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Engineering Consulting

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**SUBJECT: GEOTECHNICAL DESIGN RECOMMENDATIONS
UMPQUA VALLEY TENNIS CENTER
OUTDOOR TENNIS COURTS IMPROVEMENTS
1201 NW STEWART PARKWAY
ROSEBURG, OREGON**

Mr. Palm,

This report presents the results of our geotechnical investigation and evaluation of the site and existing court conditions for the proposed Umpqua Valley Tennis Center improvements project in Roseburg, Oregon.

The purpose of our investigation and this report was to evaluate the site area surface and subsurface conditions with a series of corings/borings through the existing outdoor tennis courts in order to assess the existing conditions of the asphaltic-concrete (AC) pavement support section and underlying subgrade soils conditions, and to provide recommendations for final design and construction/replacement of the tennis court improvements.

SITE AND PROJECT DESCRIPTION

The overall project site consists of the City of Roseburg's 236-acre Stewart Park, a large, multi-use property located on the east side of NW Stewart Parkway, in Roseburg, Oregon. See *Figure 1, Vicinity Map*. The subject tennis courts are part of the City's Umpqua Valley Tennis Center, which extends across an approximately 1.75-acre area on the west side of Stewart Park, located at the northeast corner of the intersection of NW Stewart Parkway and NW Harvey Avenue. The Umpqua Valley Tennis Center is currently developed with the outdoor tennis court area, a large metal building complex with indoor courts, and associated access roads, parking areas, sidewalks and landscaping. The ground surfaces/site grades across the tennis center are relatively flat.

The subject existing outdoor tennis court area consists of eleven (11) tennis courts in two rows (6 on the north side and 5 on the south side). The tennis court area is surrounded by approximately 12ft high chain link fences. The northern and southern courts are separated by a narrow covered and enclosed walkway. It should be noted that there were previously twelve (12) courts. It appears, however, that the southwestern most court was removed prior to May 10, 2019 and a storm water detention pond/drainage basin was constructed in that location (Google Earth, 2024).

The existing tennis courts have old asphaltic concrete (AC) surfacing. The AC appears to be moderately to severely damaged, with cracking affecting all eleven playing surfaces. Some of these cracks have evidence of past repair with the use of an asphalt sealer. Some cracks are wider, and presumably older, and currently support vegetation growth. Some cracks are narrow, and presumably more recent, with no evidence of past repairs. There are numerous semi-linear cracks extending through multiple courts, as well as shorter cracks radiating from locations where the court asphalt surfacing contacts the concrete foundations for embedded posts/poles. It appears this damage was caused by the thin AC support subsection being subjected to long-term shrink/swell cycles of the underlying expansive clay soils.

We understand the project to consist of removing/demolishing the existing outdoor courts and reconstructing a new court area consisting of both tennis courts and pickle ball courts. This study is intended to help provide recommendations for the proper design and construction of the tennis court support.

FIELD INVESTIGATION

On March 20, 2024, our Engineering Associate, Kristen S. Pierce, E.I.T, and our drilling crew, visited the site to accomplish the subsurface investigation. A total of four (4) exploratory borings were accomplished, as shown on *Figure 2, Site Plan with Boring Locations*. The drilling was accomplished using our ATV-mounted, solid-stem auger drill rig.

A utility locate was completed prior to our investigation. The president of the Umpqua Valley Tennis Center consulted with i.e. Engineering prior to our investigation to identify low traffic areas. The borings were located within the low traffic areas and away from the marked and known utility locations.

The asphaltic concrete cover of the tennis courts was first cored through at each boring location, using an 8-inch core bit. The borings were then drilled with our ATV-mounted, 4” diameter solid stem auger drill rig. In the four borings, drilling and sampling/testing extended between 3.5 feet and 8.5 feet into the subsurface. Upon completion, the borings were backfilled with the drill spoils and the surfaces were capped with cold mix asphalt patches.

Standard Penetration Testing (SPT) was accomplished in each boring as part of the exploration, to collect soil samples and data. This entails driving a 1½-inch diameter, steel split spoon sampler by dropping a 140-pound weight for a 30-inch drop. The total number of blows it takes to drive the sampler the last 12 inches of an 18-inch drive is called the SPT N-value. The results are an indication of the relative density or consistency of the soil and can be correlated with soil strength and density parameters from testing on thousands of other projects.

Our representative determined the final exploration locations, logged subsurface soils and water conditions, obtained soil samples for transport to our laboratory and supervised boring backfill and surface patching. Visual classifications of the soils were made in the field and are presented in the *Boring Logs in Appendix A*, at the end of this report. The N-values shown in the logs are raw data from the field and have not been adjusted for sampling equipment type, sampler size, or overburden pressure.

SITE SURFACE AND SUBSURFACE CONDITIONS

The AC cores at each boring showed multiple lifts/layers of asphalt and some variation in the overall thickness. There were also variations in the thickness of the base aggregate in the borings. Beneath the asphalt and base aggregate, there were two perceptible different layers of low plasticity Clay with varying amounts of silt encountered in the subsurface in all four borings. The upper Clay layer was black to dark brown in color and contained organics. The coloring of the lower Clay layer is gray and red mottled. This lower Clay layer, due to the distinct mottling color pattern, is assumed to be undisturbed. There was no free groundwater encountered in any of the borings.

The thickness and number of asphalt layers, the amount of gravel and the transition between the two clay layers and the total depth of the borings are all summarized in Table 1.

Table 1: Summary of Subsurface Conditions

Boring	Asphalt Thickness (individual layers)	Base Aggregate	Depth to Boundary Between Clay Layers	Total Boring Depth
B-1	4” (2” + 2”)	6”	3.5 ft	8.5 ft
B-2	3.5” (1.75” +1.75”)	6”	2.5 ft	4.0 ft
B-3	3.25” (1.5” + 1.75”)	4”	3.0 ft	4.5 ft
B-4	4.5” (1.5” + 1.5” + 1.5”)	6”	3.0 ft	3.5 ft

In addition to the geotechnical borings, our representative completed a visual observation of the current conditions at the project site. There was standing water in some areas of the courts at the time of the site visit. The last measurable precipitation event prior to our field investigation was on March 11, 2024, with approximately 0.2 inches of rainfall. The narrow, landscaped area along the northern outside perimeter of the tennis courts is fairly flat between the courts and the building, with a very gentle ascending slope to the northeast. This area exhibited saturated surface conditions at the time of our visit. The area along the eastern outside perimeter has very gentle descending slopes down to the east, resulting in positive drainage and the surface soil conditions in these landscaped areas were moist. The area along the southern outside perimeter has a moderate descending slope down to NW Harvey Avenue, with an approximate elevation difference of 1 foot at the southwest corner and 3 feet at the southeast corner. The surface soils along this slope were damp to dry. The western outside perimeter of the tennis courts appears to have positive drainage to the storm water detention pond that occupies what was formerly a twelfth court, with the surface soils in this area being dry to damp.

LABORATORY TEST RESULTS

The Galli Group conducted the laboratory testing on the collected soils sampled beneath the tennis courts as part of our geotechnical investigation. All soil samples were tested for natural moisture content (ASTM D2216). Two samples were tested using the Expansive Index Test (ASTM D4829). These resulted in EI_{50} values of 83 to 85. *ASTM's* classification system considers EI_{50} values of 51-90 to be in their "medium" range for expansion potential (changes in volume with change in moisture content).

