CITY OF West Linn
PLANNING AND DEVELOPMENT

STAFF REPORT
PLANNING MANAGER DECISION

DATE: May 14, 2019

FILE NO.: MIP-18-05

REQUEST: Approval of a three-parcel minor partition at 6123 Skyline Drive.

PLANNER: Darren Wyss, Associate Planner

Planning Manager AB

City Engineer EL

TABLE OF CONTENTS

GENERAL INFORMATION

APPLICANT:	Icon Construction and Development 1980 Willamette Falls Drive, Suite 200 West Linn, OR 97068
OWNER:	City of West Linn 22500 Salamo Road West Linn, OR 97068
CONSULTANT:	Rick Givens 18680 Sunblaze Drive Oregon City, OR 97045
SITE LOCATION:	6123 Skyline Drive
SITE SIZE:	35,569 square feet
LEGAL DESCRIPTION:	Assessor's Map 2S-1E-25AD Tax Lot 9900
COMP PLAN DESIGNATION:	Low-Density Residential
ZONING:	R-10, Single-Family Residential Detached (10,000 sq. ft. min. lot size)
APPROVAL CRITERIA:	Community Development Code (CDC) Chapter 11: Single-Family Residential Detached, R-10; Chapter 48: Access, Egress and Circulation; Chapter 85: Land Division, General Provisions; Chapter 92: Required Improvements; Chapter 99: Procedures for Decision Making: Quasi-Judicial.
120-DAY RULE:	The application became complete on December 17, 2018. The applicant provided a 30-day extension of the 120-day period to May 16, 2019.
PUBLIC NOTICE:	Notice was mailed to property owners within 500 feet of the subject property and to all neighborhood associations on February 14, 2019. A sign was placed on the property on February 22, 2019. The notice was also posted on the City's website on February 14, 2019. Therefore, public notice requirements of CDC Chapter 99 have been met.

EXECUTIVE SUMMARY

The applicant seeks approval for a three-parcel partition of a 35,569 square foot parcel (Parcel 1 of Partition Plat No. 1996-115) on the north side of Skyline Drive, on the east side of the Bolton Reservoir. The parcel is located in the Rosemont Summit Neighborhood. Two of the proposed parcels will be flag lots creating a private drive off of Skyline Drive. All three parcels will take access using the proposed private drive. All parcels will exceed 10,000 square feet and meet dimensional standards of the R-10 zone designation. All surrounding properties are zoned R-10.

The existing Skyline Drive right-of-way is 54 feet wide and sufficient to accommodate a Collector without Median/Center Lane as the sidewalk on the south side of Skyline Drive (adjacent to Wilderness Park) is curb-tight and doesn't have a six-foot planter strip.

The applicant has identified 17 significant trees on the subject property. Sixteen of the significant trees are located on proposed Parcel 3. The applicant proposes removing 15 of the 16 significant trees on Parcel 3 to accommodate new home construction and one significant tree on Parcel 2.

The site has a slope of 10 percent or less across Parcel 1 and 10 to 25 percent across Parcels 2 and 3. There are no environmental overlays on the property. The new homes will connect to an existing water main located in Skyline Drive. Sanitary sewer service will be extended from an existing line in Firwood Court. The new public sewer line will utilize an existing public utility easement between 5147 and 5127 Firwood Court and new easements recorded as part of the partition approval. New homes on the proposed parcels will be reviewed for compliance with setbacks at time of building permit application.

Public comments:

Public comments were submitted by Phillip and Krista Kreiger (see Exhibit PD-2). They expressed concerns and requested additional information on:

1. The impact, specifically the increased risk of wind throw, to the grove of trees located on their property from the removal of 15 significant trees from proposed Parcel 3. The applicant provided an arborist report prepared by Portland Tree Consulting on March 25, 2019 (see Exhibit PD-1). The report found the trees on Parcel 3 and the Kreiger property to be an open stand, with strong trunk taper and a high live-crown-ratio, which are more wind firm and resistant to trunk snap caused by wind. The conclusion was "*A view from Skyline Drive nearby. The mature conifers seen here are doing well as stand-alone trees. This landscape is typical of the neighborhood. Tree removal on Parcels 2 and 3 will allow increased wind velocity from the south on the trees in Tax Lot 5300. In my opinion the adverse effect will be negligible, and I do not anticipate trunk snap or blow down to result.*"
2. A geotechnical evaluation of potential impacts to their property, which is downslope, from development on Parcel 3. The applicant provided a Soil Investigation Report

prepared by GeoPacific Engineering, Inc. dated April 29, 2019 (see Exhibit PD-1). The report addressed undocumented fills, limiting grading on-site, appropriate foundation techniques, and protection of the slope between Parcel 3 and the Kreiger property. The applicant will be required to submit the GeoPacific Report during site development/building permit review. The report also recommended *"Proposed stormwater ponds near the tops of steep slope areas should be lined with an appropriate liner so that the ponds are impermeable. In no case shall stormwater be directed or allowed to flow freely over the slope faces."* The applicant has revised the stormwater plan to incorporate liners and direct overflow to the existing stormwater facility located on the Bolton Reservoir property to the east.

3. Potential location of their fence on portions of Parcels 2 and 3 and the possibility of it remaining in place. The Kreigers were provided with the applicant's contact information and advised to work with them to find resolution as it is outside of the scope of the land use application.

DECISION

The Planning Manager (designee) approves this application (MIP-18-05), based on: 1) the findings submitted by the applicant, which are incorporated by this reference, 2) supplementary staff findings included in the Addendum below, and 3) the addition of conditions of approval below. With these findings, the applicable approval criteria are met. The conditions are as follows:

1. **<u>Site Plan</u>. With the exception of modifications required by these conditions, the final plat shall conform to the Partition Site Plan dated September 2018 (Exhibit PD-1).**

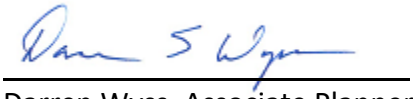
2. **<u>Engineering Standards</u>. All public improvements and facilities associated with the approved site design, including but not limited to street improvements, driveway approaches, curb cuts, utilities, grading, onsite and offsite stormwater, street lighting, easements, easement locations, and connections for future extension of utilities are subject to conformance with the City Municipal Code and Community Development Code. These must be designed, constructed, and completed prior to final plat approval.**

3. **<u>Reciprocal Access Easement</u>. Prior to final plat approval, the applicant shall record a reciprocal access easement and a mutual maintenance agreement for the shared use of the driveway located in the access easement. The easement recording number shall be provided on the face of the final plat. The final plat shall show the access easement at a width of 24 feet for the portion of drive that serves four homes.**

4. **<u>Shared Access Drive</u>. The shared driveway must be installed and measure a minimum of 20 feet in width for the portion where four homes take access and a minimum of 14 feet in width for the portion where only two homes take access, and include any fire apparatus turnaround required by the fire district, prior to final plat approval.**

5. **<u>GeoPacific Engineering Report</u>. The applicant shall submit a copy of the Soils Investigation Report prepared by GeoPacific Engineering, Inc. dated April 29, 2019 (see Exhibit PD-1) as part of the site development review/building permit application and shall provide any supplemental reports required by the Building Official. The report must be submitted prior to application for site development review/building permits.**

The provisions of the Community Development Code Chapter 99 have been met.



Darren Wyss, Associate Planner

May 14, 2019
Date

Appeals to this decision must be filed with the West Linn Planning Department within 14 days of mailing date. Cost is $400. An appeal to City Council of a decision by the Planning Director shall be heard on the record. The appeal must be filed by an individual who has established standing by submitting comments prior to the decision date. Approval will lapse 3 years from effective approval date if the final plat is not recorded.

Mailed this 15th day of May, 2019.

Therefore, the 14-day appeal period ends at 5 p.m., on May 29, 2019.

ADDENDUM
APPROVAL CRITERIA AND FINDINGS
MIP-18-05

This decision adopts the findings for approval contained within the applicant's submittal, with the following exceptions and additions:

I. CHAPTER 11, SINGLE FAMILY RESIDENTIAL DETACHED, R-10
11.030 PERMITTED USES
(...)

Staff Finding 1: Staff incorporates the findings found on page 2 in the applicant submittal (Exhibit PD-1). The criteria are met.

11.070 DIMENSIONAL REQUIREMENTS, USES PERMITTED OUTRIGHT AND USES PERMITTED UNDER PRESCRIBED CONDITIONS
1. The minimum lot size shall be 10,000 square feet for a single-family detached unit:

Staff Finding 2: The applicant proposes three parcels sized at 10,005 square feet for Parcel 1, 10,327 square feet for Parcel 2, and 10,040 square feet for Parcel 3. The criteria is met.

2. The minimum front lot line length or the minimum lot width at the front lot line shall be 35 feet.
3. The average minimum lot width shall be 50 feet.

Staff Finding 3: The applicant proposes Parcel 1 to have a front lot line width of 109.2 feet and average lot width of 109.2 feet. Parcel 2 is a flag lot and has a front lot line width of eight feet as allowed per CDC 85.200(B)(7) and an average lot width of approximately 56 feet. Parcel 3 is a flag lot and has a front lot line width of eight feet as allowed per CDC 85.200(B)(7) and an average lot width of approximately 77 feet. The criteria are met.

4. Repealed by Ord. 1622
(...)
10. The sidewall provisions of Chapter 43 CDC shall apply.

Staff Finding 4: Staff incorporates the findings found on pages 3-4 in the applicant submittal (Exhibit PD-1). The criteria are met.

II. CHAPTER 48, ACCESS CONTROL
48.025 ACCESS CONTROL
B. Access Control Standards
1. Traffic impact analysis requirements. The City or other agency with access jurisdiction may require a traffic study prepared by a qualified professional to determine access, circulation and other transportation requirements. (See also CDC 55.125, Traffic Impact Analysis.)

Staff Finding 5: No traffic impact analysis (TIA) is required since none of the criteria of 85.170.B(2) are met. For example, an Average Daily Trip count (ADT) increase of 250 is typically required before a TIA is needed. The addition of three additional/new homes generates an ADT increase of 28.71 trips per day according to the Institute of Traffic Engineers (ITE) trip generation manual. The criteria is met.

