THE GALLI GROUP
Engineering Consulting

GEOTECHNICAL INVESTIGATION REPORT
ELMER NELSON WAY BRIDGE
GRANTS PASS, OREGON

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1.0 INTRODUCTION

The current bridge over a small stream on Elmer Nelson Way in Grants Pass, Oregon has a limited width and load limit. To help this bridge support the traffic expected for Elmer Nelson Way new foundation support is required. This investigation and report is focused on abutment foundations for the bridge. The report includes the investigation data and design recommendations for the bridge abutment support.

2.0 SITE AND PROJECT DESCRIPTION

2.1 SITE DESCRIPTION

The subject site is a relatively flat area located along Elmer Nelson Way, just east of Hubbard Lane, in southwest Grants Pass, Oregon. The current bridge is elevated some 3 to 4 feet above the road on the west end and fairly level on the east end. Please see Figure 1 and Site Photos in Appendix A for site location and usage just prior to this project.

This area is generally underlain by surficial fill and sandy silt soils to depths of 6 to 12 feet. Underneath these soils, sand and gravels and dense gravels are usually encountered. Groundwater can at times be found at 8 to 12 feet. Vegetation is moderate but a bit heavy along the stream.

2.2 PROJECT DESCRIPTION

The proposed project consists of constructing a new bridge with new concrete abutments and abutment support. Abutment design loads have been estimated at between 300 kips and 400 kips.

3.0 SITE EXPLORATION

The site subsurface conditions were investigated by drilling one (1) exploratory boring at each bridge abutment, for a total of two (2) borings. On October 16, 2015 our field engineer, Mel Galli, EIT, was at the site to document drilling by Subsurface Technologies from Banks, Oregon. The holes were advanced by hollow stem auger drilling methods. The subsurface information for each boring was logged and samples were obtained for transport to our office and laboratory. These borings penetrated to depths of between 21.5 feet in B-1 to 23 feet in B-2. All borings terminated in very dense gravels and cobbles. Each boring was located approximately in the center of the north lane of the asphalt.

Soil samples and soil density data were obtained by performing the Standard Penetration Test (SPT) at various depths as drilling advanced in each boring. The SPT involved driving a 2-inch O.D. split spoon steel sample tube into the bottom of the boring by dropping a 140-lb. weight a distance of 30 inches. The SPT N-Value is the total number of blows it takes to drive the sampler the last 12 inches of an 18-inch drive. These values are listed on the Boring Logs in Appendix B. This data can be used to correlate with other data from thousands of jobs to help in determining and verifying soil strength and density parameters.

Depths and general location of the borings are as follows:

Borings	**Est. Ground Elevation (ft.)**	**Depth of Boring (ft.)**	**Depth to Dense Gravels (ft.)**	**General Location**
B-1	≈ 10' Above Creek	21.5	16.0	W Abut. Bridge
B-2	≈ 10' Above Creek	23.0	16.0	E Abut. Bridge

Please see Figure 2 for locations of the Borings.

Our representative selected the Boring locations, observed and logged the soils encountered, obtained samples for transport back to the laboratory, collected drilling and groundwater data and verified all holes were filled and sealed properly. When completed the holes were backfilled with hole plug and crushed rock. The top was compacted full of cold patch asphalt. Logs of all borings are presented in Appendix B at the end of this report.

4.0 LABORATORY TESTING

Due to the nature of the structure, lab testing on nearby projects and the soils encountered, minimal laboratory testing was accomplished. Laboratory tests accomplished included the following:

- Natural Moisture Content

These tests, along with testing and experience on nearby sites, were used to help with soil classification purposes and design recommendations in later sections.

5.0 SUBSURFACE CONDITIONS

5.1 SOIL

The subsurface soils conditions were similar in both borings. Conditions consisted of an upper layer (3½ feet) of fill over alluvial decomposed granitics. All borings terminated in dense gravels. Depths to the top of the dense gravels are presented in the table on the previous page.

5.2 GROUNDWATER

No free groundwater was found in the borings. Based on soil samples obtained, the soils were moist below a depth of 10 feet. It appears water from the creek bed was slowly seeping into the boring areas.

Regional groundwater is relatively deep. However, it is likely that perched groundwater will be present during the wet months of the year and especially during the irrigation season. It is possible this could rise 2 to 4 feet above the creek bottom as the area becomes saturated. Seepage into deep construction excavations could be moderately rapid but handled by open sumps embedded in rock filled sump holes. However, the seepage could cause unstable cut slopes and possible breach into the streambed.

6.0 SEISMIC DESIGN

6.1 IBC AND 2014 OSSC DESIGN EARTHQUAKE

The design earthquake for the project area is based upon established values and methodologies in the Oregon Structural Specialty Code (OSSC; 2014), International Building Code (IBC; 2012), and ASCE 07-10. 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 were established as set forth in Section 1613 (IBC, 2012) and Section 11.4 (ASCE 7-10), and were obtained from the online USGS Seismic Design Maps (USGS, 2015b).