Expansive soils such as these can cause shrink/swell related foundation movements if exposed to alternating wetting and drying conditions and, therefore, will provide poor support if not mitigated. An Atterberg Limits Test (ASTM D4318) was performed on one sample. These tests were accomplished to aid in determining the classification, engineering properties and characteristics of the onsite soils. The results of the laboratory testing are summarized in Table 1. Please see *Appendix B,* attached with this letter, for individual laboratory test results sheets.

Table 2: Laboratory Testing

Boring #, Sample # (depth; feet bgs)	Soil Description	Test	Results
B-2, S-1 (1.0-2.5)	Dark brown to black, silty Clay	ASTM D4318, Atterberg Limits	Liquid Limit: 34 Plastic Limit: 25 Plasticity Index: 9
B-1, S-2 (2.0-3.5)	Dark brown to black, silty Clay	ASTM D4829, Expansion Index	EI_{50} = 85 Medium to high expansion potential
B-1, S-3 (4.5-6.0)	Gray and Red, silty Clay	ASTM D4829, Expansion Index	EI_{50} = 83 Medium to high expansion potential

SITE GEOLOGY & GEOLOGIC HAZARDS

The primary geologic hazard to consider for this project is the expansive soils, the mitigation for which are detailed in our recommendations. The proximity of the project site to the South Umpqua River also makes flooding a potential hazard for this project. The 500-year floodplain of the South Umpqua River extends past the project site, however the extent of the 100-year flood plain is just under 1,000-feet to the southwest of the project site, according to FEMA online mapping (HazVu, 2018). Therefore, the risk of damage to the planned development due to flooding is considered very low. As with most areas in western Oregon, there are seismic activity related hazards to consider in the event of a Cascadia Subduction Zone (CSZ) megathrust earthquake. The project site is not susceptible to seismic induced landslides, ground rupture, lateral spread or liquefaction. However, ground shaking at the site for a CSZ originating earthquake is predicted to be strong to very strong, with a 10-20% probability of damage sustained to structures from shaking. Embedded structures with heights greater than 6-feet should be designed to withstand horizontal ground accelerations (PGA_M) of 0.55 g, based on a Site Class C designation (ASCE, 2024).

DISCUSSION & CONCLUSIONS

The existing court area exhibits structural cracks (large cracks which penetrate the pavement), radial cracks (which appear where concrete net post, light pole or fence post footings meet the asphalt court surface) and settlement cracks (which result from paving over expansive soils with inadequate base support cover and/or poorly compacted or poorly drained subbase). The cracking patterns seen on the existing courts can be attributed to a variety of factors that can be mitigated with standard construction methods. The primary cause of the existing cracking and uneven court surfaces is the expansive, stiff to medium stiff, silty Clay soils encountered beneath the court. The expansion potential and plasticity of these soils are in the medium to high range, which can cause significant damage as they go through numerous shrink/swell cycles (changes in volume with change in moisture content). We assume that the new tennis courts will be finish surfaced with asphalt. Removal and/or replacement of the existing net posts, perimeter fence posts and light poles and their concrete foundations will also be accomplished as part of this project. Proper subgrade preparation and providing adequate support and drainage is essential to the longevity of the new outdoor court area.

Our recommendations are focused on providing a stable base for the tennis courts and structures to be constructed upon, to mitigates the adverse impact of expansive soils. We have provided recommendations for the support of above ground elements/court structures in subsequent sections of this report.

We have provided two options for design concepts that provide a stable base for the new court surfacing and foundation support:

Option 1: Cement Treated Subgrade section
Option 2: Rock Base section

Both tennis court support section options will mitigate the adverse impacts of the expansive soils to a great degree:

OPTION 1: Tennis Court Support Section – Cement Treated Subgrade
Based on the expansive Clay subgrade soil conditions beneath the existing/new court location we recommend creating a cement treated subgrade (CTS). This option of utilizing cement mixed into the existing clay soils is likely the most effective and cost-efficient method for this project. The cement additive, when thoroughly mixed/treated, will react with the clay mineralogy, and chemically stabilize the subgrade soils treated. Chemical stabilization of fine-grained soils can decrease the detrimental effects of expansion and increase the soil strength. The CTS option is an alternative to the typical/conventional "remove and replace" method of expansive soil mitigation and allows for a reduction in the amount of imported crushed rock material and removed soils haul off for the project. This method has been used

for many decades beneath roadways, parking lots and structures throughout portions of the west coast with great success.

The cement in the clay soil CTS section will bond the soils together and will create a very strong, dense, and much less permeable (nearly impermeable) support layer. The CTS section can be sloped/graded at the time of installation for drainage purposes. This CTS will also help protect the deeper subgrade and provide adequate long-term support for the overlying tennis courts.

Section Design and Cement Treatment Method. From our experience and testing on many cement treated soil projects throughout southern Oregon, the results of our site investigation and laboratory test results indicate that a significant improvement in subgrade strength will be gained for support of the proposed courts when the on-site silty Clay soil subgrades are altered/improved with a 6% CTS section that is at least 14 inches thick. The following Tennis Court Cement Treated Subgrade Section Design has been determined:

Tennis Court Cement Treated Subgrade Section Design

3" AC (Class B or Type II Dense Graded HMAC)
6" Base Rock (3/4" or 1" Minus Crushed Rock)
14" Cement Treated (6% by weight) Subbase/Subgrade
Stable Subgrade

In order to reduce potential CTS mixing/contamination and to minimize the potential impacts of shrink swell cycles of the clay beneath the cement treated section, the following overall method/sequence of construction should be utilized:

1. Remove the exiting asphaltic concrete surfacing and waste off site.
2. Remove the existing sandy Gravel fill and top of the clay soils as needed to establish the CTS subgrade. Stockpile for use later in the project (if needed).
3. Cement-treat 14" of the clay subgrade soils with 6% cement, tilled, wetted, and compacted into a dense monolithic layer.
4. Place the 6" crushed rock base section over the top of the cement-treated clay soils. Moisture condition and place and compact the crushed rock to 98% of ASTM D-698.
5. Place the new AC surfacing over the newly compacted crushed rock layer.

Cement Treatment Subgrade (CTS) Recommendations. For construction of the CTS the following method should be utilized:

1. Excavate the site to the top elevation of the desired cement treated soil subgrade (bottom of foundation/sub-base rock support section and as described above).
2. These untreated subgrade soils shall be well compacted and must be maintained in a moist condition. Allow for 3/4-inch of "swell" in the soil to accommodate the addition of cement powder and water.
3. Till the subgrade to 14" depth to thoroughly pulverize the subgrade soils. Make multiple passes if necessary for good pulverization.

4. The tilled and CTS soil section must extend at least 24 inches beyond the edges of the planned surface wearing area of the court, for added underlying edge support.
5. Spread cement evenly across the entire tilled area with 7 pounds of cement per square foot of area (6% by weight for soil with unit weight of 100 pcf, to a depth of 14 inches)
6. Till cement into pulverized soil to achieve an even distribution of cement throughout the tilled soil.
7. Dry out or add water during the tilling process to achieve near optimum moisture content for cement treatment and compaction.
8. During final tilling and initial compaction, grade the surface to very close to slope and elevation as required by the project drawings.
9. Use proper equipment to achieve full compaction of the CTS soil. Must achieve at least 95% compaction per ASTM D698 throughout the entire 14-inch section of the cement treated material.
10. Roll the surface smooth and “tight” to help prevent additional disturbance and moisture penetration during precipitation events.
11. Upon completion of cement treatment (1 through 10 above), no construction traffic should be allowed to traverse compacted CTS areas nor should any placement or compaction of the base rock be accomplished until after the prescribed curing/waiting period stipulated by the cement treatment contractor, or a minimum of 4 days.