2. The City or other agency with access permit jurisdiction may require the closing or consolidation of existing curb cuts or other vehicle access points, recording of reciprocal access easements (i.e., for shared driveways), development of a frontage street, installation of traffic control devices, and/or other mitigation as a condition of granting an access permit, to ensure the safe and efficient operation of the street and highway system. Access to and from off-street parking areas shall not permit backing onto a public street.

Staff Finding 6: The proposal does not require backing onto a public street. All three proposed parcels, as well as neighboring Taxlot 9901, are owned by the applicant and will take access via the shared access easement as required by Condition of Approval 3. The applicant has volunteered to consolidate the access for Taxlot 9901 to meet driveway spacing requirements for Skyline Drive, a collector street. A reciprocal access easement and mutual maintenance agreement will be recorded and include the three proposed parcels and neighboring Taxlot 9901 per Condition of Approval 3. Subject to the Conditions of Approval, the criteria are met.

3. Access Options. When vehicle access is required for development (i.e., for off-street parking, delivery, service, drive-through facilities, etc.), access shall be provided by one of the following methods (planned access shall be consistent with adopted public works standards and TSP). These methods are "options" to the developer/subdivider.
a) Option 1. Access is from an existing or proposed alley or mid-block lane. If a property has access to an alley or lane, direct access to a public street is not permitted.
b) Option 2. Access is from a private street or driveway connected to an adjoining property that has direct access to a public street (i.e., "shared driveway"). A public access easement covering the driveway shall be recorded in this case to assure access to the closest public street for all users of the private street/drive.
c) Option 3. Access is from a public street adjacent to the development lot or parcel. If practicable, the owner/developer may be required to close or consolidate an existing access point as a condition of approving a new access. Street accesses shall comply with the access spacing standards in subsection (B)(6) of this section.

Staff Finding 7: Proposed Parcels 1, 2, and 3 will take access from Skyline Drive, a public street adjacent to the development. The proposed parcels will share access via an access easement. A reciprocal access easement and mutual maintenance agreement will be recorded per Condition of Approval 3. Subject to the Conditions of Approval, the criteria is met.

4. Subdivisions fronting onto an arterial street.
(...)
5. Double frontage lots.

Staff Finding 8: Staff incorporates the findings found on pages 16-17 in the applicant submittal (Exhibit PD-1). The criteria are met.

6. Access Spacing.
a. The access spacing standards found in the adopted TSP shall be applicable to all newly established public street intersections...variance section in the adopted TSP.
b. Private drives and other access ways are subject to the requirements of CDC 48.060.

Staff Finding 9: The applicant proposal does not include any new public street intersections. The proposal is for one private drive access to Skyline Drive, which will be shared by Parcels 1, 2, and 3. Please see Staff Findings 16 to 18 for compliance with CDC 48.060. The criteria are met.

7. Number of access points. For single-family (detached and attached), two-family, and duplex housing types, one street access point is permitted per lot or parcel, when alley access cannot otherwise be provided; except that two access points may be permitted corner lots…

Staff Finding 10: The applicant proposes one access to Skyline Drive, a public street, via a private drive located in an easement shared by Parcels 1, 2, and 3. The criteria are met.

8. Shared driveways. The number of driveway and private street intersections with public streets shall be minimized by the use of shared driveways with adjoining lots where feasible. (...)
a. Shared driveways and frontage streets may be required to consolidate access onto a collector or arterial street. When shared driveways or frontage streets are required, they shall be stubbed to adjacent developable parcels to indicate future extension. "Stub" means that a driveway or street temporarily ends at the property line, but may be extended in the future as the adjacent lot or parcel develops. "Developable" means that a lot or parcel is either vacant or it is likely to receive additional development (i.e., due to infill or redevelopment potential).
b. Access easements (i.e., for the benefit of affected properties) shall be recorded for all shared driveways, including pathways, at the time of final plat approval or as a condition of site development approval.
c. Exception. Shared driveways are not required when existing development patterns or physical constraints (e.g., topography, lot or parcel configuration, and similar conditions) prevent extending the street/driveway in the future

Staff Finding 11: Proposed Parcels 1, 2, and 3 will take access from Skyline Drive, a public street with a functional classification as a collector in the West Linn Transportation System Plan. The proposed parcels will share access via a shared private drive located in an access easement. A reciprocal access easement and mutual maintenance agreement will be recorded per Condition of Approval 3. All surrounding properties of the proposal are fully

developed and prevent extension of the shared access or the development of a new street. The criteria are met.

C. Street connectivity and formation of blocks required. In order to promote efficient vehicular and pedestrian circulation throughout the City, land divisions and large site developments shall produce complete blocks bounded by a connecting network of public and/or private streets, in accordance with the following standards:
1. Block length and perimeter. The maximum block length shall not exceed 800 feet or 1,800 feet along an arterial.
2. Street standards. Public and private streets shall also conform to Chapter 92 CDC, Required Improvements, and to any other applicable sections of the West Linn Community Development Code and approved TSP.
3. Exception. Exceptions to the above standards may be granted when blocks are divided by one or more pathway(s), in conformance with the provisions of CDC 85.200(C), Pedestrian and Bicycle Trails, or cases where extreme topographic (e.g., slope, creek, wetlands, etc.) conditions or compelling functional limitations preclude implementation, not just inconveniences or design challenges.

Staff Finding 12: The proposed land division includes no new public street and has one private street serving as access for four units of land. Adjacent properties on all sides are fully developed and provide a functional limitation to implementation of block length standards as no new block can be formed. The criteria are met.

48.030 MINIMUM VEHICULAR REQUIREMENTS FOR RESIDENTIAL USES
A. Direct individual access from single-family dwellings and duplex lots to an arterial street, as designated in the transportation element of the Comprehensive Plan (...)

Staff Finding 13: Staff incorporates the findings found on pages 18-19 in the applicant submittal (Exhibit PD-1). The criteria are met.

B. When any portion of any house is less than 150 feet from the adjacent right-of-way, access to the home is as follows:
(...)
2. Two to four single-family residential homes equals a 14 to 20 foot-wide paved or all-weather surface. Width shall depend on adequacy of line of sight and number of homes.
3. Maximum driveway grade shall be 15 percent…
4. The driveway shall include a minimum of 20 feet in length between the garage door and the back of sidewalk, or, if no sidewalk is proposed, to the paved portion of the right-of-way.

C. When any portion of one or more homes is more than 150 feet from the adjacent right-of-way, the provisions of subsection B of this section shall apply in addition to the following provisions.
1. A turnaround may be required as prescribed by the Fire Chief.
2. Minimum vertical clearance for the driveway shall be 13 feet, six inches.
3. A minimum centerline turning radius of 45 feet is required unless waived by the Fire Chief.

4. There shall be sufficient horizontal clearance on either side of the driveway so that the total horizontal clearance is 20 feet.

Staff Finding 14: The applicant proposes a shared access drive via easement for Parcels 1, 2, and 3, as well as the neighboring property (Taxlot 9901), per the Preliminary Plat found on page 173 of Exhibit PD-1 and Condition of Approval 3. A new home located on proposed Parcel 1 and the neighboring property (Taxlot 9901) will be less than 150 feet from Skyline Drive. New homes located on proposes Parcels 2 and 3 will be more than 150 feet from Skyline Drive. The access drive grade will be 7.3 percent. The applicant proposal does not contain any structures or trees that impede the 13 foot 6 inch vertical clearance. The applicant shall install a shared access drive located in the access easement per Condition of Approval 4. No structures are located adjacent to the shared access drive and a total horizontal clearance of 20 feet will be maintained. Tualatin Valley Fire & Rescue will require appropriate turnaround infrastructure during building permit review. Subject to the Conditions of Approval, the criteria are met.

D. Access to five or more single-family homes...waived by variance.
(...)
I. Gated accessways to residential development other than a single-family home are prohibited.

Staff Finding 15: Staff incorporates the findings found on page 20 in the applicant submittal (Exhibit PD-1). The criteria are met.

48.060 WIDTH AND LOCATION OF CURB CUTS AND ACCESS SEPARATION REQUIREMENTS
A. Minimum curb cut width shall be 16 feet.
B. Maximum curb cut width shall be 36 feet...
(...)
C. No curb cuts shall be allowed any closer to an intersecting street right-of-way line than the following:
(...)
5. On a collector when intersected by another collector or local street, 35 feet.
(...)
D. There shall be a minimum distance between any two adjacent curb cuts on the same side of a public street, except for one-way entrances and exits, as follows:
(…)
2. On a collector street, 75 feet.
(…)

Staff Finding 16: The applicant proposes one 24 foot curb cut to accommodate the shared access drive. The proposed curb cut is located on Skyline Drive, a collector, and the closest intersecting street, Firwood Drive, is a local street and located 205 feet to the west. The adjacent property (Taxlot 9901) currently has a temporary access from Skyline Drive during home construction. The temporary access to Taxlot 9901 will be closed after completion of

the shared access drive per Condition of Approval 4. Subject to the Conditions of Approval, the criteria are met.

E. A rolled curb may be installed...
F. Curb cuts shall be kept at a minimum...

Staff Finding 17: Staff incorporates the findings found on page 22 in the applicant submittal (Exhibit PD-1). The criteria are met.

G. Adequate line of sight pursuant to engineering standards should be afforded at each driveway or accessway.

Staff Finding 18: The applicant proposes one shared access drive and vision clearance will be confirmed by planning staff during the building permit process. The criteria are met.

III. CHAPTER 85, GENERAL PROVISIONS
85.080 SUBSTANTIAL DEVIATTION FROM APPROVED PLAN PROHIBITED
A. Approval of the tentative plan shall require the final plat to be in substantial conformance...however
B. Approval of the tentative plan...shall not constitute final acceptance of the plat of the proposed subdivision or partition for recording.

Staff Finding 19: The City will ensure the final plat substantially conforms to the approved tentative plan by satisfaction of Condition of Approval 1. The criteria are met.

85.200 APPROVAL CRITERIA
No tentative subdivision or partition plan shall be approved unless adequate public facilities will be available to provide service to the partition or subdivision area prior to final plat approval and the Planning Commission or Planning Director, as applicable, finds that the following standards have been satisfied, or can be satisfied by condition of approval.
A. Streets.
1. General. The location, width and grade of streets shall be considered in their relation to existing and planned streets...Internal streets are the responsibility of the developer. All streets bordering the development site are to be developed by the developer with, typically, half-street improvements or to City standards prescribed by the City Engineer. Additional travel lanes may be required to be consistent with adjacent road widths or to be consistent with the adopted Transportation System Plan (TSP) and any adopted updated plans.

