depths of 15 and 25 feet, respectively.

### ***Sign Walls:***

Test Hole 3 was advanced through a layer of aggregate road base, approximately 6 inches thick. Fill materials were encountered beneath the aggregate road base materials to depths of approximately 2 feet below existing grades. Beneath the fill materials, the subsurface conditions encountered consisted of overburden sands and clays with gravels that extended to depths of approximately 13 feet below the existing grades. These overburden materials were underlain by weathered claystone that extended to a depth of approximately 33 feet below existing grades in Test Hole 3. Claystone bedrock was encountered beneath the weathered claystone and extended to the Test Hole 3 termination depth of approximately 35 feet below existing grades.

### ***Additional Parking Lots:***

Test Holes P1 through P4 were advanced through a layer of pavement consisting of approximately 4 to 5.5 inches of asphalt over approximately 4.5 to 6 inches of aggregate road base materials. Test Holes P5, and P6 were advanced through a layer of aggregate road base, approximately 6 inches thick, associated with the gravel parking lots. Fill materials were also encountered beneath the asphalt and aggregate road base materials to depths of approximately 1 to 2 feet below existing grades. Beneath the fill materials, the subsurface conditions encountered in the test holes generally consisted of overburden sands and clays with gravels that extended to test hole termination depths of approximately 5 feet below the existing grades.

**30<sup>th</sup> Street Bus Stop:**

Test Holes P7, and P8 were advanced through a layer of pavement consisting of approximately 3 inches of asphalt over approximately 8 to 9 inches of aggregate road base materials. Fill materials were also encountered beneath the asphalt and aggregate road base materials to depths of approximately 2 to 3 feet below existing grades. Beneath the fill materials, the subsurface conditions encountered in the test holes generally consisted of overburden sands and clays with gravels that extended to test hole termination depths of approximately 5 feet below the existing grades.

***Subsurface Condition Descriptions:***

***Fill*** materials encountered consisted of sandy clays with gravels. They were fine to coarse grained with gravels, low to moderately plastic, loose to compact, slightly moist to moist, and red-brown in color.

***Sands and Clays*** encountered consisted of sandy clays with gravels and clayey sands with gravels. They were fine to coarse grained with gravels, low to moderately plastic, stiff/loose to very stiff/medium dense, moist to wet, and red-brown to gray-brown in color with occasional iron staining.

***Weathered Claystone*** was fine grained, highly plastic, stiff to very stiff, moist to wet, and gray to gray-brown in color with iron staining.

***Claystone Bedrock*** was fine grained, highly plastic, very hard, moist, and gray to gray-brown in color with iron staining.

***Groundwater*** was encountered at depths of approximately 8 to 9 feet below the existing grades at the time (January 13, 2010) of GROUND's subsurface exploration. Water levels can vary, however, in response to annual and longer-term cycles of precipitation, irrigation, drainage, and other factors.

***Swell-Consolidation Testing*** suggested a low potential for swell in the tested on-site materials. Percent swell values and surcharge pressures at which the swell/consolidation tests were performed are presented on Table 1.

## **DRY WELL RECOMMENDATIONS**

The shallow soils encountered in the test holes consisted primarily of clayey sands and sandy clays. USCS Classifications of SC, SM-SC, and CL were encountered in the overburden soils. The underlying weathered bedrock typically was classified as CH, as shown in GROUND's April 22, 2009 dated report.

Groundwater was relatively shallow at the site and was encountered at approximately 7 feet below existing grades and approximately 5.5 feet below existing grades at Test Holes 1 and 2, respectively, during the time of our fieldwork. Near Test Hole 1, there was approximately 11 feet of the clayey sands to sandy clays above the weathered bedrock. Near Test Hole 2, there was approximately 5 feet of the clayey sands to sandy clays above the weathered bedrock. Furthermore, we consider these overburden soils to be NRCS Hydrologic Soil Group B soils. Hydraulic Soil Group B soils typically are considered marginal for the use in dry well construction.

Slug testing indicated hydraulic conductivities of approximately  $3.3 \times 10^{-4}$  cm/s in the clayey sands at Test Hole 1 and about  $1.4 \times 10^{-5}$  cm/s in the weathered bedrock at Test Hole 2. These values can be correlated approximately to 7.0 gallons/day per ft<sup>2</sup> and 0.30 gallons/day per ft<sup>2</sup>, respectively. Values on the order of  $10^{-4}$  to  $10^{-3}$  cm/s are typical for sands with significant proportions of silt and clay.

We have assumed that the dry wells will be 2 feet in diameter and extend with effectively impermeable walls to a depth of 5 feet, below which will extend as a gravel column into the water table. We estimate that under steady-state conditions – i.e., with the well filled, infiltration cone extending outward from the dry well saturated, and groundwater at the elevation encountered during our fieldwork – a single dry well can release about 850 gallons per day into the subgrade overburden materials. Draining the well after influx has stopped will be somewhat slower, however.

## **SIGN FOUNDATION SYSTEMS**

Based on the data obtained during the subsurface exploration, results of the laboratory testing, the nature of the proposed structure and the anticipated foundation elevations, the proposed structure may be supported on a shallow foundation system.

We estimate that potential post-construction movements, if the recommendations in this report are implemented effectively, to be approximately 1 inch. Differential movements likely will be of similar magnitude across distances of approximately 50 feet.

The remedial earthworks and related measures outlined below will not eliminate the potential for future settlement, but if implemented effectively and maintained, they will tend to make the movements more uniform, and reduce resultant damage to the facility if such movement occurs.

The criteria presented below may be observed for footing foundation systems. The construction details should be considered when preparing project documents.

1. The footings should bear on undisturbed, firm native sands and clays or on similar soils placed as a uniform thickness layer of properly compacted fill. Recommendations for fill placement are provided in the *Project Earthworks* section of GROUND's April 22, 2009 dated report.
2. Footings bearing on undisturbed, firm native sands and clays or on similar soils placed as a uniform thickness layer of properly compacted fill may be designed for an allowable soil bearing pressure (Q) of 2,500 psf under drained conditions. This value may be increased by 1/3 for transient loads such as wind or seismic loading. Based on this allowable bearing capacity, we anticipate post-construction heave and/or settlements of approximately 1 inch. For other estimated settlements, allowable bearing pressure values can be obtained from Figure 7.
3. The recommended allowable bearing pressure was based on an assumption of drained conditions. If foundation materials become wet, the effective bearing capacity will be reduced and larger post-construction movements than those estimated above may result.
4. Soft and wet subgrade conditions may be encountered at the bottom of the excavations. In addition, firm materials may be disturbed by the excavation process or by the construction traffic. All such unsuitable materials should be stabilized prior to fill placement. The recommendations for potential stabilization techniques that may be used are provided in the *Project Earthworks* section of GROUND's April 22, 2009 dated report.

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5. The Contractor should take extreme care not to allow water to pond within the foundation excavations. If water is allowed to pond within the foundation excavations, significant re-working, placement of geotextile reinforcement and imported granular fill may also be necessary to achieve stable platform for footing placement.
6. Footings should have a minimum footing dimension of 16 inches. Actual footing dimensions, however, should be determined by the Structural Engineer, based on the design loads.
7. Footings should be provided with adequate soil cover above their bearing elevation for frost protection. Footings should be placed at a bearing elevation 3 or more feet below the lowest adjacent exterior finish grades.
8. Continuous foundation walls should be reinforced top and bottom to span an unsupported length of at least 10 feet.
9. The lateral resistance of spread footings will be developed as sliding resistance of the footing bottoms on the foundation materials and by passive soil pressure against the sides of the footings. Sliding friction at the bottom of footings may be taken as 0.35 times the vertical dead load. An allowable passive soil pressure of 360 psf per foot of embedment may be used, to a maximum of 3,600 psf. The upper 1-foot of embedment should not be relied upon for passive resistance, however.
10. Compacted fill placed against the sides of the footings should be compacted to at least 95 percent relative compaction in accordance with the recommendations in the *Project Earthworks* section of GROUND's April 22, 2009 dated report.
11. Care should be taken when excavating the foundation to avoid disturbing the supporting materials. Hand excavation or careful backhoe soil removal may be required in excavating the last few inches.
12. Foundation soils may be disturbed or deform excessively under the wheel loads of heavy construction vehicles as the excavations approach footing levels. Construction equipment should be as light as possible to limit development of this condition. The use of track-mounted vehicles is suggested because they

exert lower contact pressures. The movement of vehicles over proposed foundation areas should be restricted.

13. All footing areas should be compacted with a vibratory plate compactor prior to placement of concrete.

A Geotechnical Engineer should be retained to observe and test all footing excavations prior to placement of reinforcing steel or concrete.

### **WATER-SOLUBLE SULFATES**

The concentrations of water-soluble sulfates measured in selected samples obtained from Test Hole 3, P4, and P8 ranged up to 0.02 percent by weight (see Table 1). Such concentrations of water-soluble sulfates represent a negligible degree of sulfate attack on concrete exposed to these materials. Degrees of attack are based on the scale of 'negligible,' 'moderate,' 'severe' and 'very severe' as described in the "Design and Control of Concrete Mixtures," published by the Portland Cement Association.

Based on these data GROUND, makes no recommendation for use of a special, sulfate-resistant cement in project concrete.

### **PAVEMENT SECTIONS**

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of the subgrade soils and traffic loadings. Pavement sections were developed in general accordance with the design guidelines and procedures of the American Association of State Highway and Transportation Officials (AASHTO), Colorado Department of Transportation (CDOT), and University of Colorado at Boulder construction practice.

#### ***Subgrade Materials***

Based on the results of our field and laboratory studies, subgrade materials in the proposed pavement areas consisted predominantly of clayey sands and sandy clays. These materials were ranged between A-4, A-6, and A-7-6 soils and generally consisted of A-6 soils in accordance with the AASHTO classification system.

Resilient Modulus ( $M_R$ ) testing (AASHTO T-307) was performed on a representative composite sample of the subgrade materials encountered at the site. Typically, the R-value, unconfined compressive strength, California Bearing Ratio (CBR), or other index properties of subgrade materials have been obtained and the resilient modulus obtained only by correlation. However, due to the variability in the correlations, subjecting representative samples of the subgrade to the actual resilient modulus testing is the most accurate way to determine soil support characteristics for use in pavement design.

A dynamic load test, the resilient modulus measures the elastic rebound stiffness of flexible pavement materials, base courses and subgrades under repeated loading. The loading cycles were applied under various confining and deviatoric stresses as specified in AASHTO T-294. The material was compacted to 95 percent of maximum dry density at optimum moisture content, and at 2 percent and 4 percent above the optimum, based on AASHTO T-99 (the "standard Proctor") for cohesive soils, or AASHTO T-180 (the "modified Proctor") for granular soils.

The resilient modulus of a material at 2 percent above optimum moisture content typically is used for the pavement design for fine-grained soils that classify as A-4, A-6, or A-7. The resilient modulus at the optimum moisture content is typically used for granular soils that classify as A-1 or A-2. For the shallow site soils, the resilient modulus at 2 percent above optimum moisture content was taken as representative of the subgrade materials. Therefore, a resilient modulus 4,160 psi was used for pavement design calculations obtained at 2 percent above optimum moisture content.

It is important to note that significant decreases in soil support as quantified by the resilient modulus have been observed as the moisture content increases above the optimum. Therefore, pavements that are not properly drained may experience a loss of the soil support and subsequent reduction in pavement life.

### ***Pavement Distress Survey***

#### ***30<sup>th</sup> Street Bus Stop***

The 30<sup>th</sup> Street Bus Stop consisted of a concrete paved apron at the bus stop location and asphalt paved drive lane and automobile parking areas.

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The concrete pavement section exhibited severe distress, indicative of subgrade support failure, at two separate locations. The curb and gutter adjacent to these areas were also distressed. Photos 1 and 2 show the distress of the concrete paved apron and curb and gutter. With the exception of the above-mentioned distressed areas, the concrete pavement section appeared to be in fair condition.



The asphalt portion of the pavement section exhibited moderate to high severity distress, including rutting, alligator cracking, fatigue cracking, potholes, etc.) Signs of some preventative maintenance observed consisted of moderate severity asphalt patching in areas of previous apparent utility work and repair of some of the higher distressed areas. The areas of patching also exhibited distress ranging from moderate to high severity. Photos 3 and 4 show the distress observed on the asphalt-paved portions of the 30<sup>th</sup> Street Bus Stop. Overall, the asphalt-paved areas were in poor condition and exhibited signs of subgrade failure and pavement failure, likely due to heavy traffic.



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It appears that the subgrade failures observed are a result of excessive wetting of the subgrade materials combined with high traffic loads imposed by the buses.

Based on the extent and severity of distresses observed, the measured thickness of the asphalt pavements within the bus drive lane, desired age of the rehabilitated pavements (greater than 10 years), and our experience; sufficient rehabilitation of the pavement by commonly used rehabilitation methods, such as mill and overlay, chip-seal, fog-seal, patching, etc., will likely not provide adequate design life for the rehabilitated pavement section and are not recommended. GROUND recommends reconstruction of the existing pavement section for the 30<sup>th</sup> Street Bus Stop in addition to improving subgrade support and surface drainage. GROUND has provided geotechnical pavement section recommendations below, which can be utilized for reconstruction of the existing 30<sup>th</sup> Street Bus Stop alignment.

*Asphalt Surfaced Parking Areas*

The asphalt-surfaced parking lots exhibited distresses, including fatigue, longitudinal, transverse and edge cracking, with severity levels ranging from low to high. Moderate to high severity distresses were observed primarily in the drive lanes, and low to moderate distresses were observed in the parking stalls. Signs of some preventative maintenance observed consisted of sealing of a small portion of the longitudinal and transverse cracking, moderate severity asphalt patching in areas of previous apparent utility work, and repair of some of the higher distressed areas. These areas of patching also exhibited distress ranging from low to moderate severity. Photos 5 and 6 show the distress observed on the asphalt paved portions of the paved parking areas and driveline. Overall, the asphalt-paved parking areas located to the north of the proposed building footprint were in poor condition.



Recommendations for possible rehabilitation and reconstruction of the existing asphalt pavements, as well as pavement sections for the gravel surface parking lots are provided below.

### ***Design Traffic***

#### ***30<sup>th</sup> Street Bus Stop***

For the 30<sup>th</sup> Street Bus Stop, the following traffic count information was provided by the University of Colorado at Boulder Facilities Management Department:

- Based on current bus routes and schedules, 190 buses drive through the 30<sup>th</sup> Street Bus Stop on a typical day.
- This number could increase by 30 to 50 percent during the life cycle of the improvement planned.

Based on this information and assuming a life cycle of 20-years, we estimate that the number of buses that drive through the bus stop in 20 years could be as much as 285 buses per day. Typically for a 20-year pavement design, the midpoint traffic volume value is used for design. Therefore, GROUND used the midpoint daily bus traffic of 238 buses per day to calculate the equivalent 18-kip single-axle loading (ESAL<sub>18</sub>) of 1,261,352 for the 30<sup>th</sup> Street Bus Stop. Our calculations are provided in Appendix A. Please note that the calculated concrete pavement section thickness was increased by 0.25 inches to account for diamond grinding at 10 years.

#### ***35<sup>th</sup> Street, Access Drives, and Automobile Parking Areas***

Specific traffic loadings for 35<sup>th</sup> Street, Access Drives, and Automobile Parking areas were not available at the time of this reports preparation. As requested, the recommended Equivalent (18 Kip) Daily Load Application (EDLA) values that are provided on page 3 under Section 02513 of University of Colorado at Boulder Standards were utilized to calculate the ESAL<sub>18</sub> values for 20-year design lives. Therefore, an equivalent 18-kip daily load application (EDLA) value of 50 was used for the 35<sup>th</sup> Street (campus collector street), an EDLA value of 30 was used for access drives, and an EDLA value of 10 was used for automobile parking areas. The EDLA values of 50, 30,

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If design traffic loadings differ significantly from these assumed values, GROUND should be notified to re-evaluate the pavement recommendations below.

***Pavement Design***

***New Pavement Sections***

The soil resilient modulus and the design ESAL values were used to determine the required design structural number for the project pavements. The required structural number was then used to develop recommended pavement sections. Pavement designs were based on the DARWin™ computer program that solves the 1993 AASHTO pavement design equation. Pavement design parameters and calculations are summarized in Appendix A. Reliability Levels of 85 and 95 percent was utilized for design of the flexible and rigid Portland cement concrete pavement (PCCP) sections, respectively. Structural coefficients of 0.40 and 0.12 were used for hot bituminous asphalt (HBA) and high quality aggregate base course (ABC), respectively. The table provided below summarizes the recommended minimum pavement sections:

**Recommended Minimum Pavement Section Summary**

Location	Design Traffic 18-kip Equivalent Single Axle Loads	Flexible Pavement (inches)		Rigid Pavement (inches)
		Full Depth	Composite	Portland Cement Concrete
	<i>ESAL<sub>20</sub></i>	<i>(HBA)</i>	<i>(HBA / ABC)</i>	<i>(PCCP / ABC)</i>
Automobile Parking Areas	73,000	7.0	4.5 / 10	-
Drive Lanes	219,000	8.0	5.0 / 10	-
35th Street (Campus Collector)	365,000	9.0	6.0 / 9.0	-
30th Street Bus Stop	1,261,352	-	-	8.0 / 6.0

*Please refer to the Subgrade Preparation section below for additional information regarding pavement subgrade preparation.*

***Existing Paved Parking Area Rehabilitation Alternate***

*Existing Paved Parking Area Rehabilitation Alternate*

As an alternate to reconstruction, the existing paved automobile parking areas could be rehabilitated by standard pavement mill and overlay in order to extend the life of the existing pavement. The Owner must be aware that if rehabilitation is chosen for the existing northern parking lots, the life of the pavement will be of a significantly shorter duration than any newly constructed pavement.

Based on our calculations (see Appendix A) a 2-inch mill and a 3-inch overlay would provide the pavement with the required structural number for a 5-year design life for the automobile parking areas. It may be necessary to further mill the existing asphalt to meet proposed and existing elevations, where necessary. In addition, milling may be complicated by the level of distress and existing thickness of the asphalt.

Prior to asphalt placement, all cracks in the milled surface should be sealed. The Contractor should anticipate that local areas would require more extensive repairs in areas where moderate to high severity distresses were observed. Use of a geotextile paving fabric may be beneficial to reduce reflective cracking, especially over areas where moderate to high severity fatigue cracking was observed. The Geotechnical Engineer should be retained to observe the milled surface and identify areas for greater depths of removal and replacement.

If a new overlay will be performed, it should be placed in conjunction with the last lift of the pavement section for any construction/reconstructed portion(s) of the proposed northern parking lots.

***Hot Bituminous Asphalt (HBA)***

The asphalt pavement shall consist of a bituminous plant mix composed of a mixture of high quality aggregate and bituminous material, which meets the requirements of a job-mix formula established by the University of Colorado at Boulder. The asphalt material used should be based on a SuperPave Gyration Design Revolution.

***Portland Cement Concrete Pavement (PCCP)***

Concrete pavements should consist of a plant mix composed of a mixture of aggregate, portland cement and appropriate admixtures meeting the requirements of a job-mix

formula established by a qualified engineer as well as applicable design requirements of *University of Colorado at Boulder*. Concrete should have a minimum modulus of rupture of third point loading of 650 psi. Normally, concrete with a 28-day compressive strength of 4,500 psi should develop this modulus of rupture value. The concrete should be air-entrained with approximately 6 percent air and should have a minimum cement content of 6 sacks per cubic yard. Maximum allowable slump should be 4 inches. To reduce surficial spalling resulting from freeze-thaw cycling, we suggest that pavement concrete meet the requirements of CDOT Class P concrete. In addition, the use of de-icing salts on concrete pavements during the first winter after construction will increase the likelihood of the development of scaling. Placement of flatwork concrete during cold weather so that it is exposed to freeze-thaw cycling before it is fully cured also increases its vulnerability to scaling. Concrete placed during cold weather conditions should be blanketed or tented to allow full curing. Depending on the weather conditions, this may result in 3 to 4 weeks of curing, and possibly more.

The concrete pavement should contain effective sawed or formed joints. In areas of repeated turning stresses including the 30<sup>th</sup> Street Bus Stop, we recommend that the concrete pavement joints be fully tied and doweled. If the concrete pavement joints are not fully tied and doweled, the recommended concrete pavement section thickness should be increased by 1-inch. We suggest that civil design consider joint layout in accordance with CDOT's M Standards. Standard plans for placement of ties and dowels, etc., (CDOT M Standards) for concrete pavements can be found at the CDOT website: <http://www.dot.state.co.us/DesignSupport/>

### ***Aggregate Base Course (ABC)***

The aggregate base material should meet the criteria of CDOT Class 6 aggregate base course. Base course should be placed in uniform lifts not exceeding 8 inches in loose thickness and compacted to at least 95 percent of the maximum dry density a uniform moisture contents within 3 percent of the optimum as determined by ASTM D1557 / AASHTO T-180, the "modified Proctor."

### ***Subgrade Preparation***

Shortly before placement of pavement, including aggregate base, the exposed subgrade soils should be scarified to a depth of at least 12 inches, thoroughly mixed, moisture

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conditioned and replaced as properly compacted fill as outlined in the *Project Earthworks* section of GROUND's April 22, 2009 report. Subgrade preparation should extend the full width of the pavement from back-of-curb to back-of-curb. As recommended in the *Exterior Flatworks and Project Hardscaping* sections of GROUND's April 22, 2009 dated report, sidewalks and other hardscaping also should be underlain by at least 12 inches of properly moisture conditioned and compacted fill.

Due to the nature of the majority of site soils, some post-construction movements of the pavements should be anticipated. A greater depth of excavation and replacement will result in improved pavement performance over its design life. Recommendations in this regard can be provided upon request.

Recommendations for placement and compaction of fill soils are provided in the *Project Earthworks* section of GROUND's April 22, 2009 dated report.

The Contractor should be prepared either to dry the subgrade materials or moisten them, as needed, prior to compaction. Some site soils likely will "pump" or deflect during compaction if moisture levels are not carefully controlled. The Contractor should be prepared to process and compact such soils to establish a stable platform for paving, including use of chemical stabilization, if necessary.

The Contractor should be prepared to perform additional stabilization of the subgrade during the reconstruction of the 30<sup>th</sup> Street Bus Stop pavement sections. As previously mentioned under the pavement distress observation section, several areas within the bus stop has experienced distress due to inadequate subgrade support. Our experience indicates that the subgrade materials in these areas are likely present at moisture contents that are several points above their optimum and will likely pump and deflect excessively under construction traffic load. These areas will likely need additional stabilization measures. Recommendations for stabilizing soft, wet, and unstable subgrade constructions were previously provided in GROUND's April 22, 2009 report, on Page 30. These recommendations should also be followed to stabilize the subgrade, as necessary, within the 30<sup>th</sup> Street Bus Stop and elsewhere on this site.

Immediately prior to paving, the pavement subgrade should be proof rolled with a heavily loaded, pneumatic tired vehicle. The proof roll should be performed in accordance with Section 203.09 of the current CDOT Standard Specifications for Road

and Bridge. Areas that show excessive deflection during proof rolling should be excavated and replaced and stabilized. Areas allowed to pond prior to paving will require significant re-working prior to proof rolling.

### ***Pavement Maintenance and Performance***

The collection and diversion of surface drainage away from paved areas is extremely important to satisfactory performance of the pavements. The subsurface and surface drainage systems should be carefully designed to ensure removal of the water from paved areas and subgrade soils. Allowing surface waters to pond on pavements will cause premature pavement deterioration. Where topography, site constraints or other factors limit or preclude adequate surface drainage, pavements should be provided with edge drains to reduce loss of subgrade support.

GROUND's experience indicates that longitudinal cracking is common in asphalt-pavements generally parallel to the interface between the asphalt and concrete structures such as curbs, gutters or drain pans. Distress of this type is likely to occur even where the subgrade has been prepared properly and the asphalt has been compacted properly.

The standard care of practice in pavement design describes the recommended flexible pavement section as a "20-year" design pavement; however, most pavements will not remain in satisfactory condition without routine, preventive maintenance and rehabilitation procedures performed throughout the life of the pavement. Preventive pavement treatments are surface rehabilitation and operations applied to improve or extend the functional life of a pavement. These treatments preserve, rather than improve, the structural capacity of the pavement structure. In the event the existing pavement is not structurally sound, the preventive maintenance will have no long-lasting effect. Therefore, a routine maintenance program to seal cracks, repair distressed areas, and perform thin overlays throughout the life of the pavement is recommended.

A crack sealing and fog seal / chip seal program should be performed on the roadway alignment on a regular basis. After approximately 8 to 10 years, patching, additional crack sealing, and asphalt overlay may be required. Prior to future overlays, it is important that all transverse and longitudinal cracks be sealed with a flexible, rubberized crack sealant in order to reduce the potential for propagation of the crack through the

overlay. Traffic volumes that exceed the values utilized by this report will likely necessitate the need of pavement maintenance practices on a schedule of shorter timeframe than that stated above. "The greatest benefit of preventive maintenance is achieved by placing the treatments on sound pavements that have little or no distress".

## **CLOSURE**

### ***Geotechnical Review***

The poor performance of foundations and subsurface structures has been directly attributed to inadequate geotechnical review and earthwork quality control. Therefore, a Geotechnical Engineer should be retained to review project plans and specifications to evaluate whether they comply with the intent of the recommendations in this report. The author of this report and/or the reviewing engineer should be contacted directly to provide this review. The review should be reported in writing.

Project earthwork construction operations should be observed by the Geotechnical Engineer. All excavations should be observed by the Geotechnical Engineer prior to placement of fill or backfill soils, installation of shoring, or foundation construction. Placement of fill/backfill soils should be observed by the Geotechnical Engineer, and the soils tested.

The geotechnical recommendations presented in this report are highly contingent upon observation and testing of project earthworks by representatives of GROUND. If another geotechnical consultant is selected to provide construction observation and quality control, then that consultant must assume all responsibility for the geotechnical aspects of the project by concurring in writing with the recommendations in this report, or by providing alternative recommendations.

### ***Limitations***

This report has been prepared for University of Colorado at Boulder as it pertains to design of the subject University of Colorado Boulder Williams Village Phase IIA project as described herein. It may not contain sufficient information for other parties or other purposes. In addition, GROUND has assumed that project construction will commence by Spring 2011. Changes in project plans or schedule should be brought to the

attention of the Geotechnical Engineer, in order that the geotechnical recommendations may be re-evaluated and, as necessary, modified.

The geotechnical conclusions and recommendations in this report relied upon subsurface exploration at a limited number of exploration points, as shown in Figure 1. Subsurface conditions were interpolated between and extrapolated beyond these locations. It is not possible to guarantee the subsurface conditions are as indicated in this report. Actual conditions exposed during construction should be anticipated to differ, somewhat, from those encountered during site exploration.

Contractors should review all available project information prior to providing construction/service bids. If during construction, surface, soil, bedrock, or groundwater conditions appear to be at significant variance with those described herein, the Geotechnical Engineer should be advised at once, so that re-evaluation of the recommendations may be made in a timely manner. Findings were dependent on the limited amount of direct evidence obtained at the time of this geotechnical evaluation. Our recommendations were developed for the site conditions as described above.

A Contractor who relies upon this report for development of his scope of work or cost estimates may consider the geotechnical information in this report to be inadequate for his purposes or find the geotechnical conditions described herein to be at variance with his experience in the greater project area. In such cases, the Contractor should obtain the additional geotechnical information that he considers necessary to develop his workscope and cost estimates with sufficient precision.

The recommendations and criteria provided in this report were based on the data presented herein, and our experience in the general project area with similar projects, and our engineering judgment with regard to the applicability of the data and methods of forecasting future performance. A variety of engineering parameters were considered as indicators of potential future soil movements. Our recommendations were based on our judgment of "likely movement potentials," (i.e., the amount of movement likely to be realized if site drainage is effective, estimated to a reasonable degree of engineering certainty) as well as our assumptions about the Owner's willingness to accept geotechnical risk. "Maximum possible" movement estimates necessarily will be larger than those presented herein. They also have a lower likelihood of being realized in our opinion, and generally require more expensive measures to address. We encourage

University of Colorado at Boulder  
Williams Village Phase II A – Extended Scope  
Boulder, Colorado

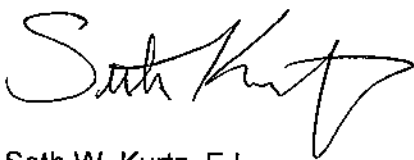
the Owner or prospective future owners, upon receipt of this report, however, to discuss these risks and the geotechnical alternatives with us.

Engineering consulting and design practice necessarily involves weighing the risks inherent in a given design approach against the construction and maintenance costs associated with reducing those risks. The Owner (and subsequent prospective future owners) must, therefore, understand the risks and remedial approaches presented in this report (and the risk-cost trade-offs addressed by the Civil Engineer and Structural Engineer) in order to direct his Design Team to the portion of the Higher Cost / Lower Risk – Lower Cost / Higher Risk spectrum in which this project should be designed. If the Owner or a prospective future owner does not understand these risks, it is critical that he request additional information or clarification so that his expectations reasonably can be met.

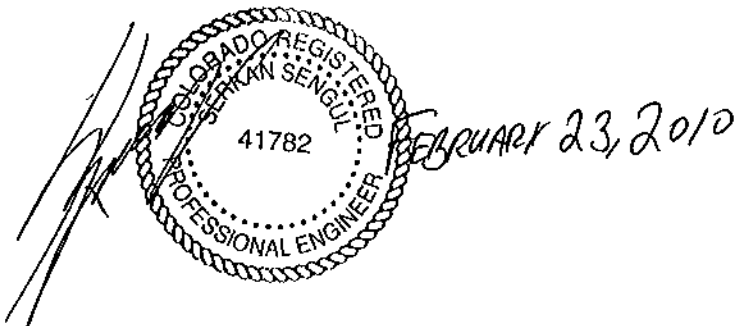
This report was prepared in accordance with generally accepted soil and foundation engineering practice in the Boulder, Colorado area at the date of preparation. GROUND makes no other warranties, either express or implied, as to the professional data, opinions or recommendations contained herein.

Sincerely,

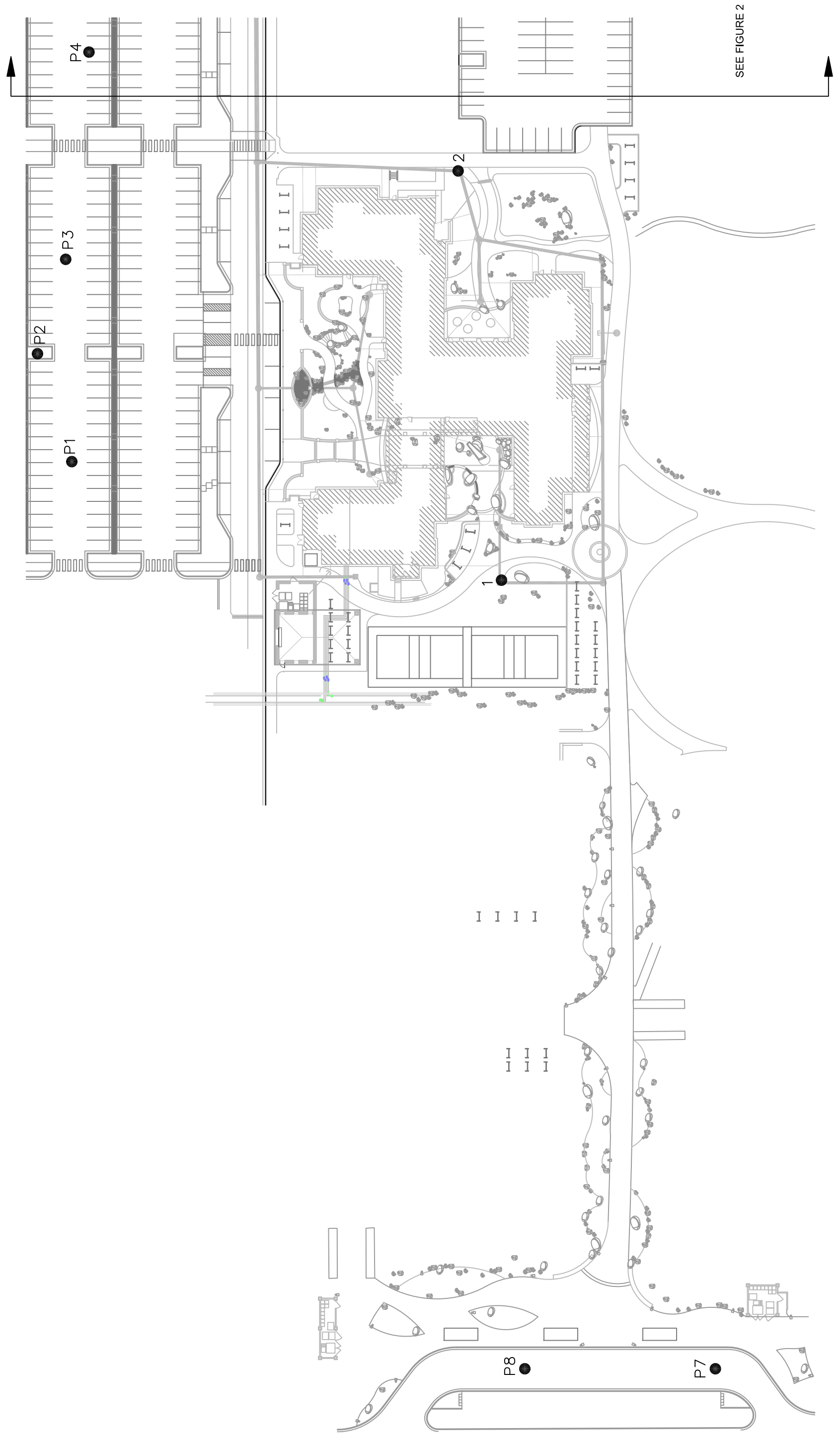
**GROUND Engineering Consultants, Inc.**



Seth W. Kurtz, E.I.



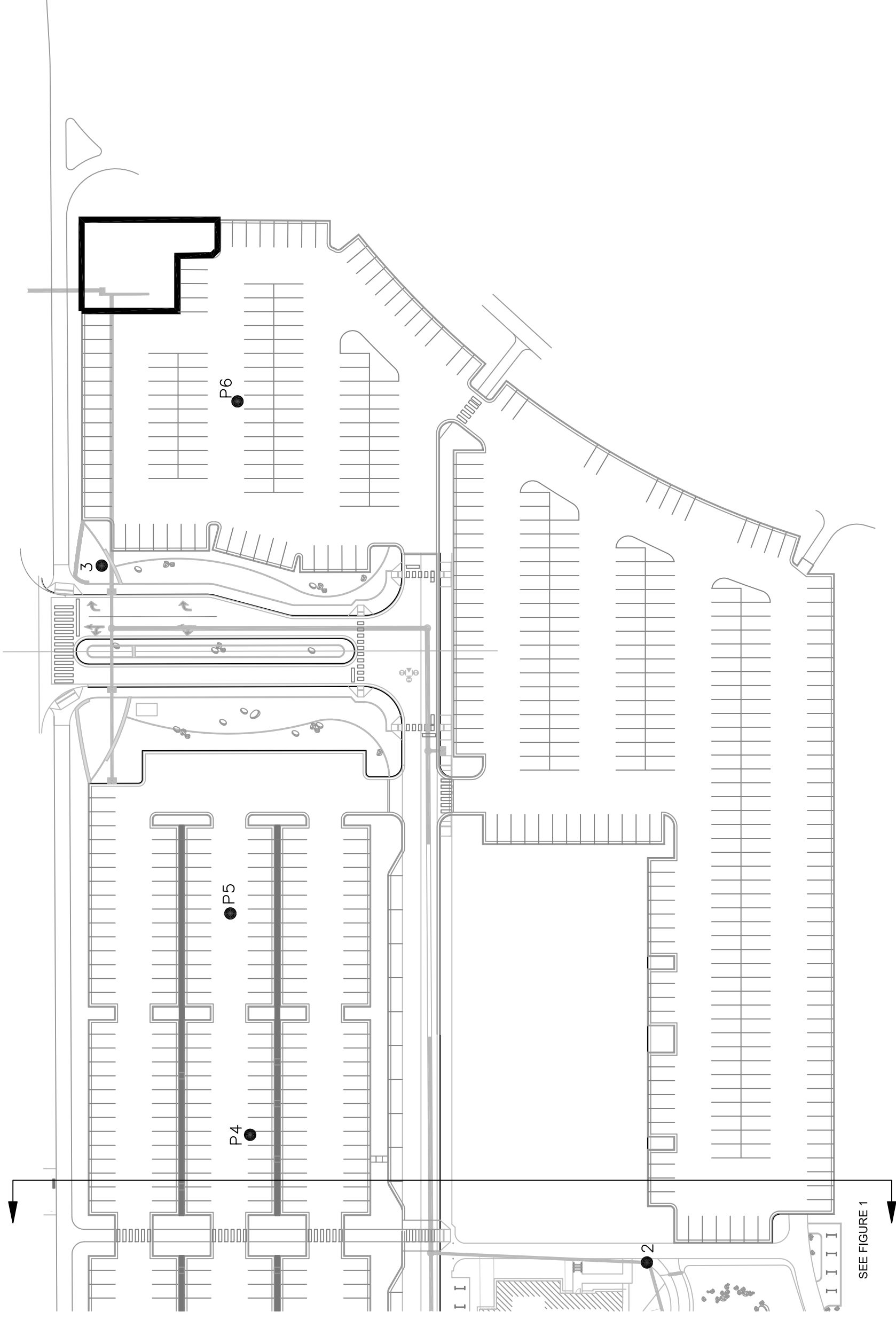
Reviewed by Serkan Sengul, P.E.



1 ● Indicates test hole number and approximate location.

(Not to Scale)

<b>GROUND</b> ENGINEERING CONSULTANTS	
LOCATION OF TEST HOLES	
JOB NO.: 09-3011	FIGURE: 1
CADFILE NAME: 3011ASITE.DWG	

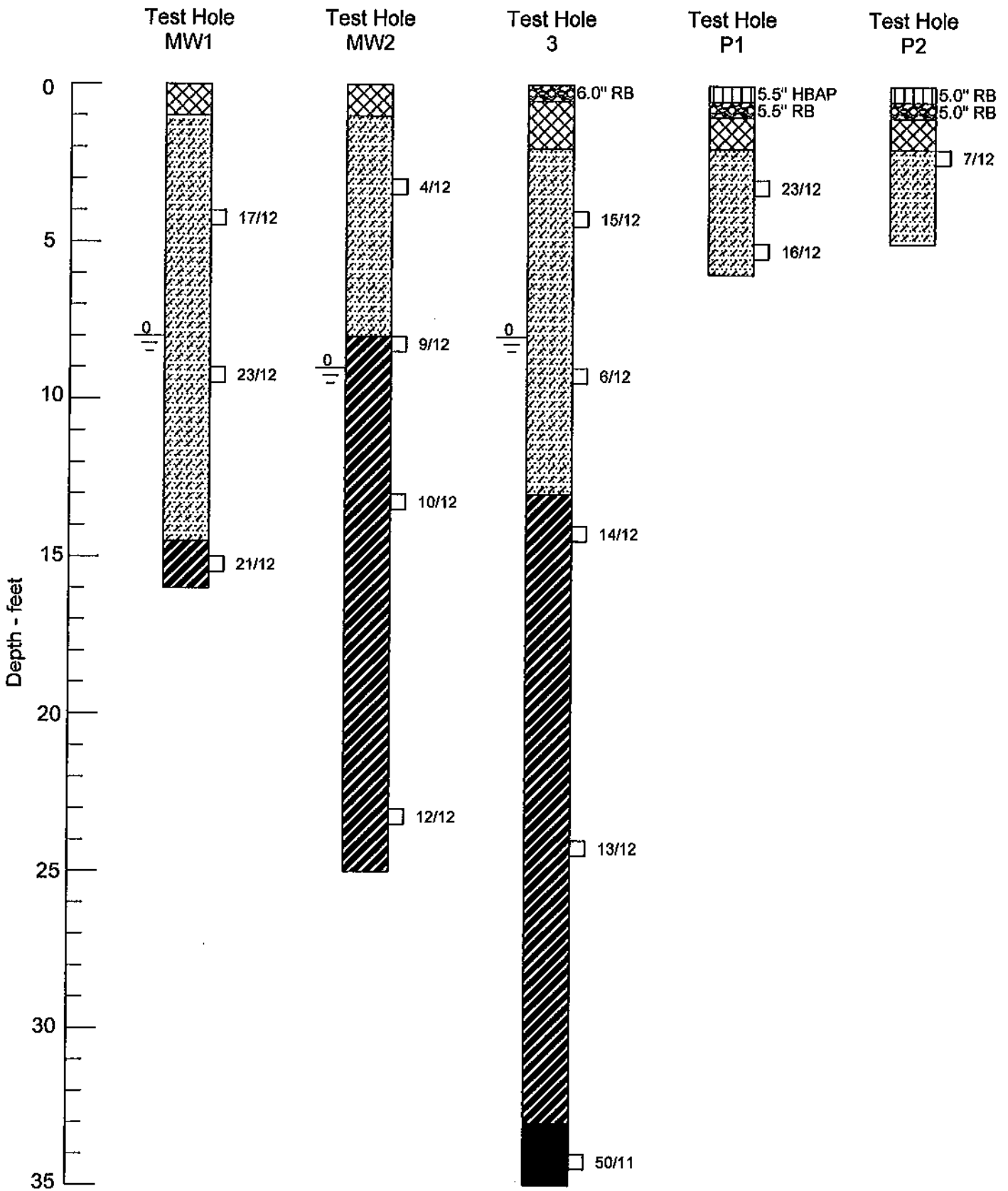


SEE FIGURE 1



(Not to Scale)

- 1 ● Indicates test hole number and approximate location.



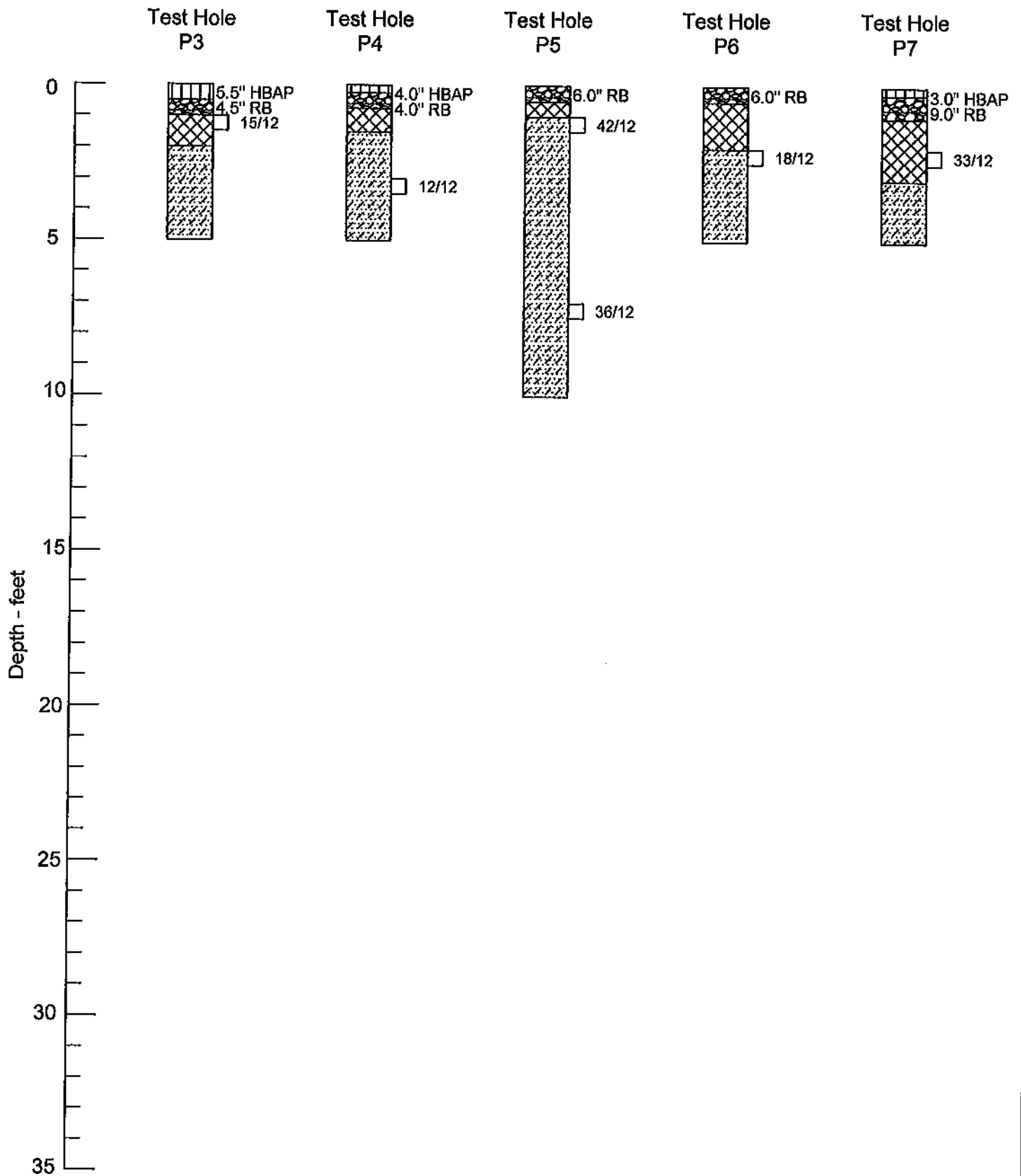
**GROUND**  
ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

JOB NO.: 09-3011

FIGURE: 3

CADFILE NAME: 3011A LOG01.DWG



# GROUND

ENGINEERING CONSULTANTS

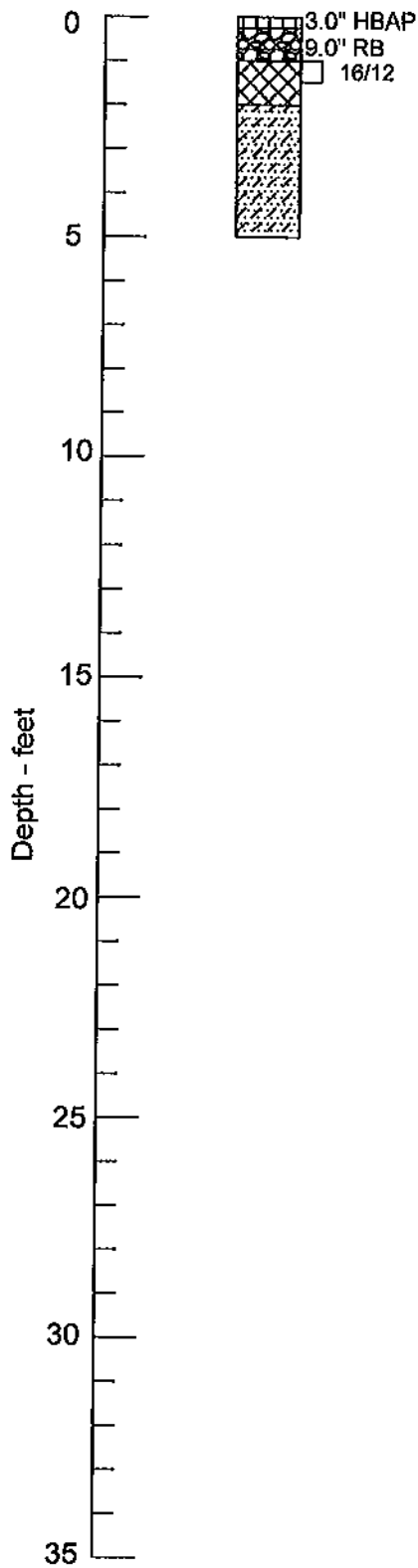
## LOGS OF TEST HOLES

JOB NO.: 09-3011

FIGURE: 4

CADFILE NAME: 3011ALOG02.DWG

Test Hole  
P8



**GROUND**

ENGINEERING CONSULTANTS

LOGS OF TEST HOLES

JOB NO.: 09-3011

FIGURE: 5

CADFILE NAME: 3011ALOG03.DWG

LEGEND:



Hot Bituminous Asphalt Pavement (HBAP)



Road Base (RB)



Fill: Consisted of sandy clays with gravels. They were fine to coarse grained with gravels, low to moderately plastic, loose to compact, slightly moist to moist, and red-brown in color.



Sand and Clay: Consisted of sandy clays with gravels and clayey sands with gravels. They were fine to coarse grained with gravels, low to moderately plastic, stiff/loose to very stiff/medium dense, moist to wet, and red-brown to gray-brown in color with occasional iron staining.



Weathered Claystone: Fine grained, highly plastic, stiff to very stiff, moist to wet, and gray to gray-brown in color with iron staining.



Claystone Bedrock: Fine grained, highly plastic, very hard, moist, and gray to gray-brown in color with iron staining.



Drive sample, 2-inch I.D. California liner sample

23/12

Drive sample blow count, indicates 23 blows of a 140-pound hammer falling 30 inches were required to drive the sampler 12 inches.



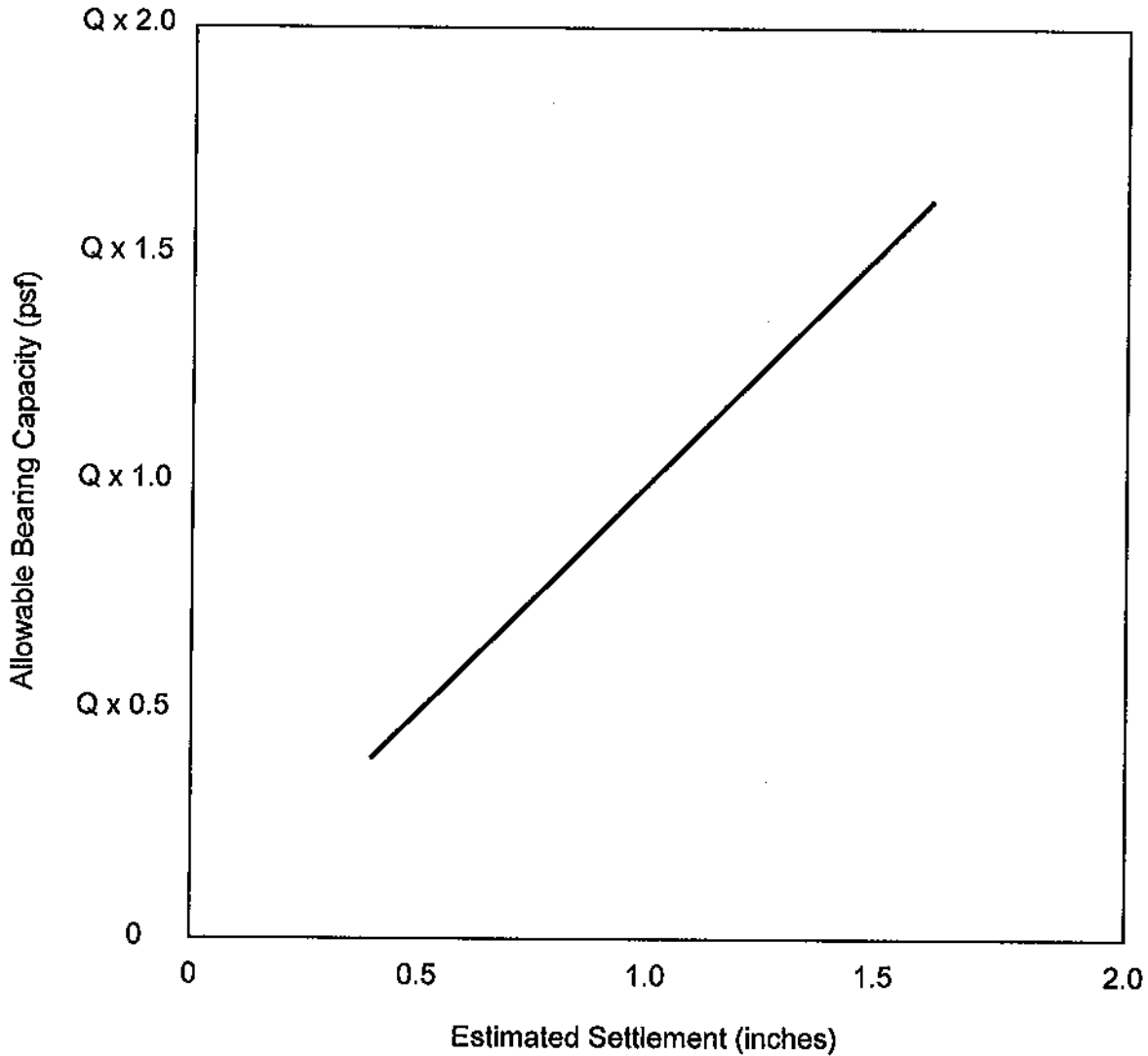
Depth to water level and number of days after drilling that measurement was taken.

NOTES:

- 1) Test holes were drilled on 01/13/10 with 4-inch diameter continuous flight power augers.
- 2) Locations of the test holes were measured approximately by pacing from features shown on the site plan provided.
- 3) Elevations of the test holes were not measured and the logs of the test holes are drawn to depth.
- 4) The test hole locations and elevations should be considered accurate only to the degree implied by the method used.
- 5) The lines between materials shown on the test hole logs represent the approximate boundaries between material types and the transitions may be gradual.
- 6) Groundwater level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in the water level may occur with time.

The material descriptions on this legend are for general classification purposes only. See the full text of this report for descriptions of the site materials and related recommendations.

<b>GROUND</b> ENGINEERING CONSULTANTS	
LEGEND AND NOTES	
JOB NO.: 09-3011	FIGURE: 6
CADFILE NAME: 3011ALEG.DWG	



$Q = 2,500 \text{ psf}$

Note: Design should be controlled by settlement. Estimated settlement values indicated above are based on drained conditions. If foundation materials become wet, the allowable bearing capacity will be reduced and result in larger estimated settlement. This relationship is based on footing widths of 1 to 4 feet. If the footing width is to be greater than 4 feet, we should be notified to reevaluate these recommendations.

<b>GROUND</b> ENGINEERING CONSULTANTS	
BEARING CAPACITY CHART	
JOB NO.: 09-3011	FIGURE: 7
CADFILE NAME: 3011ABRCP.DWG	

# COMPACTION TEST REPORT

Curve No.: 2819

Project No.: 09-3011  
 Project: Williams Village Phase IIA

Date: 1/19/2010

Location: Test Holes P1 - P8

Elev./Depth: 0 to 5 feet

Sample No. 2819

Remarks:

## MATERIAL DESCRIPTION

Description: Composite Bulk Sample

Classifications -

USCS: (SC)g

AASHTO: A-6(1)

Nat. Molst. =

Sp.G. =

Liquid Limit = 30

Plasticity Index = 12

% > No.4 = 15.0 %

% < No.200 = 37.2 %

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 122.3 pcf	117.5 pcf
Optimum moisture = 10.3 %	11.5 %

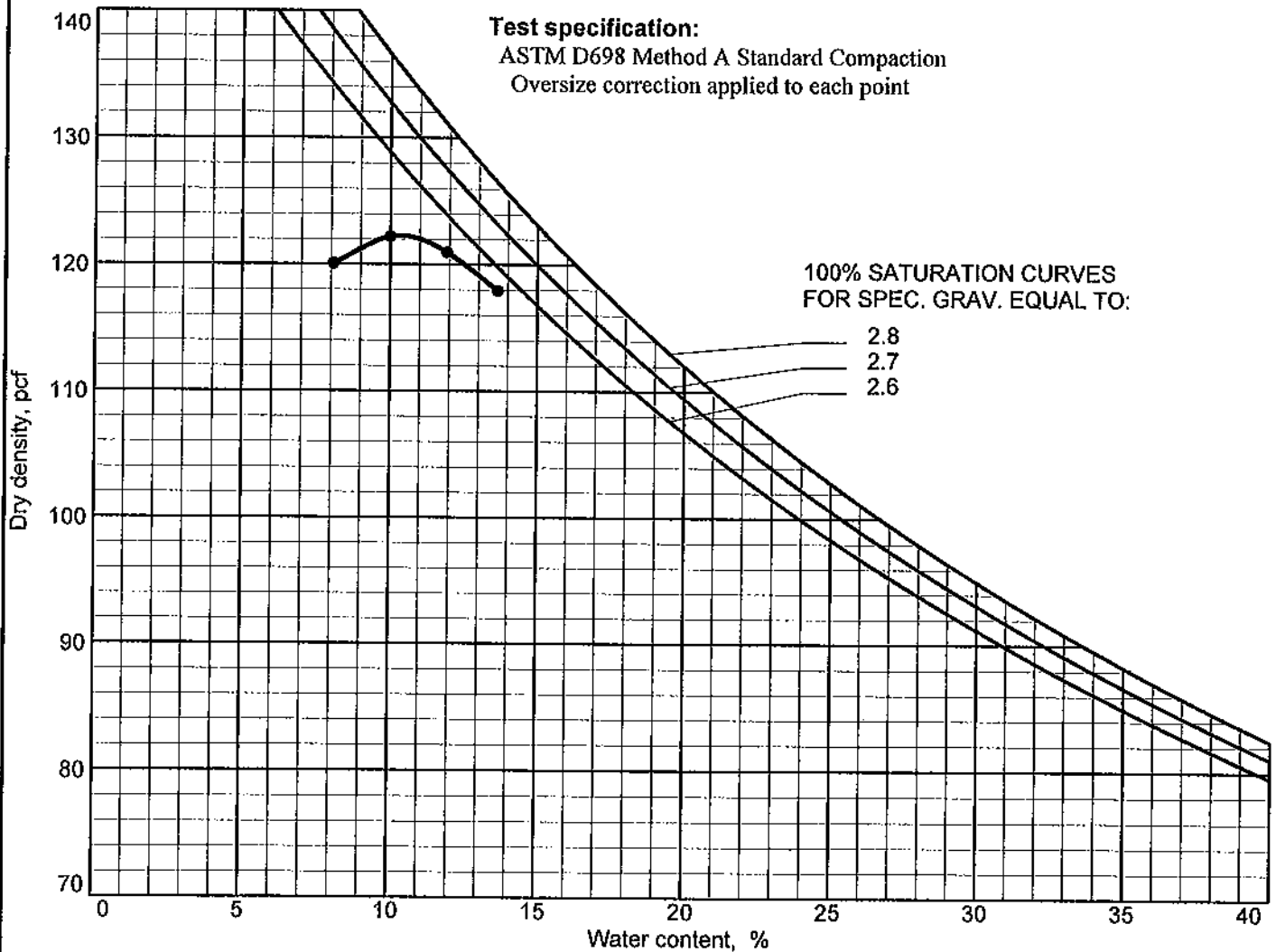


TABLE 1  
SUMMARY OF LABORATORY TEST RESULTS

Sample Test Hole No.	Location Depth (feet)	Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Percent Passing No. 200 Sieve	Atterberg Limits		Percent Swell (200 psf Surcharge)	Water Soluble Sulfates (%)	USCS Classification	AASHTO Classification (GI)	Soil or Bedrock Type
				Gravel (%)	Sand (%)	Silt (%)		Liquid Limit (%)	Plasticity Index (%)					
1	4	7.3		31	58		11	27	11			(SW-SC)g	A-2-6	Well-Graded SAND with Clay and Gravel
1	9	20.0		38	37	19	26	28	9			(GC)s	A-2-4	Clayey GRAVEL with Sand
1	15	20.7	106.1				85	44	20			(CL)s	A-7-6(18)	Weathered CLAYSTONE
2	3	26.9	94.1	10	40		50	45	22			s(CL)	A-7-6(7)	Sandy Lean CLAY
2	8	11.2		33	46	16	22	27	9			(SC)g	A-2-4	Clayey SAND with Gravel
2	13	21.9		30	31	30	39	37	17			(SC)g	A-6(2)	Clayey SAND with Gravel
2	23	28.9	93.6				96	48	23			CL	A-7-6(26)	Weathered CLAYSTONE
3	4	18.3	109.5				67	43	20		0.01	s(CL)	A-7-6(12)	Sandy Lean CLAY
3	9	17.8	106.5				32	38	16			SC	A-2-6	Clayey SAND
3	14	19.8	108.1				71	47	24			(CL)s	A-7-6(16)	Weathered CLAYSTONE
P1	3	12.9	113.8				54	36	15			s(CL)	A-6(5)	Sandy Lean CLAY
P4	3	21.0	105.8				78	41	21	1.3	<0.01	(CL)s	A-7-6(16)	Lean CLAY with Sand
P8	1	11.4	123.9				40	22	9	0.5	0.02	SC	A-4	Fill: Clayey SAND

## **APPENDIX A**

### **PAVEMENT SECTION CALCULATIONS**

1993 AASHTO Pavement Design  
**DARWin Pavement Design and Analysis System**

A Proprietary AASHTOWare  
 Computer Software Product  
 Network Administrator

**Flexible Structural Design Module**

Job No. 09-3021  
 University of Colorado at Boulder  
 Williams Village Phase IIA  
 Automobile Parking Areas  
 Full Depth Asphalt Section

**Flexible Structural Design**

18-kip ESALs Over Initial Performance Period	73,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	85 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	4,160 psi
Stage Construction	1
 Calculated Design Structural Number	 2.66 in

**Specified Layer Design**

<u>Layer</u>	<u>Material Description</u>	Struct Coef. <u>(Ai)</u>	Drain Coef. <u>(Mi)</u>	Thickness <u>(Di)(in)</u>	Width <u>(ft)</u>	Calculated <u>SN (in)</u>
1	Hot Bituminous Asphalt	0.4	1	7	-	2.80
Total	-	-	-	7.00	-	2.80

1993 AASHTO Pavement Design  
**DARWin Pavement Design and Analysis System**

A Proprietary AASHTOWare  
 Computer Software Product  
 Network Administrator

**Flexible Structural Design Module**

Job No. 09-3021  
 University of Colorado at Boulder  
 Williams Village Phase IIA  
 Automobile Parking Areas  
 Composite Section

**Flexible Structural Design**

18-kip ESALs Over Initial Performance Period	73,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	85 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	4,160 psi
Stage Construction	1
Calculated Design Structural Number	2.66 in

**Specified Layer Design**

<u>Layer</u>	<u>Material Description</u>	Struct Coef. <u>(Ai)</u>	Drain Coef. <u>(Mi)</u>	Thickness <u>(Di)(in)</u>	Width <u>(ft)</u>	Calculated <u>SN (in)</u>
1	Hot Bituminous Asphalt	0.4	1	4.5	-	1.80
2	Aggregate Base Course	0.12	1	10	-	1.20
Total	-	-	-	14.50	-	3.00

1993 AASHTO Pavement Design  
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**Flexible Structural Design Module**

Job No. 09-3021  
 University of Colorado at Boulder  
 Williams Village Phase IIA  
 Drive Lanes  
 Full Depth Asphalt Section

**Flexible Structural Design**

18-kip ESALs Over Initial Performance Period	219,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	85 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	4,160 psi
Stage Construction	1
Calculated Design Structural Number	3.16 in

**Specified Layer Design**

<u>Layer</u>	<u>Material Description</u>	Struct Coef. <u>(Ai)</u>	Drain Coef. <u>(Mi)</u>	Thickness <u>(Di)(in)</u>	Width <u>(ft)</u>	Calculated <u>SN (in)</u>
1	Hot Bituminous Asphalt	0.4	1	8	-	3.20
Total	-	-	-	8.00	-	3.20

1993 AASHTO Pavement Design  
DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare  
Computer Software Product  
Network Administrator

Flexible Structural Design Module

Job No. 09-3021  
University of Colorado at Boulder  
Williams Village Phase IIA  
Drive Lanes  
Composite Section

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	219,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	85 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	4,160 psi
Stage Construction	1
Calculated Design Structural Number	3.16 in

Specified Layer Design

<u>Layer</u>	<u>Material Description</u>	Struct Coef. <u>(A<sub>i</sub>)</u>	Drain Coef. <u>(M<sub>i</sub>)</u>	Thickness <u>(D<sub>i</sub>)(in)</u>	Width <u>(ft)</u>	Calculated <u>SN (in)</u>
1	Hot Bituminous Asphalt	0.4	1	5	-	2.00
2	Aggregate Base Course	0.12	1	10	-	1.20
Total	-	-	-	15.00	-	3.20

1993 AASHTO Pavement Design  
**DARWin Pavement Design and Analysis System**

A Proprietary AASHTOWare  
**Computer Software Product**  
 Network Administrator

**Flexible Structural Design Module**

Job No. 09-3021  
 University of Colorado at Boulder  
 Williams Village Phase IIA  
 Drive Lanes  
 Full Depth Asphalt Section

**Flexible Structural Design**

18-kip ESALs Over Initial Performance Period	365,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	85 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	4,160 psi
Stage Construction	1
 Calculated Design Structural Number	 3.42 in

**Specified Layer Design**

<u>Layer</u>	<u>Material Description</u>	<u>Struct Coef. (Ai)</u>	<u>Drain Coef. (Mi)</u>	<u>Thickness (Di)(in)</u>	<u>Width (ft)</u>	<u>Calculated SN (in)</u>
1	Hot Bituminous Asphalt	0.4	1	9	-	3.60
Total	-	-	-	9.00	-	3.60

1993 AASHTO Pavement Design  
**DARWin Pavement Design and Analysis System**

A Proprietary AASHTOWare  
 Computer Software Product  
 Network Administrator

**Flexible Structural Design Module**

Job No. 09-3021  
 University of Colorado at Boulder  
 Williams Village Phase IIA  
 Drive Lanes  
 Composite Section

**Flexible Structural Design**

18-kip ESALs Over Initial Performance Period	365,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	85 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	4,160 psi
Stage Construction	1
Calculated Design Structural Number	3.42 in

**Specified Layer Design**

<u>Layer</u>	<u>Material Description</u>	Struct Coef. <u>(Ai)</u>	Drain Coef. <u>(Mi)</u>	Thickness <u>(Di)(in)</u>	Width <u>(ft)</u>	Calculated <u>SN (in)</u>
1	Hot Bituminous Asphalt	0.4	1	6	-	2.40
2	Aggregate Base Course	0.12	1	9	-	1.08
Total	-	-	-	15.00	-	3.48

1993 AASHTO Pavement Design  
**DARWin Pavement Design and Analysis System**

A Proprietary AASHTOWare  
 Computer Software Product  
 Network Administrator

**Flexible Structural Design Module**

Job No. 09-3021  
 University of Colorado at Boulder  
 Williams Village Phase IIA  
 Automobile Parking Areas (Previously Paved)  
 2" Mill and 3" Overlay Design For 5-Year Pavement Life

**Flexible Structural Design**

18-kip ESALs Over Initial Performance Period	18,250
Initial Serviceability	4.5
Terminal Serviceability	2
Reliability Level	75 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	4,160 psi
Stage Construction	1
Calculated Design Structural Number	1.98 in

**Specified Layer Design**

<u>Layer</u>	<u>Material Description</u>	Struct Coef. <u>(Ai)</u>	Drain Coef. <u>(Mi)</u>	Thickness <u>(Di)(in)</u>	Width <u>(ft)</u>	Calculated <u>SN (in)</u>
1	Hot Bituminous Asphalt	0.4	1	3	-	1.20
2	Existing Asphalt	0.2	1	2	-	0.40
3	Aggregate Base Course	0.1	1	4	-	0.40
Total	-	-	-	9.00	-	2.00

Project Name: Williams Village Phase IIA, 30th Street Bus Stop  
 Project Number: 09-3011

Pavement Design Setup

18-Kip Factors For Pavement Design High Average Method		
Vehicle Type	Pavement Type	
	Flexible (rate/1000)	Rigid (rate/1000)
Passenger Cars	0.0008	0.0008
Pickups	0.0075	0.0076
Single Units	0.249	0.285
Combinations	1.087	1.692
Buses	0.615	0.726

0.003 total  
 (pickups/pas. Cars)

Lane Configuration Chart	
# of Lanes	Configuration #
1	1.00
2	0.60
4	0.45
6	0.30
8	0.25

Daily Traffic Load (ADT): 238 busses per day

Daily Traffic Loads	% (of Total ADT)	# (of Total ADT)	Pavement Type		Lane Config # (from above chart)
			Flexible (rate/1000)	Rigid (rate/1000)	
Buses	100	238	0.615	0.726	172.788
			146.37		1.00

Total Yearly Load (ESAL):  
 20 Year Load (ESAL): 53,425 Flexible 63,068 Rigid  
 30 Year Load (ESAL): 1,068,501  
 10 Year Load (ESAL): 1,602,752  
 1,261,352  
 1,892,029  
 534,251 630,676

# Rigid Pavement Design - Based on AASHTO Supplemental Guide

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

## Results

Project # 09-3011

Description: Williams Village Phase IIA, Boulder, Colorado

Location: 30th Street Bus Stop

### Slab Thickness Design

Pavement Type	JPCP	
18-kip ESALs Over Initial Performance Period (million)	1.25	million
Initial Serviceability	4.5	
Terminal Serviceability	2.5	
28-day Mean PCC Modulus of Rupture	650	psi
Elastic Modulus of Slab	3,400,000	psi
Elastic Modulus of Base	25,000	psi
Base Thickness	6.0	in.
Mean Effective k-Value	50	psi/in
Reliability Level	95	%
Overall Standard Deviation	0.34	
Calculated Design Thickness	<b>7.75</b>	in

### Temperature Differential

Mean Annual Wind Speed	8.8	mph
Mean Annual Air Temperature	50.3	°F
Mean Annual Precipitation	15.3	in
Maximum Positive Temperature Differential	<b>6.36</b>	°F

### Modulus of Subgrade Reaction

<u>Period</u>	<u>Description</u>	<u>Subgrade k-Value, psi</u>
---------------	--------------------	------------------------------

Seasonally Adjusted Modulus of Subgrade Reaction		psi/in
Modulus of Subgrade Reaction Adjusted for Rigid Layer and Fill Section	<b>50</b>	psi/in

**Traffic**

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

<u>Vehicle Class</u>	Percent of <u>ADT</u>	<u>Annual Growth</u>	<u>Initial Truck Factor</u>	<u>Annual Growth in Truck Factor</u>	<u>Accumulated 18-kip ESALs</u> (millions)
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Total Calculated Cumulative ESALs		million
-----------------------------------	--	---------

**Faulting**

*Doweled*

Dowel Diameter		<b>1.25</b>	in
Drainage Coefficient		<b>1.10</b>	
Average Fault for Design Years with Design Inputs		<b>0.03</b>	in
Criteria Check	<b>PASS</b>		

*Nondoweled*

Drainage Coefficient		<b>1.1</b>	
Average Fault for Design Years with Design Inputs		<b>0.06</b>	in
Criteria Check	<b>PASS</b>		