It would be prudent to accomplish the cement treatment operations when the site is not fully saturated by recent rainfall.

Note: We recommend additional sampling and lab testing of the prepared reclamation materials during construction to verify the material consistency and, for density testing purposes, to verify maximum densities and optimum moisture contents. Personnel from The Galli Group must be present to verify initial tilling, tilling in of cement and water, leveling of surface, compaction of the treated soil and compaction of the aggregate base rock (after curing).

OPTION 2: Tennis Court Support Section – Rock Base/Subbase (Remove and Replace)

The goal of this option is to provide a stable base for the tennis courts to be constructed upon and mitigate the adverse impacts of the expansive soils. This can be accomplished by 1) using a subbase and base section of sufficient thickness such that the soils left beneath the courts are deep enough below the surface to significantly decrease their moisture loss in summer months, 2) using a subbase and base section that creates a strong "buffer" between the expansive soils and surfacing materials and 3) provide a separation at the subgrade soil to subbase interface that will not allow crack penetration from the clay soils up into the subbase and base rock, and that will help limit moisture loss from the soil below the courts.

Redensification and Moisture Conditioning of the Subgrade Soils. When the subgrade has been cut to grade, it must have the surface layer redensified. The exposed subgrade must exhibit a smooth, firm, and unyielding surface. Care must be taken to not over vibrate and disturb the subgrade soils during wet weather and saturated soil conditions. To help limit movements, the exposed finish subgrade soils must be kept in a moist and fully swelled state. In no case should dried out soils, which exhibit cracks, be covered with the subbase and base rock design section provided below.

Separation and Moisture Barrier. This can be accomplished by using one or two layers of material. Separation can be achieved by covering the subgrade with a woven geotextile support fabric (ACF 180 or equivalent). This would be underlain with a plastic moisture barrier such as 6-mil visquene or a heavier one such as Stego Industries 15 mil vapor barrier (Stego Wrap). The Stego Industries, Stego Wrap vapor barrier would be the best one to use. The fabric and vapor barrier should extend at least 4 feet beyond all edges of the court surfacing to avoid edge effects.

Another way to accomplish this would be to use only a very tough moisture barrier such as Stego Industries 15 mil or 20 mil material. All seams must be sealed with Stego Tape and *care must be taken to not damage this during subbase placement.* This would then serve as both the separation and moisture loss block.

Note: Where the mitigation section extends beyond the edges of the court surfacing the outer portion of this may have a surface consisting of grass. In that case the 4 inches of aggregate base (see below) could be replaced with topsoil for promotion of grass growth.

Prior to placement of these materials, the subgrade must be cut to a clean, relatively smooth surface and redensified. *The clayey subgrade must then be kept in a moist and fully swelled condition until the vapor barrier and subbase are in place.*

Subbase and Base Support Section Design. To help maintain a stable base a deepened subbase and base rock combination should be used, as follows:

Tennis Court Rock Base/Subbase Section Design

3” AC (Class B or Type II Dense Graded HMAC)
4" Base Rock (3/4” or 1” Minus Crushed Rock)
20” Subbase (2"or 4" Minus Crushed Rock or Jaw-Run Shale)
Woven Support Fabric
Vapor Barrier (Stego Wrap 10 mil or similar)
-OR- just 15 mil Stego Wrap
Redensified & Moisture Conditioned Subgrade

Note: Care must be taken to protect the vapor barrier and fabric during placement of the subbase material. The subbase rock (2" or 4" minus) can be a low-grade rock that is well graded. However, the soil fines content may become an issue during wet weather. The first lift of this rock must be placed very carefully to minimize damage to the separation and/or moisture barrier.

FOOTING RECOMMENDATIONS

It is assumed that all footings necessary for the project (net posts, perimeter fence posts and light poles) will be supporting isolated loads no greater than 1.0 kips. We anticipate that the bottoms of planned footings will encounter the expansive soils which can cause shrink/swell related foundation movements if exposed to alternating wetting and drying conditions, and will provide poor footing support if not mitigated. Therefore, we recommend the footings be supported on at least 24 inches of structural fill (crushed rock, shale, or decomposed granite) at the bottom of an area over-excavated at least 4 feet below finish exterior grades into the native Clayey subgrade soils, as follows:

1. Footing areas designated to receive structural fill should first be excavated to a depth of at least 4 feet below finish exterior grades. The subgrade should be free of disturbed and loose soil prior to placing fabric and structural fill (redensify/prepare the subgrade soils as described in the previous sections of this report). *The clay subgrade soils must be kept wet and fully swelled prior to covering with structural fill.*
2. The width of the crushed rock structural fill placed beneath footings must extend laterally beyond the outside edges of all footings, a distance equal to at least half of the total depth of the structural fill for vertical support (i.e., for 2 feet of structural fill beneath footings, extend the fill at least 1-foot past all edges of the footings) and be constrained laterally on all sides.
3. Cover the base of the footing excavations with a woven geotextile support fabric (ACF 180, ACF S200 or equivalent).
4. Place and compact a minimum of 24" of Structural fill over the support fabric. Top of the crushed rock structural fill must extend up to the elevation of the bottom of footing. The last 6 inches of structural fill may consist of ¾" minus crushed rock.
5. The bottom of all footings shall be buried a minimum of 24 inches below finish grade in order mitigate the potential adverse impacts of the expansive Clay soils and to provide lateral support and frost protection. Footings should be backfilled over and against with crushed rock structural fill.
6. Footings placed on the redensified subgrade covered with compacted crushed rock as listed above may be designed for an allowable bearing pressure of 2,000 pounds per square foot. A 1/3 increase in this allowable bearing pressure may be used when considering short-term transitory wind and seismic loads.
7. We recommend minimum lateral dimensions of 12 inches for continuous load bearing footings and 24 inches for isolated piers constructed in this manner.

Anticipated Settlements. For properly constructed foundations, as described above, we anticipate maximum total and differential settlement to be less than 3/4-inch and 3/8-inch, respectively.

Note: Concrete and asphalt expand and contract at different rates, which can cause stress on the playing surface that manifests in cracks. Footings that are rectangular at the contact with an asphalt base are more likely to contribute to radial cracks that originate at the corners of the footings (as evidenced on these courts at several post/pole locations). Round footings for embedded structures are known to reduce radial cracking. It is important for these footings to be properly interfaced with the court base. To minimize the potential for loading/interaction/radial cracks to develop between any new proposed footings and the court surfacing, we recommend that the tops of footings be embedded to between 8" and 12" below finished grade.

We recommend constructing all footings prior to subgrade preparation and construction of either the CTS or rock supported court design sections. Also, all utility line installations should be accomplished prior to court section construction and should be embedded to depths below the bottom of the design section.

STRUCTURAL FILL PLACEMENT & COMPACTION RECOMMENDATIONS

For both court support options and the foundation support recommendations provided above, all base and subbase structural fill sections (consisting free-draining, granular material with a maximum aggregate size of 1.5-inches) must be placed and constructed in the following manner to minimize the potential impacts of differential settlement:

1. Care should be taken to not disturb/damage the CTS or soil subgrades during structural fill placement.
2. Material shall be placed in horizontal lifts, not exceeding 8-inches loose thickness (less, if necessary to obtain proper compaction) for heavy compaction equipment and 4-inches for light and hand-operated equipment.
3. Each lift must be compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM Test Method D-698 (Standard Proctor).
4. The placement and compaction shall be observed by our representative.
5. The required construction monitoring of structural fill utilizing standard nuclear density gauge testing and standard laboratory compaction curves (ASTM D-698 specified) is applicable to the court and foundation structural fill support sections.
6. Testing shall be accomplished in a systematic manner on all lifts as they are placed. *Testing only the upper lift is not adequate.*
7. These structural fill sections must extend at least 12-inches beyond the edges of the planned surfaces or structures for added underlying edge support, unless otherwise specified.

DRAINAGE RECOMMENDATIONS

The site shall be graded during construction such that surface water does not pond within existing excavations or beneath the court areas. Surface runoff shall be controlled during construction and with final site grading. All areas adjacent to the structures shall have a permanent slope away from the foundations at an inclination of at least 6 inches in eight (8) feet. **Note:** *Provision shall be made during construction to intercept runoff before it moves onto the site and into the new court areas/excavations*. This surface water must be channeled into landscape area drains or catch basins, or shall be conveyed around the work area. Where items such as landscape areas and walkways block the flow of surface water, small area drains shall be installed to collect the surface runoff. Good site design accommodates all site runoff and conveys it away from the structures and off the site to an acceptable disposal location. This would include drainage of surface water along the upslope side of the project.

The most prevalent reason for deterioration of recreational surfaces is the presence of standing water on, or free water beneath, the playing surface. Proper drainage is an essential component of a properly constructed sport court in order to minimize the potential negative impacts of varying moisture content on underlying soils and to properly intercept and divert surface water infiltration and subsurface lateral flow. Hard surfaced courts should drain end-to-end, side-to-side or corner-to-corner. We recommend the following measures be incorporated into the drainage design and construction for the courts:

1. The subgrade, and all subbase and base rock layers placed for the courts must all be graded to convey lateral water flows diagonally in a planar surface from the east side of the courts to the west side.
2. Subgrades should be sloped at 1-inch in 8 feet (1%)
3. At all points along the perimeter, the final grade of the court should be elevated/tapered to at least 4-inches above the surrounding/nearby vegetated or hardscaped ground surfaces.
4. A swale in the landscaping along the northern and western sides of the courts should be constructed to collect and direct surface water to the storm water detention pond located in the southwest corner of the project area.
5. Subsurface drainage is recommended along the northern and western perimeters of the finished court surface to intercept and divert subsurface water away from the court subgrade and foundation (see items #6-10, below).
6. The subsurface drainage should be installed by trenching 12-inches to 18-inches wide by approximately 18-inches (with the base of the drainage trench coinciding within the thickness of the CTS section) or 30-inches (coinciding with top of subgrade for the sub/base rock section) deep.
7. The trench should be a minimum distance of 2-feet (CTS section) or 4-feet (subbase/base rock section) from the edge of the court. Ideally, the drainage trench locations would coincide/follow with the landscaping ground surface invert/swale alignments. The slope of the drainage trench should be 1/8-inch per foot, drainage to the west along the northern court perimeter (parallel to the subgrade slope) and to the south along the western perimeter,
8. The entire drainage ditch must be lined on all sides with a non-woven geotextile filter fabric (Mirafi 140 or equivalent), the ends of which should overlap at least 12-inches at the upper portion of the drain after the installation of the pipe and drain rock. The subsurface drainage pipe should be a 3-inch or 4-inch diameter, smooth interior, solid wall, perforated pipe, capable of being cleaned by a typical drain auger. The drain pipe should be oriented with perforations facing down directly on the filter fabric. The drainage ditch should be filled with clean drain rock or pea gravel, with aggregate size compatible with perforation size to within 6” of grade. See *Figure 3 - Typical French Drain Detail.*
9. It is recommended that a cleanout be installed for the drain pipe at the northeastern terminus of the pipe and additional cleanouts be installed at intervals along the drain pipe, with a separation distance of no more than 100-feet between cleanouts.

10. The drain pipe should daylight and terminate at (discharge into) the detention pond located in the southwest corner of the project area.
11. The drain trench should be covered with well-draining top soil prior to landscaping and should not be covered with the low permeability, expansive, on-site soils.

MATERIAL SPECIFICATIONS

The following materials specifications shall apply to the materials as used on this project.

On-Site Silty Clay

- Used in landscaping areas only and general backfill portion of utility trenches in landscape areas only.

On-Site Gravel

- May be used as structural fill immediately following demolition.

Aggregate Subbase Rock (Acceptable for Structural Fill)

- Angular Clean Crushed (jaw run) hard “Shale” (4" Minus Jaw-Run) or Crushed Rock (2" to 4” Minus); R=50 or greater; Angular and Reasonably Well Graded.
- At Least 60% retained on the No. 4 Sieve.
- Maximum passing the No. 200 sieve ≤ 10% Total; ≤ 3% Clay Size.
- During wet weather; passing No. 200 sieve ≤ 5%.
- Compacted to 95% of the maximum dry density as determined by ASTM D698 or AASHTO T-99; initial lift may not attain 95% due to soft subgrade; Engineer to decide in the field.
- Care must be taken to avoid very silty subbase that will not support construction loads, especially when wet (will not meet specifications).

Aggregate Base Rock (Acceptable for Structural Fill)

- Angular Crushed Rock (3/4”, 1” or 1-1/2” Minus); R=80 or greater; Well Graded (No Gaps and at least 60% retained on the No. 4 sieve).
- Maximum passing the No. 200 sieve ≤ 5%.
- Compacted to 95% of the maximum dry density as determined by ASTM D698 or AASHTO T-99.

Drain Rock (For Drainage Sections)

- Clean washed rounded or angular openwork drain rock.
- Gradation to be 1/4" and greater, sized to not move into and through perforations in the pipe.
- 1/4" to 3/4" clean crushed, 3/4" to 1" clean rounded rock, and 1" to 2" clean angular rock are all acceptable.
- Clean means washed rock with NO coating of silt, clay or sand; less than 2% passing No. 200 sieve.

Note: All types may be used in all applications of drain rock that are not beneath the court areas. The foundation/sub-base may consist of angular clean drain rock, round drain rock must not be used for the court support.

Non-Woven Geotextile Filter Fabric

- Non-woven geotextile filter fabric for wrapping drainage sections and separation of openwork rock from sands or soils fines.
- Meet specifications as per Mirafi 140N or equivalent (unless otherwise specified).
- Overlap all edges at least 24 inches (12" for drainage section envelope).
- Secure in place such that overlaps will not move during covering operation.

Woven Geotextile Support Fabric

- Woven geotextile support fabric designed for separation of crushed rock and subgrade soil and for rock section support.
- Meet specifications as per ACF180 OR S200 woven support fabric.
- Overlap edges at least 2 feet and ends at least 5 feet.
- Align roll lengthwise with direction of traffic in all drive lanes.
- Pull tight full length and keep tight during placement of crushed rock above fabric.
- Do not drive on the fabric until it is covered with rock.

Perforated Pipe

- 3" or 4" rigid wall, smooth interior perforated pipe.
- Secure all joints with solvent weld glue. DO NOT use only compression push together fittings.
- Slope to drain per specifications in report or on plan sheets (minimum 1%).
- Align perforations in the downward direction.
- Must always be placed within filter fabric wrap unless specified otherwise.
- Protect from construction traffic until buried at least 2 times pipe diameter.

CONSTRUCTION MONITORING

Personnel from The Galli Group must be present during subgrade preparation and cement treatment and foundation/sub-base construction operations to verify the following.