Staff Finding 20: The proposal does not include any internal public streets. The applicant proposes installation of half-street improvements along the property's Skyline Drive frontage to meet the cross-section for a Collector without Median/Center Lane per City of West Linn Public Works Standards. The criteria is met.

2. Right-of-way widths shall depend upon which classification of street is proposed. The right-of-way widths are established in the adopted TSP.

Staff Finding 21: The proposal is located on Skyline Drive, a collector street that has a 54 foot right-of-way width adjacent to the subject property. City standards for a Collector without Median/Center Lane are found in Exhibit 7 of the 2016 West Linn Transportation System Plan. The standards require a 58 foot right-of-way (6 foot sidewalks, 6 foot planter strips, 5 foot bike lane, and 12 foot travel lanes). No additional right-of-way dedication is required as the presence of Wilderness Park across Skyline Drive dictated a curb-tight sidewalk, thus reducing the right-of-way need by six-feet. The criteria is met.

3. Street widths. Street widths shall depend upon which classification of street is proposed. The classifications and required cross sections are established in the adopted TSP…

Staff Finding 22: The proposal is located on Skyline Drive, a collector street that has a 54 foot right-of-way width adjacent to the subject property. City standards for a Collector without Median/Center Lane are found in Exhibit 7 of the 2016 West Linn Transportation System Plan. The standards require a 58 foot right-of-way (6 foot sidewalks, 6 foot planter strips, 5 foot bike lane, and 12 foot travel lanes). No additional right-of-way dedication is required as the presence of Wilderness Park across Skyline Drive dictated a curb-tight sidewalk, thus reducing the right-of-way need by six-feet. The criteria is met.

4. The decision-making body shall consider the City Engineer's recommendations on the desired right-of-way width, pavement width and street geometry of the various street types within the subdivision after consideration by the City Engineer of the following criteria:
(…)
5. Additionally, when determining appropriate street width, the decision-making body shall consider the following criteria:
(...)

Staff Finding 23: The proposal is located on Skyline Drive, a collector street that has a 54 foot right-of-way width adjacent to the subject property. City standards for a Collector without Median/Center Lane are found in Exhibit 7 of the 2016 West Linn Transportation System Plan. The standards require a 58 foot right-of-way (6 foot sidewalks, 6 foot planter strips, 5 foot bike lane, and 12 foot travel lanes). No additional right-of-way dedication is required as the presence of Wilderness Park across Skyline Drive dictated a curb-tight sidewalk, thus reducing the right-of-way need by six-feet. The criteria are met.

6. Reserve strips. Reserve strips or street plugs controlling the access to streets are not permitted unless owned by the City.
7. Alignment. All streets other than local streets or cul-de-sacs, as far as practical, shall be in alignment with existing streets by continuations of the centerlines thereof. The staggering of street alignments...

8. Future extension of streets. Where necessary to give access to or permit a satisfactory future subdivision of adjoining land, streets shall be extended to the boundary of the subdivision...
9. Intersection angles. Streets shall be laid out to intersect angles as near to right angles as practical...

Staff Finding 24: The applicant proposes no reserve strips, no new streets or intersections, and is precluded from any potential future extension of streets by existing development patterns. The criteria are met.

10. Additional right-of-way for existing streets. Wherever existing street rights-of-way adjacent to or within a tract are of inadequate widths based upon the standards of this chapter, additional right-of-way shall be provided at the time of subdivision or partition.

Staff Finding 25: The proposal is located on Skyline Drive, a collector street that has a 54 foot right-of-way width adjacent to the subject property. City standards for a Collector without Median/Center Lane are found in Exhibit 7 of the 2016 West Linn Transportation System Plan. The standards require a 58 foot right-of-way (6 foot sidewalks, 6 foot planter strips, 5 foot bike lane, and 12 foot travel lanes). No additional right-of-way dedication is required as the presence of Wilderness Park across Skyline Drive dictated a curb-tight sidewalk, thus reducing the right-of-way need by six-feet. The criteria is met.

11. Cul-de-sacs.
a. New cul-de-sacs and other closed-end streets...
(...)
f. All cul-de-sacs/closed-end streets shall terminate with a turnaround built to one of the following specifications (measurements are for the traveled way and do not include planter strips or sidewalks).

Staff Finding 26: The applicant does not propose any new cul-de-sacs or closed-end streets. The criteria are met.

12. Street Names
13. Grades and Curves
14. Access to local streets. Intersection of a local residential street with an arterial street may be prohibited...
15. Alleys

Staff Finding 27: The applicant does not propose any new public streets. The proposed partition is adjacent to Skyline Drive, a City street with a functional classification of collector, and does not require access on to an arterial street. No alley is proposed. The applicant shall install a shared access drive located in the access easement per Condition of Approval 4. Tualatin Valley Fire & Rescue will require appropriate turnaround infrastructure prior to final plat approval. Subject to the Conditions of Approval, the criteria are met.

16. Sidewalks. Sidewalks shall be installed per CDC 92.010(H), Sidewalks. The residential sidewalk width is six feet plus planter strip…or to match existing sidewalks or right-of-way limitations.
17. Planter strip. The planter strip is between the curb and sidewalk providing space for a grassed or landscaped area and street trees. The planter strip shall be at least 6 feet wide…or in response to right-of-way limitations.

Staff Finding 28: The applicant will install 6 foot sidewalk and 6 foot planter strip along the subject property's frontage of Skyline Drive. The criteria are met.

18. Streets and roads shall be dedicated without any reservations or restrictions.
19. All lots in a subdivision shall have access to a public street. Lots created by partition may have access to a public street via an access easement pursuant to the standards and limitations set forth for such accessways in Chapter 48 CDC.

Staff Finding 29: The application is for a partition of an existing parcel. The applicant proposes a shared private access drive, located in an access easement, to Skyline Drive for the three newly created Parcels. Please see Applicant findings as well as Staff Findings 5 through 18. No new streets or roads are proposed. Subject to the Conditions of Approval, the criteria is met.

20. Gated streets. Gated streets are prohibited in all residential areas on both public and private streets. A driveway to an individual home may be gated.
21. Entryway treatments and street isle design...

Staff Finding 30: The applicant does not propose any gated streets or driveways, nor entryway treatments or street isle designs. These criteria are met.

22. Based upon the determination of the City Manager or the Manager's designee, the applicant shall construct or cause to be constructed, or contribute a proportionate share of the costs, for all necessary off-site improvements identified by the transportation analysis commissioned to address CDC 85.170(B)(2) that are required to mitigate impacts from the proposed subdivision…

Staff Finding 31: The proposal is not a subdivision. This criteria does not apply. However, the proposal will add three additional dwelling units. The addition of three additional dwelling units should only generate an ADT of 28.71 new trips per day according to the Institute of Traffic Engineers (ITE) trip generation tables at 9.57 trips per household. The proposal is in compliance with the City's Transportation System Plan and will not create impacts to existing off-site facilities that would trigger the need for improvements. The criteria is met.

B. Blocks and Lots.
1. General. The length, width, and shape of blocks shall be designed with due regard for the provision of adequate building sites for the use contemplated; consideration of the need for traffic safety, convenience, access, circulation, and control; and recognition of limitations and opportunities of topography and solar access.
2. Sizes. The recommended block size is 400 feet in length to encourage greater connectivity within the subdivision. Blocks shall not exceed 800 feet in length between street lines... Block sizes and proposed accesses must be consistent with the adopted TSP.

Staff Finding 32: The applicant does not propose new street connections or new blocks. Existing development patterns preclude a new street connection between across the subject property. The criteria are met.

3. Lot size and shape. Lot or parcel size, width, shape, and orientation shall be appropriate for the location of the subdivision or partition... Depth and width of properties reserved or laid out for commercial and industrial purposes shall be adequate to provide for the off-street parking and service facilities required by the type of use proposed.

Staff Finding 33: Staff incorporates the findings found on page 11 in the applicant submittal (Exhibit PD-1). Please also see Staff Findings 2 through 3. The criteria are met.

4. Access. Access to subdivisions, partitions, and lots shall conform to the provisions of Chapter 48 *CDC, Access, Egress and Circulation.*

Staff Finding 34: Please see Staff Findings 5 through 18. Subject to the Conditions of Approval, the criteria is met.

5. Double Frontage lots and parcels.
6. Lot and parcel side lines.

Staff Finding 35: Staff incorporates the findings found on page 12 in the applicant submittal (Exhibit PD-1). The criteria are met.

7. Flag lots. Flag lots can be created where it can be shown that no other reasonable street access is possible to achieve the requested land division...Where two to four flag lots share a common accessway, the minimum street frontage and accessway shall be eight feet in width per lot...The following dimensional requirements shall apply to flag lots:
a. Setbacks applicable to the underlying zone shall apply to the flag lot.
(...)
f. If the use of a flag lot stem to access a lot is infeasible... access easement of a minimum 15-foot width across intervening property

Staff Finding 36: Staff incorporates the findings found on pages 12-13 in the applicant submittal (Exhibit PD-1). The proposed flag lots are the only reasonable street access and the

stems will be 8 feet wide for Parcels 2 and 3 at the Skyline Drive right-of-way. The applicant proposal includes an access easement over the flag lot stems. The applicant is required to construct a 14 to 20 foot wide shared access private drive and record a mutual maintenance and reciprocal access agreement per Conditions of Approval 3 and 4. Subject to the Conditions of Approval, the criteria are met.

8. Large lots or parcels.

Staff Finding 37: Staff incorporates the findings found on page 13 in the applicant submittal (Exhibit PD-1). The criteria are met.

C. Pedestrian and bicycle trails.
(…)
D. Transit Facilities
(...)

Staff Finding 38: The West Linn Transportation System Plan does not identify any pedestrian or bicycle facilities on or adjacent to the subject property. The closest bus route is more than one-half mile distance and there are currently no adopted plans to add transit service to Skyline Drive. The applicant is not required to install any improvements outside of required street improvements. The criteria are met.

E. Grading. Grading of building sites shall conform to the following standards unless physical conditions demonstrate the propriety of other standards:
1. All cuts and fills shall comply with the excavation and grading provisions of the Uniform Building Code
(...)
4. The proposed grading shall be the minimum grading necessary to meet roadway standards, and to create appropriate building sites, considering maximum allowed driveway grades.