6.2 SEISMIC DESIGN TABLE

The following table shall be used by the structural engineer in the design of all structures.

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

Parameter	Value
Project Latitude/ Longitude- Elmer Nelson Bridge; Grants Pass (Project 02-4861)	Lat. 42.418678N Long 123.385678W
Occupancy/Risk Category (Table 1.5-1 ASCE/SEI 7-10)	I, II or III Risk Category I, II or III
Mapped Spectral Response Acceleration (MCE_R) - Short Period (S_S)	0.825g
Mapped Spectral Response Acceleration (MCE_R) - 1-Second Period (S_1)	0.436g
Site Class - *(Table 20-3-1 ASCE/SEI 7-10)*	**C**
Short Period Site Coefficient based on Site Class - (F_a)	1.071
1-Second Site Coefficient based on Site Class - (Fv)	1.364
MCE_R Spectral Response Acceleration - (S_{MS})	$S_{MS} = F_a \cdot S_S =$ 0.883g
MCE_R Spectral Response Acceleration for 1-Second - (S_{M1})	$S_{M1} = Fv * S_1 =$ 0.595g
Design Spectral Response Acceleration for Short Periods - (S_{DS})	**S_{DS} = 2/3 S_{MS}= 0.588g**
Design Spectral Response Acceleration for 1-Second - (S_{D1})	**S_{D1} = 2/3 S_{M1} = 0.397g**
PGA= MCE_G PGA (Section 11.8.3.2; and Figures 22-7; ASCE/SEI 7-10)	PGA= 0.411g
F_{PGA} (Table 11.8-1 ASCE/SEI 7-10)	F_{pga} =1.000
$PGA_M = F_{pga}$*PGA (EQ 11.8-1; ASCE/SEI 7-10)	PGA_M= **0.411g**
Design PGA= $PGA_D = PGA_M$*2/3	PGA_D= **0.274g**
Seismic Design Category (Section 11.6 and Table 11.6-1 and Table 11.6-2; ASCE/SEI 7-10)	D

7.0 GEOTECHNICAL RECOMMENDATIONS

The primary design requirements are for foundation support of the bridge abutments and design loads for the abutment walls. Associated construction recommendations are also included.

The dense soils encountered are moderately shallow. However, potential construction difficulties could preclude the use of reinforced concrete spread footings instead of driven piles for some of the abutments. We have provided design recommendations for both shallow and deep foundations, including potential construction difficulties.

7.1 SHALLOW FOUNDATIONS

Based on the subsurface investigation both bridge abutments could be supported on spread footings founded on the dense underlying gravels. This would require excavations to depths of between 14 feet and 16 feet below the road surface. The following sections provide information for spread footings at the bridge.

Auto Bridge Foundations. This bridge, with B-1 and B-2 at the two abutments, encountered stiff fill, medium dense to dense sand and silt and then dense to very dense gravels and cobbles at depths of 13½ feet and 15½ feet respectively. Water in the creek can be at depths of 8 to 10 feet. Therefore, based on the borings at these two abutments, spread footings for these abutments may be designed and constructed as follows.

1. Excavate into the dense gravels at least 1 foot. Redensify the base of the excavation.
2. Place 12 inches of compacted structural rock fill. Use 4" minus crushed rock compacted in two lifts to at least 98% of ASTM D-698.
3. The excavation and compacted structural rock fill shall be at least 1 foot wider than the abutment footings, on all sides.
4. Footings placed on the prepared subgrade described above may be designed for a bearing pressure of 3,500 pounds per square foot (psf). This may be increased to 4,500 psf for transitory seismic loads.
5. Footing widths should be at least 24 inches wide.

Settlement. Anticipated settlement of footings designed as described above should be less than approximately 1 inch for total settlement and less than ¾ inches differential settlement across a 35 foot width.

Excavations. To embed the footings as described above excavations to depths of 14 and 16 feet below the existing road surface will be required. The base of these could be on the order of 4 and 6 feet below the bottom of the stream bottom. The borings encountered dampness at near the base of the creek. We did not encounter a static water level in the borings. The 12 inches of compacted rock should help keep the base of the excavation stable. However, this water level could increase during the wetter months of the year which can cause excavation and cut slope problems.

Excavations to the depths above would have to be sloped at between 1H and 1¼ H:1V to be stable. If seepage occurs out of the base of the cut slopes they would likely to have to be cut at 1½H to 1¾ H:1V or be shored. This can result in wide excavations, with the streamside slope being close to or into the bottom of the stream. Therefore, increased seepage and possible breach from the creek could take place. Pumping from open sumps along the outside edges of the excavations, particularly along the creek side of the excavations, could adequately remove the water during low flows.