A. Verify all footings for planned embedded structure support are in place prior to cement treatment.
B. Verify compaction of the prepared subgrade: visual proofroll (A successful proofroll is when there is less than ¼" deflection below the tires of a loaded truck).
C. Verify cement powder placement quantities: measure/sample in-place cement powder for height/weight (coordinate verification with tilling/cement treatment contractor) - (Option 1 only).
D. Verify tilling/mixing depth of soil, cement and water: visual inspection - (Option 1 only).

E. Final grading and compaction verification of the CTS (Option 1 only): visual proofroll or Nuclear Density Testing.
F. Lift placement, compaction and final grading of the base/subbase materials: visual proofroll or Nuclear Density Testing.

Limitations. This report was prepared for the use of i.e. Engineering and the City of Roseburg for their review and use in the final design and construction of the subject recreational sport courts. It should be made available to others for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

We have performed these services in accordance with generally accepted geotechnical engineering practices in southern Oregon, at the time the study was accomplished. No other warranties, either expressed or implied are provided.

Please do not hesitate to call if you have any questions. We look forward to working with you to make this a successful project.

Respectfully,
THE GALLI GROUP
GEOTECHNICAL CONSULTING

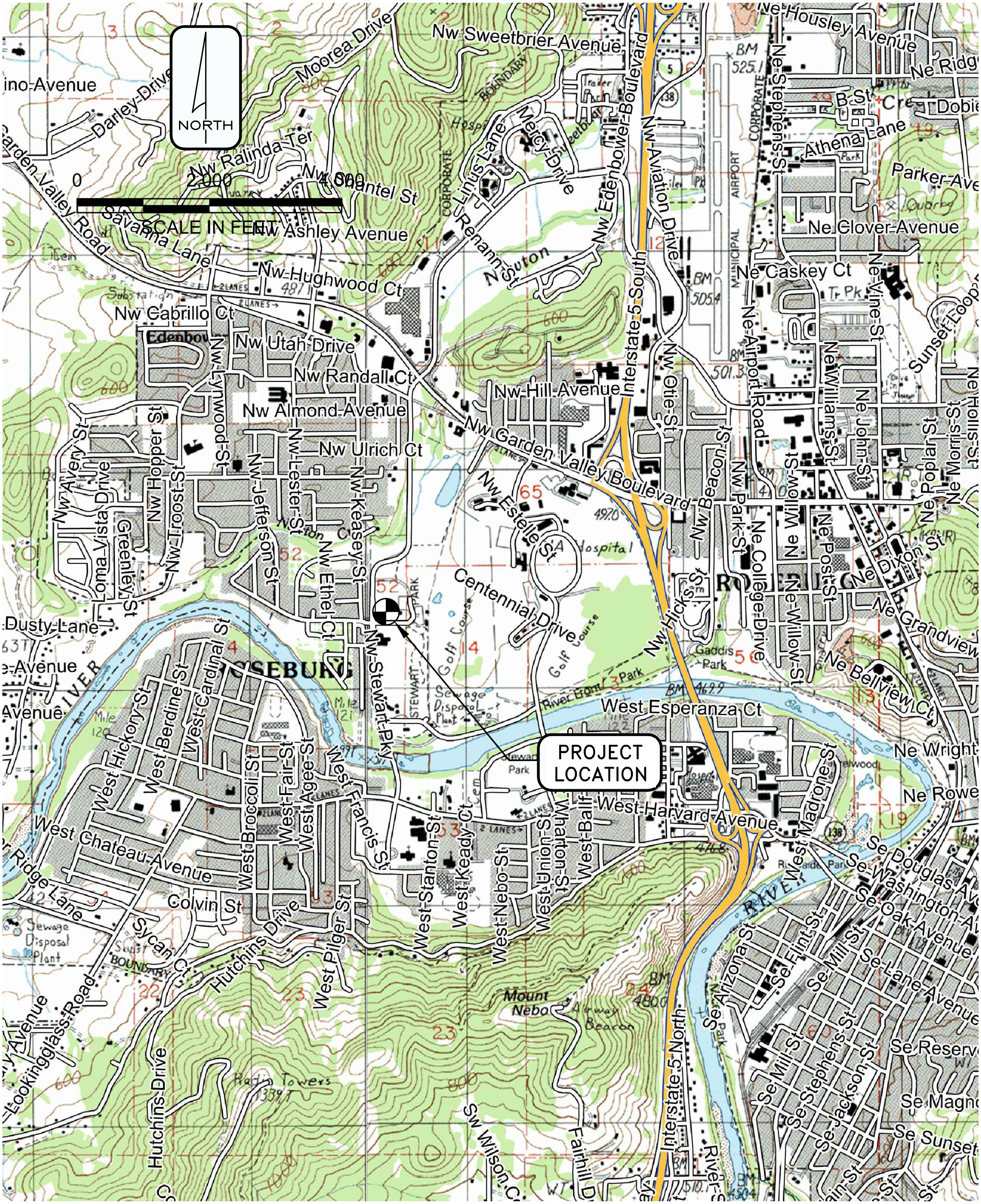
Kristen Pierce, EIT
Engineering Associate

Dennis Duru, P.E., CEG, RG
Senior Engineering Geologist

Mel Galli, III, P.E.
Sr. Principal Engineer

EXPIRES: 06/30/2025

Attachments:
Figure 1: Vicinity Map
Figure 2: Site Plan with Boring Locations
Figure 3: Typical French Drain Detail
Appendix A: Boring Logs
Appendix B: Laboratory Test Results



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612 NW 3rd Street
Grants Pass, OR 97526