Staff Finding 39: Staff incorporates the findings found on pages 26-30 in the applicant submittal (Exhibit PD-1). The applicant submitted a Soils Investigation Report prepared by GeoPacific Engineering, Inc. The GeoPacific Report made specific recommendations to remediate any potential for geotechnical hazards related to grading, undocumented fill, and stormwater disposal. The applicant, per Condition of Approval 5, is required to submit the GeoPacific Report as part of the site development review/building permit application and shall provide any supplemental reports required by the Building Official. The report must be submitted prior to application for site development review/building permits. The report found all three proposed parcels to be buildable. Subject to the Conditions of Approval, the criteria are met.

5. Type I lands shall require a report submitted by an engineering geologist, and Type I and Type II lands shall require a geologic hazard report.

6. Per the submittals required by CDC 85.170(C)(3), the applicant must demonstrate that the proposed methods of rendering known or potential hazard sites safe for development, including proposed geotechnical remediation, are feasible and adequate to prevent landslides or other damage to property and safety. The review authority may impose conditions, including limits on type or intensity of land use, which it determines are necessary to mitigate known risks of landslides or property damage.

Staff Finding 40: The subject property is 53 percent Type III lands. The applicant submitted a Geotechnical Report prepared by GRI that includes general site characteristics, geologic descriptions, and a review of the West Linn Natural Hazard Mitigation Plan's applicability for the area surrounding the Bolton Reservoir property. The applicant also submitted a Soils Investigation Report prepared by GeoPacific Engineering, Inc. The GeoPacific Report made specific recommendations to remediate any potential for geotechnical hazards related to grading, undocumented fill, and stormwater disposal. The applicant, per Condition of Approval 5, is required to submit the GeoPacific Report as part of the site development review/building permit application and shall provide any supplemental reports required by the Building Official. The report must be submitted prior to application for site development review/building permits. The report found all three proposed parcels to be buildable. Subject to the Conditions of Approval, the criteria are met.

F. Water.

Staff Finding 41: Water is available in Skyline Drive to serve the proposed development. A registered civil engineer prepared the water provision plan. The City's public water system has sufficient capacity and pressure in this area. The criteria are met.

G. Sewer.

Staff Finding 42: The applicant has submitted a plan prepared by a registered civil engineer that will gravity flow to the existing sanitary sewer line in Firwood Court. The applicant proposes to extend a public sewer line from Firwood Court through an existing public utility easement between 5147 and 5127 Firwood Court to the boundary of the subject property. The new public sewer line will be located in new easements on the subject property. The easement locations are shown on the Partition Site Plan and will be recorded as part of the partition approval. The City public sanitary sewer system has sufficient capacity to service the proposed use. The applicant shall comply with the requirements and install improvements to meet the West Linn Public Works Design Standards per Condition of Approval 2. Subject to the Conditions of Approval, the criteria are met.

H. Storm detention and treatment. All proposed storm detention and treatment facilities comply with the standards for the improvement of public and private drainage systems located in the West Linn Public Works Design Standards, there will be no adverse off-site impacts caused by the development (including impacts from increased intensity of runoff downstream or

constrictions causing ponding upstream), and there is sufficient factual data to support the conclusions of the submitted plan.

Staff Finding 43: The applicant has submitted a Preliminary Drainage Report, prepared by a licensed engineer, which complies with the West Linn Public Works Design Standards, shows no adverse off-site impacts, and provides sufficient factual data to support the conclusions of the plan. The applicant also submitted a Soils Investigation Report prepared by GeoPacific Engineering, Inc. The GeoPacific Report made specific recommendations to remediate any potential for geotechnical hazards related to grading, undocumented fill, and stormwater disposal. The applicant shall comply with the requirements and install improvements to meet the West Linn Public Works Design Standards per Condition of Approval 2. Subject to the Conditions of Approval, the criteria are met.

I. Utility Easements. Subdivisions and partitions shall establish utility easements to accommodate the required service providers as determined by the City Engineer. The developer of the subdivision shall make accommodation for cable television wire in all utility trenches and easements so that cable can fully serve the subdivision.

Staff Finding 44: The applicant will record a reciprocal access easement per Condition of Approval 3. Per the Partition Site Plan, the applicant will record a new eight-foot wide public utility easement adjacent to the proposal's entire frontage of Skyline Drive. The applicant will also add four-feet to a portion of the existing public utility easement located along the rear property line and record a 15 foot sanitary sewer easement as shown on the Partition Site Plan. Subject to the Conditions of Approval, the criteria is met.

J. Supplemental Provisions
1. Wetland and Natural Drainage Ways.
2. Willamette and Tualatin Greenways.
3. Street trees. Street trees are required as identified in the appropriate section of the municipal code and Chapter 54 CDC.
4. Lighting. All subdivision or alley lights shall meet West Linn Public Works Design Standards.

Staff Finding 45: Staff incorporates the findings found on page 14 in the applicant submittal (Exhibit PD-1). The criteria are met.

5. Dedications and exactions.
The City may require an applicant to dedicate land and/or construct a public improvement that provides a benefit to property or persons outside the property that is the subject of the application when the exaction is roughly proportional. No exaction shall be imposed unless supported by a determination that the exaction is roughly proportional to the impact of development.

Staff Finding 46: Skyline Drive has sufficient right-of-way width to accommodate required street improvements. The proposal is in compliance with the City's Transportation System

Plan and will not create impacts to existing off-site facilities that would trigger the need for improvements. The City's sanitary sewer and water systems have sufficient capacity to serve the site. The criteria are met.

6. Underground utilities.
All utilities…that may at times be above ground or overhead shall be buried underground in the case of new development. The exception would be in those cases where the area is substantially built out and adjacent properties have above-ground utilities and where the development site's frontage is under 200 feet and the site is less than one acre…

Staff Finding 47: The subject property meets all three exemption criteria. The area is built out with adjacent properties having above-ground utilities, 125 feet of site frontage, and less than an acre (35,569 square feet). The applicant is not required to underground existing utilities. This criterion is met.

7. Density requirement. Density shall occur at 70 percent or more of the maximum density allowed by the underlying zoning. These provisions would not apply when density is transferred from Type I and II lands as defined in CDC 02.030. Development of Type I or II lands are exempt from these provisions. Land divisions of three lots or less would also be exempt.
8. Mix requirement. The "mix" rule means that developers shall have no more than 15 percent of the R-2.1 and R-3 development as single-family residential. The intent is that the majority of the site shall be developed as medium high density multi-family housing.

Staff Finding 48: The subject property is 35,569 square feet. The property contains no Type I or II lands. The property is zoned R-10, which requires a minimum 10,000 square feet per parcel. The maximum number of parcels that can be created from the subject property is three (35,569/10,000), thus the proposal is for 100% of the maximum density. The "mix" rule does not apply. The criteria are met.

9. Heritage trees/significant tree and tree cluster protection.
All heritage trees, as defined in the Municipal Code, shall be saved. Diseased heritage trees, as determined by the City Arborist, may be removed at his/her direction. All non-heritage trees and clusters of trees (three or more trees with overlapping dripline; however, native oaks need not have an overlapping dripline) that are considered significant by virtue of their size, type, location, health, or numbers shall be saved pursuant to CDC 55.100(B)(2). Trees are defined per the municipal code as having a trunk six inches in diameter or 19 inches in circumference at a point five feet above the mean ground level at the base of the trunk.

Staff Finding 49: The subject property contains no heritage trees. The City Arborist identified a grove of significant trees on proposed Parcel 3. The applicant proposes to remove 16 of the 17 significant trees. The applicant submitted an Arborist Report prepared by Portland Tree Consulting that concluded any adverse effect on the northern neighboring property will be negligible. Staff also incorporates the findings found in the applicant submittal (Exhibit PD-1). The criteria are met.

V. CHAPTER 92, REQUIRED IMPROVEMENTS
92.020 IMPROVEMENTS IN PARTITIONS
The same improvements shall be installed to serve each lot of a partition as are required of a subdivision. However, if the approval authority finds that the nature of development in the vicinity of the partition makes installation of some improvements unreasonable, at the written request of the applicant those improvements may be waived. If the street improvement requirements are waived, the applicant shall pay an in-lieu fee for off-site street improvements, pursuant to the provisions of CDC 85.200(A)(1).

In lieu of accepting an improvement, the Planning Director may recommend to the City Council that the improvement be installed in the area under special assessment financing or other facility extension policies of the City.

Staff Finding 50: The applicant proposes to install half-street improvements, which meet the City of West Linn Public Works Standards, on Skyline Drive adjacent to the subject property. The applicant will install the half-street improvements to meet the cross-section for a Collector without Median/Center Lane per the 2016 West Linn Transportation System Plan. Since the applicant has agreed to install the improvements, no nexus and proportionality analysis is required. The criteria are met.

PD-1 APPLICANT SUBMITTAL

City of West Linn
Planning & Development • 22500 Salamo Rd #1000 • West Linn, Oregon 97068
Telephone 503.656.4211 • Fax 503.656.4106 • westlinnoregon.gov

DEVELOPMENT REVIEW APPLICATION

For Office Use Only		
STAFF CONTACT Jennifer Arnold	PROJECT NO(S). MIP-18-05	
NON-REFUNDABLE FEE(S) 500-	REFUNDABLE DEPOSIT(S) 2800	TOTAL 3300

Type of Review (Please check all that apply):

- ☐ Annexation (ANX)
- ☐ Appeal and Review (AP) *
- ☐ Conditional Use (CUP)
- ☐ Design Review (DR)
- ☐ Easement Vacation
- ☐ Extraterritorial Ext. of Utilities
- ☐ Final Plat or Plan (FP)
- ☐ Flood Management Area
- ☐ Hillside Protection & Erosion Control
- ☐ Historic Review
- ☐ Legislative Plan or Change
- ☐ Lot Line Adjustment (LLA) */**
- ☒ Minor Partition (MIP) (Preliminary Plat or Plan)
- ☐ Non-Conforming Lots, Uses & Structures
- ☐ Planned Unit Development (PUD)
- ☐ Pre-Application Conference (PA) */**
- ☐ Street Vacation
- ☐ Subdivision (SUB)
- ☐ Temporary Uses *
- ☐ Time Extension *
- ☐ Variance (VAR)
- ☐ Water Resource Area Protection/Single Lot (WAP)
- ☐ Water Resource Area Protection/Wetland (WAP)
- ☐ Willamette & Tualatin River Greenway (WRG)
- ☐ Zone Change

Home Occupation, Pre-Application, Sidewalk Use, Sign Review Permit, and Temporary Sign Permit applications require different or additional application forms, available on the City website or at City Hall.