Note: While it is likely pumping will control water entering the excavations, it is also possible such pumping could dewater the creek to some degree and could require cofferdams to control the stream flow.

Shoring. It would be possible to achieve near vertical excavations and limit water intrusion by the use of shoring. Driven sheet pile shoring or other method could be used to provide a reasonably dry excavation for footing construction. Design parameters could be provided if such shoring is to be considered.

7.2 DEEP FOUNDATIONS

If the cut slope and dewatering and stream issues indicate using spread footings presents significant construction difficulties, then deep foundations, such as driven piles, should be considered. It is likely the best deep foundation system for this bridge would be driven piles, likely steel pipe piles or steel H-piles. Piles driven into the dense gravels or to refusal into the top of the soft rock would provide excellent support for the bridge abutments.

Driven Piles. Driven steel pipe piles or steel H-piles may be used for abutment support. These must be driven to refusal criteria into the dense gravels. Embedment depths for these piles are assumed to be as follows.

Abutment	Depth Below Grade (ft.)
W (B-1)	22 to 24
E (B-2)	25 to 27

Piles driven into the very dense gravels will have the minimum capacities listed below:

Pile Type and Size	Allowable Design Load (kips)*
10" Steel Pipe Pile	45
12" Steel Pipe Pile	60
HP 10x42 Steel Pile	55
HP 12x53 Steel Pile	80

*This assumes FS of 2.5, pile to pile spacing of at least 3 feet.

Pile Protection. These piles will have to be driven deep into the very dense cobbly gravels. This can cause significant damage to unprotected piles. Therefore, it is recommended that all piles be provided with welded on armored tips.

Pile Driving. Penetration of the upper zones of the cobbly gravels will take a reasonable amount of driving energy. We strongly recommend that the pile driving subcontractor carefully consider the subsurface conditions to assure the pile hammer selected will be able to drive the piles to the required penetration depths. Our design engineer must review the hammer specifications prior to work at the site.

Pile Settlements. Piles driven into the very dense gravels will likely experience settlements on the order of ½ to ¾ inch.

Final Pile Design. We recommend we be involved in reviewing the final pile design and layout for the abutments. It is also likely that different pile contractors could use different hammers and/or driving methods. This can result in differing "set" values for pile driving and penetration. Our engineer should review the data for the selected piles, tip armoring and proposed pile hammer prior to construction. A set value, along with minimum embedment criteria into the gravels of 10 feet, would be provided for use by the contractor during pile driving operations.

Pile Installation Documentation. In order to verify pile installation is accomplished in accordance with the design recommendations our field personnel should observe and document all driven piles. Inconsistencies will be discussed with the geotechnical design engineer and appropriate changes made to insure proper pile support for the bridge abutments.

7.3 ABUTMENT RETAINING WALL DESIGN

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

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

Low Grade Angular Rock EFP	40 pcf
Crushed Rock EFP	35 pcf
Seismic (up to 10 feet tall)	0.27g.

- Walls that are fixed at the top (restrained) when backfilled should be designed for the following loads.

Low Grade Angular Rock EFP	50 pcf
Crushed Rock EFP	45 pcf
Seismic (up to 10 feet tall)	0.27g.

- The walls all must have full drainage as shown in Figure 3.
- These equivalent fluid pressures are to be used for the soil through which the anticipated failure plane will develop (assume envelope beginning 4 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".
- These values are for properly compacted, free draining, non-expansive, granular soils, free of organics and other debris or for imported granular backfill. The onsite organic topsoil or very Silty and clayey soils should not be used for wall backfill materials. Imported crushed rock or jaw-run "shale" works 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 the anticipated traffic.
- We recommend designing retaining walls to resist seismic loading. A horizontal acceleration component of at least 0.27g 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 0.8H, where H is the height of the wall. This relates to an equivalent uniform load over the entire back of the wall of approximately 17 pounds per square foot for each foot of backfill, for walls up to 10 feet tall (i.e. for an 8-foot wall, fully backfilled, uniform seismic load will be on the order of 140 psf over the entire back of the wall).
- 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 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.

Wall Drains. Wall drains should 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 to 18 inches of the surface or be backfilled with free draining material. A preformed, fabric-wrapped, polymer sheet drain, such as

Amerdrain, Linq Drain or Enkamat may be used in lieu of the vertical drainage zone, provided this is backfilled with clean, free-draining material. Exterior wall drains, which will not be sealed on top by asphalt or concrete, should have the upper 12 to 18 inches backfilled with compacted onsite silt soils to minimize intrusion of surface waters into the wall drain system. Please see Figure 3.

Walls that should not pass water vapor must be fully sealed (with a bitumen-based sealer that will not harden or crack).

All drains should be tightlined and positively sloped to an approved stormwater disposal location at the creek. All perforated pipe shall 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 drainage system.

7.4 LATERAL LOAD RESISTANCE

Lateral loads exerted upon these structures can be resisted by passive pressure acting on buried portions of the foundations and retaining walls and by friction between the bottom of structural elements of the wall and footings and the underlying soil and by driven piles.

We recommend the following design:

Passive Equivalent Fluid Pressures (EFP)

- Against Compacted Structural Rock Fill — 450 pcf
- Against Medium stiff to Medium Dense Native Soils — 250 pcf
- Against Dense Gravels — 350 pcf

We also recommend that the first 12 inches below the ground surface be ignored when computing the passive resistance of the native soils.

Frictional Coefficients

- Against Native Sand and Silt — 0.30
- Against Native Sand and Gravels — 0.40
- Against Crushed Rock (at least 12" thick) — 0.50

Pile Tops; 10 inch pipe (minimum).

- Per pile, 4 kips each (verify by field load test).

If added lateral resistance is needed, deeper footings, buried concrete walls as deadman anchors, embedded structural walls or batter piles could be used.

7.5 STRUCTURAL FILL PLACEMENT AND COMPACTION

7.5.1 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 the structure, sidewalks and other load-bearing areas or that will create fill slopes. It appears that portions of the site will require structural fill. The subgrade needs to be prepared properly and the structural fill 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 inches. The material should be reasonably well-graded with less than 8 percent fines (silt and clay size passing the No. 200 mesh sieve). During dry weather, any organic-free, non-expansive, compactable, reasonably well graded granular material, meeting the maximum size criteria, is typically acceptable for this purpose. Locally available crushed rock and jaw-run crushed shale have performed adequately for most applications of structural fill. We recommend the excavated soil at the site not be used as structural fill. *Crushed rock must be used for fill beneath the footings.* We recommend our representative sample proposed fill for approval prior to use at the site.

Structural Fill Placement. 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 three to 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 laboratory Test Method D-698 (Standard Proctor).

We strongly recommend the contractor utilize a large vibratory roller when compacting the imported granular fill. A large smooth drum roller may be utilized when compacting rock materials such as imported crushed rock or jaw-run "shale".

Structural fill placed beneath footings or other structural elements must extend beyond all sides of such elements a distance equal to at least 1 foot for vertical support. **Note:** The structural fill must be located as accurately as the footings unless added width of structural fill is placed to accommodate inaccuracies in its location.

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. We recommend our representative observe and document placement and compaction of all structural fill in a systematic manner.

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" or crushed rock 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 or less (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 (less than ¼ inch deflection is required). All areas which exhibit movement or compression of the rock material, 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, shale 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.

7.5.2 Non-Structural Fill

Any waste soil, organic strippings or other deleterious soil 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 (which are not on slopes greater than 10%). 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.

8.0 ADDITIONAL SERVICES & LIMITATIONS

8.1 ADDITIONAL SERVICES

Additional services by The Galli Group are recommended to verify that design recommendations are correctly interpreted in final project design and to help monitor compliance with project specifications during the construction process. For this project we anticipate additional services could include the following:

1) Discussion and redesign for final project location and foundation type.

2) Review of final construction plans and specifications for compliance with geotechnical recommendations (including foundation type selection and design, subdrains and retaining walls).

3) Possible project team meetings and/or phone discussions to clarify issues and proceed smoothly into and through the construction process.

4) Preconstruction meeting to discuss major issues and begin project; establish lines of communication.

5) Observation of site excavation slopes, overexcavation and subgrade proofrolling.

6) Observation and testing of structural fill placement and compaction.

7) Observation of completed footing excavations, structural fill and/or installation of all driven piles.

8) Verification of wall drains.

9) Periodic reports as requested by the client and/or required by the building department.

10) Any other geotechnical related items requested by the client.

We would provide these additional services on a time-and-expense basis in accordance with the current Standard Fee Schedule and General Conditions at the time these additional services are provided. Please note that if we are not retained to perform these services The Galli Group cannot be held responsible for the geotechnical items we did not review, inspect or confirm on the site. The owner and contractor will accept responsibility for all geotechnical items.

Construction Materials Testing. The Galli Group can also provide several qualified technicians with engineer oversight to perform the special inspection and testing services required by this structure. These services could include rebar inspection, epoxy installation inspection, concrete inspection, sampling and testing, nuclear density and laboratory testing of fill soils and other testing and inspection as required by the structural engineer, the owner or the City.

8.2 LIMITATIONS

The analyses, conclusions and design recommendations contained in this report are based on site conditions and development plans as they existed at the time of the site visit, and assume soils and groundwater conditions exposed and observed in the borings are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions, code requirements 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, 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 City and their design and construction team in the design and construction of the proposed new Elmer Nelson Way Bridge. 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 and geology practices in southern Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

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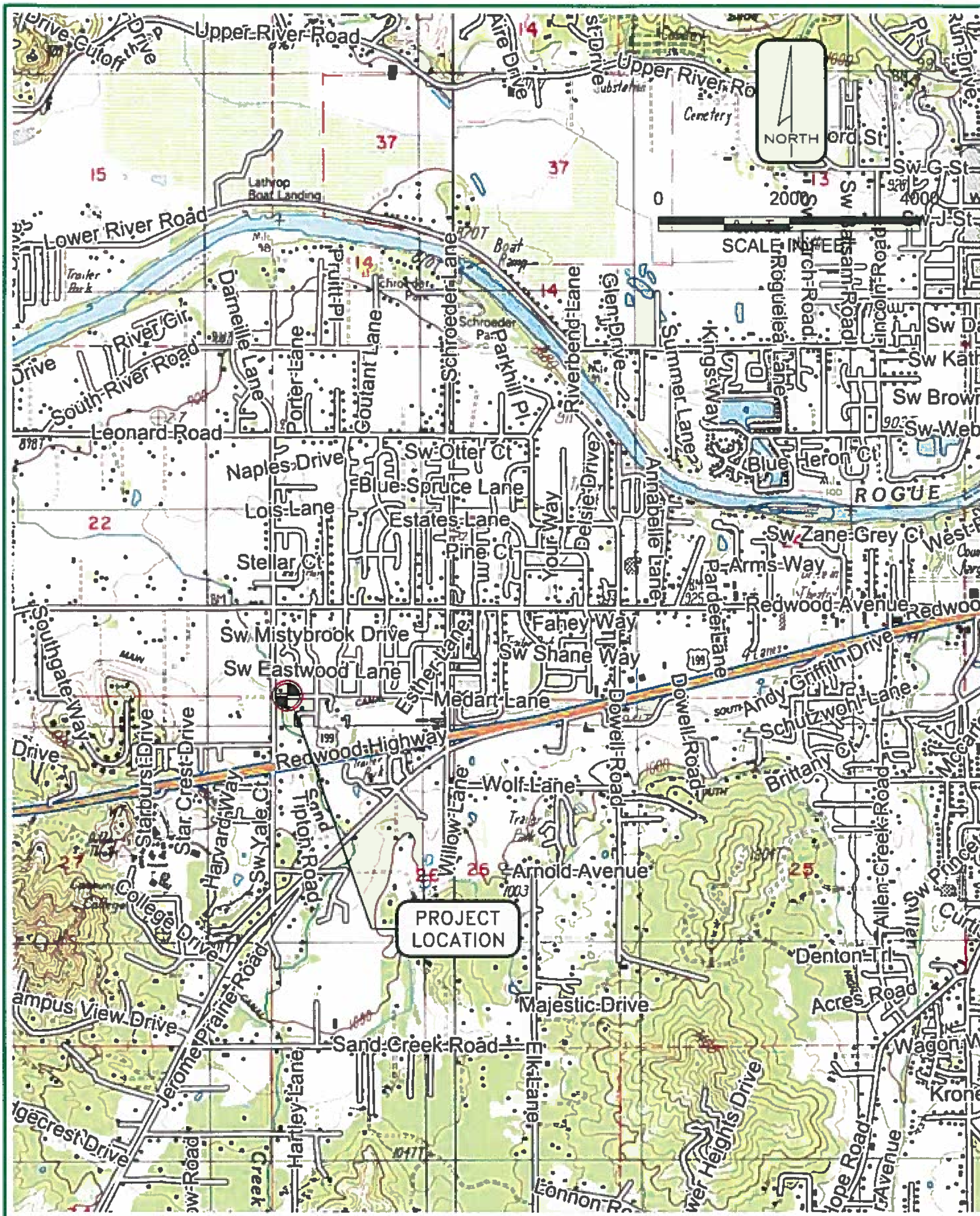
REFERENCES

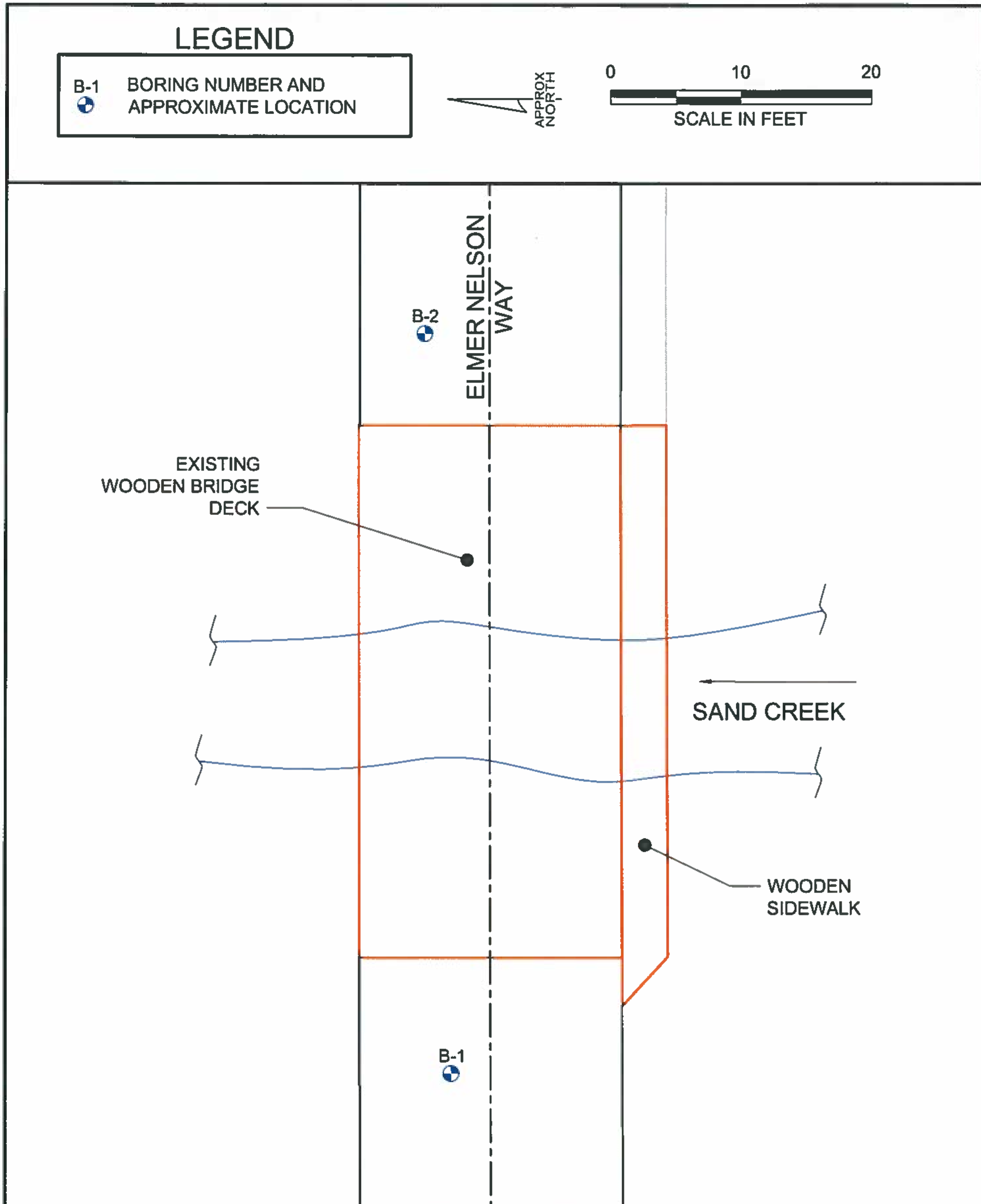
ASCE; 2010; American Society of Civil Engineers; ASCE 7-10 Minimum Design Loads for Buildings and Other Structures

OSSC; 2014; Oregon Structural Specialty Code; International Code Council, Inc.

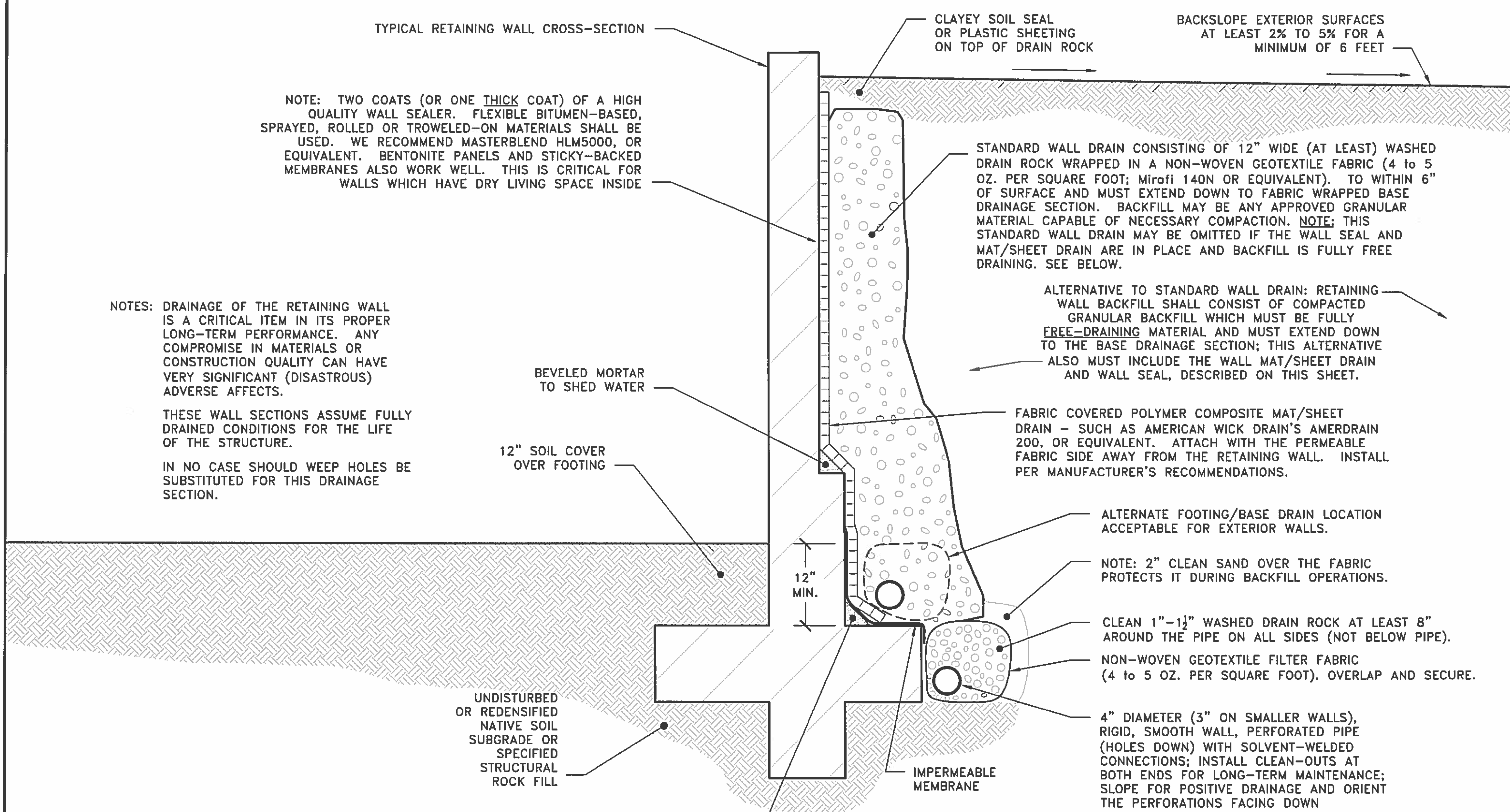
USGS, 2015 (http://earthquake.usgs.gov/designmaps/us/application.php



THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	VICINITY MAP ELMER NELSON BRIDGE GRANTS PASS, OREGON	DATE: NOVEMBER 2015 JOB NO: 02-4861-02 REV: 11/9/2015 1:30 PM PREPARED BY: MG3 4861 Elmer Nelson Bridge - 01 - Vicinity dwg	FIGURE: 1



THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	SITE PLAN WITH BORING LOCATIONS	DATE: NOVEMBER 2015	FIGURE:
		JOB NO: 02-4861-02	2
	ELMER NELSON BRIDGE GRANTS PASS, OREGON	REV: 11/9/2015 2:02 PM	
		PREPARED BY: MG3	
		4861 Elmer Nelson Bridge - 02 - Site Plan.dwg	



FOR ILLUSTRATION PURPOSES ONLY

NOT TO SCALE

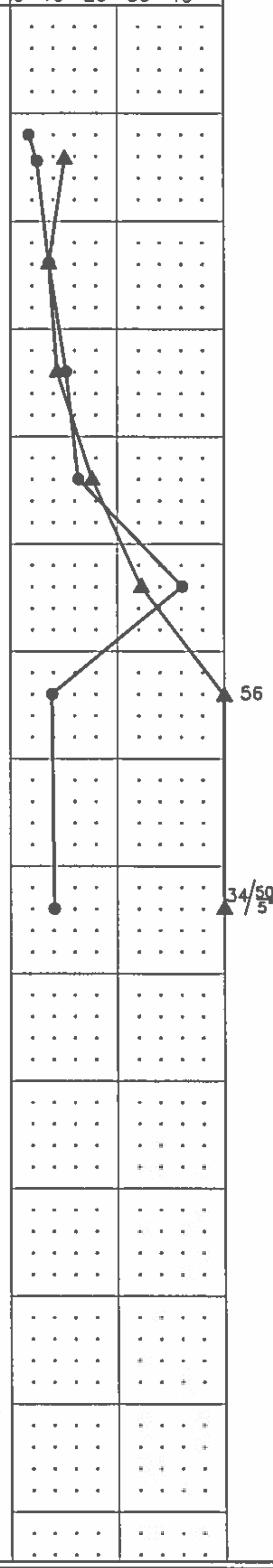
THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	EXTERIOR RETAINING WALL DRAINAGE CROSS-SECTION ELMER NELSON BRIDGE GRANTS PASS, OREGON	DATE: NOVEMBER 2015 JOB NO: 02-4861-02 REV: 11/9/2015 2:06 PM PREPARED BY: MG3 4861 Elmer Nelson Bridge - 03 - retwall drain-ext.dwg	FIGURE: 3

APPENDIX A

BORING LOGS

DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEV. (FT.)	SAMPLES NO.	S.P.T.	GROUND WATER DEPTH IN FEET
½	2½" Asphaltic Concrete (A.C.), over 3" 3/4-inch minus Crushed Rock FILL.			
	(FILL) Dense, orange tan to gray, silty, gravelly SAND; fine sand, dry to damp.			2½
3½	Loose to medium dense, brown to light brown, silty SAND (decomposed granite); occasional gravels to clean, reducing silt with depth, damp.	S-1	13	
				5
		S-2	9	
				7½
		S-3	11	
9½	Medium dense, brown to light brown, silty SAND; with gravels and some clay, very moist to wet.			10
		S-4	19	
11½	Hard, red brown, fine sandy, clayey SILT; clay and fine sand varies somewhat, very moist.			12½
		S-5	31	
13½	Dense to very dense, dark red brown & gray, silty, sandy GRAVELS; varies somewhat, coarse sand & gravels increasing with depth, moist to saturated.			15
		S-6	56	
				17½
				20
		S-7	34/50/5"	▼10/16/15
21½	Bottom of boring at 21½ feet. Wet soil conditions encountered from approximately 10 to 11½ feet. Groundwater level was estimated at 21 feet.			22½
				25
				27½
				30
				32½
				35

STANDARD PENETRATION TEST
▲ BLOWS PER FOOT



LEGEND

- 2" S.P.T. SAMPLE
- GRAB SAMPLE
- 3" DAMES & MOORE SAMPLE (140 LB SPT HAMMER)
- * SAMPLE NOT RECOVERED
- ESTIMATED EXTENTS OF PERCHED WATER TABLE
- 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

DRILLER SUBSURFACE TECHNOLOGIES
DATE START 10/16/15 FINISH 10/16/15
DRILLING TECHNIQUE 8"ø HSA

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

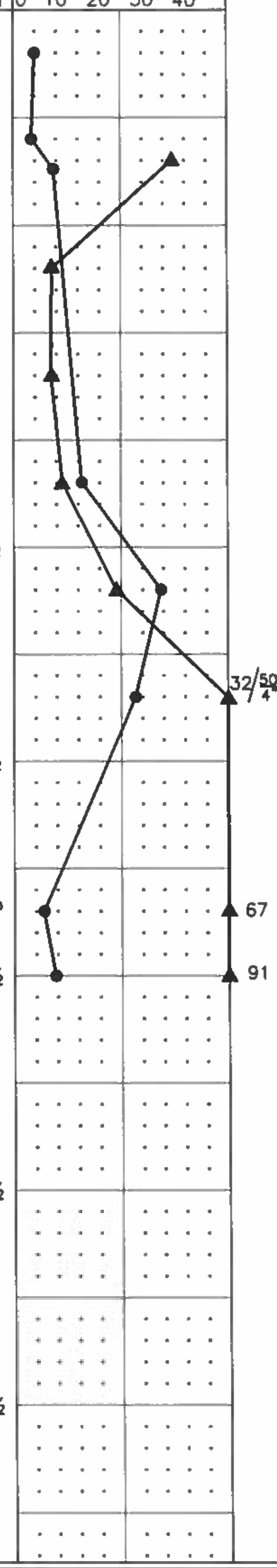
SUMMARY BORING LOG
B-2

ELMER NELSON BRIDGE
GRANTS PASS, OREGON

DATE NOV. 2015
JOB NO. 02-4861-02
FIG. A2

DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEV. (FT.)	SAMPLES NO.	S.P.T.	GROUND WATER DEPTH IN FEET
½	1½" Asphaltic Concrete (A.C.), over 1½" 3/4-inch minus Crushed Rock FILL.	S-1		
	(FILL) Dense, orange tan to gray, silty, SAND; fine sand & scattered rounded gravels, dry to damp.			2½
3½	Loose to medium dense, brown to light brown, silty SAND (decomposed granite); occasional to scattered gravels, damp to moist.	S-2	37	5
		S-3	9	7½
		S-4	9	
	... silty Sand becomes wet to saturated in bottom 6 inches of layer.			10
11	Very stiff to hard, dark brown & gray, clayey, fine sandy, SILT; clay and fine sand varies somewhat, trace fine gravels, moist to very moist.	S-5	11	12½
		S-6	24	15
		S-7	32/50/4	
17	Dense to very dense, brown & black, sandy GRAVELS (to gravelly SAND - varies/layered); medium to coarse sand, gravels increasing with depth, moist to saturated.			17½
				20
		S-8	67	▼10/16/15
		S-9	91	22½
23	Bottom of boring at 23 feet. Wet to saturated conditions (perched groundwater - est.) encountered from approximately 10½ to 11 feet. Groundwater level was measured at 21 feet.			25 27½ 30 32½ 35

STANDARD PENETRATION TEST
▲ BLOWS PER FOOT



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SUMMARY BORING LOG
B-1
ELMER NELSON BRIDGE
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FIG. A1