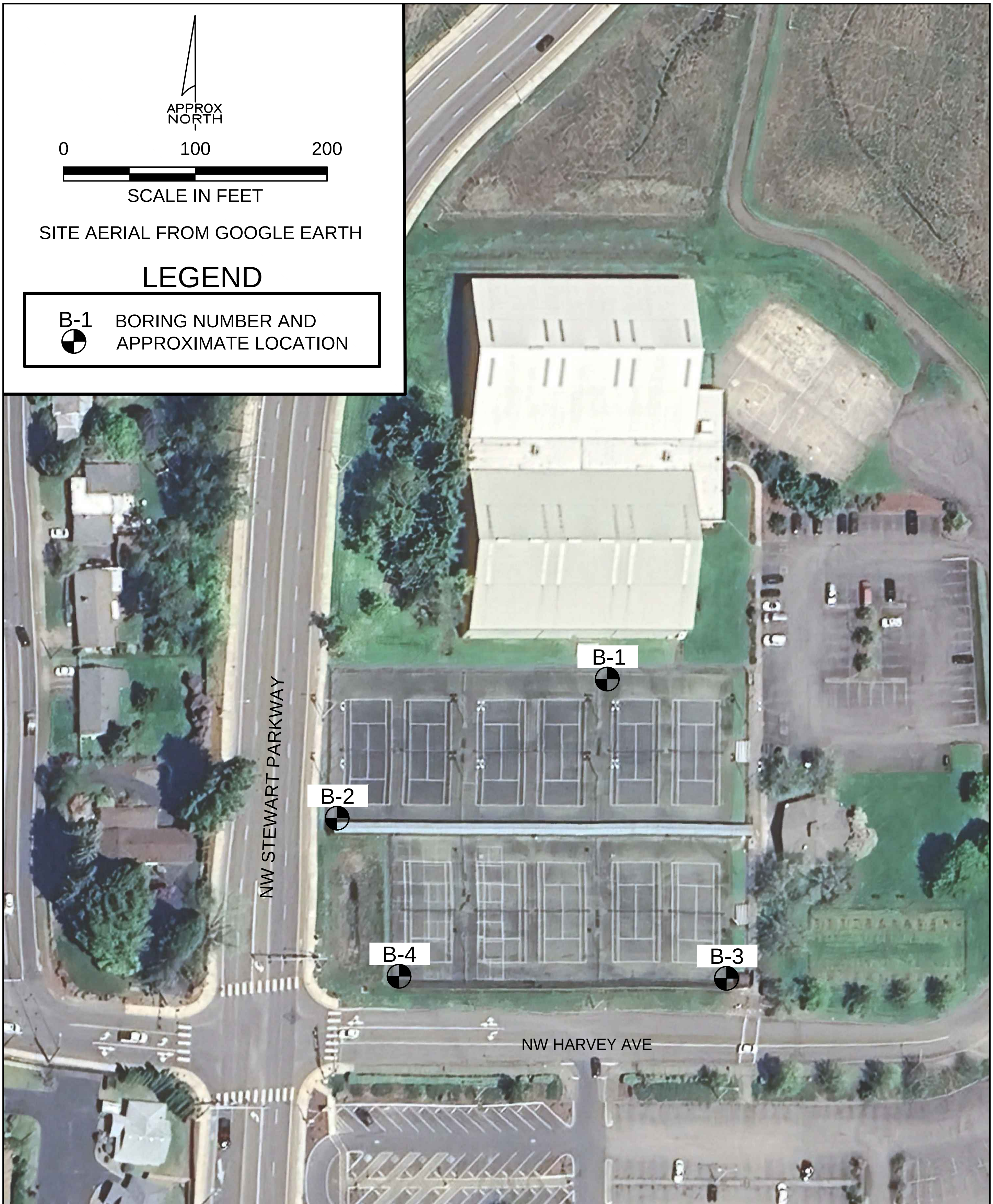
VICINITY MAP

UMPQUA VALLEY TENNIS CENTER COURTS
ROSEBURG, OREGON

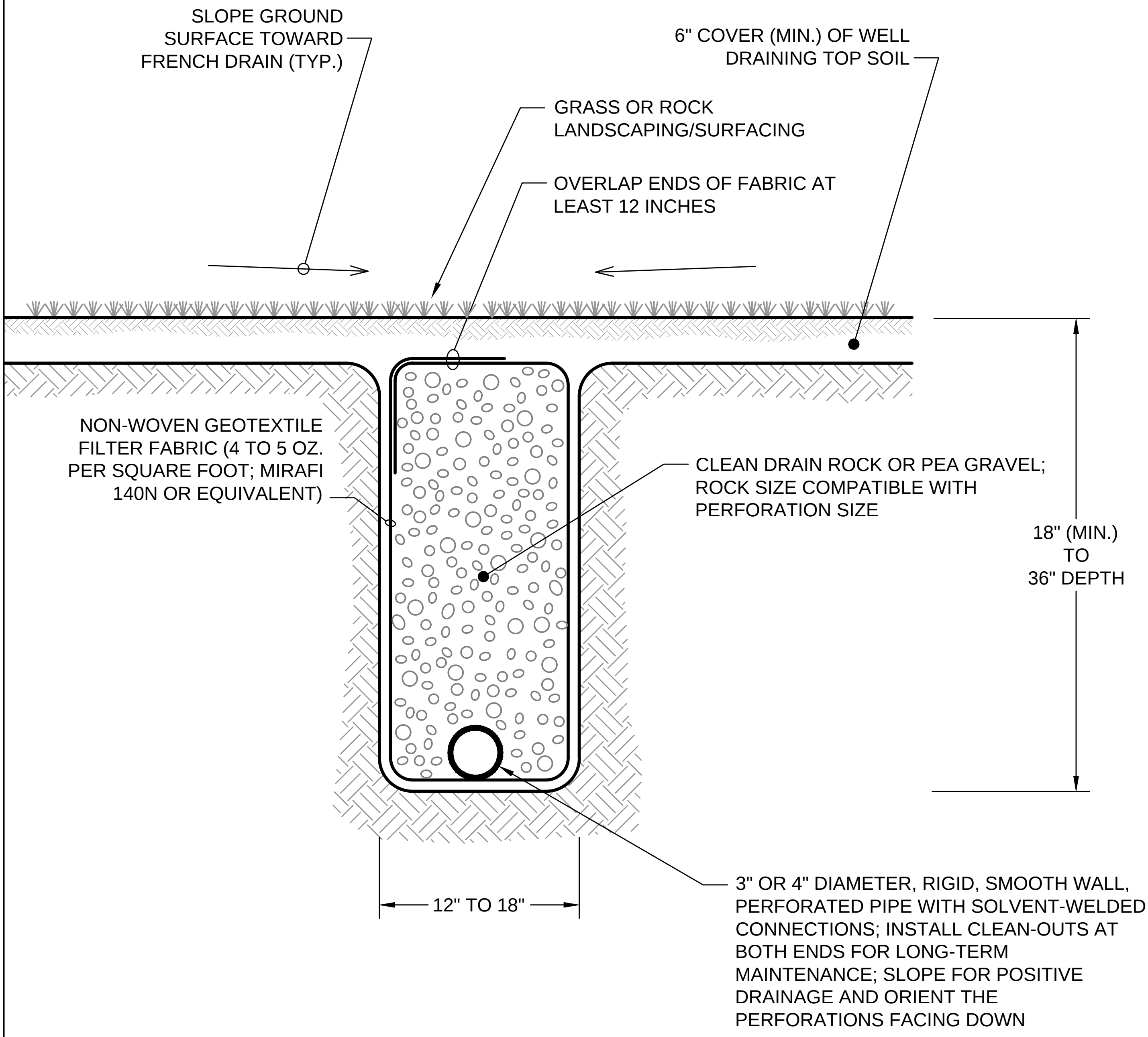
DATE:	APRIL 2024
JOB NO:	02-6385-01
REV:	4/18/2024 6:21 AM
PREPARED BY:	MG3

6385 Tennis Ctr 1201 Stewart Prkwy - 01 - Vicinity.dwg

FIGURE: 1



THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	SITE PLAN WITH BORING LOCATIONS UMPQUA VALLEY TENNIS CENTER COURTS ROSEBURG, OREGON	DATE: APRIL 2024 JOB NO: 02-6385-01 REV: 4/18/2024 6:22 AM PREPARED BY: MG3 6385 Tennis Ctr 1201 Stewart Prkwy - 02 - Site Plan.dwg	FIGURE: 2



FOR ILLUSTRATION PURPOSES ONLY
NOT TO SCALE

THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	TYPICAL FRENCH DRAIN DETAIL	DATE: APRIL 2024	FIGURE: 3
		JOB NO: 02-6385-01	
	UMPQUA VALLEY TENNIS CENTER COURTS ROSEBURG, OREGON	REV: 4/18/2024 12:21 PM	
		PREPARED BY: MG3	
		6385 Tennis Ctr 1201 Stewart Prkwy - 03 - Typ French Drain.dwg	

APPENDIX A

BORING LOG

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

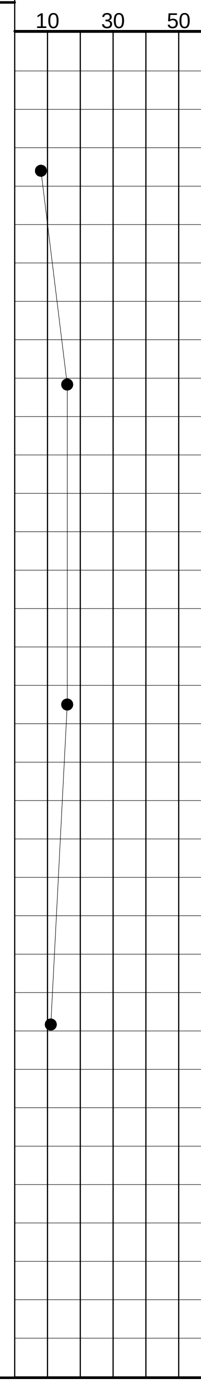
BORING LOG
B1

Project: Umpqua Valley Tennis Center - 1201 NW Stewart Parkway, Roseburg
Project No.: 02-6385-01
Client: i.e. Engineering
Date: 03/20/2024
Location: North edge of courts (see Figure 2 - Site Plan)
Elevation:
Driller: TGG (Ken P., Nate C.)
Logged By: Kristen S. Pierce
Drill Rig: ATV mounted 4" diameter SSA
Depth To Water> Initial ▽ : At Completion ▼ :

USCS	Description	Depth (ft)	Sample No. and Type	NMC	N
FILL	Asphaltic Concrete (AC): 4" total in two lifts, 2" each.	.33			
GW-SW	Loose, gray, sandy Gravel with some silt and clay; wet (fill).	.33 – .83	S1-G		
CL	Medium stiff to stiff, black Clay; with trace silt and organics, molist to damp.	.83 – 3.5	S1(1)		8
			S1(2)	21%	
			S2	31%	16
CL-ML	Stiff, gray and red mottled, Clay; with trace silt, damp.	3.5	S3	33%	16
	Stiff, brown mottled with gray and red, Clay; with trace silt, moist.	– 8.5	S4	40%	11
	Botom of boring at 8.5 ft. No free groundwater was encountered.				

Depth scale: 0, 1.5, 3, 4.5, 6, 7.5, 9, 10.5

Standard Penetration Test CURVE



Legend of Samplers: Grab sample SPT sample Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
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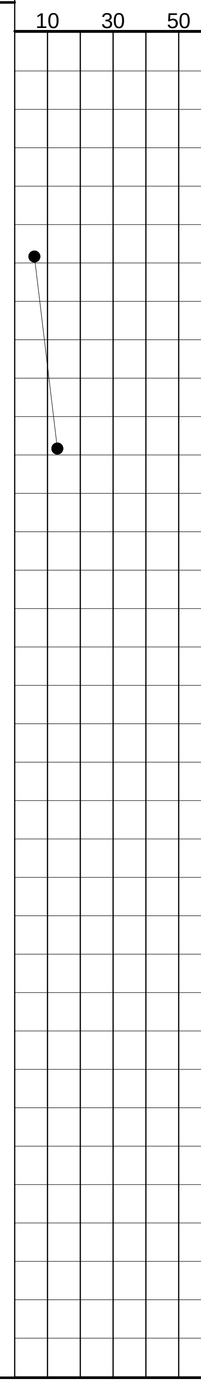
BORING LOG
B2

Project: Umpqua Valley Tennis Center - 1201 NW Stewart Parkway, Roseburg
Project No.: 02-6385-01
Client: i.e. Engineering
Date: 03/20/2024
Location: south west corner of north courts (see Figure 2:site plan)
Elevation:
Driller: TGG (Ken, Nate C.)
Logged By: Kristen S. Pierce
Drill Rig: ATV mounted 4" diameter SSA
Depth To Water> Initial : At Completion :

USCS	Description	Depth	Sample No. and Type	NMC	Standard Penetration Test N
FILL	Asphaltic Concrete (AC): 3.5" total in two lifts, 1.75" each.	0 – .3			
GW-SW	Loose, gray, sandy Gravel with some silt and clay; wet (fill).	.3 – 0.83			
CL	Medium stiff, black, Clay; with trace silt and organics, damp.	0.83 – 2.5	S1 (SPT)	33%	6
CL-ML	Stiff, gray and red mottled, Clay; with trace silt, damp.	2.5 – 4.0	S2 (SPT)	30%	13
	Botom of boring at 4.0 ft. No free groundwater was encountered.				

Depth scale: 0, 1.5, 3, 4.5, 6, 7.5, 9, 10.5

Standard Penetration Test CURVE



Legend of Samplers: Grab sample SPT sample Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG B3

Project: Umpqua Valley Tennis Center - 1201 NW Stewart Parkway, Roseburg
Project No.: 02-6385-01
Client: i.e. Engineering
Date: 03/20/2024
Location: south east corner of south courts (see Figure 2:site plan)
Elevation:
Driller: TGG (Ken, Nate C.)
Logged By: Kristen S. Pierce
Drill Rig: ATV mounted 4" diameter SSA
Depth To Water> Initial : At Completion :

USCS	Description	Depth	Sample No. and Type	NMC	Standard Penetration Test N
FILL	Asphaltic Concrete (AC): 3.25" total in two lifts, 1.5" lift over 1.75" lift.	0 – .27			
GW-SW	Loose, gray, sandy Gravel with some silt and clay; wet (fill).	.27 – .625	S1(1)		
CL	Medium stiff, black, Clay; with trace silt and organics, damp.	.625 – 3.0	S1(2)	21%	7
CL-ML	Stiff, gray and red mottled, Clay; with trace silt and coarse grained easily friable inclusions, moist.	3.0 – 4.5	S2	32%	11
	Botom of boring at 4.5 ft. No free groundwater was encountered.	4.5			

Depth scale: 0, 1.5, 3, 4.5, 6, 7.5, 9, 10.5
SPT CURVE scale: 10, 30, 50

Legend of Samplers: Grab sample · SPT sample · Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B4

Project: Umpqua Valley Tennis Center - 1201 NW Stewart Parkway, Roseburg
Project No.: 02-6385-01
Client: i.e. Engineering
Date: 03/20/2024
Location: south west corner of south courts (see Figure 2:site plan)
Elevation:
Driller: TGG (Ken, Nate C.)
Logged By: Kristen S. Pierce
Drill Rig: ATV mounted 4" diameter SSA
Depth To Water> Initial ▽ : At Completion ▼ :

USCS	Description	Layer bottom	Depth	Sample No. and Type	NMC	Standard Penetration Test N
FILL	Asphaltic Concrete (AC): 4.5" total in three lifts, 1.5" each.	.38	0			
GW-SW	Loose, gray, sandy Gravel; with some silt and clay, wet. (fill)	1.0		S1(1)		5
CL	Medium stiff, black, Clay; with trace silt and organics, damp.	3.0	1.5	S1(2)	24%	
				S2(1)	33%	8
CL-ML	Medium stiff, gray, brown and red, Clay; with trace silt and coarse grained easily friable inclusions, moist.	3.5	3	S2(2)	30%	

Botom of boring at 3.5 ft.
No free groundwater was encountered.

Depth scale: 0, 1.5, 3, 4.5, 6, 7.5, 9, 10.5
Standard Penetration Test CURVE: 10, 30, 50

Legend of Samplers: Grab sample · SPT sample · Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

APPENDIX B

LABORATORY TEST RESULTS

Expansion Index Worksheet
(ASTM D4829)

THE GALLI GROUP
Geotechnical Consulting

Client:	i.e. Engineering
Project	Umpqua Valley Tennis Center
Job No:	02-6385-01
Test Date:	3/26/2024
Sample Location:	B-1 S-2
Sample Date:	3/20/2024
Description of Soil:	Black, silty Clay

Expansion Index measured (EIm):

$EI_m = \Delta H/H_{orig} * 1000$

begin dial :	0.0143
end dial:	0.0904
EI_m:	**76**

Weight of ring (g):	365.1
Wt. Wet sample in ring(g):	729.3
Sample Wet Weight (g):	364.23
Sample Length (in.):	1
Sample Diameter (in.):	4.01
Volume of sample (ft^3):	0.007309
Sample Unit Wt. (PCF):	**109.8**
Sample Dry Unit Wt. (PCF):	**93.1**

Saturation (S):

$S=(SG)(w)\gamma d)/(SG)*62.4)-\gamma d$

SG:	2.7
γd:	93.1
%*w* :	17.9
S=	**60**

As prepared for testing:

can no.	D-10
wet weight of soil + can (g)	431.61
dry weight of soil + can (g)	385.36
weight of can (g)	127.49
weight of dry soil (g)	257.87
weight of water (g)	46.25
moisture content (% of dry weight)	17.9

EI_{50} Calculation:

$EI_{50} = EIm - (50-Sm)*[(65+EIm)/(220-Sm)]$

EI_M	76
S	60
EI_{50} =	**85**

After testing:

can no.	G-6
wet weight of soil + can (g)	599.31
dry weight of soil + can (g)	497.29
weight of can (g)	190.30
weight of dry soil (g)	306.99
weight of water (g)	102.02
moisture content (% of dry weight)	33.2

#4 + (dry wt.)	0
#4 - (dry wt.)	621.9
% Passing #4 Sieve =	**100.0**

Tested By: Nathan Chand

Expansion Index Worksheet
(ASTM D4829)

THE GALLI GROUP
Geotechnical Consulting

Client:	i.e. Engineering
Project	Umpqua Valley Tennis Center
Job No:	02-6385-01
Test Date:	3/26/2024
Sample Location:	B-1 S-3
Sample Date:	3/20/2024
Description of Soil:	red-gray, silty Clay

Expansion Index measured (EIm):

$EI_m = \Delta H / H_{orig} * 1000$

begin dial :	0.0353
end dial:	0.1126
EI_m:	**77**

Weight of ring (g):	191.6
Wt. Wet sample in ring(g):	527.4
Sample Wet Weight (g):	335.81
Sample Length (in.):	1
Sample Diameter (in.):	4.01
Volume of sample (ft^3):	0.007309
Sample Unit Wt. (PCF):	**101.2**
Sample Dry Unit Wt. (PCF):	**83.5**

Saturation (S):

$S=(SG)(w)\gamma d)/(SG)*62.4)-\gamma d$

SG:	2.7
γd:	83.5
%*w* :	21.2
S=	**56**

As prepared for testing:

can no.	A-6
wet weight of soil + can (g)	548.28
dry weight of soil + can (g)	479.46
weight of can (g)	155.12
weight of dry soil (g)	324.34
weight of water (g)	68.82
moisture content (% of dry weight)	21.2

EI_{50} Calculation:

$EI_{50} = EIm - (50-Sm)*[(65+EIm)/(220-Sm)]$

EI_M	77
S	56
EI_{50} =	**83**

After testing:

can no.	G-3
wet weight of soil + can (g)	576.67
dry weight of soil + can (g)	464.13
weight of can (g)	190.44
weight of dry soil (g)	273.69
weight of water (g)	112.54
moisture content (% of dry weight)	41.1

#4 + (dry wt.)	0
#4 - (dry wt.)	839.96
% Passing #4 Sieve =	**100.0**

Tested By: Nathan Chand

THE GALLI GROUP
Geotechnical Consulting
612 Northwest Third Street,
Grants Pass, Oregon 97526

Atterberg Limits Testing
ASTM D4318

Client: i.e. Engineering
Project: Umpqua Valley Tennis Center
Job No. 02-6385-01
Date Sampled: 3/20/2024
Sample Location B - 2 S - 1
Depth of Sample: 1.0' - 2.5'
Description of Soil: **Dark brown, clayey Silt**
Date Tested: 3/25/2024

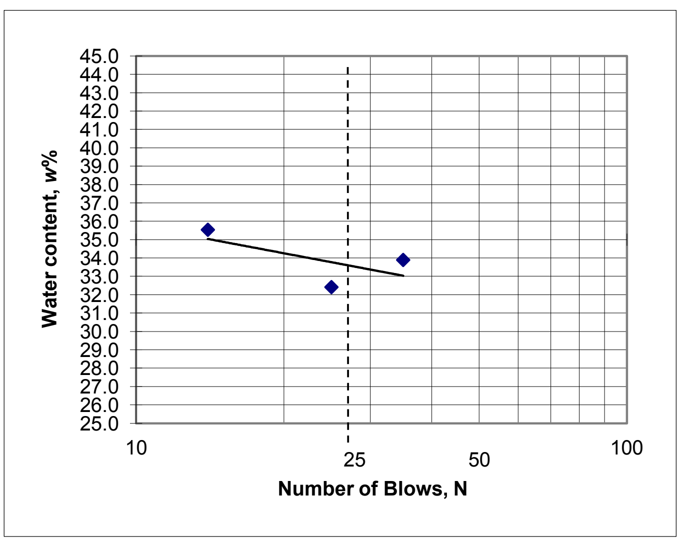
Liquid Limit Determination

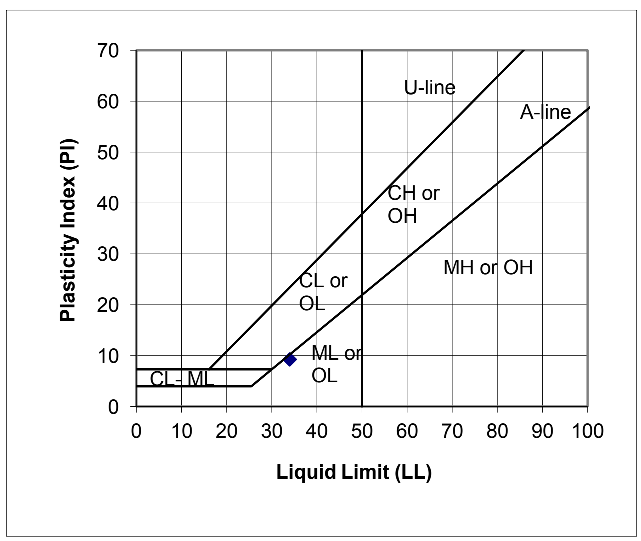
Can No.	203	xx	H
Wt. of wet soil + can (g)	27.86	22.02	23.11
Wt. of dry soil + can (g)	25.03	19.33	20.18
Wt. of can (g)	16.68	11.76	11.14
Wt. of dry soil (g)	8.35	7.57	9.04
Wt. of Moisture (g)	2.83	2.69	2.93
Water content, w%	33.9	35.5	32.4
No. of blows, N	35	14	25

Plastic Limit Determination

Can No.	x	z	I
Wt. of wet soil + can (g)	14.10	14.49	15.51
Wt. of dry soil + can (g)	13.65	13.89	14.76
Wt. of can (g)	11.78	11.53	11.65
Wt. of dry soil (g)	1.87	2.36	3.11
Wt. of Moisture (g)	0.45	0.60	0.75
Water content, w%	24.1	25.4	24.1

LIQUID LIMIT (LL)= 34
PLASTIC LIMIT (PL)= 25
PLASTICITY INDEX (PI)= 9





Tested by: Ken Perry