Site Location/Address:		
6123 Skyline Drive	Assessor's Map No.:	2S 1E 25AD
	Tax Lot(s):	9900
	Total Land Area:	0.75 acres

Brief Description of Proposal:

Partition application to create three parcels for construction of single-family homes.

Applicant Name: (please print)	Icon Construction & Development, LLC	Phone:	(503) 657-0406
Address:	1980 Willamette Falls Drive, Suite 200	Email:	mark@iconconstruction.net
City State Zip:	West Linn, OR 97068		
Owner Name (required): (please print)	City of West Linn	Phone:	503-657-0331
Address:	22500 Salamo Road	Email:	
City State Zip:	West Linn OR 97068		
Consultant Name: (please print)	Rick Givens, Planning Consultant	Phone:	503-479-0097
Address:	18680 Sunblaze Dr.	Email:	rickgivens@gmail.com
City State Zip:	Oregon City, OR 97045		

1. All application fees are non-refundable (excluding deposit). **Any overruns to deposit will result in additional billing.**
2. The owner/applicant or their representative should be present at all public hearings.
3. A denial or approval may be reversed on appeal. No permit will be in effect until the appeal period has expired.
4. **Three (3) complete hard-copy sets (single sided) of application materials must be submitted with this application. One (1) complete set of digital application materials must also be submitted on CD in PDF format. If large sets of plans are required in application please submit only two sets.**

* No CD required / ** Only one hard-copy set needed

The undersigned property owner(s) hereby authorizes the filing of this application, and authorizes on site review by authorized staff. I hereby agree to comply with all code requirements applicable to my application. Acceptance of this application does not infer a complete submittal. All amendments to the Community Development Code and to other regulations adopted after the application is approved shall be enforced where applicable. Approved applications and subsequent development is not vested under the provisions in place at the time of the initial application.

Applicant's signature — Date: 9/19/18

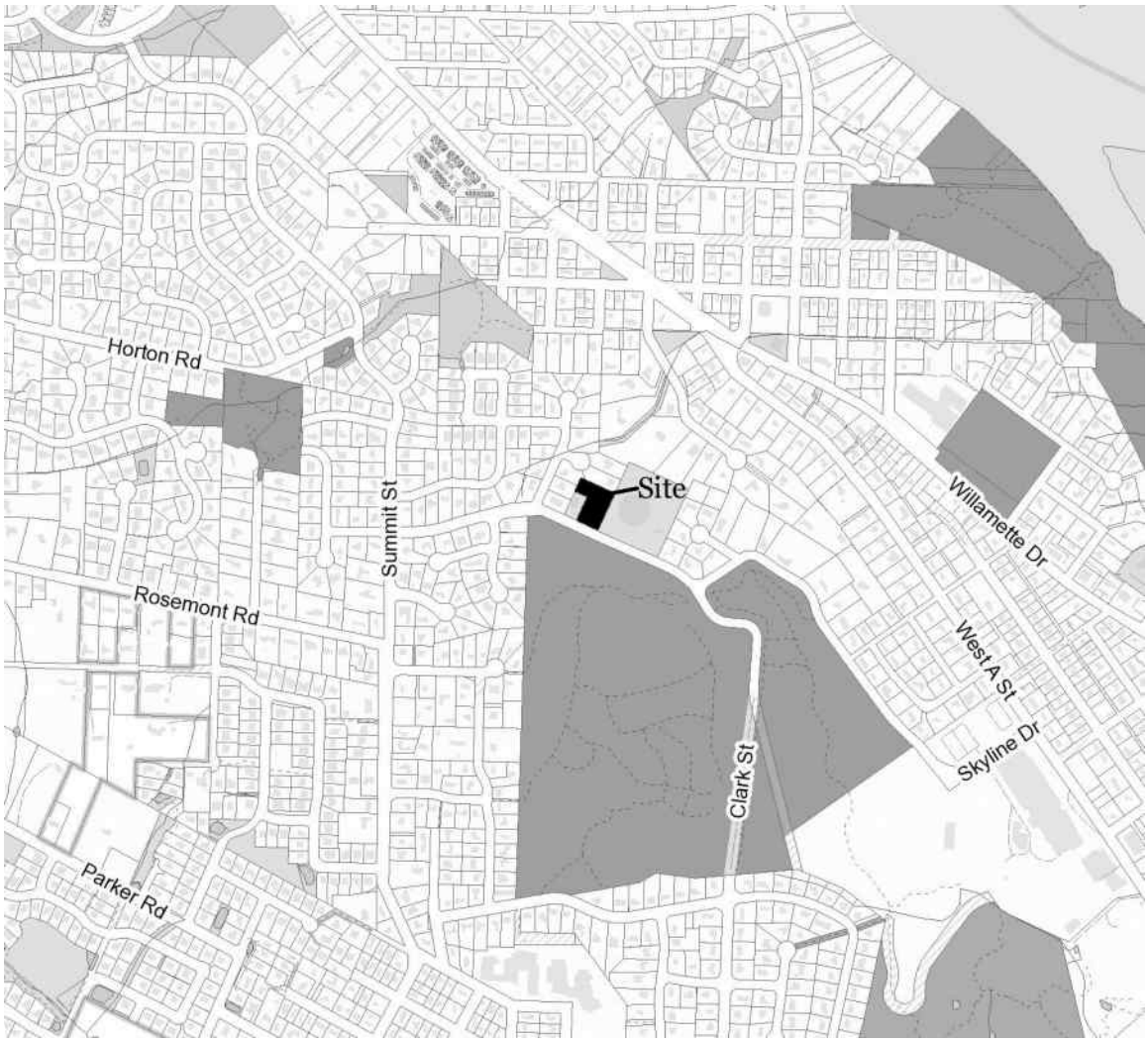
Owner's signature *(required)* — Date: 10/2/18

Partition Narrative

6123 Skyline Dr., West Linn

Icon Construction & Development, LLC

Proposal: This application requests approval of a three-lot partition for property located at 6123 Skyline Dr. in West Linn. The property is situated on the north side of the street, Firwood Drive and Clark Street. The subject property is 0.75 acres in area and is vacant. The City of West Linn's water reservoir is located immediately to the east of the subject property.The proposed partition will divide the property into 3 parcels, with two of the lots being flag lots situated behind the lot fronting directly onto Skyline Dr. The subject property is zoned R-10. The property is described as Tax Lot 9900 of Clackamas County Assessor's Map 2-1E-25AD.



Vicinity Map

The proposed development conforms to the applicable provisions of the CDC as follows:

CHAPTER 11 SINGLE-FAMILY RESIDENTIAL DETACHED, R-10

11.030 PERMITTED USES

The following are uses permitted outright in this zoning district

1. Single-family detached residential unit. (….)

Comment: The application is for the creation of three parcels to accommodate three new single-family detached residences. This use is permitted use by this section. The criterion is met.

11.070 DIMENSIONAL REQUIREMENTS, USES PERMITTED OUTRIGHT AND USES PERMITTED UNDER PRESCRIBED CONDITIONS

Except as may be otherwise provided by the provisions of this code, the following are the requirements for uses within this zone:

1. The minimum lot size shall be 10,000 square feet for a single-family detached unit.

Comment: As shown on the site plan, all three parcels exceed the 10,000 sq. ft. minimum lot size. This criterion is met.

2. The minimum front lot line length or the minimum lot width at the front lot line shall be 35 feet.

Comment: Parcel 1 has a front lot line length of 110 feet, which exceeds the minimum standard of 35 feet. Parcels 2 and 3 meet the minimum flag lot stem width per CDC 85.200 (B) (7) and comply with the 35’ width requirement at the building line.

3. The average minimum lot width shall be 50 feet.

Comment: All three parcels exceed the minimum lot width standard. This standard is met.

4. Repealed by Ord. 1622.

5. *Except as specified in CDC 25.070(C)(1) through (4) for the Willamette Historic District, the minimum yard dimensions or minimum building setback area from the lot line shall be:*

    a. *For the front yard, 20 feet; except for steeply sloped lots where the provisions of CDC 41.010 shall apply.*
    b. *For an interior side yard, seven and one-half feet.*
    c. *For a side yard abutting a street, 15 feet.*
    d. *For a rear yard, 20 feet.*

Comment: The property is not in the Willamette Historic District. Setbacks for the homes to be constructed on these lots will conform to these standards and will be reviewed for compliance at the time of building permit application.

6. *The maximum building height shall be 35 feet, except for steeply sloped lots in which case the provisions of Chapter 41 CDC shall apply.*

Comment: Building height for the new home will comply with the height standard and will be reviewed for compliance with the building permit application.

7. *The maximum lot coverage shall be 35 percent.*

Comment: Lot coverage for the homes to be built on these parcels will comply with this standard, as will be demonstrated at the time of building permit application.

8. *The minimum width of an accessway to a lot which does not abut a street or a flag lot shall be 15 feet.*

Comment: The accessway to Parcels 2 and 3 measures 16 feet in width.

9. *The floor area ratio shall be 0.45. Type I and II lands shall not be counted toward lot area when determining allowable floor area ratio, except that a minimum floor area ratio of 0.30 shall be allowed regardless of the classification of lands within the property. That 30 percent shall be based upon the entire property including Type I and II lands. Existing residences in excess of this standard may be replaced to their prior dimensions when damaged without the requirement that the homeowner obtain a non-conforming structures permit under Chapter 66 CDC.*

Comment: The floor area for the new homes to be built on these parcels will comply with this standard. Compliance will be reviewed with the building permit.

10. *The sidewall provisions of Chapter 43 CDC shall apply.*

Comment: Compliance of the new home with the provisions of Chapter 43 will be reviewed with the building permit.

Chapter 85 GENERAL PROVISIONS (Land Division)

85.200 APPROVAL CRITERIA

No tentative subdivision or partition plan shall be approved unless adequate public facilities will be available to provide service to the partition or subdivision area prior to final plat approval and the Planning Commission or Planning Director, as applicable, finds that the following standards have been satisfied, or can be satisfied by condition of approval.

A. Streets.

1. General. The location, width and grade of streets shall be considered in their relation to existing and planned streets, to the generalized or reasonable layout of streets on adjacent undeveloped lots or parcels, to topographical conditions, to public convenience and safety, to accommodate various types of transportation (automobile, bus, pedestrian, bicycle), and to the proposed use of land to be served by the streets. The functional class of a street aids in defining the primary function and associated design standards for the facility. The hierarchy of the facilities within the network in regard to the type of traffic served (through or local trips), balance of function (providing access and/or capacity), and the level of use (generally measured in vehicles per day) are generally dictated by the functional class. The street system shall assure an adequate traffic or circulation system with intersection angles, grades, tangents, and curves appropriate for the traffic to be carried. Streets should provide for the continuation, or the appropriate projection, of existing principal streets in surrounding areas and should not impede or adversely affect development of adjoining lands or access thereto.

To accomplish this, the emphasis should be upon a connected continuous pattern of local, collector, and arterial streets rather than discontinuous curvilinear streets and cul-de-sacs. Deviation from this pattern of connected streets should only be permitted in cases of extreme topographical challenges including excessive slopes (35 percent-plus), hazard areas, steep drainageways, wetlands, etc. In such cases, deviations may be allowed but the connected continuous pattern must be reestablished once the topographic challenge is passed. Streets should be oriented with consideration of the sun, as site conditions allow, so that over 50 percent of the front building lines of homes are oriented within 30 degrees of an east-west axis.

Internal streets are the responsibility of the developer. All streets bordering the development site are to be developed by the developer with, typically, half-street improvements or to City standards prescribed by the City Engineer. Additional travel lanes may be required to be consistent with adjacent road widths or to be consistent with the adopted Transportation System Plan (TSP) and any adopted updated plans.

An applicant may submit a written request for a waiver of abutting street improvements if the TSP prohibits the street improvement for which the waiver is requested. Those areas with numerous (particularly contiguous) under-developed or undeveloped tracts will be required to install street improvements. When an applicant requests a waiver of street improvements and the waiver is granted, the applicant shall pay an in-lieu fee equal to the estimated cost, accepted by the City Engineer, of the otherwise required street improvements. As a basis for this determination, the City Engineer shall consider the cost of similar improvements in recent development projects and may require up to three estimates from the applicant. The amount of the fee shall be established prior to the Planning Commission's decision on the associated application. The in-lieu fee shall be used for in kind or related improvements.

Streets shall also be laid out to avoid and protect tree clusters and significant trees, but not to the extent that it would compromise connectivity requirements per this subsection (A)(1), or bring the density below 70 percent of the maximum density for the developable net area. The developable net area is calculated by taking the total site acreage and deducting Type I and II lands; then up to 20 percent of the remaining land may be excluded as necessary for the purpose of protecting significant tree clusters or stands as defined in CDC 55.100(B)(2).

Comment: The subject property is an infill property, with properties to the north, east and west being fully developed without any street stubs provided to this site. To the west and north, R-10 lots abut the site within the Bridge View Estates plat and are developed with single-family homes. The City water reservoir is adjacent to the site along its east boundary. Skyline Drive forms the south boundary. As a result of this preexisting development pattern, it is not feasible to provide for greater street connectivity from this site. The proposed shared private driveway from Skyline Drive provides for access to all three parcels. Skyline Drive is fully improved to collector street standards along the project's frontage, except for sidewalks. Sidewalks will be provided in conjunction with the construction of a home on Parcel 1.

2. Right-of-way widths shall depend upon which classification of street is proposed. The right-of-way widths are established in the adopted TSP.

Comment: Skyline Drive is listed as a collector street in the West Linn TSP. The existing 60' right-of-way along the site's frontage is consistent with collector street standards.

3. Street widths. Street widths shall depend upon which classification of street is proposed. The classifications and required cross sections are established in the adopted TSP.

The following table identifies appropriate street width (curb to curb) in feet for various street classifications. The desirable width shall be required unless the applicant or his or her engineer can demonstrate that site conditions, topography, or site design require the reduced minimum width. For local streets, a 12-foot travel lane may only be used as a shared local street when the available right-of-way is too narrow to accommodate bike lanes and sidewalks.

Comment: The existing street section along Skyline Drive provides for 30 feet of right-of-way and 18' of paving from centerline, allowing for a 13' travel lane and 5' bike lane, plus the proposed 6' sidewalk, consistent with the collector street improvements specified in this section's table. No parking is provided on Skyline Drive.

4. The decision-making body shall consider the City Engineer's recommendations on the desired right-of-way width, pavement width and street geometry of the various street types within the subdivision after consideration by the City Engineer of the following criteria:

a. The type of road as set forth in the Transportation Master Plan.
b. The anticipated traffic generation.
c. On-street parking requirements.
d. Sidewalk and bikeway requirements.
e. Requirements for placement of utilities.
f. Street lighting.
g. Drainage and slope impacts.
h. Street trees.
i. Planting and landscape areas.
j. Existing and future driveway grades.
k. Street geometry.
l. Street furniture needs, hydrants.

Comment: The applicant will work with the City Engineer in developing final construction plans consistent with his recommendations on these improvements.

5. Additionally, when determining appropriate street width, the decision-making body shall consider the following criteria:

a. When a local street is the only street serving a residential area and is expected to carry more than the normal local street traffic load, the designs with two travel and one parking lane are appropriate.

b. Streets intended to serve as signed but unstriped bike routes should have the travel lane widened by two feet.

c. Collectors should have two travel lanes and may accommodate some parking. Bike routes are appropriate.

d. Arterials should have two travel lanes. On-street parking is not allowed unless part of a Street Master Plan. Bike lanes are required as directed by the Parks Master Plan and Transportation Master Plan.

Comment: Skyline Dr. is a collector so a) does not apply. The bike lanes on Skyline Drive are striped, so b) does not apply. Skyline Drive is developed with two travel lanes and has bicycle lanes. No parking is provided on this street. Skyline Drive is not an arterial street so d) is not applicable.

6. Reserve strips. Reserve strips or street plugs controlling the access to streets are not permitted unless owned by the City.

Comment: No dead end streets are proposed so reserve strips or street plugs are not needed.

7. Alignment. All streets other than local streets or cul-de-sacs, as far as practical, shall be in alignment with existing streets by continuations of the centerlines thereof. The staggering of street alignments resulting in "T" intersections shall, wherever practical, leave a minimum distance of 200 feet between the centerlines of streets having approximately the same direction and otherwise shall not be less than 100 feet.

Comment: No new public streets are proposed so this subsection does not apply.

8. Future extension of streets. Where necessary to give access to or permit a satisfactory future subdivision of adjoining land, streets shall be extended to the boundary of the subdivision and the resulting dead-end streets may be approved without turnarounds. (Temporary turnarounds built to Fire Department standards are required when the dead-end street is over 100 feet long.)

Comment: Not applicable. Adjacent properties are fully developed and no future extensions of streets are practicable.

9. Intersection angles. Streets shall be laid out to intersect angles as near to right angles as practical, except where topography requires lesser angles, but in no case less than 60 degrees unless a special intersection design is approved. Intersections which are not at right angles shall have minimum corner radii of 15 feet along right-of-way lines which form acute angles. Right-of-way lines at intersections with arterial streets shall have minimum curb radii of not less than 35 feet. Other street intersections shall have curb radii of not less than 25 feet. All radii shall maintain a uniform width between the roadway and the right-of-way lines. The intersection of more than two streets at any one point will not be allowed unless no alternative design exists.

Comment: No new public street intersections are proposed. The private drive serving Parcels 1 through 3 intersects Skyline Drive at a 90 degree angle.

10. Additional right-of-way for existing streets. Wherever existing street rights-of-way adjacent to or within a tract are of inadequate widths based upon the standards of this chapter, additional right-of-way shall be provided at the time of subdivision or partition.

Comment: The existing right-of-way of Skyline Drive measures 60 feet in width (30' on each side of the centerline) and is consistent with TSP requirements for this collector street. No additional right-of-way is needed.

11. Cul-de-sacs.

Comment: No new public cul-de-sac streets are proposed so 11) does not apply.

12. *Street names. No street names shall be used which will duplicate or be confused with the names of existing streets within the City. Street names that involve difficult or unusual spellings are discouraged. Street names shall be subject to the approval of the Planning Commission or Planning Director, as applicable. Continuations of existing streets shall have the name of the existing street. Streets, drives, avenues, ways, boulevards, and lanes shall describe through streets. Place and court shall describe cul-de-sacs. Crescent, terrace, and circle shall describe loop or arcing roads.*

Comment: Not applicable. No new streets are proposed.

13. *Grades and curves. Grades and horizontal/vertical curves shall meet the West Linn Public Works Design Standards.*

Comment: Skyline Drive is existing along the project frontage. The street grade is approximately one percent and the road is straight in the section fronting this site. Therefore, the existing condition satisfies Public Works Design Standards.

14. *Access to local streets. Intersection of a local residential street with an arterial street may be prohibited by the decision-making authority if suitable alternatives exist for providing interconnection of proposed local residential streets with other local streets. Where a subdivision or partition abuts or contains an existing or proposed major arterial street, the decision-making authority may require marginal access streets, reverse-frontage lots with suitable depth, visual barriers, noise barriers, berms, no-access reservations along side and rear property lines, and/or other measures necessary for adequate protection of residential properties from incompatible land uses, and to ensure separation of through traffic and local traffic.*

Comment: Not applicable. The subdivision does not abut or contain an existing or proposed arterial street.

15. *Alleys. Alleys shall be provided in commercial and industrial districts unless other permanent provisions for access to off-street parking and loading facilities are made as approved by the decision-making authority. While alley intersections and sharp changes in alignment should be avoided, the corners of necessary alley intersections shall have radii of not less than 10 feet. Alleys may be provided in residential subdivisions or multi-family projects. The decision to locate alleys shall consider the relationship and impact of the alley to adjacent land uses. In determining whether it is appropriate to require alleys in a subdivision or partition, the following factors and design criteria should be considered:*

    a. *The alley shall be self-contained within the subdivision. The alley shall not abut undeveloped lots or parcels which are not part of the project proposal. The alley will not stub out to abutting undeveloped parcels which are not part of the project proposal.*

    b. *The alley will be designed to allow unobstructed and easy surveillance by residents and police.*

c. *The alley should be illuminated. Lighting shall meet the West Linn Public Works Design Standards.*

d. *The alley should be a semi-private space where strangers are tacitly discouraged.*

e. *Speed bumps may be installed in sufficient number to provide a safer environment for children at play and to discourage through or speeding traffic.*

f. *Alleys should be a minimum of 14 feet wide, paved with no curbs.*

Comment: Not applicable. The proposed land use is single-family residential and no alleys are proposed.

16. *Sidewalks. Sidewalks shall be installed per CDC 92.010(H), Sidewalks. The residential sidewalk width is six feet plus planter strip as specified below. Sidewalks in commercial zones shall be constructed per subsection (A)(3) of this section. See also subsection C of this section. Sidewalk width may be reduced with City Engineer approval to the minimum amount (e.g., four feet wide) necessary to respond to site constraints such as grades, mature trees, rock outcroppings, etc., or to match existing sidewalks or right-of-way limitations.*

Comment: A 6' Sidewalk will be provided along the site's frontage at the time of construction of the home on Parcel 1, as shown on the Tentative Plan.

17. *Planter strip. The planter strip is between the curb and sidewalk providing space for a grassed or landscaped area and street trees. The planter strip shall be at least 6 feet wide to accommodate a fully matured tree without the boughs interfering with pedestrians on the sidewalk or vehicles along the curbline. Planter strip width may be reduced or eliminated, with City Engineer approval, when it cannot be corrected by site plan, to the minimum amount necessary to respond to site constraints such as grades, mature trees, rock outcroppings, etc., or in response to right-of-way limitations.*

Comment: A planter strip will be provided, as shown on the Tentative Plan.

18. *Streets and roads shall be dedicated without any reservations or restrictions.*

Comment: No street or road right-of-way is proposed to be dedicated with this project so this criterion does not apply.

19. *All lots in a subdivision shall have access to a public street. Lots created by partition may have access to a public street via an access easement pursuant to the standards and limitations set forth for such accessways in Chapter 48 CDC.*

Comment: All lots in the proposed partition will have access to Skyline Drive, a public street, via an access easement per the standards of Chapter 48.

20. *Gated streets. Gated streets are prohibited in all residential areas on both public and private streets. A driveway to an individual home may be gated.*

Comment: No gated streets are proposed.

21. Entryway treatments and street isle design. When the applicant desires to construct certain walls, planters, and other architectural entryway treatments within a subdivision, the following standards shall apply:

a. All entryway treatments except islands shall be located on private property and not in the public right-of-way.

b. Planter islands may be allowed provided there is no structure (i.e., brick, signs, etc.) above the curbline, except for landscaping. Landscaped islands shall be set back a minimum of 24 feet from the curbline of the street to which they are perpendicular.

c. All islands shall be in public ownership. The minimum aisle width between the curb and center island curbs shall be 14 feet. Additional width may be required as determined by the City Engineer.

d. Brick or special material treatments are acceptable at intersections with the understanding that the City will not maintain these sections except with asphalt overlay, and that they must meet the Americans with Disabilities Act (ADA) standards. They shall be laid out to tie into existing sidewalks at intersections.

e. Maintenance for any common areas and entryway treatments (including islands) shall be guaranteed through homeowners association agreements, CC&Rs, etc.

f. Under Chapter 52 CDC, subdivision monument signs shall not exceed 32 square feet in area.

Comment: Not applicable. No special entry treatments are proposed.

22. Based upon the determination of the City Manager or the Manager's designee, the applicant shall construct or cause to be constructed, or contribute a proportionate share of the costs, for all necessary off-site improvements identified by the transportation analysis commissioned to address CDC 85.170(B)(2) that are required to mitigate impacts from the proposed subdivision. The proportionate share of the costs shall be determined by the City Manager or Manager's designee, who shall assume that the proposed subdivision provides improvements in rough proportion to identified impacts of the subdivision. Off-site transportation improvements will include bicycle and pedestrian improvements as identified in the adopted City of West Linn TSP.

Comment: Not applicable. No off-site improvements are anticipated

B. Blocks and lots.

1. General. The length, width, and shape of blocks shall be designed with due regard for the provision of adequate building sites for the use contemplated; consideration of the need for traffic safety, convenience, access, circulation, and control; and recognition of limitations and opportunities of topography and solar access.

Comment: As previously mentioned, the development pattern in this area is already established. There is no opportunity for additional local street connections. No new blocks are proposed.

2. Sizes. The recommended block size is 400 feet in length to encourage greater connectivity within the subdivision. Blocks shall not exceed 800 feet in length between street lines, except for blocks adjacent to arterial streets or unless topographical conditions or the layout of adjacent streets justifies a variation. Designs of proposed intersections shall demonstrate adequate sight distances to the City Engineer's specifications. Block sizes and proposed accesses must be consistent with the adopted TSP.

Comment: Same as for B1, above.

3. Lot size and shape. Lot or parcel size, width, shape, and orientation shall be appropriate for the location of the subdivision or partition, for the type of use contemplated, for potential utilization of solar access, and for the protection of drainageways, trees, and other natural features. No lot or parcel shall be dimensioned to contain part of an existing or proposed street. All lots or parcels shall be buildable. "Buildable" describes lots that are free of constraints such as wetlands, drainageways, etc., that would make home construction impossible. Lot or parcel sizes shall not be less than the size required by the zoning code unless as allowed by planned unit development (PUD).

Depth and width of properties reserved or laid out for commercial and industrial purposes shall be adequate to provide for the off-street parking and service facilities required by the type of use proposed.

Comment: The proposed lots are consistent with the dimensional standards of the R-10 zone and provide reasonable building sites for single-family detached homes. The lots are deep enough on their north-south axes to provide for the opportunity to orient the homes for solar access. The lots do not include portions of existing streets. The flagstrip area for the access drive has not been included in the computation of lot size for purposes of meeting R-10 standards.

4. Access. Access to subdivisions, partitions, and lots shall conform to the provisions of Chapter 48 CDC, Access, Egress and Circulation.

Comment: See discussion of Chapter 48, below.

5. Double frontage lots and parcels. Double frontage lots and parcels have frontage on a street at the front and rear property lines. Double frontage lots and

parcels shall be avoided except where they are essential to provide separation of residential development from arterial streets or adjacent non-residential activities, or to overcome specific disadvantages of topography and orientation. A planting screen or impact mitigation easement at least 10 feet wide, and across which there shall be no right of access, may be required along the line of building sites abutting such a traffic artery or other incompatible use.

Comment: No double frontage lots or parcels are proposed.

6. Lot and parcel side lines. The lines of lots and parcels, as far as is practicable, should run at right angles to the street upon which they face, except that on curved streets they should be radial to the curve.

Comment: The proposed side lot lines are roughly perpendicular to the street right-of-way.

7. Flag lots. Flag lots can be created where it can be shown that no other reasonable street access is possible to achieve the requested land division. A single flag lot shall have a minimum street frontage of 15 feet for its accessway. Where two to four flag lots share a common accessway, the minimum street frontage and accessway shall be eight feet in width per lot. Common accessways shall have mutual maintenance agreements and reciprocal access and utility easements. The following dimensional requirements shall apply to flag lots:

a. Setbacks applicable to the underlying zone shall apply to the flag lot.

b. Front yard setbacks may be based on the rear property line of the lot or parcel which substantially separates the flag lot from the street from which the flag lot gains access. Alternately, the house and its front yard may be oriented in other directions so long as some measure of privacy is ensured, or it is part of a pattern of development, or it better fits the topography of the site.

c. The lot size shall be calculated exclusive of the accessway; the access strip may not be counted towards the area requirements.

d. The lot depth requirement contained elsewhere in this code shall be measured from the rear property line of the lot or parcel which substantially separates the flag lot from the street from which the flag lot gains access.

e. As per CDC 48.030, the accessway shall have a minimum paved width of 12 feet.

f. If the use of a flag lot stem to access a lot is infeasible because of a lack of adequate existing road frontage, or location of existing structures, the proposed lot(s) may be accessed from the public street by an access easement of a minimum 15-foot width across intervening property.

Comment: Due to the lack of street frontage or streets that are stubbed to the property line, access to the rear portion of the subject property may only feasibly be provided via the use of a flag lot development pattern. The property to the north and west is platted and developed as a part of the Bridge

View subdivision plat. Property to the east is developed as a City water reservoir. The subject property has only 125 feet of road frontage, which is not sufficient to develop a City-standard street with a circular cul-de-sac. Flag lots with a shared accessway are the only feasible development option for this site. Setbacks will be reviewed at the time of building permit application. All parcels exceed the minimum 10,000 sq. ft. lot size standard of the R-10 district, exclusive of area within the access strip. All lots proposed exceed the minimum lot depth standard of the R-10 zone. The proposed access drive serving Parcels 2 and 3 will be 12 feet in width and is located in the 16 foot access easement.

8. Large lots or parcels. In dividing tracts into large lots or parcels which, at some future time, are likely to be redivided, the approval authority may:

a. Require that the blocks be of such size and shape, and be so divided into building sites, and contain such easements and site restrictions as will provide for extension and opening of streets at intervals which will permit a subsequent division of any tract into lots or parcels of smaller size; or

b. Alternately, in order to prevent further subdivision or partition of oversized and constrained lots or parcels, restrictions may be imposed on the subdivision or partition plat.

Comment: Not applicable. None of the parcels proposed are large enough to be capable of being redivided.

C. Pedestrian and bicycle trails.

Comment: Not applicable. No pedestrian or bicycle trails exist or are planned in this area.

D. Transit facilities.

Comment: Not applicable. There are no Tri-Met bus services in this area so there is no need for transit facilities.

E. Grading. Grading of building sites shall conform to the following standards unless physical conditions demonstrate the propriety of other standards:

Comment: No grading of building sites is planned at this time. Grading plans will be reviewed at the time of building permit application.

F. Water.

Comment: Water service will be provided from the existing water line in Skyline Dr. No new public water lines are proposed. Water meters for Parcels 2 and 3 will be provided in the public right-of-way, with private water service lines extending to the parcels via the access strip.

G. Sewer.

Comment: Sewer service will be provided from the existing sewer line in Firwood Ct., to the north of the subject property. A new sewer line will be extended via an existing easement along the common lot line of Tax Lots 5300 and 5400 of Assessor's Map 21E25AC. This line will be extended to serve the new parcels, as shown on the Preliminary Utility Plan.

H. Storm detention and treatment.

All proposed storm detention and treatment facilities comply with the standards for the improvement of public and private drainage systems located in the West Linn Public Works Design Standards, there will be no adverse off-site impacts caused by the development (including impacts from increased intensity of runoff downstream or constrictions causing ponding upstream), and there is sufficient factual data to support the conclusions of the submitted plan.

Comment: A storm water report has been prepared by Theta Engineering and is included with this application submittal. As shown the Preliminary Utility Plan, water from the shared private driveway will be collected and directed to an underground infiltration system so that there will be no surface runoff from the driveway to other downstream properties. Rain gardens are proposed to be provided on each lot to handle runoff from roofs of new homes.

I. Utility easements.

Comment: Easements for public utilities will be provided as shown on the Preliminary Utility Plan.

J. Supplemental provisions.

1. Wetland and natural drainageways.

Comment: There are no wetlands or drainageways on the subject property or on adjacent parcels.

2. Willamette and Tualatin Greenways.

Comment: The subject property is not located within the Willamette or Tualatin Greenway areas. There are no Habitat Conservation Areas on the property.

3. Street trees. Street trees are required as identified in the appropriate section of the municipal code and Chapter 54 CDC.

Comment: Street trees will be provided along the frontage of Parcel 1, as shown on the Tentative Plan.

4. Lighting.

Comment: There is existing street lighting on Skyline Dr.

5. Dedications and exactions.

Comment: Five feet of right-of-way was dedicated along the Skyline Drive frontage of the subject property when the land was partitioned in 1996. This dedication provides for a half-street width from centerline of 30 feet along the property's frontage, consistent with what was discussed at the pre-application conference. A public utility easement will be provided as required along the street frontage. No other exactions are warranted.

6. Underground utilities.

Comment: All new utilities will be place underground.

7. Density requirement.

Comment: The subject property measures 32,569 square feet in site area. The access strip accounts for 2,199 sq. ft. and does not count towards density. Deducting this area from the site area leaves a net area of 30,058 sq. ft. Dividing by the minimum 10,000 sq. ft. lot size of the R-10 zone yields a maximum density of 3 lots. Three lots are proposed so both the minimum and maximum density standards are met.

8. Mix requirement. The "mix" rule means that developers shall have no more than 15 percent of the R-2.1 and R-3 development as single-family residential. The intent is that the majority of the site shall be developed as medium high density multi-family housing.

Comment: The subject property is not in the R-2.1 or R-3 zones so this provision does not apply.

9. Heritage trees/significant tree and tree cluster protection.

Comment: There are no heritage trees on the site. There is a cluster of trees on Parcel 3 that the City Arborist has determined to be significant. See discussion of Chapter 55, below.

Chapter 48 - ACCESS, EGRESS AND CIRCULATION

48.025 ACCESS CONTROL

B. Access control standards.

1. Traffic impact analysis requirements. The City or other agency with access jurisdiction may require a traffic study prepared by a qualified professional to determine access, circulation and other transportation requirements. (See also CDC 55.125, Traffic Impact Analysis.)

Comment: Because of the small size of this project, the City did not require a traffic impact analysis. The project will result in less than 30 new vehicle trips per day based on ITE data.

2. The City or other agency with access permit jurisdiction may require the closing or consolidation of existing curb cuts or other vehicle access points,

recording of reciprocal access easements (i.e., for shared driveways), development of a frontage street, installation of traffic control devices, and/or other mitigation as a condition of granting an access permit, to ensure the safe and efficient operation of the street and highway system. Access to and from off-street parking areas shall not permit backing onto a public street.

Comment: There are no existing curb cuts that need to be closed. All lots will access onto the proposed shared private drive.

3. Access options. When vehicle access is required for development (i.e., for off-street parking, delivery, service, drive-through facilities, etc.), access shall be provided by one of the following methods (planned access shall be consistent with adopted public works standards and TSP). These methods are "options" to the developer/subdivider.

a) Option 1. Access is from an existing or proposed alley or mid-block lane. If a property has access to an alley or lane, direct access to a public street is not permitted.

b) Option 2. Access is from a private street or driveway connected to an adjoining property that has direct access to a public street (i.e., "shared driveway"). A public access easement covering the driveway shall be recorded in this case to assure access to the closest public street for all users of the private street/drive.

c) Option 3. Access is from a public street adjacent to the development lot or parcel. If practicable, the owner/developer may be required to close or consolidate an existing access point as a condition of approving a new access. Street accesses shall comply with the access spacing standards in subsection (B)(6) of this section.

Comment: Access will be via the shared private driveway.

4. Subdivisions fronting onto an arterial street. New residential land divisions fronting onto an arterial street shall be required to provide alleys or secondary (local or collector) streets for access to individual lots. When alleys or secondary streets cannot be constructed due to topographic or other physical constraints, access may be provided by consolidating driveways for clusters of two or more lots (e.g., includes flag lots and mid-block lanes).

Comment: Not applicable. The site does not front onto an arterial street. Skyline Drive is classified as a collector street in the West Linn Transportation Systems Plan.

5. Double-frontage lots. When a lot or parcel has frontage onto two or more streets, access shall be provided first from the street with the lowest classification. For example, access shall be provided from a local street before a collector or arterial street. When a lot or parcel has frontage opposite that of the adjacent lots or parcels, access shall be provided from the street with the lowest classification.

Comment: Not applicable. No double-frontage lots are proposed.

6. Access spacing.

a. The access spacing standards found in Chapter 8 of the adopted Transportation System Plan (TSP) shall be applicable to all newly established public street intersections and non-traversable medians.

b. Private drives and other access ways are subject to the requirements of CDC.

Comment: All parcels will be accessed via the proposed private drive, which conforms to City access spacing requirements.

7. Number of access points. For single-family (detached and attached), two-family, and duplex housing types, one street access point is permitted per lot or parcel, when alley access cannot otherwise be provided; except that two access points may be permitted corner lots (i.e., no more than one access per street), subject to the access spacing standards in subsection (B)(6) of this section. The number of street access points for multiple family, commercial, industrial, and public/institutional developments shall be minimized to protect the function, safety and operation of the street(s) and sidewalk(s) for all users. Shared access may be required, in conformance with subsection (B)(8) of this section, in order to maintain the required access spacing, and minimize the number of access points.

Comment: All lots will make use of the private drive, which will satisfy this standard.

8. Shared driveways. The number of driveway and private street intersections with public streets shall be minimized by the use of shared driveways with adjoining lots where feasible. The City shall require shared driveways as a condition of land division or site design review, as applicable, for traffic safety and access management purposes in accordance with the following standards:

a. Shared driveways and frontage streets may be required to consolidate access onto a collector or arterial street. When shared driveways or frontage streets are required, they shall be stubbed to adjacent developable parcels to indicate future extension. "Stub" means that a driveway or street temporarily ends at the property line, but may be extended in the future as the adjacent lot or parcel develops. "Developable" means that a lot or parcel is either vacant or it is likely to receive additional development (i.e., due to infill or redevelopment potential).

b. Access easements (i.e., for the benefit of affected properties) shall be recorded for all shared driveways, including pathways, at the time of final plat approval or as a condition of site development approval.

c. Exception. Shared driveways are not required when existing development patterns or physical constraints (e.g., topography, lot or parcel configuration, and similar conditions) prevent extending the street/driveway in the future.

Comment: The proposed shared driveway will have an easement shown on the partition plat.

C. Street connectivity and formation of blocks required. In order to promote efficient vehicular and pedestrian circulation throughout the City, land divisions and large site developments shall produce complete blocks bounded by a connecting network of public and/or private streets, in accordance with the following standards:

1. Block length and perimeter. The maximum block length shall not exceed 800 feet or 1,800 feet along an arterial.

2. Street standards. Public and private streets shall also conform to Chapter 92 CDC, Required Improvements, and to any other applicable sections of the West Linn Community Development Code and approved TSP.

3. Exception. Exceptions to the above standards may be granted when blocks are divided by one or more pathway(s), in conformance with the provisions of CDC 85.200(C), Pedestrian and Bicycle Trails, or cases where extreme topographic (e.g., slope, creek, wetlands, etc.) conditions or compelling functional limitations preclude implementation, not just inconveniences or design challenges. (Ord. 1635 § 25, 2014; Ord. 1636 § 33, 2014)

Comment: Adjacent property is fully developed and no street stubs are provided to the subject property. Because of this, it is not possible to extend a local street through the site to create a new block.

48.030 MINIMUM VEHICULAR REQUIREMENTS FOR RESIDENTIAL USES

A. Direct individual access from single-family dwellings and duplex lots to an arterial street, as designated in the transportation element of the Comprehensive Plan, is prohibited for lots or parcels created after the effective date of this code where an alternate access is either available or is expected to be available by imminent development application. Evidence of alternate or future access may include temporary cul-de-sacs, dedications or stubouts on adjacent lots or parcels, or tentative street layout plans submitted at one time by adjacent property owner/developer or by the owner/developer, or previous owner/developer, of the property in question.

In the event that alternate access is not available as determined by the Planning Director and City Engineer, access may be permitted after review of the following criteria:

1. Topography.

2. Traffic volume to be generated by development (i.e., trips per day).

3. Traffic volume presently carried by the street to be accessed.

4. Projected traffic volumes.

5. *Safety considerations such as line of sight, number of accidents at that location, emergency vehicle access, and ability of vehicles to exit the site without backing into traffic.*

6. *The ability to consolidate access through the use of a joint driveway.*

7. *Additional review and access permits may be required by State or County agencies.*

Comment: Figure 17 in the TSP designates Skyline Drive as a collector street. This section does not apply.

B. When any portion of any house is less than 150 feet from the adjacent right-of-way, access to the home is as follows:

1. *One single-family residence, including residences with an accessory dwelling unit as defined in CDC 02.030, shall provide 10 feet of unobstructed horizontal clearance. Dual-track or other driveway designs that minimize the total area of impervious driveway surface are encouraged.*

2. *Two to four single-family residential homes equals a 14- to 20-foot-wide paved or all-weather surface. Width shall depend upon adequacy of line of sight and number of homes.*

3. *Maximum driveway grade shall be 15 percent. The 15 percent shall be measured along the centerline of the driveway only. Variations require approval of a Class II variance by the Planning Commission pursuant to Chapter 75 CDC. Regardless, the last 18 feet in front of the garage shall be under 12 percent grade as measured along the centerline of the driveway only. