THE GALLI GROUP
Engineering Consulting

GEOTECHNICAL DESIGN REPORT
SAVAGE STREET BRIDGE
GRANTS PASS, OREGON

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02-4952-01
July 22, 2014

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1.0 INTRODUCTION

The Galli Group has conducted a site subsurface investigation and design studies for the proposed new bridge over Gilbert Creek at Savage Street, in Grants Pass, Oregon. The purpose of the study was to investigate the site soils and provide recommendations for design and construction of the project.

2.0 SITE AND PROJECT DESCRIPTION

The project site is where NW Savage Street crosses Gilbert Creek in NW Grants Pass. The current crossing is a traffic-lane-only width, at-grade crossing. The bridge-like older culvert structure has a wide downstream apron that falls into a deep plunge pool. It also has a concrete cross wall some 30 to 40 feet downstream to help pond water and limit the drop down from the culvert apron.

The stream channel upstream and downstream consists of steep banks and a channel embedded 10 to 16 feet below adjacent property. The banks are generally covered with understory and blackberry vines with abundant scattered trees.

The area upslope of the site consists of decomposed granite soil over weathered granite rock at shallow depth. Closer to the stream, which is located about ½ block out onto the flatter portion of this area of Grants Pass, the soils can consist of old alluvial silty sands and fine gravel over gravels and then weathered granite rock at depth.

The subject project consists of replacing a narrow culvert-like crossing of Gilbert Creek with a new much wider bridge or bottomless culvert structure. The design will allow for a full street section with bike lanes and sidewalks. It will also include reconstructing the stream channel to allow for juvenile fish passage and increase flood flow capacity and widened approaches along Savage Street.

3.0 SITE EXPLORATION

On June 21, 2014, personnel from The Galli Group accomplished two (2) exploratory borings at the site. One boring was drilled on each side of the creek with a hollow stem auger drill rig provided by Lawrence & Associates out of Redding, CA. The two borings

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penetrated to depths of 42 ½ feet and 38 ½ feet, into the dense, underlying weathered granite rock.

Due to the site constraints one lane of traffic had to be closed during drilling operations. Appropriate signage and two flaggers were used to provide safe traffic control. No incidents were reported.

When completed each hole was sealed with hole plug and the surface was patched with cold mix AC patch, pounded into the top of the hole. All drill spoils were carried away from the site.

4.0 SUBSURFACE CONDITIONS

4.1 SOIL

The site is in an area of Grants Pass that is generally underlain by weathered granitic rock or residual soil deposits of that material. The foothills, which begin about ½ block to the west, are comprised of the weathered, dense granitics. As you move out onto the flatter areas they tend to have colluvium and alluvium deposits of silty Sand and sandy Silt with scattered fine gravels over the weathered granite rock.

The two borings encountered the following generalized units.

<u>Road Section</u>

Both holes penetrated 4 inches of asphaltic concrete over approximately 12 inches of ¾" minus crushed rock base rock materials.

<u>Silt, Sand and Gravels</u>

Both borings encountered multiple layers of soils in combinations of silt, sand and occasionally gravels. These varied from loose to medium dense. These extended to depths of 21 feet in B-1 and 22 feet in B-2. It should be noted that both borings had a loose to very loose zone in the 14 to 17 foot depth range. These soils would <u>NOT</u> be suitable for foundation or wingwall support. The soils became saturated at between 7 and 9 feet.

<u>Gravels and Cobbles</u>

Both borings then encountered a layer of dense to very dense, sandy Gravels and Cobbles which extended to 33 feet in B-1 and 28 feet in B-2.

<u>Weathered Granitic Rock</u>

A very dense unit of silty Sand (weathered granitic rock) then extended to the bottom of the borings. This unit varies a bit but is uniformly dense and would be expected to stay that way to great depth.

Please see the Boring Logs in Appendix A at the end of this report for more subsurface detail. Note that soils conditions can change between borings and near the streambank.

4.2 WATER

Free water was encountered at depths of 8 feet and 9 feet in B-1 and B-2, respectively. These water levels are influenced by the stream level and runoff from storms which can saturate the overlying soil deposits adjacent to the stream.

Construction during wetter months of the year is likely to encounter seepage out of cuts as shallow as 5 or 6 feet. This seepage will likely cause instability and sloughing or failures in steeper cuts into the upper soils.

Deeply embedded footings could encounter significant water flow through the loose soil unit in the 14 to 17 foot range. Excavations into this water bearing zone would likely have to be fully shored and are not recommended due to cost and probable complications.

5.0 GEOLOGIC HAZARDS REVIEW

The subject site consists of both banks of Gilbert Creek. Therefore, there is the possibility of sloughing and failure of these steep banks during and after construction. Properly designed foundation and abutment support, as well as scour protection, should alleviate threats to the new bridge. These do not present a threat to adjacent structures.

There are no expansive soils in the area and risk of tsunami and sieche damage is almost zero. No known Quaternary faults cross the site (USGS, 2014; online database). Therefore, damage due to seismic ground offset is very low.

Moderately severe seismic shaking can be expected at the site. The site is basically Quaternary fan material over the granitic bedrock, based on the DOGAMI GIS geology overlay. The fan material appears to be about 30 feet deep on average. For this site we believe a Site Class B would be appropriate for design. Therefore, based on ASCE 7-10 methods:

$PGA_m = 0.39g$
Design PGA = 0.26g

The design PGA should be used for the dynamic seismic lateral earth pressures for design of the bridge abutments and other backfilled walls.

Liquefiable Soils. Both borings encountered a layer of potentially liquefiable soils between depths 14 feet and 22 feet below existing street grades. These are loose to very loose sands below the water table. They will be susceptible to liquefaction during a seismic event. During such liquefaction it is likely that these soils will undergo lateral spread and possibly "flow" out of the streambank face (if exposed). This could also lead to seismic induced settlement, which could cause up to 3 or 4 inches of surface subsidence. In our professional opinion, spread footings to support the bridge and its abutments cannot be placed above or within this zone. Methods to deal with this issue are incorporated later in this report.

6.0 GEOTECHNICAL RECOMMENDATIONS

This section provides recommendations for design and construction of the subject bridge, abutments, roadways, streambanks and in-stream wiers and walls.

6.1 SITE PREPARATION AND GRADING

6.1.1 Clearing, Grubbing and Stripping

All areas proposed for structures, roadways, bike paths and sidewalks or structural fill beneath these items should be cleared and grubbed of all trees, stumps, brush and other debris and/or deleterious materials (including old culvert parts). The site should then be stripped and cleared of all asphaltic concrete, vegetation, sod and organic topsoil. It appears that a stripping depth of from 4 to 8 inches will be required adjacent to roadways and sidewalks to remove the organic topsoil and rootzone. Additional stripping (or excavations) will most likely be required to remove root balls beneath larger bushes, any waste fill areas encountered close to the existing culvert, old culvert fill and old structures. The stripped materials, old fill soils and debris removed should be hauled from the site. This material should not be used in structural fill. All old, undocumented fill must be removed beneath the bridge area, roadways, sidewalks and other hard surfaces, due to the possibility of it consolidating/densifying under new load.

Holes or depressions resulting from the removal of underground obstructions and old debris or excavations for stump removal or old fill that extend below the finish subgrade and will be beneath structures, access or walkways, shall be cleared of all loose material and dished to provide access for compaction equipment. These areas shall then be backfilled and compacted to finish subgrade with structural fill, as described later in this report.

Note: The project will require removal of the existing culvert/bridge, wingwalls, plunge pool wall and associated fills and debris. This would normally be removed down to undisturbed native soils where other structure or fills will be placed.

It is recommended that grubbing and stripping of the site, removal of old fill and structures, and backfill and compaction of depressions below finish subgrade be observed by the project engineer or his representative from The Galli Group.

6.1.2 Structure Removal Issues

When the existing culvert/bridge (including the wing walls) is removed, support for both banks will be significantly decreased. This will also undoubtedly include removal of materials from below the current water level in the stream. This removal will also expose the full height of each bank (between 12 and 15 feet tall). It is likely that seepage could be present in these areas. Such seepage out of the face of the excavations can cause unstable conditions. Therefore, the project design and the methods used by the contractor must be compatible with maintaining bank stability and preventing large scale loss of soil into the stream.

Culverting the streamflow through the work area during such removal work and foundation work would be very beneficial.

6.1.3 Subgrade Proofrolling

The exposed subgrade throughout the site which will support structures, access, fills and sidewalks (and that can be reached reasonably) should be proofrolled (after grubbing and stripping and overexcavation where required) under the observation of a representative from The Galli Group. This will mostly be roadway areas back away from the top of banks. The proofrolling may be accomplished with a loaded dump truck, loaded water truck or large heavy roller (no vibration). Proofrolling should not be attempted in wet weather and should be discontinued if it appears the operation is pumping moisture up to the surface or otherwise disturbing the in-place soils. When proofrolling, the tires of a loaded truck should not deflect the soils more than ⅜ inch.

Where soils are disturbed or do not demonstrate a firm, unyielding condition when proofrolled, the soil should be removed and replaced with imported granular fill. The imported fill material should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM Test Method D-698 (Standard Proctor). All soft and/or unstable areas should be over-excavated and backfilled with granular structural fill.

6.2 UTILITY AND SITE EXCAVATIONS

During the construction of the project, we anticipate utility excavations will be required for construction of utility lines. The utility excavations and cuts will generally encounter the sandy Silt or silty Sand soils.

Excavations. In our opinion all excavators of larger size should be able to remove these materials, to depths that would be used on this project (probably 15 feet or less).

Trench Excavations. Trench excavations during dry weather should stand for short periods of time (several hours) in shallow trenches in the soils (less than 3 feet) which are not subjected to emerging groundwater seepages or surface water. Seepage or wet weather could cause the silt soils to cave and slough into the trench. *Excavations deeper than 3 feet in soil could require the use of temporary shoring, trench boxes and/or temporary cut slopes.* Pumping from open sumps should be able to handle the majority of water in trenches. This assumes trenches are not deep (greater than 7 feet) and into the water bearing loose soils. If deep trenches are anticipated, wall collapse and cave-ins are likely. This is especially true where seepage is present. *We strongly recommend that if* *at all possible such deep trenches not be used as part of the project.* To help decrease sloughing, construction in the drier weather periods would be better. Workmen must still be protected in all trenches.

Please note that while we have commented on the anticipated stability of the soil in trenches, we are not responsible for job site safety. The contractor is at all times responsible for job site safety, including excavation safety. We recommend all local, state and federal safety regulations be adhered to.

6.3 STRUCTURAL FILL PLACEMENT AND COMPACTION

6.3.1 Beneath Structures and Roadways

Structural fill is defined as any fill placed and compacted to specified densities and used in areas that will be under structures, roads and sidewalks and other load-bearing areas or that will create fill slopes. It appears that the roadway and sidewalks and all wing wall areas will have structural fill below or behind them. The subgrade in these areas needs to be prepared properly and the soils must be placed and compacted correctly for proper long-term performance.

Structural Fill Materials. Ideally, and particularly for wet weather construction, structural fill should consist of a free-draining granular material (non-expansive) with a maximum particle size of six to eight inches (larger is allowable if trenching through it is not required). The material should be reasonably well-graded with less than 7 percent fines (silt and clay size passing the No. 200 mesh sieve). During dry weather, any organic-free, non-expansive, compactable granular material, meeting the maximum size criteria, is typically acceptable for this purpose. Locally available crushed rock, jaw-run crushed shale, and sandy decomposed granite (DG) have performed adequately for most applications of structural fill.

Note: It is likely there will be several types of structural fill used at the site. Some need to provide good support for the roadway and bike path, others need to perform well as wall backfill or to create stable streambank areas. Each of these types of fill should be specific for the task it will be used on. We assume these would be as follows:

Streambed Fill	8" to 24" angular rock with sand and fine gravel infill; less than 25% passing the 1" sieve.
Streambed Fill; Above Water Line	12" minus dirty, jaw-run shale compacted to 95% of ASTM-D698; less than 15% passing No. 200 sieve
Wingwall (dry) Fill	4" minus jaw-run shale that is reasonably free draining; less than 5% passing the No. 200 sieve.
Wingwall (wet) Fill	Clean, ¼" to ¾" crushed rock or larger, OR 2-inch minus crushed rock placed and compacted in the dry.
Roadway/Walkway Subbase	4" minus jaw-run shale or crushed rock; less than 10% passing No. 200 sieve.

Roadway Base Rock	¾" or 1" minus crushed rock; less than 7% passing the No. 200 sieve.
Wall Drainage Material	¼" to ¾" clean, crushed rock or 1" to 1½" drain rock.
Overexcavation Backfill	¾" or 1" minus crushed rock; less than 7% passing the No. 200 sieve.

Structural Fill Placement. Structural fill should be placed in horizontal lifts not exceeding 8 inches (or 1½ times the largest stone for materials larger than 4" minus) loose thickness (less, if necessary to obtain proper compaction) for heavy compaction equipment and four inches for light and hand-operated equipment. Each lift should be compacted to a minimum of 98 percent of the maximum dry density, as determined by ASTM Test Method D-698 (Standard Proctor).

Structural fill placed beneath footings or other structural elements must extend beyond all sides of such elements a distance equal to at least ½ the total depth of the structural fill beneath the structural element in question for vertical support (i.e. for 3 feet of structural fill beneath footings, extend the fill at least 1½ feet past all edges of the footing). These fills must extend further beyond edges of footings if lateral support is required (generally in the order of 5 to 6 feet or more).

To facilitate the earthwork and compaction process, the earthwork contractor should place and compact fill materials at or slightly above their optimum moisture content. If fill soils are too high on the wet side of optimum, they can be dried by continuous windrowing and aeration or by intermixing lime or Portland Cement to absorb excess moisture and improve soil properties. If soils become dry during the summer months, a water truck should be available to help keep the moisture content at or near optimum during compaction operations.

Fill Placement Observation and Testing Methods. The required construction monitoring of the structural fill utilizing standard nuclear density gauge testing and standard laboratory compaction curves (ASTM D-698 specified) is applicable to materials 2-inch size and under. Larger (2½" or above) jaw-run "shale", crushed rock and the pulverized DG do not yield consistent results with this type of testing. The high percentage of rock particles greater than ¾'s of an inch in these materials causes laboratory and field density test results to be erratic and does not provide an adequate representation of the density achieved. Therefore, construction specifications for this type of material typically specify method of placement and compaction coupled with visual observation during the placement and compaction operations, instead of nuclear density testing.

For these larger rock materials, we recommend the 8-inch lift (after being "worked in" with a dozer) be compacted by a minimum of 3 passes with a heavy vibratory roller. One "pass" is defined as the roller moving across an area once in both directions. The placement and compaction should be observed by our representative. After compaction, as specified above, is completed the entire area should be proofrolled with a loaded dump truck to verify density has been achieved. All areas which exhibit movement or compression of the rock material more than ¼ inch, under proofrolling, should be reworked or removed and replaced as specified above.

Field density testing by nuclear methods would be adequate for verifying compaction of 2-inch to ¾-inch minus crushed base rock, Decomposed Granite and other materials 2 inches or smaller in size. Therefore, typical specifications would suffice. Testing should be accomplished in a systematic manner on all lifts as they are placed. Testing only the upper lifts is not adequate.

6.3.2 Non-Structural Fill

Any waste soil, organic strippings or other deleterious soil (such as wet or dried out expansive clay) would be considered non-structural fill. These materials may make reasonable landscape soils and lawn topsoil material. This material may be placed in landscape areas and waste soil areas such as berms with slopes at 3.5H:1.0V or flatter. It should not be placed under structures, sidewalks, roadways, parking areas or as part of a structural fill slope. It is recommended that when these soils are used they be given a moderate level of compaction (90 to 92 percent) to help seal them from surface water.

Note: It is unlikely there will be room on the site for such undocumented fill zones. This material should not be used to rebuild the banks.

6.4 CUT AND FILL SLOPES

Cut and fill slopes will be incorporated into the project and utilized during construction. Care must be taken to design and construct these for short-term safety and good long-term performance. These should be designed as described below.

6.4.1 Cut Slopes

Permanent Cut Slopes. Across the site, we anticipate that some permanent cut slopes ranging from 3 to 10 feet in height could be needed. We recommend permanent cuts made into these slopes be varied in steepness depending upon the strata type being excavated (or all be sloped at the flattest inclination given). The general permanent cut slope rule for this site should be as follows:

Strata Type	Cut Slope Inclination
Clayey Silt	3H:1V
Silt, Sand and Gravel	2¼H:1V

It should be noted that at the slope angles provided above some sloughing and slumping near the top of the cut slopes should be expected. The upper portions of the slopes could be flattened in an attempt to alleviate more sloughing of the weaker materials, but that would typically increase the areas of the cut slopes and potential construction costs significantly. The above-listed cut slope recommendations also assume that concentrated surface water flows are not present and do not "run" down these slopes. Excessive amounts of surface water will most likely result in surficial sloughing of the upper clayey silt soil unit. They also assume no subsurface seepage is exiting the slopes.

Temporary Cut Slopes. Temporary cut slopes will likely be used during construction. For this site we recommend the following:

Strata Type	Cut Slope Inclination
Silt, Sand, Gravel Soil	1½H:1V

Please note that these cut slope angles could be required to construct the project. They could also be subject to smaller sloughs and failures, especially in wet weather and where seepage exists. Rockfall also may take place when larger stones appear. In that case the cut must be "scaled" of all pieces longer than 2 inches and/or a chain link fence across the toe should be installed to protect the workmen. The contractor must inspect all cut slopes every day for signs of instability or loose stones.

Note: It is possible some of these cuts could be made at 1H:1V. However, based on the borings these could be very unstable. In the case where site constraints necessitate steep cuts these will likely have to be shored to protect workmen and the stream or adjacent property.

We strongly recommend that we discuss all temporary cut slopes with the contractor before excavation and be on site when they are cut and periodically during project construction.

6.4.2 Fill Slopes

All fill slopes should be placed and compacted as structural fill as described earlier in this report and be no steeper than 3H:1V for native clayey Silt and 2½H:1V for other silt soil structural fill. Imported DG may be placed as steep as 2.0H:1V; and angular shale or rock may be placed at 1.75H:1V.

Fill slopes that will be inundated by stream flow will have to be flatter than listed above. We recommend these be constructed of angular rock or dirty crushed rock of larger sizes, 12-inch minus or larger. These slopes should be constructed at 2H:1V or flatter. This is for slopes not subject to swift currents or steam flow scour attack. In those situations heavy rip rap will be required and could be placed at 1H:1V if interlocked properly and is well embedded (to below scour depth).

Compaction of the fill being placed is critical to its stability and to the stability of adjoining or upslope and downslope areas. Therefore, fills should be placed and compacted as structural fill as described earlier in this report.

We recommend, in order to decrease sloughing and erosion of any fill slopes that all fills be overbuilt laterally and the face cut back to a compacted fill face. This would not be required of slopes constructed of hard rock fill materials. It is critical to decrease long-term settlements that these fills be placed and compacted properly. All materials should be placed and compacted as described earlier in this report.

Fill on Slopes. Fills placed on slopes steeper than 10% must be keyed and benched into the native slope. Subdrains in the key and part way upslope beneath the fill may also be required. Figure 3, Fill on Steep Slope, provides recommendations of how this should be accomplished.

In areas where a fill slope will be built over a cut slope or dense native slope the following is recommended:

1. Establish, with the help of the geotechnical engineer, that the underlying slope is dense and stable.
2. Cut an initial bench that is at least as wide as the proposed fill height to be placed. Backslope the bench into the slope at 10% to 15%.
3. Cut benches into slope the rest of the way back up the slope. These should be at least level or slightly backsloped and run along the contours.
4. Inspect entire cut out areas and establish where subdrains are needed.
5. Install subdrains as shown in Figure 4 (where needed).
6. Construct the fill above as structural fill.

6.5 BRIDGE FOUNDATION RECOMMENDATIONS

The subject bridge across Gilbert Creek must be supported on 1) spread footings founded in dense soils below the liquefiable soil zone and anticipated scour depth or 2) deep foundations such as auger cast piles, drilled piers or driven piles.

A review of the borings shows silty Sand and clayey or sandy Silt with some gravels down to about 10 feet or so. At 10 to 11 feet appears to be more gravels with SPT blow counts up to 21 and 26 in B-1 and B-2, respectively. But then at about 14 feet they encounter a very loose layer with SPT N-values of 3 and 5. *This layer is subject to liquefaction.*

It appears to get footings embedded below scour depth they would be in the loose zone at 14 to 21 or 22 feet. That means footings would have to be close to 25 feet deep in order to be embedded into the denser gravels. That may be too deep to construct for this project.

It appears that piles driven into the denser, gravelly zones between 21 and 30 to 35 feet would be the best foundation support. Then the pile caps can be placed where best suits the rest of the bridge.

It appears a pile cap at a depth of 10 to 12 feet with 15 foot long piles below that would be a good option.

The pile sections, embedded to a depth of 28 to 30 feet below existing street grade, could be designed to support axial compression loads. Based on standard end bearing computations below the water table, various pile sizes and types will have the following capacity (combined end bearing and friction).

Pile Size	Axial Compression (KIPS)
10-inch Steel Pipe (closed end)	30
12-inch Steel Pipe (closed end)	40
16-inch Steel Pipe (closed end)	70
HP 10X42 or less*	35
HP 12X53 or less*	50
HP 14X63 or less*	70

Assumes: Final embedment depth of approx. 28 to 30 ft. below existing road grades.
Soil profile as found in attached boring logs.
Water Table at 8 feet.
Factor of Safety of 2.0.
Armored tips required.
Spacing is 4 feet minimum.
*Structural Engineer Decision

The steel sections of these piles likely can withstand much greater axial loads. In our opinion, the subject very dense gravel and cobble zone will likely provide better end bearing capacity than provided by the computations. However, to verify this we recommend a pile load test be performed.

It should be noted that the driven piles will not provide uplift resistance; therefore, the structural engineer should utilize the weight of structure and embedded foundations to resist uplift loads.

Pile to pile spacing should be at least four (4) pile diameters. This assumes that heavy, pile sections are driven with a pile hammer with at least 24,000 foot pounds of energy (needs to be sized by the contractor). **Note:** Depending upon which pile size and pile hammer are selected for the project, the pile "refusal" criteria will change somewhat. We would be available to provide the criteria needed for the specific hammer and pipe size selected for the project. Piles other than armored steel should not be utilized for this project as they will likely not be able to withstand the driving force required to penetrate the underlying, dense gravel and cobble unit.

Anticipated Pile Settlements. Piles driven to end-bearing in the underlying dense granitic sands will likely experience small amounts of settlement (or deformation) under load. At this time, we estimate that total pile settlements should be less than ¾-inch.

Abutment and Pile Cap Foundations. Due to the fact that the upper native silty soils are easily disturbed and can soften when exposed during wet weather, it is prudent to "protect" the subgrade soils during wet weather construction by placing at least 12 inches of 8-inch minus crushed rock structural fill over firm, undisturbed subgrade soils. In this manner, the abutment footing subgrades will be "protected" during rebar and form placement which will help protect them against possible future washouts.

Pile Design and Installation Verification. The Geotechnical Engineer must be included in the design process in order to discuss and review the various foundation support schemes and to provide additional geotechnical recommendations for foundation support. We recommend that our engineers be allowed the opportunity to review and comment on the proposed foundation support design prior to final plans submittal.

Our representative should also be present to observe and document all pile installation. Depths of embedment and "set values" should be recorded for each pile. All piles should be accepted based on field pile driving set values, established by the FHWA dynamic formula (Gates formula) and required depths of embedment. These records should be immediately reviewed by our design engineer to verify that each pile is capable of supporting the required design load. At the end of pile installation, we would provide a pile installation summary report for submittal to the local building department.

6.6 SCOUR PROTECTION

The subject bridge abutments will require protection against the moderate and high streamflows of Gilbert Creek. Typical streamflow velocities for such bridge sections on a stream like Gilbert Creek range from 10 feet per second to 15 feet per second. Localized turbulent areas could also be somewhat greater. To resist the scouring affects of this flow the banks around the abutments will have to be protected by an angular rip rap protected section. This should be constructed as follows:

1. Cut bank adjacent to abutment to at least 4 feet below the bottom of the reconstructed streambed (scour embedment).
2. Line area with a geotextile fabric (6 oz. per square yard or more).
3. Cover with 8-inch layer of 6-inch minus angular crushed rock or shale. Cover with a 36-inch thick layer of 12-inch to 24-inch angular rip rap, placed for maximum interlock at slope of 1H:1V or flatter.
4. Infill with 4-inch minus dirty crushed rock (helps for rooting vegetation).
5. Plant with scores of live willow stakes.

This scour protection blanket should be shaped around the upstream and downstream ends and along the front of the abutments. This should extend to above the 25-year flood line.

6.7 LATERAL LOAD RESISTANCE

Lateral loads exerted upon this bridge and its foundations can be resisted by passive pressure acting on buried portions of the foundation and other buried structures and by friction between the bottom of concrete elements of the foundations and the underlying soil.

We recommend the use of passive equivalent fluid pressures of the following values for portions of the structure and foundations embedded into the onsite soils.

Soil Unit	Depth (ft)	Passive Equivalent Fluid Pressure (psf)	Coefficient of Friction (f)
Silt, Sand/Gravel	1'-4'	200	0.35
Native silty Sands	4'-12'	300	0.35

We recommend that at a minimum the first 12 inches (local frost depth) below the native ground surface be ignored when computing the passive resistance. The liquefiable sands between depths of 14 and 22 feet should also be ignored when computing passive resistance.

The piles may also be used to resist lateral loads. Typical lateral resistance values would be 5 kips per pile (allowing up to 1 inch of movement at the top). **Note:** This will likely not be available through the liquefiable zone during a seismic event. This could decrease the lateral capacity at the top.

6.7.1 Seismic Design Considerations

The pile foundations and bridge will be subjected to loads from severe ground shaking in a seismic event. This could result in the loose sand layer at 14 to 22 feet experiencing liquefaction and will likely increase the overall loading on the underlying subgrade soils. Using the current IBC & OSSC methodology the Design PGA was determined to be 0.26g. This should be used in the design of the structure.

These values should provide adequate strength for protection of egress during a seismic event. Please note this does not mean that damage would not occur as a result of such a seismic event.

6.8 RETAINING WALLS OR ABUTMENT WALLS

Lateral earth pressures will be imposed on all below ground and backfilled structures or walls, including foundations which do not have uniform heights of fill on both sides. The following recommendations are provided for design and construction of conventional reinforced concrete retaining walls:

- We recommend walls which are free to rotate at the top (unrestrained) when backfilled, be designed for an equivalent fluid pressure (EFP) of 40 pcf.
- Walls that are fixed at the top (restrained) when backfilled should be designed for an equivalent fluid pressure of 60 pcf.
- A wet soil unit weight of 135 pcf should be used for design of retaining walls which are backfilled with crushed rock, jaw-run "shale" or good quality sandy decomposed granite.
- These values are for properly compacted, free draining, non-expansive, granular backfill, free of organics and other debris or for imported granular backfill. Imported crushed rock or jaw-run "shale" works well for wall backfill materials.
- These design values assume the wall or structure is fully drained and has no surcharge loads from traffic or other structures. The structural designer should include surcharge loading from traffic, building loads and/or sloped backfill. A typical traffic load would be a uniform load on the back of the wall of 75 pounds per square foot (assumes 250 psf surface traffic load surcharge).
- We recommend designing retaining walls to resist seismic loading. A horizontal acceleration component of 0.26g (Design PGA) should be applied to the mass of an enlarged active wedge of soil behind the walls and utilized in a pseudo-static analysis. The wedge length back from the wall along the ground surface may be taken to be 0.6H, where H is the height of the wall. This relates to an equivalent uniform load over the entire back of the wall of approximately 10 pounds per square foot for each foot of backfill, for walls up to 10 feet tall (i.e. for a 6-foot wall, uniform seismic load will be 48 psf).
- The backfill should be placed in lifts at near the optimum moisture content and compacted to between 93 and 95 percent of the maximum dry density as determined by laboratory procedure ASTM D-698 (Standard Proctor). Loosely placed backfill will exert greater pressures on the wall than the pressures provided above and <u>must</u> be avoided.
- To prevent damage to the wall, backfill and compaction against walls or embedded structures should be accomplished with lighter hand-operated equipment within a distance of 1/2 h to 1/3 h (h being the vertical distance from the level being compacted down to the surface on the opposite side of the wall). Outside this distance, normal compaction equipment may be used.

While proper compaction of wall backfill is critical to the proper performance of the walls, care should be taken to not over-compact the backfill materials. Over-compaction can induce greater lateral loads on the wall or structure than the design pressures given above.

6.9 WALL AND ABUTMENT DRAINS

All retaining walls and embedded structures should have proper drainage.

Wall drains would typically have a minimum 12-inch wide drainage zone of drain rock wrapped in non-woven filter fabric immediately behind the wall extending up from the drainage section to within 6 inches of the surface. A preformed, fabric-wrapped, polymer sheet drain, such as Amerdrain, Linq Drain or Enkamat may be used on the back of the wall in lieu of the vertical drainage zone, provided this is backfilled with clean, free-draining material. The retaining walls must have the dirt side sealed with approved sealer if seepage through the wall is to be prevented. Exterior wall drainage sections, which will not be sealed on top by asphalt or concrete, should have the upper 12 inches backfilled with compacted onsite silt soils to minimize intrusion of surface waters into the wall drain system. Please see Figure 5 for details of typical wall drainage methods.

All drains should be tightlined and positively sloped to an approved stormwater disposal location into the public storm drain system. **Note:** In no case shall water be collected and/or directed or discharged close to or on slopes. Such water discharge can cause added slope instability and erosion.

6.10 ASPHALTIC CONCRETE PAVEMENTS

It is our understanding that this section of Savage Street will likely consist of Hot Mix Asphaltic Concrete (HMAC) paved surface. This would include parking and any bike lanes. The following sections provide recommendations for asphaltic concrete section design and construction. We have also included one Portland Cement concrete design in case it is included in the project.

6.10.1 Pavement Subgrade & Traffic Loading

The subject site is underlain by sandy Silt and silty Sand soils. Based on typical asphalt design methods we have assumed an R-Value of 10 for these soils. This is consistent with R-Values from other sites that have similar soils.

The following asphalt sections were designed utilizing a Crushed Rock Equivalent (CRE) method. This is a typical design method utilized and/or required by many state and local jurisdictions. Sufficient thickness of asphaltic concrete and rock materials are used to provide the computed crushed rock equivalent needed to protect the subgrade soils and successive rock layers from anticipated traffic loads.

We anticipate the traffic loading to consist of autos, pick-ups, delivery trucks, occasional trash trucks and some heavy trucks needed for moving or construction. Only minor (2% to 3%) heavy truck traffic is anticipated for this roadway. In our professional opinion, the project should have the design based on the use of the Traffic Indice (TI) as listed below. The TI values should be verified by the project designer or the City.

- Residential Street; TI = 7.2

6.10.2 Pavement Design

We have designed the pavement sections using the Traffic Indices (TI) listed above. Based on this TI and R-values of 10, 50 and 90, (subgrade soil, 4" minus and ¾" minus), we have computed asphalt design sections (utilizing the Crushed Rock Equivalent Method) with the following results.

Residential Street (TI = 7.2)

4" AC
6" AB (3/4" minus Crushed Rock)
14" ASB (4" minus Crushed Rock or Jaw-Run Shale)
Woven Geotextile Support Fabric (ACF 200 or Equivalent)

6.10.3 Portland Cement Concrete Pavements

It is possible the site will use reinforced Portland Cement Concrete Pavements. These can have varying amounts of traffic and number of trucks. Therefore, we have provided a section that can support minor truck traffic.

PCC Pavements

8" Portland Cement Concrete (4,000 psi mix)
6" Aggregate Base (3/4" or 1" minus Crushed Rock)
10" Aggregate Subbase (4" minus Crushed Rock or Jaw-Run Shale)
Woven Geotextile Support Fabric (ACF 200 or Equivalent)

Note: The Portland Cement concrete section design assumes the subgrade is prepared properly and that the woven fabric is used to help distribute construction loads and provide some added protection to the subgrade.

The following items should be part of the concrete design and construction.

Aggregate Base: Extend beyond edges of concrete at least 18 inches.

Reinforcing: No. 5's @ 12" O.C. each way. Rebar at center of concrete. Include continuous edge bars at 3" to 4" from all edges. Reinforcing to be continuous across all different pours or joints. Overlap all bars at least 24 inches.

Concrete: 4,000 psi, 28-day strength mix; 5% ± 2% entrained air; place at 4" slump or use admixtures to keep same water/cement ratio for higher slump. Do not over trowel surface early and trap bleed moisture below the finish, which can lead to freeze-thaw damage.

Surface Jointing: Surface jointing at 6 to 8 feet on center each way will help decrease cracking in the "field". If saw cutting is used it must be done as soon as the surface will support the work to make sure cracks do not develop within the concrete mass prior to the surface cutting. **Note:** This is probably not practical except for cross jointing at 10 foot intervals.

6.10.4 General Recommendations

Subgrade Preparation. Subgrade preparation should begin with removal of debris and loose and disturbed soils. All debris and organic material should be disposed of properly and is not permitted as subgrade or fill material.

The subgrade should be shaped to a uniform surface running reasonably true to established line and grade described in the contract documents. Areas so specified must be redensified and/or backfilled with structural fill. It is important that dense, stable conditions of the subgrade be maintained until the subgrade is covered with the woven fabric and subbase rock. Subgrade preparation should include cleaning and proofrolling to identify soft and disturbed subgrade areas.

