



**GEOTECHNICAL & GEOLOGICAL
INVESTIGATION REPORT
SEISMIC RETROFIT DESIGN
HOOVER ELEMENTARY SCHOOL
MEDFORD, OREGON**

For: Ron Havniar
Facilities Manager
Medford School District
815 S Oakdale Ave
Medford, OR 97501

By: **THE GALLI GROUP**
612 NW Third Street
Grants Pass, OR 97526
(541) 955-1611

02-5561-01
October 19, 2018

Table of Contents

<u>Section</u>	<u>Page</u>
1.0 INTRODUCTION	1
2.0 SITE AND PROJECT DESCRIPTION.....	1
3.0 FIELD EXPLORATION	1
4.0 LABORATORY TESTING	2
5.0 SUBSURFACE CONDITIONS	2
5.1 SOIL	2
5.2 GROUNDWATER.....	2
6.0 GEOLOGIC AND SEISMIC DESIGN	3
6.1 REGIONAL GEOLOGIC SETTING	3
6.2 SITE GEOLOGY	4
6.3 IBC AND 2014 OSSC DESIGN EARTHQUAKE.....	4
6.4 GEOLOGIC OR SEISMIC INDUCED HAZARDS	5
7.0 CONCLUSIONS.....	7
8.0 GEOTECHNICAL RECOMMENDATIONS	7
8.1 SITE PREPARATION AND GRADING.....	7
8.1.1 Manmade Fill & Debris Considerations	7
8.1.2 Clearing, Grubbing, Stripping and Site Excavation	8
8.2 STRUCTURAL FILL PLACEMENT AND COMPACTION	8
8.2.1 Beneath Structures	8
8.2.2 Non-Structural Fill.....	9
8.3 FOUNDATION SUPPORT	9
8.4 LATERAL LOAD RESISTANCE	10
8.4.1 Foundation Members	10
8.4.2 Buttress or Thrust Block.....	11
8.5 RETAINING WALLS	11
8.6 FOUNDATION, FLOOR AND WALL DRAINS	12
8.7 MATERIALS SPECIFICATIONS	14
9.0 ADDITIONAL SERVICES AND LIMITATIONS	16
9.1 ADDITIONAL SERVICES	16
9.2 LIMITATIONS	17

LIST OF FIGURES

Figure 1	Vicinity Map
Figure 2	Site Plan with Boring Locations
Figure 3	Typical Foundation Drain, Slab on Grade Floor
Figure 4	Floor Subdrain Detail
Figure 5	Exterior Retaining Wall Drainage

APPENDIX A: Boring Logs

APPENDIX B: Laboratory Test Results



GEOTECHNICAL & GEOLOGY INVESTIGATION REPORT SEISMIC RETROFIT DESIGN HOOVER ELEMENTARY SCHOOL MEDFORD, OREGON

1.0 INTRODUCTION

This report presents results of our geotechnical and geology evaluation of the site for the seismic retrofit to the Hoover Elementary School in Medford, Oregon. The subject property is located at 2323 Siskiyou Boulevard in the southeast portion of Medford, Oregon. This is a relatively flat area between Siskiyou Boulevard and the foothills to the north. Please see Figure 1, Vicinity Map, for a more precise site location.

The purpose of this investigation and report was to evaluate the site surface and subsurface conditions with a series of four (4) borings and conduct office studies in order to provide geotechnical and geologic recommendations for design and construction of the seismic retrofit. This may include improving the foundations and/or adding embedded footings/butresses for lateral and vertical resistance of loads generated in a seismic event.

2.0 SITE AND PROJECT DESCRIPTION

The site is currently occupied by a functioning elementary school, with a large multi-wing structure, sports fields, drive lanes and parking lots.

We understand the project to consist of accomplishing a Seismic Retrofit at the school in order to bring it up to current code requirements for public schools. This could require structural upgrades including improved foundations and/or embedded footings/butresses for resistance of lateral and vertical loads generated in a seismic event.

3.0 FIELD EXPLORATION

On October 9 and 12, 2018, Senior Technician, Blake Reichenbach and Associate Geologist, Dennis Duru, and our drilling crew, visited the site to accomplish the subsurface investigation. A total of four (4) exploratory borings were drilled to depths between 2.8 feet and 5.5 feet throughout the site at the locations shown on Figure 2, Site Plan. The drilling was accomplished with our ATV-mounted solid stem auger drill rig.

All borings encountered the clayey Sand and then dense sand or weathered Sandstone. All holes were refilled after drilling with spoil soils from drilling. The areas were left clean of most soil debris. Borings were advanced with sample collection and testing being accomplished at various depths.

Standard Penetration Testing (SPT) was accomplished in each boring. This entails driving a 1-1/2-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. These can be correlated with soil strength parameters from testing on thousands of other projects. This data can be used to develop design recommendations for this project.

Our representative identified the exploration locations, logged subsurface soils and water conditions and obtained soil samples for transport to our laboratory. 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. Please note that in the logs, soil changes are depicted as distinct layers, while in nature they may be more gradual.

4.0 LABORATORY TESTING

The soils encountered above the weathered sandstone were mostly clayey Sand that could be expansive. Therefore, Expansion Index (EI) testing and moisture content testing were accomplished. Results of the testing were EI=25, which means the soils encountered were only mildly expansive.

5.0 SUBSURFACE CONDITIONS

5.1 SOIL

The site areas investigated had very similar soils conditions. The borings encountered from 0.5 to 3.0 feet of clayey Sand. This was underlain by dense Sand and then dense weathered Sandstone. The upper soils were slightly expansive. All borings terminated in the dense weathered Sandstone. Please see more specific soils information in the Boring Logs in Appendix A. Please note that the soils are shown as distinct layers in the Boring Logs while in nature they may change more gradually. Soils conditions may also change somewhat between the locations investigated.

5.2 GROUNDWATER

Generally, the soils encountered were mostly moist. Seepage or standing water was not encountered. Due to the dense nature of the underlying weathered Sandstone and local geology it is likely perched water levels could rise to within 1 to 3 feet of the surface during the wetter months of most years. These site soils are likely to perform moderately well during mild wetter periods of the year. Heavy rains could cause them to become

disturbed and unworkable, unless excavations penetrate into the dense Sand or Sandstone. All water would be able to be removed by periodic sumps into the sandstone.

6.0 GEOLOGIC AND SEISMIC DESIGN

6.1 REGIONAL GEOLOGIC SETTING

The project is located in southwestern Oregon, within the central portion of the city of Medford. The site is on the northeast edge of the broad Bear Creek Valley, formed by Bear Creek, the Rogue River, and their smaller tributaries. Bear Creek Valley is bounded on the west by mountains within Oregon's Klamath Mountain Physiographic Province, and, immediately east of the valley, begins the foothills of the Cascade Volcanic Physiographic Province.

The Klamath Mountain Province consists of exotic terranes originating in island archipelago environments during the Paleozoic to Mesozoic Eras. The terranes were transported eastward by plate motions, where they were accreted as individual east-dipping lithologic units against the North American Plate. Accretion of the terranes began in middle to late Jurassic and ended by early Cretaceous Period. The province contains several northeast trending intrusive granitic belts which were typically intruded after accretion of the individual terranes. The Mount Ashland, Gold Hill, Jacksonville, and Grants Pass plutons are examples of the intrusive units. Seven individual terranes are identified in the Klamath Province, which covers approximately 12,000 square miles in northern California and southern Oregon (Orr and Orr, 2012).

The Hayfork subterrane occurs in the western portion of the Bear Creek Valley area, and is comprised of volcanoclastic arc rocks (cherts, argillites, limestone, and meta-andesite). The Hornbrook Formation, Cretaceous marine sedimentary rocks ranging from sandstone, siltstone, mudstone, and conglomerate, overlies the Hayfork subterrane, and forms the "bedrock" unit for the project area. The Hornbrook Formation was deposited in an extensive seaway which transgressed into what is now southwestern Oregon in early to middle Cretaceous time (approximately 100 to 75 Ma). The Payne Cliff formation, upper Eocene fluvial braided-stream deposits of sandstone, conglomerate, mudstone and minor coal deposits, overlies the Hornbrook Formation, and begins approximately 1.5 miles to the east of the project (Wiley and Smith, 1993; Wiley et al, 2011).

The Western Cascade sub-province of Oregon's Cascade Physiographic Province begins in the foothills just east of the project site. Deposition of the Western Cascade volcanic units in this region began in early Oligocene (approximately 36 million years ago) and ended in early to middle Miocene (approximately 25 million years ago), (Wiley and Smith, 1993). The Western Cascades are faulted and mildly folded and have a regional dip of 10-15 degrees to the east. Softer volcanic units are highly dissected and drainages are well established along structure and the more easily eroded geologic units.

Four stages of Quaternary alluvial fans and valley fill have been mapped in the Bear Creek Valley area, and several of these units form the surficial deposits immediately west of the project location (Wiley and Smith, 1993; Parsons and Herriman; 1976). Those Quaternary alluvial units pertinent to the project area are shown on the Geologic Map (Figure 3).

Oregon's Klamath Mountain Province experienced regional uplift and faulting into the Tertiary Period. Faults are observed to offset formations as young as late Miocene in the Rogue Valley area (Wiley and Smith, 1993). No Quaternary fault activity, however, has been established for the immediate project area within the Rogue Valley (Walker and MacLeod, 1991; Wiley and Smith, 1993; Madin and Mabey, 1996; Wiley et al, 2011; USGS; 2014a).

6.2 SITE GEOLOGY

Mapped geologic units at the project area consist primarily of loose clayey Sand, of the Terrace Deposits Formation (OGDC-6, 2015) over the dense Hornbrook Sandstone.

A total of four auger borings were completed on the project site. These borings ranged in depth from 2.8 feet to 5.5 feet below ground surface (bgs). All the borings encountered the clayey Sand. All borings terminated in the dense weathered Sandstone.

Seasonal perched zones of groundwater, with relatively small yields, may occur throughout the soil unit, and at the interface between overlying loose to medium dense soil and the dense weathered Sandstone. The potential for such seepage will be addressed in the geotechnical report with regard to subsurface drainage design for the proposed project.

6.3 IBC AND 2014 OSSC DESIGN EARTHQUAKE

The design earthquake for the project area is based upon established values and methodologies in the International Building Code (IBC; 2012), ASCE 07-10, and Oregon Structural Specialty Code (OSSC; 2014). The OSSC adopted the IBC 2012 on July 1, 2014, and an online version of OSCC 2014 is now available. Seismic design information referenced in this report is from the 2012 IBC and ASCE 07-10.

The Maximum Considered Earthquake (MCE_R) and spectral response accelerations, as set forth in Section 1613 (IBC, 2012) and Section 11.4 (ASCE 7-10), were obtained from the online USGS Seismic Design Maps (USGS, 2014b). They are summarized in Table 1.

Table 1- DESIGN EARTHQUAKE (IBC, 2012; ASCE 7-10; OSSC, 2014)

Parameter	Value
Project Latitude/ Longitude Hoover Elementary School	Lat. 42.319986 Long. -122.838169-
Occupancy/Risk Category (Table 1.5-1 ASCE/SEI 7-10)	III (elementary school) Risk Category III
Mapped Spectral Response Acceleration (MCE_R) - Short Period (S_S)	0.609g
Mapped Spectral Response Acceleration (MCE_R) - 1-Second Period (S_1)	0.325g
Site Class - (Table 20-3-1 ASCE/SEI 7-10)	C
Short Period Site Coefficient based on Site Class - (F_a)	1.156
1-Second Site Coefficient based on Site Class - (F_v)	1.475
MCE_R Spectral Response Acceleration - (S_{MS})	$S_{MS} = F_a * S_S = 0.704g$
MCE_R Spectral Response Acceleration for 1-Second - (S_{M1})	$S_{M1} = F_v * S_1 = 0.480g$
Design Spectral Response Acceleration for Short Periods - (S_{DS})	$S_{DS} = 2/3 S_{MS} = 0.469g$
Design Spectral Response Acceleration for 1-Second - (S_{D1})	$S_{D1} = 2/3 S_{M1} = 0.320g$
PGA= MCE_G (Section 11.8.3.2; and Figures 22-7; ASCE/SEI 7-10)	PGA= 0.284g
F_{PGA} (Table 11.8-1 ASCE/SEI 7-10)	1.116
$PGA_M = F_{pga} * PGA$ (EQ 11.8-1; ASCE/SEI 7-10)	= 1.116*0.284= 0.317g
Design PGA ($PGA_M * 2/3$)	$PGA_D = 0.317 * 2/3 = 0.211g$
Seismic Design Category (Section 11.6 and Table 11.6-1 and Table 11.6-2; ASCE/SEI 7-10)	D

6.4 GEOLOGIC OR SEISMIC INDUCED HAZARDS

Expansive Soil. The near surface top soil (~2 feet) during auger boring encountered was clayey Sand. This soil is only mildly expansive. Expansion Index testing had the result of EI=25. Therefore, based on the borings and because of all foundation support will be below 3 feet, expansive clay will not be an issue in this site. All of the thin surficial soil, where clay is present, will likely be removed during site preparation. Site preparation and footing subgrade preparation are discussed in the geotechnical report. Together with the general underlying dense sand and weathered rock conditions at the site, this will mitigate any possible expansive soil conditions.

Landslides/Slope Instability. The site is relatively flat. No landslides were observed in the immediate project area during our field reconnaissance or upon review of aerial photos (JCGIS, 2005; Google Earth Pro, 2017). No slope instability that might impact the project was noted on the Oregon landslide inventory database (SLIDO;2011), or on LIDAR imagery of the project area available at on-line mapping (DOGAMI, 2017). The site is not in a mapped hazard zone for rapidly moving landslides (Hofmeister and others; 2002). Therefore, damage to the site due to landslides, mudflows and rockfall is considered zero.

Flooding. The project is not within the 100-year flood zone as mapped on the Jackson County FEMA Special flood Hazard Area (SFHA), effective May 3, 2011 (JC; 2014).

Liquefaction. A general screening of liquefaction hazard includes evaluation of the following: historic occurrence of liquefaction; seismic source potential to cause liquefaction; depth to the water table; and geologic age and composition of subsurface material, including density of material.

The project area is subject to seismic shaking from local Basin and Range faults as well as the Cascadia Subduction Zone megathrust. A seismic source potential is certainly present.

No historic or paleoseismic evidence was found in a literature search indicating the occurrence of liquefaction in the alluvial units within and adjacent to stream channels in the Rogue Valley. Similarly, no paleoseismic data is available to indicate liquefaction has occurred in the project area.

When thin, surficial soil does overly the bedrock, these soils were determined from auger drilling and SPT sampling to be medium dense clayey Sand. This is underlain by dense rock at very shallow depth. No general water table was noted in the surficial soil units during project drilling. Groundwater elevation can be expected to vary seasonally, and may be present as isolated, perched zones within the thin soil units.

It can be concluded from the above discussion that no liquefaction hazard exists in the thin surficial soil units at the project site.

Seismic Ground Amplification or Resonance. No unusually hazardous amplification or resonance effects of seismic waves have been associated with soil/bedrock subsurface conditions in the project area.

The peak horizontal acceleration (PGA_M), established from EQ 11.8-1 of ASCE/SEI 7-10, is 0.353g. This value is adjusted for the Site Class C designation determined for the project from subsurface drilling and evaluation of SPT data. The PGA design value can be used with an appropriate seismic coefficient in pseudostatic analysis for design of any fills being placed at the project. Such design criteria should properly compensate for ground shaking.

Tsunami/Seiche Hazard. The project is located nearly 80 miles inland and is not subject to tsunami hazard. The project site is not located adjacent to any large lake or body of water. Therefore, no seismically induced seiche hazard exists for the project. No large reservoirs are located in a drainage area upslope from the project site. Therefore, the project site is not subject to hazard from seismically induced reservoir failure.

Surface Rupture. No active fault traces or local faults are mapped within the project site (Walker and MacLeod, 1991; Wiley and Smith, 1993; Madin and Mabey, 1996; Wiley et al, 2011; USGS; 2014). The risk of surface rupture is considered to be very low at the project.

7.0 CONCLUSIONS

In our professional opinion, based on our field investigation, and office review, and previous work in the area, the soils conditions at the site are suitable for the proposed seismic retrofit, provided the recommendations of our report are incorporated in the design and construction of the project. The dense sand and weathered Sandstone will provide good foundation and buttress support.

8.0 GEOTECHNICAL RECOMMENDATIONS

The subject site has good soils for support of the new structure footings or buttresses. The following sections provide methods and data for proper design of the seismic retrofit items.

8.1 SITE PREPARATION AND GRADING

This site has structures that may require structural and foundation upgrades for the seismic upgrade. Therefore, normal methods of debris removal, clearing, grubbing, stripping for organic removal and subgrade soil preparation will apply in areas of the work.

8.1.1 Manmade Fill & Debris Considerations

All old fill and debris encountered during construction must be removed. Soil that is clear of debris could be used in landscape berms. All other debris or debris laden soil must be wasted off site or used in landscape berms. The full extent of any waste fill removal (if any) will be determined during site stripping operations. This would include demolition debris from areas where portions of existing foundations may have to be removed.

8.1.2 Clearing, Grubbing, Stripping and Site Excavation

All areas for new footings and/or lateral buttresses shall be founded on, or embedded into, the dense Sand and/or dense weathered Sandstone. Therefore, full removal of all overlying clayey Sand soils down to the dense Sand or Sandstone, which includes all overlying organic soils, vegetation and root zones, will be required.

8.2 STRUCTURAL FILL PLACEMENT AND COMPACTION

8.2.1 Beneath Structures

Structural fill is defined as any fill placed and compacted to specified densities and used in areas that will be under foundations, floors or structure support items. It appears that new footings and buttresses could have structural fill below, beside and above them. The subgrade needs to be prepared properly and the fill must be placed and compacted correctly for proper long-term performance.

Structural Fill Materials. Ideally, and particularly for wet weather construction, structural fill must consist of a free-draining granular material (non-expansive) with a maximum particle size of six inches. The material shall be reasonably well-graded with less than 5 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" (low-grade rock and good quality Sandy Decomposed Granite [DG]) have performed adequately for most applications of structural fill. See Section 8.7 for import fill specifications (aggregate base, aggregate subbase, sandy Gravels and Decomposed Granite).

Structural Fill Placement. All structural fill should be placed in horizontal lifts not exceeding 8 inches loose thickness (less, if necessary to obtain proper compaction) for heavy compaction equipment and four inches for light and hand-operated equipment. Each lift must be compacted by mechanical means to a minimum of 98 percent of the maximum dry density, as determined by ASTM Test Method D-698 (Standard Proctor).

Beneath Footings. Structural fill placed beneath footings or other structural elements must extend beyond all sides of such elements a distance equal to at least 1/2 of the total depth of the structural fill beneath the structural element in question 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 footing).

To facilitate the earthwork and compaction process, the earthwork contractor shall 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 smaller. Larger (2-1/2" or above) jaw-run "shale", crushed rock or larger broken decomposed granite (DG) do not yield consistent results with this type of testing. The high percentage of rock particles greater than 3/4'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 and proofrolling of lifts, instead of nuclear density testing.

Observation of Fill Placement. For these larger rock materials, we recommend the 8-inch lift be compacted by a minimum of 2 passes of a large vibratory hoepak on a moderately large excavator. The placement and compaction must be observed by our representative. After compaction, as specified above, is completed the entire area shall be proofrolled with a loaded dump truck to verify density has been achieved if the area is large enough for the vehicle. *All areas which exhibit movement or compression of the rock material more than 1/4 inch, under proofrolling, must be reworked or removed and replaced as specified above.*

Nuclear Density Testing of Fill. Field density testing by nuclear density gage would be adequate for verifying compaction of 2-inch to 3/4-inch minus crushed base rock, expansive clay and silt soils, Decomposed Granite and other materials 2 inches or smaller in size. Therefore, typical % compaction specifications would suffice. Testing must be accomplished in a systematic manner on all lifts as they are placed. Testing only the upper lifts is not adequate.

8.2.2 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 2.5H:1.0V or flatter. It must not be placed under footings or other structural members. It is recommended that when these soils are used they be given a moderate level of compaction (92 to 94 percent) to help seal them from surface water.

8.3 FOUNDATION SUPPORT

During the site investigation, borings encountered clayey Sand and then dense sand and weathered rock. The upper soils must be removed down to dense sand or weathered sandstone. Therefore, foundations must be founded on a layer of compacted structural fill which extends down to these dense materials.

Footings on Compacted Crushed Rock

1. Footing excavations shall penetrate down to the dense sand or weathered Sandstone (2.5 feet to 4.75 feet).
2. All areas over-excavated beneath footings must extend beyond all edges of the footing a distance equal to at least 1/2 the depth of the over-excavation (i.e., 3-foot deep over-ex requires the width to be 1 1/2 feet wider on each side of the footing).
3. Redensify the footing subgrade disturbed during excavation.
4. Backfill with at least 6 inches of compacted crushed rock structural fill compacted to at least 98% of ASTM D-698. Backfill with structural fill up to base of footing elevation if needed.
5. Footings placed on the crushed rock as listed above may be designed for a bearing pressure of 3,000 pounds per square foot. This may be increased by 33% for transitory wind and seismic loading.
6. The base of spread footings shall be buried a minimum of 12 inches below finish exterior grade in order to provide lateral support and frost protection.
7. We recommend minimum lateral dimensions of 16 inches for continuous load bearing footings and 24 inches for isolated piers constructed in this manner.

Anticipated Settlements. For properly constructed foundations founded on redensified and structural fill covered native soils, we anticipate maximum total and differential settlement to be less than approximately 1/2-inch and 1/4-inch, respectively.

Foundation Drains. We typically recommend all footings be installed with a footing drain to intercept groundwater seepage. Footing drains consisting of a rigid, smooth-wall perforated pipe surrounded by drain rock (sides and above), all wrapped in a non-woven geotextile fabric and should be placed adjacent to the footings. This is addressed more fully later in this report (Section 8.6). Please see Figure 3.

8.4 LATERAL LOAD RESISTANCE

8.4.1 Foundation Members

Lateral loads exerted upon these members can be resisted by passive pressure acting on buried portions of the foundations, retaining walls and other buried structures and by friction between the bottom of structural elements 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 native soils.

- Native Clayey Sand (below 12") 200 pcf
- Dense Sand/Sandstone 400 pcf
- Dense Compacted Crushed Rock (4' wide minimum) 450 pcf

A coefficient of friction of 0.55 can be used for elements poured neat against angular crushed rock structural fill. These should be reduced to 0.20 for areas over a vapor barrier or 0.45 over native sand/sandstone.

8.4.2 Buttress or Thrust Block

Concrete buttresses or thrust blocks may also be used for lateral resistance. These consist of an embedded concrete deadman block of reinforced concrete that can resist loads in all directions. These can be designed utilizing the lateral bearing capacities provided below.

Lateral Bearing Capacity	
Depth	Capacity (ksf)
1 to > 2 feet (Clayey)	Not Recommended
2 to > 4 feet (Clayey)	0.6 up to 6 kips
2 to > 4 feet (Sand)	1.0 to 15 kips
2 to > 4 feet (Sandstone)	2.0 to 20 kips
4 to > 7 feet (Sandstone)	3.0 up to 30 kips

- (1) These must be poured “neat” against the dense clean native excavation sidewall.
- (2) These may also be formed and poured and then backfilled around with well compacted crushed rock or sand/cement slurry.

8.5 RETAINING 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 and grade separation retaining walls. The following recommendations are provided for design and construction of conventional reinforced concrete or CMU block retaining walls:

- We recommend walls which are free to rotate at the top (unrestrained) when backfilled, be designed for the following loads.

Native Soil Fill (Sand or Sandstone) EFP	45 pcf
Low Grade Angular Rock EFP	35 pcf
Crushed Rock EFP	30 pcf
Seismic (up to 8 feet tall)	0.24g
- Walls that are fixed at the top (restrained) when backfilled should be designed for the following loads.

Native Soils Fill EFP	60 pcf
Low Grade Angular Rock EFP	45 pcf
Crushed Rock EFP	40 pcf
Seismic (up to 8 feet tall)	0.24g
- The walls all must have full drainage as described in section 8.6 and as shown on Figure 5.

- These equivalent fluid pressures are to be used for the soil through which the anticipated failure plane will develop (assume envelope beginning 2 feet behind base of wall and rising up and away from wall at 60 degrees off the horizon).
- A wet soil unit weight of 135 pcf should be used for design of retaining walls which are backfilled with crushed rock or jaw-run “shale”. Use 130 pcf for native soil backfill.
- These values are for properly compacted, free draining walls. Imported crushed rock or clean jaw-run “shale” work well for wall backfill materials.
- These design values assume the wall or structure is fully drained, has a flat backfill 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.
- We recommend designing retaining walls to resist seismic loading. A horizontal acceleration component of at least 0.24g shall 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 0.8H, where H is the height of the wall. This relates to an equivalent uniform load over the entire back of the wall of approximately 14 pounds per square foot for each foot of backfill, for walls up to 8 feet tall (i.e. for an 8-foot wall, fully backfilled, uniform seismic load will be on the order of 110 psf over the entire back of the wall).
- The backfill must be placed in lifts at near the optimum moisture content (clayey soils at 2% to 3% above optimum) 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 must 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.

8.6 FOUNDATION, FLOOR AND WALL DRAINS

All exterior foundations and embedded structures should have proper drainage.

Footing Drains. Foundation drainage should consist of a rigid smooth wall perforated pipe surrounded by at least 8 inches of drain rock on top and sides, all wrapped in a non-woven geotextile designed as a filter fabric (such as Mirafi 140N or equivalent). The

perforated pipe should be located on the footing next to the stem wall (or beside the footing), provided this is at least 12 inches below underslab drain rock. Please see Figure 3.

Floor Drains. Where the drain rock layer below slabs will be lower than the adjacent exterior grades, water will tend to accumulate in this low area. One method to drain this water is to include a series of subdrains at the bottom of the drain rock layer beneath the slab. The drain rock section shall be thickened to at least 8-inches for such lower areas. The subdrain lines typically consist of 3-inch diameter, smooth interior, solid wall, perforated pipe at spacing of 15 feet (or less) across the structure (and around the interior perimeter). The perforated pipe is placed in a deepened zone of the drain layer as shown on Figure 4. The pipes are sloped to drain and collected by a tightline which leads to the stormwater disposal system. We recommend we be allowed to review the subdrain system design prior to final plan submittal or construction bidding.

Retaining Wall Drainage. Wall drains must also 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 12 inches of the surface. A preformed, fabric-wrapped, polymer sheet drain, such as Amerdrain, Linq Drain or Enkamat must be placed against the wall. Exterior wall drains, which will not be sealed on top by asphalt or concrete, shall 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.

Walls that shall not pass water vapor must be fully sealed (with a bitumen-based sealer that will not harden or crack) before the sheet drain is attached. Wall seal such as MasterBlend HLM5000 or equivalent, should be used and applied per the manufacturer's recommendations.

All drains shall be tightlined and positively sloped to an approved stormwater disposal location into the public storm drain system or detention pond. **Note:** In no case shall water be collected and/or directed or discharged close to the foundations. Such improper water discharge can cause added water related problems.

We strongly recommend against connecting roof drains or surface area drains to wall, foundation or floor subdrains unless to a discharge line away from the structure. Foundation drains should consist of rigid smooth-wall perforated pipe. The rigid smooth-wall pipe can be cleaned out by means of a "roto-rooter" type system should it become plugged with sediment or fine roots. We recommend cleanouts be placed periodically by the designer to facilitate cleaning and maintenance of the drains.

8.7 MATERIALS SPECIFICATIONS

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

Aggregate Base Rock (Acceptable for Structural Fill)

- Angular Crushed Rock (3/4 or 1" Minus); R=85 or greater; Well Graded (No Gaps and at least 60% retained on the No. 4 sieve).
- 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 $\leq 5\%$.
- Compacted to 98% of the maximum dry density as determined by ASTM D698 or AASHTO T-99.

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.
- 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 $\leq 10\%$ Total; $\leq 3\%$ Clay Size.
- During wet weather; passing No. 200 sieve $\leq 5\%$.
- Compacted to 98% 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).

Embankment Fill (Acceptable for Structural Fill During Dry Weather)

- Reasonably well graded (not open work).
- Has at least 60% retained on the No. 4 sieve.
- Has no more than 30% passing No. 200 sieve.
- Passing No. 200 sieve must have less than 20% clay size.

On-Site Soil Fill (Acceptable as Specified in Report)

- Dense Sand, Pulverized Sandstone.
- Place when weather allows proper compaction.

Other Acceptable Fill

- Sandy Decomposed Granite; avoid orange or pink clayey material; $< 10\%$ passing No. 200 sieve and $> 50\%$ retained on No. 10 sieve.

Clean Sand

- Clean washed sand or sand and gravel, less than 1% passing No. 200.
- Gravel to be rounded or subrounded (no fracture faces), 1" or less.
- Must have less than 30% gravel by weight.

Note: Some fill materials will be difficult to nearly impossible to compact during wet weather. *The contractor must select the type of structural fill that will be able to be placed and compacted to specified conditions during the weather conditions that can take place during the construction schedule.*

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, 1" to 2" clean angular rock are all acceptable.
- Clean means washed rock with NO coating of silt, clay or sand.

Note: All types may be used in all applications of drain rock that are not beneath Asphaltic Concrete paved areas. In all AC areas angular clean drain rock must be used for AC support.

Note: Drainage layer drain rock that is beneath the floor slab must be the angular clean drain rock.

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.
- Overlap all edges at least 24 inches (12" for drainage section envelope).
- Secure in place such that overlaps will not move during covering operation.

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 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", 4" or 6" 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.
- Align perforations in the downward direction.
- Must always be placed within filter fabric wrap unless specifically specified otherwise.
- Protect from construction traffic until buried at least 2 times pipe diameter (minimum 8 inches) of angular rock fill.

Wall Sheet Drain

- Polymer sheet drain with filter fabric attached 1 or 2 sides, designed for drainage of vertical embedded foundation or retaining walls.
- For walls up to 10 feet tall. Must meet specifications as for American Wick Drain's AMERDRAIN 200 or 220.
- Install and splice and patch per manufacturer's recommendations.
- Install with fabric side towards the backfill.
- Attach to wall per manufacturer's recommendations.
- Extend down wall all the way to bottom of drainage section around perforated pipe.
- Protect from damage when backfilling with crushed rock larger than 2-inch minus.
- Repair all damaged areas prior to final backfill.

These materials shall be used on this project as specified in this report and on project plans or specifications.

NOTE: DEVIATIONS FROM SPECIFIED MATERIALS MUST BE APPROVED IN WRITING BY THE GEOTECHNICAL ENGINEER, OWNER AND OWNER'S OTHER CONSULTANTS/DESIGN ENGINEERS PRIOR TO USE AT THE SITE.

9.0 ADDITIONAL SERVICES AND LIMITATIONS**9.1 ADDITIONAL SERVICES**

We should review construction plans and specifications for this project as they are being 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. Additionally, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, all construction operations dealing with earthwork, foundations and rock placement and compaction 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 excavations to verify stability and subgrade density are acceptable.
- Observation and/or testing of subgrade preparation, subgrade proofrolling, structural fill placement and compaction, subdrains, footing subgrade, site grading, footing, wall and floor 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 for geotechnical related issues in the constructed product that we have not verified.

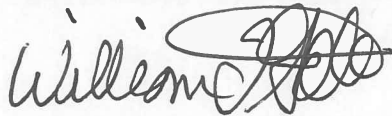
9.2 LIMITATIONS

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.

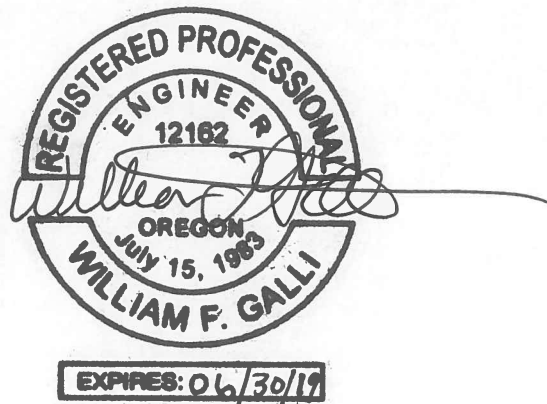
This report was prepared for the use of the School District and their design and construction team for the design and construction of the project. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

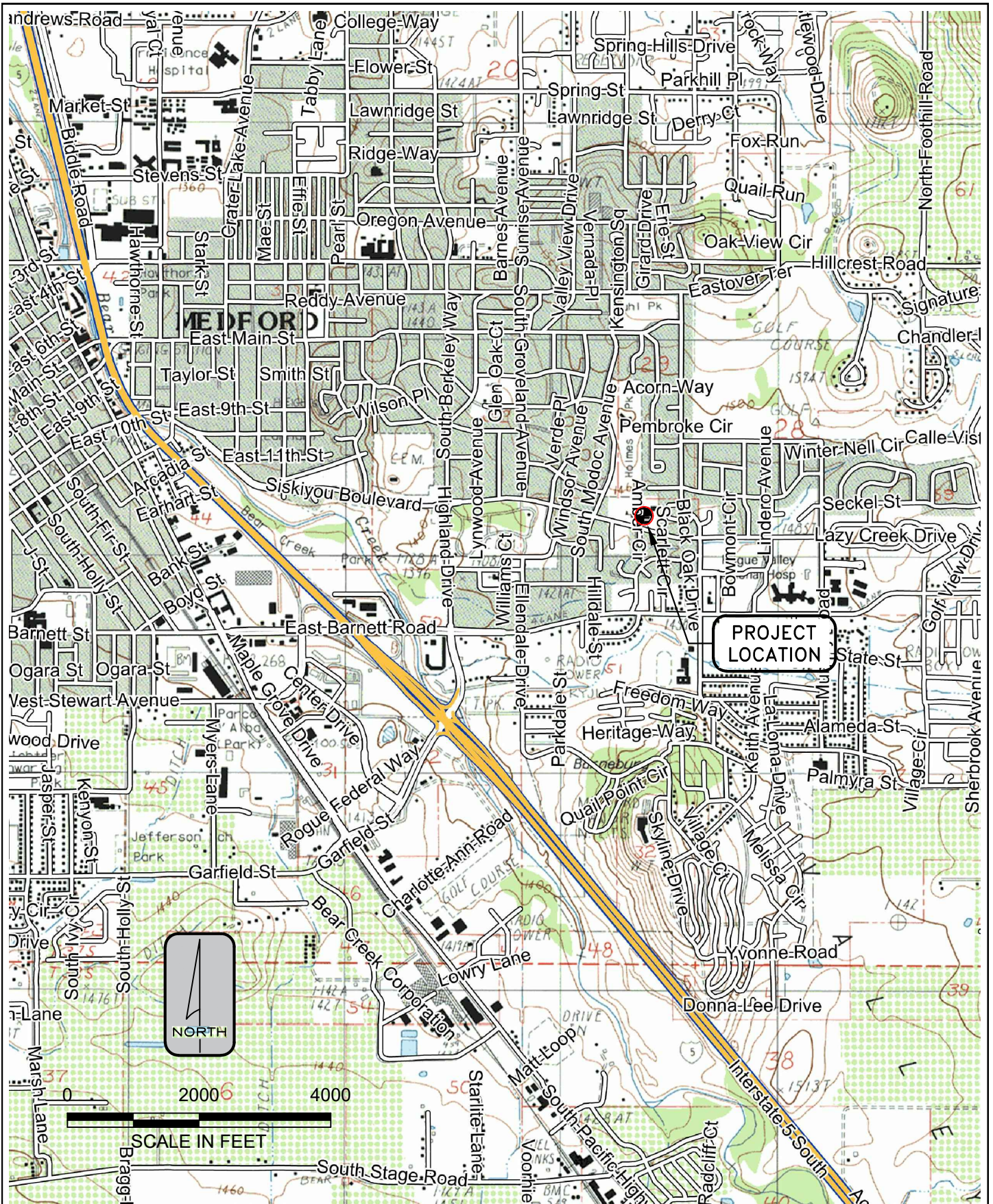
We have performed these services in accordance with generally accepted geotechnical engineering practices in southern Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

THE GALLI GROUP GEOTECHNICAL CONSULTING



William F. Galli, P.E., G.E.
Principal Engineer





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

VICINITY MAP

HOOVER ELEMENTARY SCHOOL
 MEDFORD, OREGON

DATE: OCTOBER 2018
 JOB NO: 02-5561-01
 REV: 10/04/2018
 PREPARED BY: MG3
 5561 Hoover Elementary - 01 - Vicinity.dwg

FIGURE:
1

LEGEND

B-1 BORING NUMBER AND APPROXIMATE LOCATION

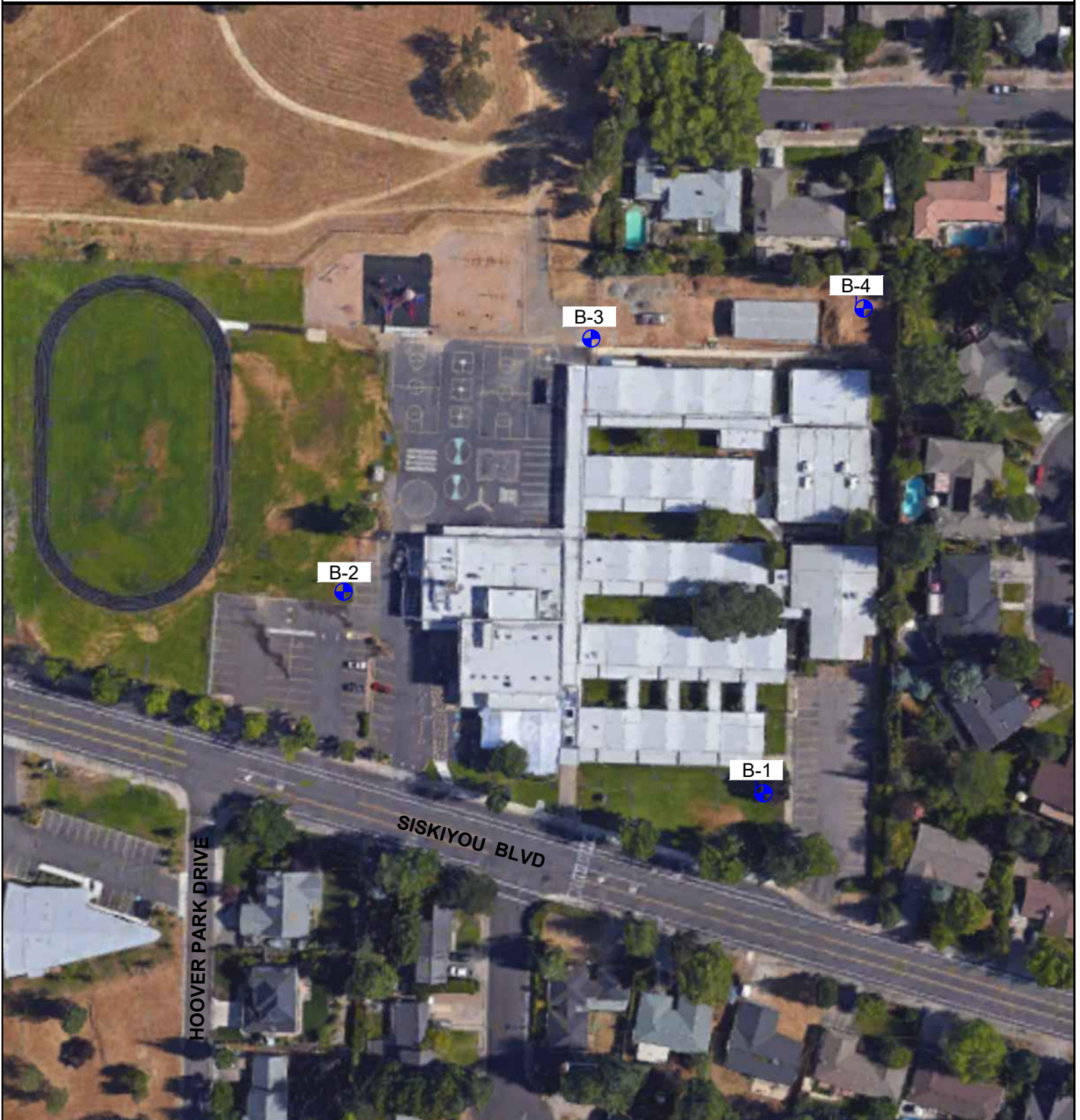


0 120 240



SCALE IN FEET

SITE AERIAL PROVIDED BY GOOGLE EARTH



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

SITE PLAN WITH BORING LOCATIONS

HOOVER ELEMENTARY SCHOOL
MEDFORD, OREGON

DATE: OCTOBER 2018

JOB NO: 02-5561-01

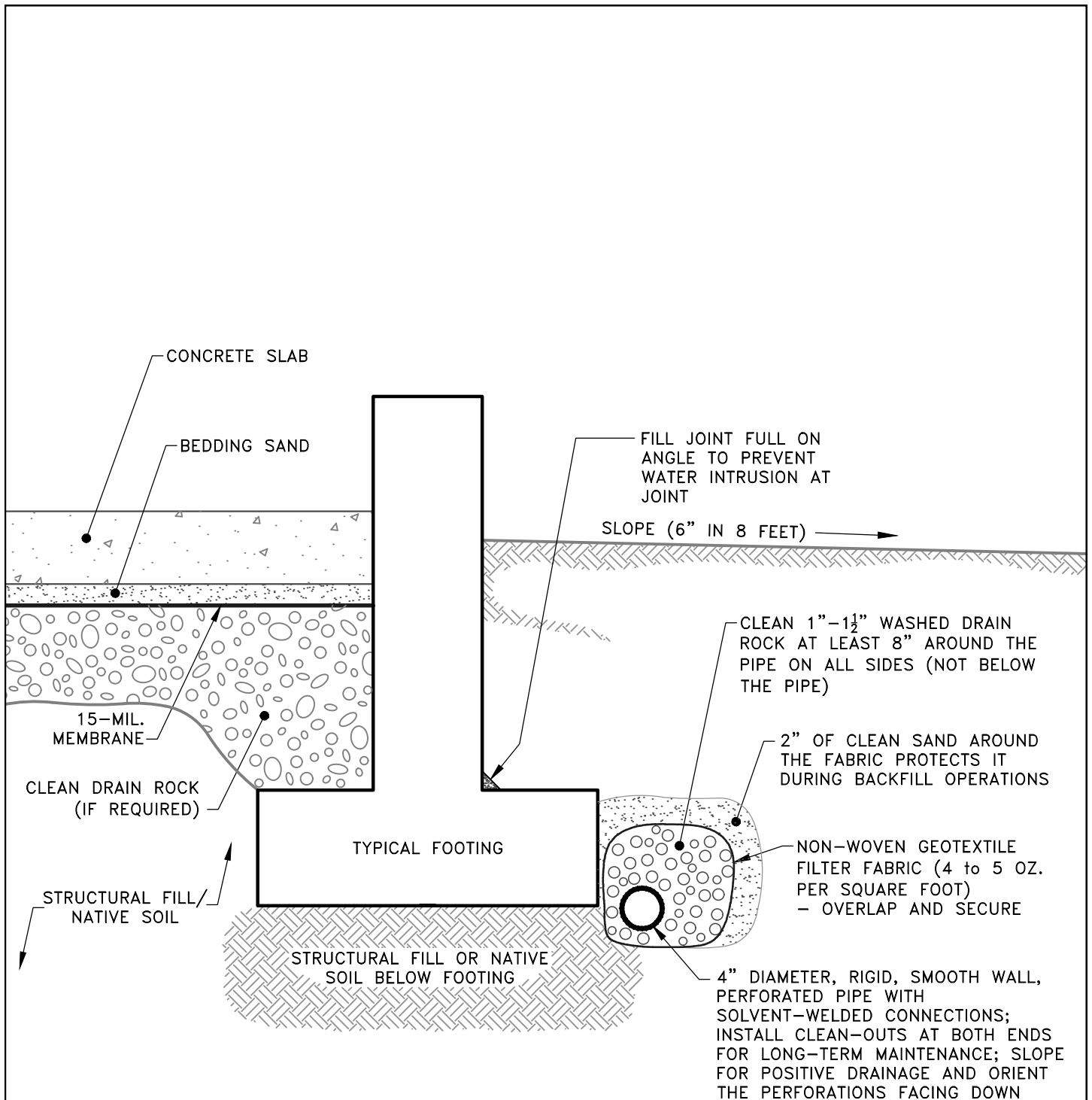
REV: 10/04/2018

PREPARED BY: LMD

5561 Hoover Elementary - 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

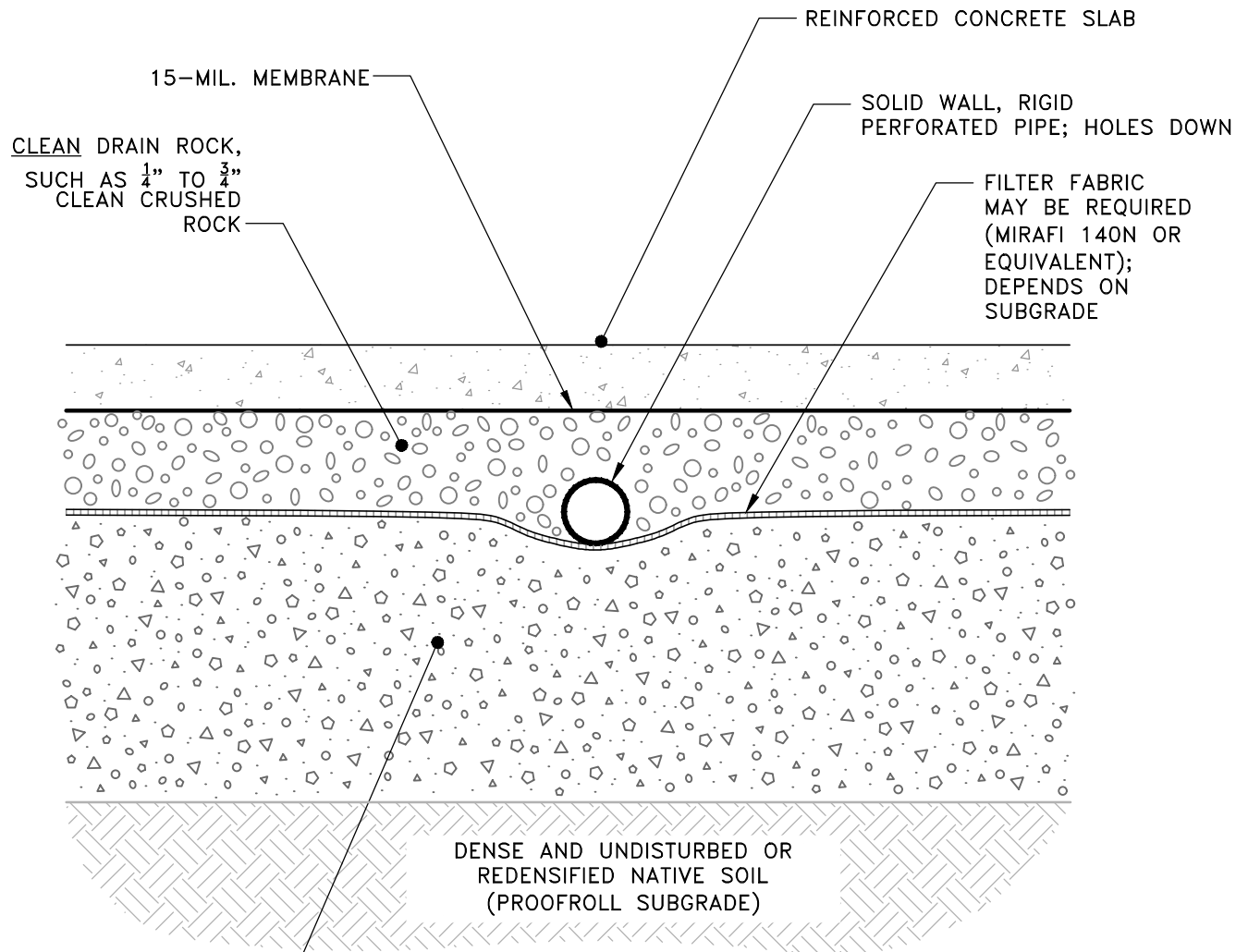
TYPICAL FOUNDATION DRAIN
SLAB ON GRADE FLOOR

HOOVER ELEMENTARY SCHOOL
MEDFORD, OREGON

DATE: OCTOBER 2018
JOB NO: 02-5561-01
REV: 10/04/2018
PREPARED BY: LMD
5561 Hoover Elementary - 03 - found drain-slab.dwg

FIGURE:

3



POSSIBLE
STRUCTURAL FILL
(THICKNESS VARIES)
- OR NATIVE SOIL
COMPACTED TO AT
LEAST 98% OF ASTM
D-698

NOTES:

- (1) MAXIMUM SPACING IS 10 TO 15 FEET.
- (2) ORIENT PIPE PERFORATIONS TO BOTTOM.
- (3) ASSEMBLE PIPE USING SOLVENT-WELDED CONNECTIONS.
- (4) DO NOT DRIVE OVER DRAIN LINES.
- (5) DRAIN ROCK AND STRUCTURAL FILL TO MEET SPECS. IN REPORT BODY - SLOPE PIPE TO DRAIN.
- (6) MAY REQUIRE FILTER FABRIC ON NATIVE SUBGRADE OR IF STRUCTURAL FILL IS VERY SILTY OR SANDY.
- (7) BEST VAPOR BARRIER IS STEGO INDUSTRIES 15mil STEGO WRAP (OR EQUIVALENT). OWNER MAY CHOOSE TO USE 6mil VISQUENE WITH THE UNDERSTANDING IT DOES NOT PERFORM NEARLY AS WELL.

FOR ILLUSTRATION PURPOSES ONLY
NOT TO SCALE



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

FLOOR SUBDRAIN DETAIL

HOOVER ELEMENTARY SCHOOL
MEDFORD, OREGON

DATE: OCTOBER 2018
JOB NO: 02-5561-01
REV: 10/09/2018
PREPARED BY: LMD
5561 Hoover Elementary - 04 - subdrain sog.dwg

FIGURE:

4

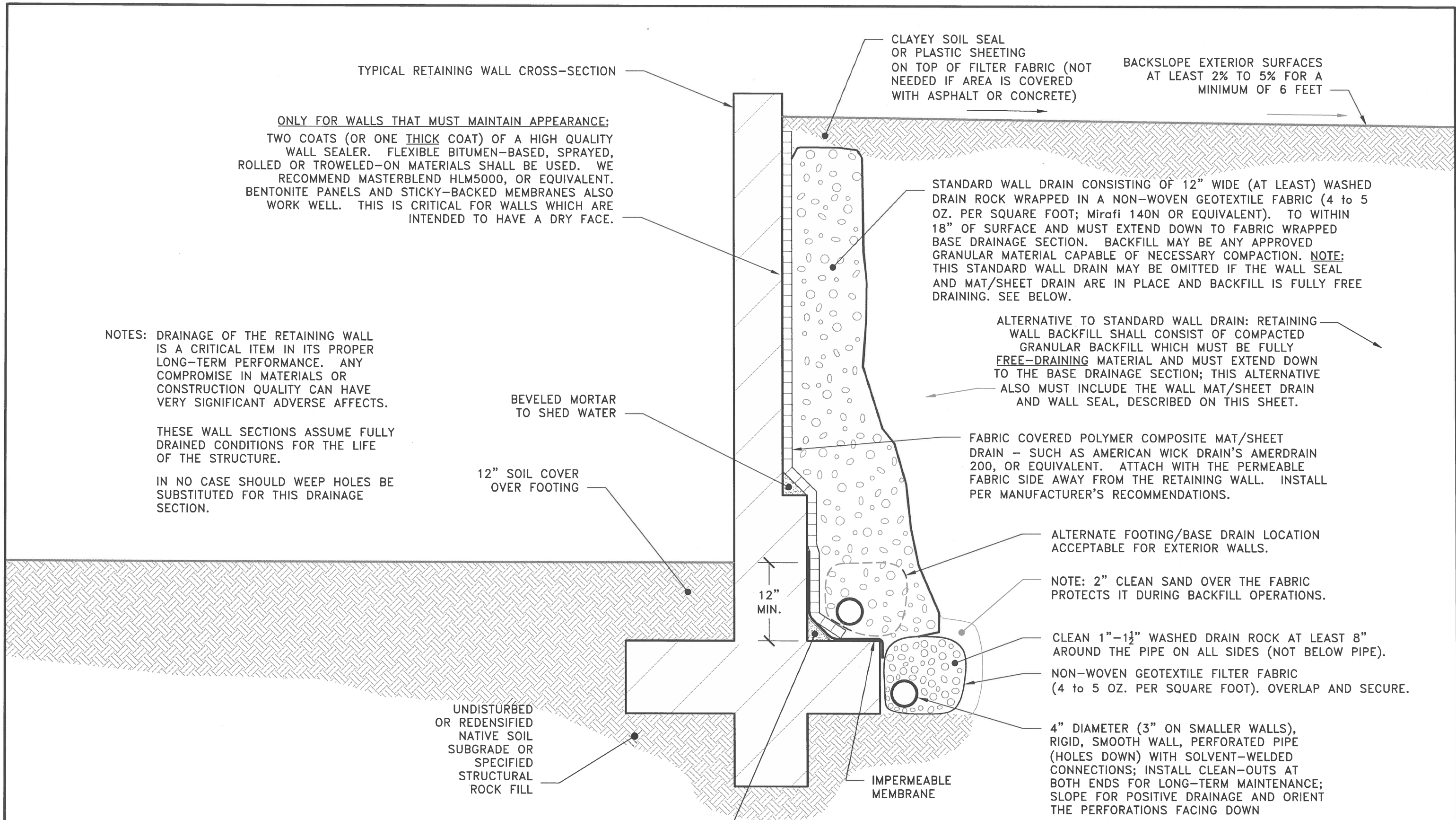
TYPICAL RETAINING WALL CROSS-SECTION

ONLY FOR WALLS THAT MUST MAINTAIN APPEARANCE:
TWO COATS (OR ONE THICK COAT) OF A HIGH QUALITY WALL SEALER. FLEXIBLE BITUMEN-BASED, SPRAYED, ROLLED OR TROWELED-ON MATERIALS SHALL BE USED. WE RECOMMEND MASTERBLEND HLM5000, OR EQUIVALENT. BENTONITE PANELS AND STICKY-BACKED MEMBRANES ALSO WORK WELL. THIS IS CRITICAL FOR WALLS WHICH ARE INTENDED TO HAVE A DRY FACE.

NOTES: DRAINAGE OF THE RETAINING WALL IS A CRITICAL ITEM IN ITS PROPER LONG-TERM PERFORMANCE. ANY COMPROMISE IN MATERIALS OR CONSTRUCTION QUALITY CAN HAVE VERY SIGNIFICANT ADVERSE AFFECTS.

THESE WALL SECTIONS ASSUME FULLY DRAINED CONDITIONS FOR THE LIFE OF THE STRUCTURE.

IN NO CASE SHOULD WEEP HOLES BE SUBSTITUTED FOR THIS DRAINAGE SECTION.



CLAYEY SOIL SEAL OR PLASTIC SHEETING ON TOP OF FILTER FABRIC (NOT NEEDED IF AREA IS COVERED WITH ASPHALT OR CONCRETE)

BACKSLOPE EXTERIOR SURFACES AT LEAST 2% TO 5% FOR A MINIMUM OF 6 FEET

STANDARD WALL DRAIN CONSISTING OF 12" WIDE (AT LEAST) WASHED DRAIN ROCK WRAPPED IN A NON-WOVEN GEOTEXTILE FABRIC (4 TO 5 OZ. PER SQUARE FOOT; Mirafi 140N OR EQUIVALENT). TO WITHIN 18" OF SURFACE AND MUST EXTEND DOWN TO FABRIC WRAPPED BASE DRAINAGE SECTION. BACKFILL MAY BE ANY APPROVED GRANULAR MATERIAL CAPABLE OF NECESSARY COMPACTION. NOTE: THIS STANDARD WALL DRAIN MAY BE OMITTED IF THE WALL SEAL AND MAT/SHEET DRAIN ARE IN PLACE AND BACKFILL IS FULLY FREE DRAINING. SEE BELOW.

ALTERNATIVE TO STANDARD WALL DRAIN: RETAINING WALL BACKFILL SHALL CONSIST OF COMPACTED GRANULAR BACKFILL WHICH MUST BE FULLY FREE-DRAINING MATERIAL AND MUST EXTEND DOWN TO THE BASE DRAINAGE SECTION; THIS ALTERNATIVE ALSO MUST INCLUDE THE WALL MAT/SHEET DRAIN AND WALL SEAL, DESCRIBED ON THIS SHEET.

FABRIC COVERED POLYMER COMPOSITE MAT/SHEET DRAIN - SUCH AS AMERICAN WICK DRAIN'S AMERDRAIN 200, OR EQUIVALENT. ATTACH WITH THE PERMEABLE FABRIC SIDE AWAY FROM THE RETAINING WALL. INSTALL PER MANUFACTURER'S RECOMMENDATIONS.

ALTERNATE FOOTING/BASE DRAIN LOCATION ACCEPTABLE FOR EXTERIOR WALLS.

NOTE: 2" CLEAN SAND OVER THE FABRIC PROTECTS IT DURING BACKFILL OPERATIONS.

CLEAN 1"-1½" WASHED DRAIN ROCK AT LEAST 8" AROUND THE PIPE ON ALL SIDES (NOT BELOW PIPE).

NON-WOVEN GEOTEXTILE FILTER FABRIC (4 TO 5 OZ. PER SQUARE FOOT). OVERLAP AND SECURE.

4" DIAMETER (3" ON SMALLER WALLS), RIGID, SMOOTH WALL, PERFORATED PIPE (HOLES DOWN) WITH SOLVENT-WELDED CONNECTIONS; INSTALL CLEAN-OUTS AT BOTH ENDS FOR LONG-TERM MAINTENANCE; SLOPE FOR POSITIVE DRAINAGE AND ORIENT THE PERFORATIONS FACING DOWN

BEVELED MORTAR TO SHED WATER

12" SOIL COVER OVER FOOTING

12" MIN.

UNDISTURBED OR REDENSIFIED NATIVE SOIL SUBGRADE OR SPECIFIED STRUCTURAL ROCK FILL

IMPERMEABLE MEMBRANE

BEVELED MORTAR TO SHED WATER

FOR ILLUSTRATION PURPOSES ONLY
NOT TO SCALE

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

EXTERIOR RETAINING WALL
DRAINAGE CROSS-SECTION
HOOVER ELEMENTARY SCHOOL
MEDFORD, OREGON

DATE: OCTOBER 2018
JOB NO: 02-5561-01
REV: 10/19/2018 7:43 AM
PREPARED BY: MG3
5581 Hoover Elementary - 05 - retwall drain-ext.dwg

FIGURE:
5

APPENDIX A

BORING LOGS

BORING LOGS

Please note that the soil descriptions given below are representative of how the field representative observed and classified them at the time of drilling. However, these should not be used as a guarantee of subsurface conditions across the site. Any interpretation or estimates made by others based on these logs, is done at their risk.

B-1

0.0 – 2.5	Topsoil/Rootzone
0.25 – 2.5	Dense, brown Sand; damp. SPT 2.0' to 2.8'; 11/20 for 2"; N=60
2.5 – 2.8	Very dense, brown, weathered Sandstone; dry.

No Free Groundwater or Seepage Observed.
Bottom of Boring at 2.8 feet.

B-2

0.0 – 0.25	Topsoil/Rootzone
0.25 – 3.0	Medium stiff, brown black, clayey Sand; moist to wet. SPT 2.5 to 4.0; 4/6/8; N=14
3.0 – 5.2	Dense, dark brown, clayey Sand; moist.
5.2 – 5.5	Very dense, brown, weathered Sandstone; damp to dry.

No Free Groundwater or Seepage Observed.
Bottom of Boring at 5.5 feet.

B-3

0.0 – 0.25	Topsoil/Rootzone
0.25 – 1.2	Dense, brown Sand; moist.
1.2 – 1.4	Very dense, brown, weathered Sandstone; moist to dry.

No Free Groundwater or Seepage Observed.
Bottom of Boring at 1.4 feet.

B-4

0.0 – 0.25	Topsoil/Rootzone
0.25 – 2.0	Loose to medium dense, brown Sand; moist, some gravel with depth.
2.0 – 3.5	Medium stiff, dark grey, silty Sand; moist, some clay.
3.5 – 4.0	Dense, brown, clayey Sand; moist.
4.0 – 4.2	Very dense, brown, weathered Sandstone; moist to dry.

No Free Groundwater or Seepage Observed.
Bottom of Boring at 4.2 feet.

APPENDIX B

LABORATORY TEST RESULTS



THE GALLI GROUP
Geotechnical Consulting

Expansion Index Worksheet (ASTM D-4829)

Client:
Project Hoover Elementary School
Job No: 02-5561-01
Test Date: 10/15/2018
Sample Location: B-6, S-1, 2.0'-2.5'
Sample Date: 12/15/2017
Description of Soil: Brown Clayey SILT

Expansion Index measured (E_m):

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

begin dial : 0.0751

end dial: 0.1094

E_m: 34

Weight of ring (g): 365.3
Wt. Wet sample in ring(g): 737.5
Sample Wet Weight (g): 372.28
Sample Length (in.): 1
Sample Diameter (in.): 4.01
Volume of sample (ft³): 0.007309
Sample Unit Wt. (PCF): **112.2**
Sample Dry Unit Wt. (PCF): **104.5**

Saturation (S):

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

SG: 2.7

γ_d : 104.5

%w : 7.4

S = 32

As prepared for testing:

can no. G5
wet weight of soil + can (g) 655.27
dry weight of soil + can (g) 628.68

weight of can (g) 267.13
weight of dry soil (g) 361.55
weight of water (g) 26.59
moisture content (% of dry weight) 7.4

E₁₅₀ Calculation:

$$E_{150} = E_m - (50 - S_m) * [(65 + E_m) / (220 - S_m)]$$

E_M 34

S 32

E₁₅₀ = 25

After testing:

can no. G7
wet weight of soil + can (g) 620.78
dry weight of soil + can (g) 507.00
weight of can (g) 191.61
weight of dry soil (g) 315.39
weight of water (g) 113.78
moisture content (% of dry weight) 36.1

#4 + (dry wt.) 934.54

#4 - (dry wt.) 91.43

% Passing #4 Sieve = 8.9

Tested By: Dennis Duru