After subgrade preparation is completed, the upper 12 inches of exposed subgrade prepared for the pavement structure should demonstrate at least 95 percent of the maximum dry density, as determined by the Standard Proctor test (ASTM D-698).

Soft or loose materials disturbed during the excavation process, incapable of achieving the compaction criteria should be removed to appropriate bearing materials prior to replacing with structural fill. Where loose or softened subgrade areas are identified, the area should be over-excavated and replaced with imported granular fill with less than 10 percent passing the number 200 sieve.

It should be noted that in no case should construction trucks be allowed to "run" directly on top of the subgrade soils until they are covered with rock. This would most likely result in the disturbance of the subgrade soils due to the heavily loaded vehicles (which would result in additional over-excavation to remove softened soils). We recommend covering the subgrade soils with at least 12 inches of crushed rock or "shale" over the woven fabric prior to construction truck traffic traversing the area. Therefore, construction traffic must be carefully coordinated in order to minimize disturbance to the underlying fine-grained soils.

Wet Weather Construction. We recommend that for construction during wet weather, in all construction roads and drive lanes, the subgrade should be covered with a woven geotextile support fabric (ACF 200 or equivalent) and a minimum of 12 inches of imported granular 4-inch minus crushed rock (contractor must use more rock if necessary to maintain stable subgrade). Compaction of the fill should not begin until a minimum of 12 inches of rock is placed above the fabric. This should provide an adequate working surface and help protect the subgrade from damage from construction traffic. Construction traffic should not be allowed to traverse the area until the minimum of 12 inches of compacted material has been placed and compacted.

Note: Preparation of subgrade and rock placement during dry weather typically yields a better asphaltic concrete section.

Geotextile Fabric Placement. When the subgrade soils have been properly prepared, the silt and clay areas should be covered with the woven geotextile support fabric. We recommend a fabric such as ACF 200 or equivalent. The fabric should be laid longitudinally with the roadway. All ends and edges should be overlapped a minimum of 5 and 2 feet, respectively. Fabric layout shall be such that it "runs" aligned with the lane traffic directions.

Care must be taken to not damage the fabric. In no case shall track vehicles be allowed on the fabric. At least 12 inches of rock should be over the fabric prior to allowing truck traffic in the area. Then the traffic should be light to protect the subgrade. Be careful not to disturb the subgrade when compacting the rock.

Materials. All materials used and construction techniques applied at the site must result in conditions as assumed for design of the pavement sections. We recommend materials used in the pavement support sections be as follows:

Aggregate Base Rock
- Crushed Rock (¾ or 1" Minus); R=90
- Exceeds the fracture, durability and sand equivalent requirements outlined in Section 00641 of the Oregon Standard Specifications for Construction
- Maximum passing the No. 200 sieve=7%
- Compacted to 95% of the maximum dry density as determined by ASTM D698 or AASHTO T-99

Aggregate Subbase Rock
- Crushed (jaw run) hard "Shale" (6" Minus) or Crushed Rock (2 to 4" Minus); R=50
- Exceeds the fracture, durability and sand equivalent requirements outlined in Section 00641 of the Oregon Standard Specifications for Construction
- Maximum passing the No. 200 sieve=10%
- Compacted to 95% of the maximum dry density as determined by ASTM D698 or AASHTO T-99

We recommend avoiding the use of soft rock or subrounded and/or sandy gravel materials for the aggregate base, since they typically do not perform well in supporting asphaltic pavement sections (i.e., usually do not meet CBR requirements).

Installation of utilities and other site work, which may compromise the integrity of the support fabric or completed base rock section, should be avoided when possible. Therefore, utilities which must cross through these areas should be placed and backfilled prior to placing the fabric and all base rock sections.

We recommend that the finished subgrade and subbase be viewed and that base rock be tested for density and stability by a representative of The Galli Group prior to placement of asphalt at the site.

Asphaltic Concrete. We recommend the project plans and specifications require the use of Dense Graded Hot Mix Asphalt Concrete (HMAC) and the contractor must provide an ODOT approved HMAC design mix. Section 00745 of the 2008 edition (or newer) of the Oregon Standard Specifications for Construction should be specified for all HMAC provided for this project. We recommend all aspects of the asphaltic paving be accomplished in accordance with applicable ODOT standards and recommendations.

Drainage. Adequate provision should be made to direct surface water away from the pavement section and subgrade. Ponded water adjacent to the roadway can saturate the subgrade resulting in loss of support. Therefore, we recommend the areas along the edge of the roadway be well drained. All paved areas should be sloped and drainage gradients maintained to carry surface water to catch basins or ditches for transmission off the roadway and parking areas.

Maintenance. Pavement life can be extended by providing proper maintenance and overlays as needed. Cracks in the pavement should be filled to prevent intrusion of surface water into the subbase. Asphalt pavements typically require seal coats or overlays after 15 to 20 years to maintain structural performance and aesthetic appearance.

6.11 SITE DRAINAGE

Site drainage will be critical to maintaining bank stability along this reach of Gilbert Creek. In all areas concentrated flow from the street, parking, bike path, sidewalk and landscape areas <u>must not</u> be allowed to flow over the top edge and down the banks. This will lead to excessive erosion and bank instability. All such storm runoff should be captured in catch basins and conveyed to the stream via storm drains.

Landscape areas that could have pesticides or fertilizers used during their life should have runoff filtered through bioswales or other medium to reduce nitrates and other contaminant concentrations before entering the stream.

7.0 EROSION CONTROL

The site soils are moderately to highly susceptible to erosion, depending upon site disturbance. The site grades are moderate to steep on the site. Therefore, site erosion could be moderate to high depending upon construction practices.

Construction Erosion Control. Typical requirements would be that all disturbed areas shall have the low side surrounded by a silt fence with the bottom edge embedded in the soil at least two (2) inches. At select locations settling ponds of hay-bale backed silt fence should be established to decrease silt content of water flowing off site. This could be difficult given the locale of the stream. It is likely ODFW will have their own requirements for controlling erosion on this site. Any downslope catch basins within 200 feet should be protected by hay bales or wattles.

Permanent Erosion Control. Permanent project landscaping and paving for the project will meet most needs of long-term erosion control. All disturbed areas on the site but outside the developed area of the project must be reseeded with local native grasses and planted with understory and overstory per ODFW requirements for erosion prevention.

8.0 ADDITIONAL SERVICES AND LIMITATIONS

8.1 ADDITIONAL SERVICES

We should be involved in final foundation/abutment wall support design. We should also review construction plans and specifications for this project when they have been developed. In addition, The Galli Group should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. Also, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, all construction operations dealing with earthwork and bridge foundations should be observed by a representative from The Galli Group.

For this project, we anticipate additional services could include the following:

- Review of final construction plans and specifications for compliance with geotechnical recommendations.
- Possible project team meetings to clarify issues and proceed smoothly into and through the construction process.
- Observation of onsite cut slopes and excavations to verify stability.
- Observation of slope benches and subdrains below fills on slopes.
- Review of pile and pile hammer data.
- Observation of pile driving operations.
- Observation of over-excavated areas, structural fill placement, fill subdrains, subgrade proofrolling, footing subgrade, crushed rock placement and compaction and wall drainage.
- Periodic construction field reports, as requested by the client and required by the building department.

We would provide these additional services on a time-and-expense basis in accordance with our current Standard Fee Schedule and General Conditions at the time of construction. If we are not retained to provide these services we cannot be held responsible for the decisions by others or geotechnical related issues in the constructed product.

8.2 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions and assumed development plans as they existed at the time of the study, and assume soils, rock and groundwater conditions exposed and observed in the borings during our investigation are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions or assumed design information is found to be different, we should be advised at once so that we can review this report and reconsider our recommendations in light of the changed conditions. If there is a significant lapse of time between submission of this report and the start of work at the site, if the project is changed, or if conditions have changed due to acts of God or construction at or adjacent to the site, it is recommended that this report be reviewed in light of the changed conditions and/or time lapse.

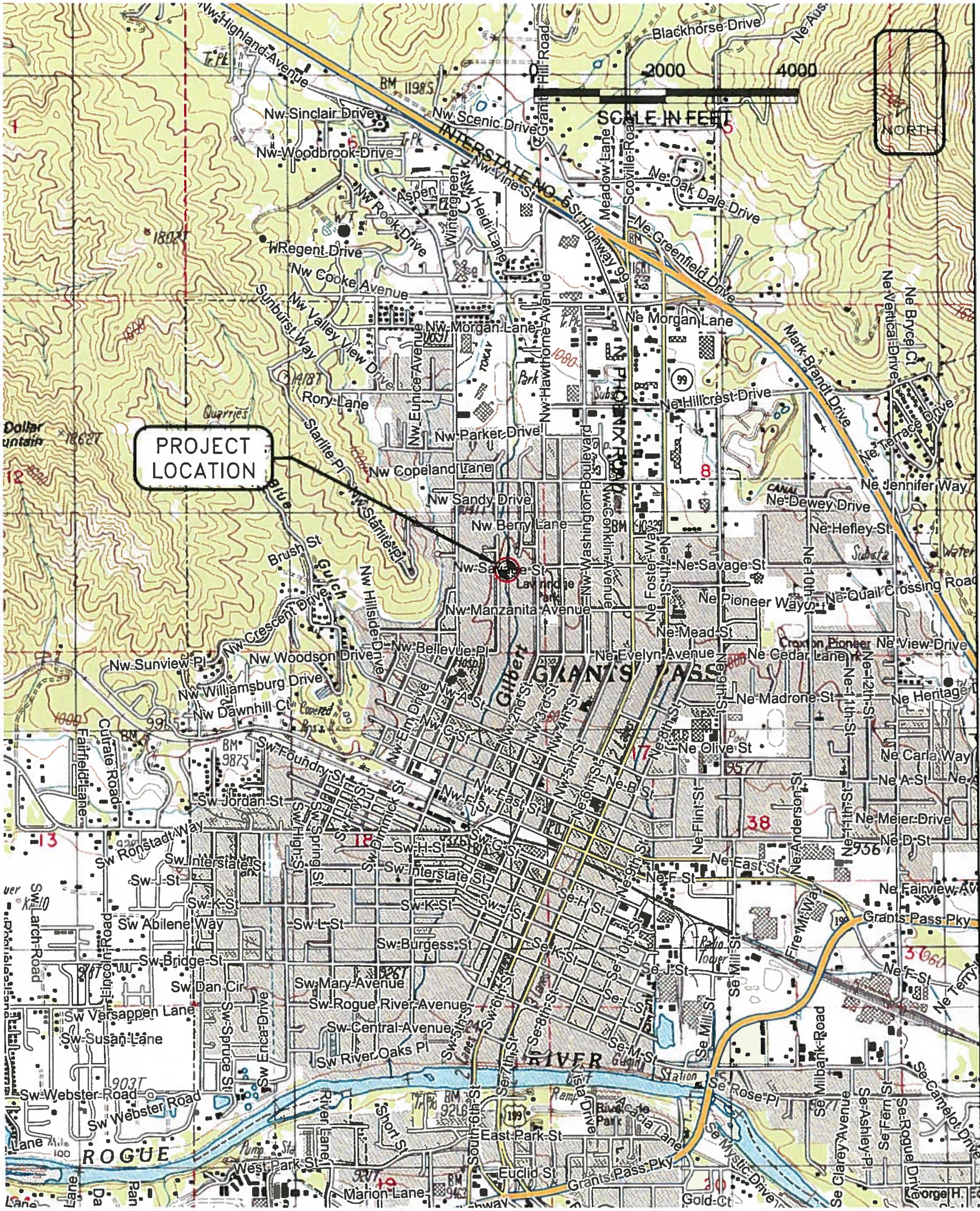
This report was prepared for the use of the owner and his design and construction team for the design and construction of the Savage Street Bridge project. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty 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.

THE GALLI GROUP
GEOTECHNICAL CONSULTING

William F. Galli, PE, GE
Senior Principal Engineer

REGISTERED PROFESSIONAL
ENGINEER
12162
OREGON
JULY 15, 1982
WILLIAM F. GALLI
EXPIRES: 06/15



THE GALLI GROUP
GEOTECHNICAL CONSULTING

612 NW 3rd Street
Grants Pass, OR 97526

VICINITY MAP

SAVAGE STREET BRIDGE
GRANTS PASS, OREGON

DATE:	JUNE 2014
JOB NO:	02-4952-01
REV:	6/10/2014 11:41 AM
PREPARED BY:	MG3
4952 Savage Street Bridge - 01 - Vicin.dwg	

FIGURE:
1

LEGEND

B-1 BORING NUMBER AND APPROXIMATE LOCATION

2004 LIDAR TOPOGRAPHY PROVIDED BY THE CITY OF GRANTS PASS; AERIAL IMAGE PROVIDED BY GOOGLE EARTH.

APPROX NORTH

0 50 100

SCALE IN FEET

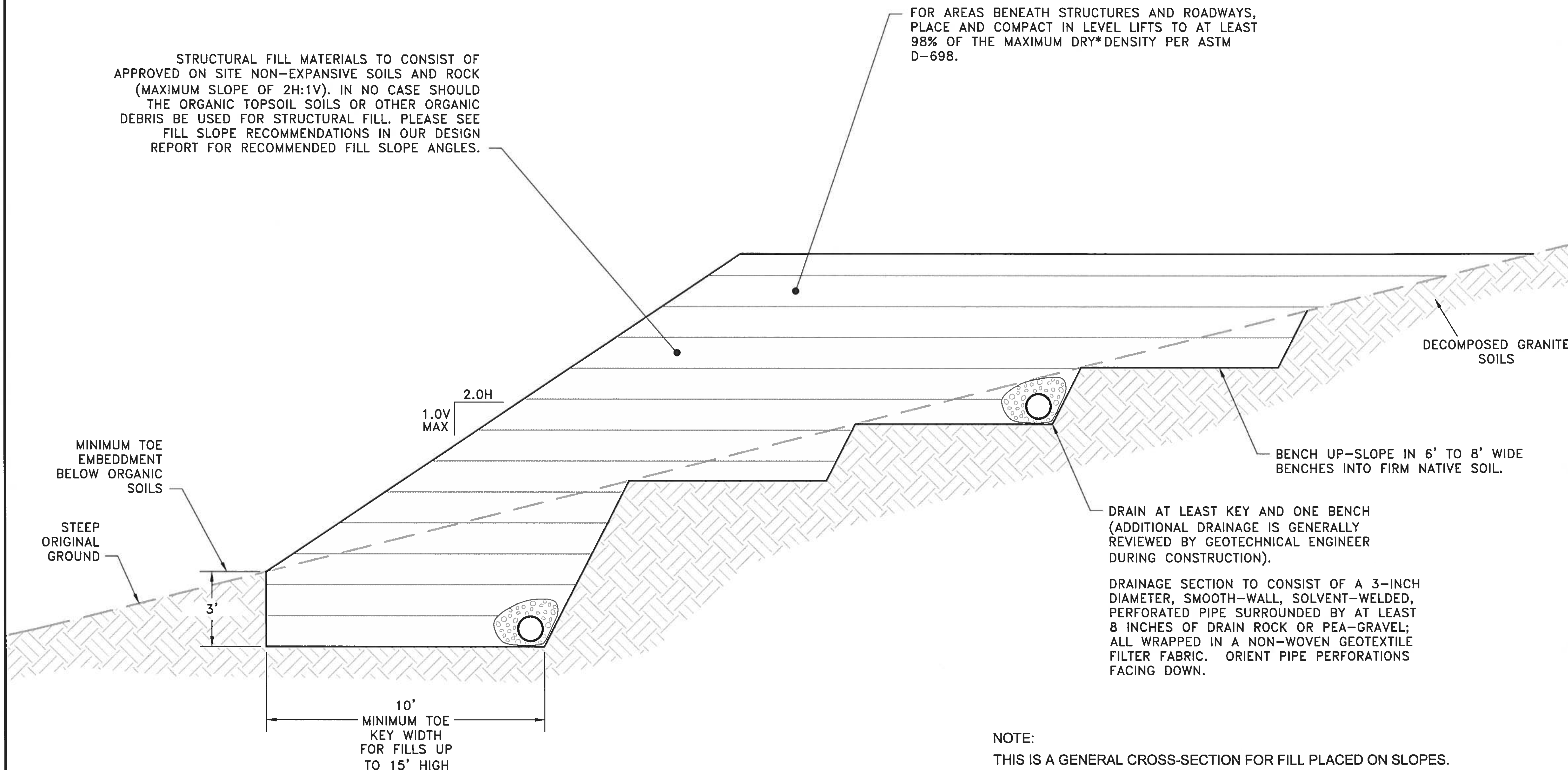
THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

SITE AERIAL & TOPOGRAPHY WITH BORING LOCATIONS

SAVAGE STREET BRIDGE
GRANTS PASS, OREGON

DATE:	JUNE 2014
JOB NO:	02-4952-01
REV:	6/10/2014 1:00 PM
PREPARED BY:	MG3
4952 Savage Street Bridge - 02 - Site Plan.dwg	

FIGURE: 2

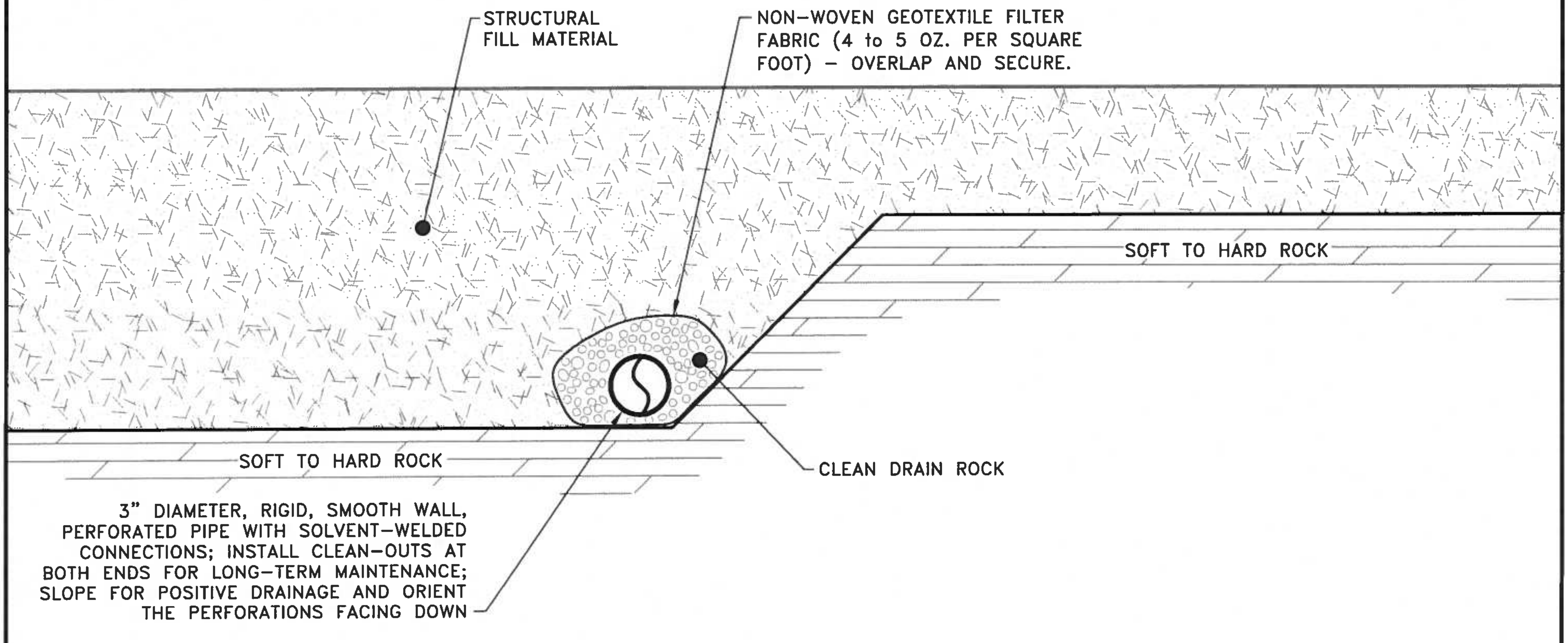


NOTE:

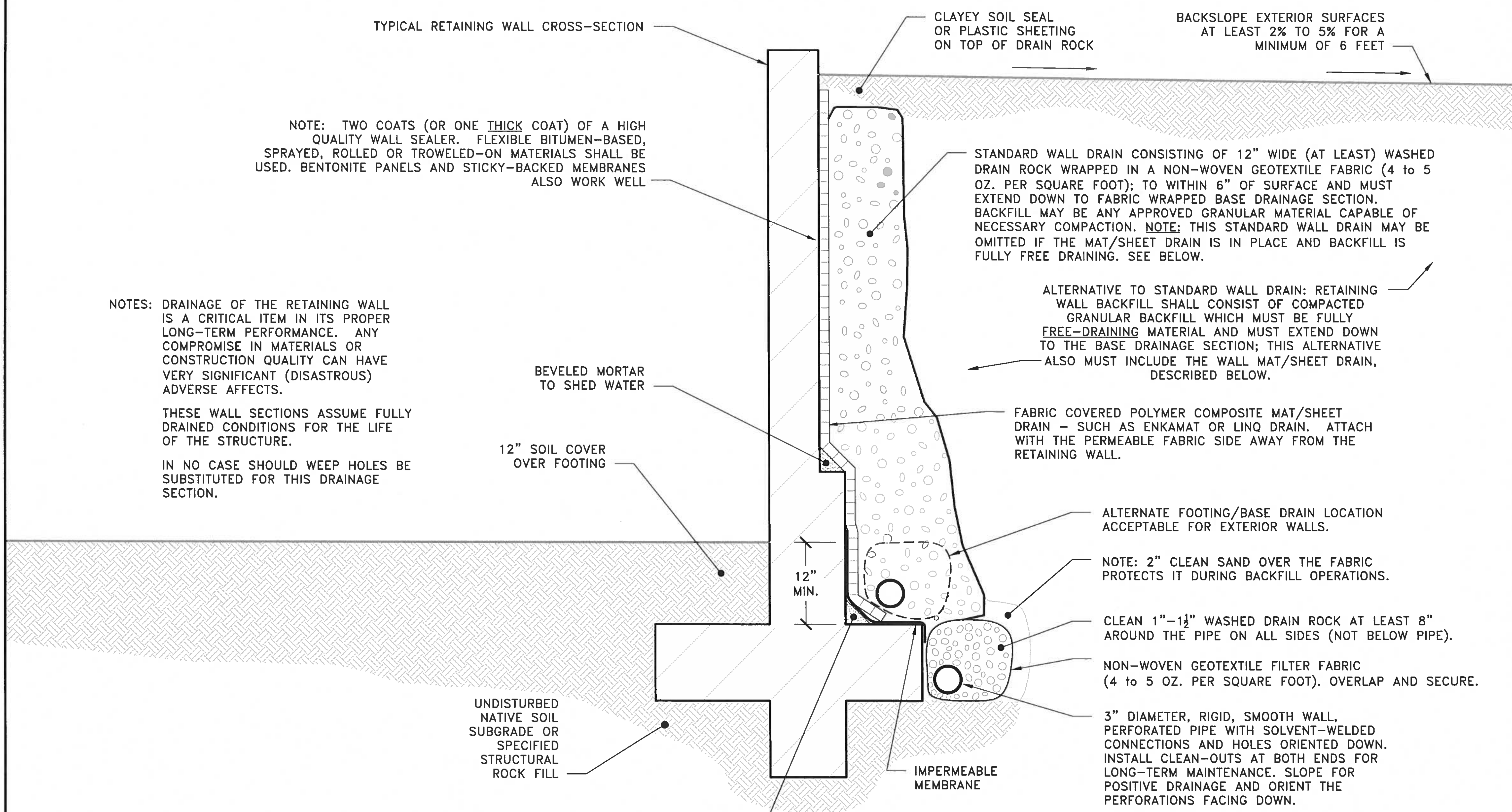
THIS IS A GENERAL CROSS-SECTION FOR FILL PLACED ON SLOPES.
IT IS NOT INTENDED AS A SPECIFIC DESIGN FOR THIS PROJECT.

FOR ILLUSTRATION PURPOSES ONLY
NOT TO SCALE

THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	FILL ON STEEP SLOPE CROSS-SECTION SAVAGE STREET BRIDGE GRANTS PASS, OREGON	DATE: JULY 2014 JOB NO: 02-4952-01 REV: 7/17/2014 2:41 PM PREPARED BY: MG3 4952 Savage Street Bridge - 03 - slope fill.dwg	FIGURE: 3



THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	BENCH DRAIN/SLOPE SUBDRAIN DETAIL SAVAGE STREET BRIDGE GRANTS PASS, OREGON	DATE: JULY 2014 JOB NO: 02-4952-01 REV: 7/17/2014 2:47 PM PREPARED BY: MG3 4952 Savage Street Bridge - 04 - subdrain-bench.dwg	FIGURE: 4



FOR ILLUSTRATION PURPOSES ONLY
NOT TO SCALE

THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

EXTERIOR RETAINING WALL/
ABUTMENT DRAINAGE CROSS-SECTION
SAVAGE STREET BRIDGE
GRANTS PASS, OREGON

DATE: JULY 2014
JOB NO: 02-4952-01
REV: 7/17/2014 2:52 PM
PREPARED BY: MG3
4952 Savage Street Bridge - 05 - retwall drain-ext-02.dwg

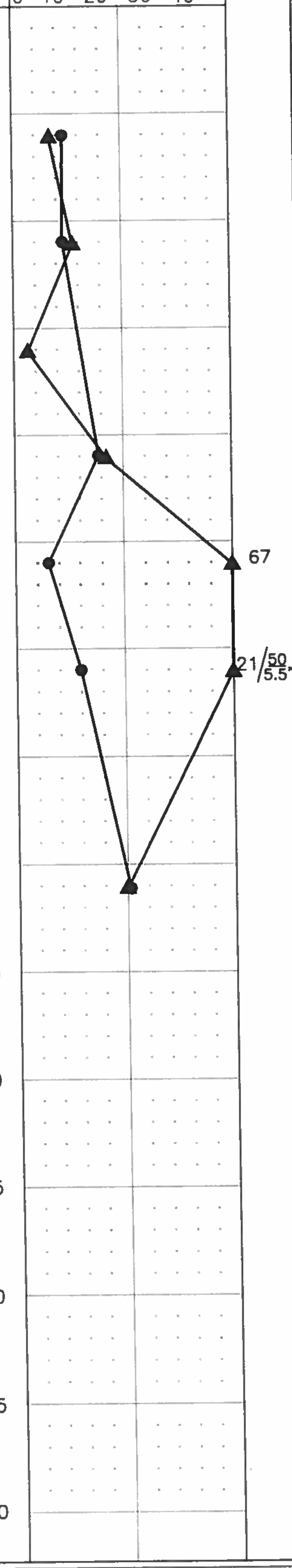
FIGURE:
5

APPENDIX A

BORING LOGS

USC	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEV. (FT.)	SAMPLES NO.	S.P.T.	GROUND WATER DEPTH IN FEET
	1.3	4" Asphaltic Concrete (A.C.), over 12" 3/4-inch minus Crushed Rock –approx.			
		Loose, tan to brown, silty SAND (D.G.); variable silts and sands, trace fine gravels, moist then saturated at 8' approx. (SM)	S-1	9	06/04/14 ▼
	9½	Medium dense, light brown and gray, silty, gravelly SAND; well graded sand, subrounded gravels vary somewhat, saturated. (SW-SP)	S-2	14	
	14	Loose, gray to blue-gray, silty fine SAND; silts and sands vary – layered, saturated. (SM-ML)	S-3	3*	
	21	Dense to very dense, gray, sandy GRAVELS & COBBLES; well graded sands, subrounded gravels, saturated [moderate scraping and chatter of drill rig]. (GP) .. driller noted heavy scraping & chatter from 28' to 33'. .. disturbed sample @ S-6 then had to redrill from 25' to 30', due to heaving sands.	S-4 S-5 S-6	21 67 21/50/5.5"	
	33	Medium dense, gray, brown & orange, silty SAND (granitic); saturated to very moist. (SM)	S-7	25	
	41½	Bottom of boring at 41½ feet. Perched (Gilbert Creek) groundwater level was measured at approximately 8 feet.			

STANDARD PENETRATION TEST
▲ BLOWS PER FOOT
0 10 20 30 40



LEGEND

- 2" S.P.T. SAMPLE
- GRAB SAMPLE
- * SAMPLE NOT RECOVERED
- WH WEIGHT OF HAMMER
- WR WEIGHT OF DRILLING RODS
- U 3" THIN WALL SAMPLE
- P PITCHER SAMPLE
- IMPERVIOUS SEAL
- ▼ WATER LEVEL
- PIEZOMETER TIP
- LIQUID LIMIT
- NATURAL WATER CONTENT
- PLASTIC LIMIT
- ● WATER CONTENT IN PERCENT
- USC UNIFIED SOIL CLASSIFICATION

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. GROUNDWATER LEVELS DO RISE AND FALL DUE TO THE SEASONS. GROUNDWATER LEVELS MAY RISE CLOSE TO GROUND SURFACE.

DRILLER LAWRENCE & ASSOCIATES
DATE START 06/04/14 FINISH 06/04/14
DRILLING TECHNIQUE 8"ø HSA

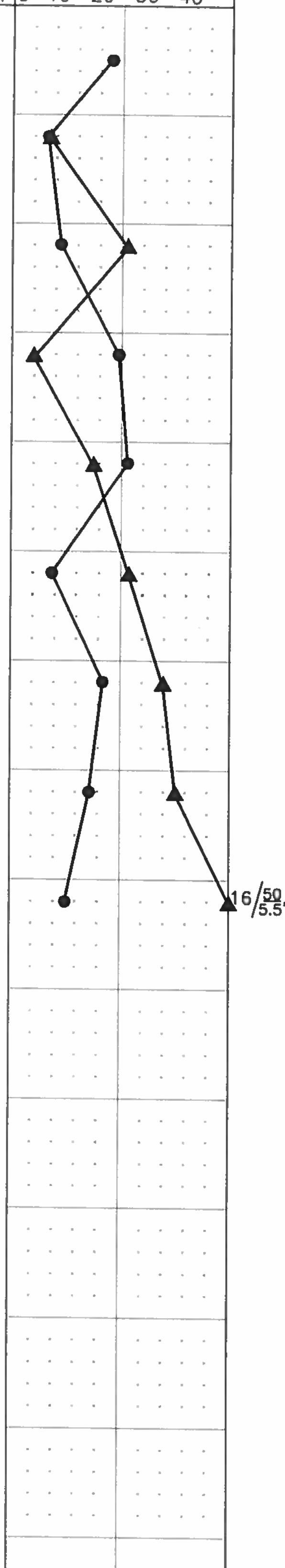
THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

SUMMARY BORING LOG
B-1
SAVAGE STREET BRIDGE
GRANTS PASS, OREGON

DATE JUNE 2014
JOB NO. 02-4952-01
FIG. A1

USC	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEV. (FT.)	SAMPLES NO.	S.P.T.	GROUND WATER DEPTH IN FEET
	1.3	4" Asphaltic Concrete (A.C.), over 12" 3/4-inch minus Crushed Rock	S-1		
	4	Soft, dark brown to black, fine sandy SILT; trace clay, very moist to wet. (ML)	S-2	9	5
		Loose, tan to brown, silty SAND (D.G.); variable silts and sands, very moist then saturated at 8½' to 9' approx. (SM)			06/04/14 ▼
	10½	Medium dense, light brown and gray, sandy GRAVELS; subrounded gravels, some cobbles, saturated.	S-3	26	10
	14½	Loose, brown to gray & blue-gray, silty fine SAND; silts and sands vary - layered, saturated. (SM-ML)	S-4	5	15
			S-5	19	20
	22	Medium dense, gray with some white, sandy GRAVELS & COBBLES; well graded sands and subrounded gravels, saturated [moderate scraping and chatter of drill rig]. (GP)	S-6	27	25
	28	Dense to very dense, blue-gray to brown-orange & white, silty SAND; silts and sands vary some, variable gravels: clean to numerous - some soft gravels (weathered, granitic), wet to very moist. (SM)	S-7	35	30
			S-8	38	35
			S-8	16/50/5.5"	40
	41½	Bottom of boring at 41½ feet. Perched (Gilbert Creek) groundwater level was measured at approximately 9 feet.			45
					50
					55
					60
					65
					70

STANDARD PENETRATION TEST
▲ BLOWS PER FOOT
0 10 20 30 40



LEGEND

- 2" S.P.T. SAMPLE
- GRAB SAMPLE
- * SAMPLE NOT RECOVERED
- WH WEIGHT OF HAMMER
- WR WEIGHT OF DRILLING RODS
- U 3" THIN WALL SAMPLE
- P PITCHER SAMPLE
- IMPERVIOUS SEAL
- ▼ WATER LEVEL
- PIEZOMETER TIP
- LIQUID LIMIT, NATURAL WATER CONTENT, PLASTIC LIMIT
- ● WATER CONTENT IN PERCENT
- USC UNIFIED SOIL CLASSIFICATION

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. GROUNDWATER LEVELS DO RISE AND FALL DUE TO THE SEASONS. GROUNDWATER LEVELS MAY RISE CLOSE TO GROUND SURFACE.

DRILLER LAWRENCE & ASSOCIATES
DATE START 06/04/14 FINISH 06/04/14
DRILLING TECHNIQUE 8"ø HSA

THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

SUMMARY BORING LOG
B-2

SAVAGE STREET BRIDGE
GRANTS PASS, OREGON

DATE JUNE 2014
JOB NO. 02-4952-01
FIG. A2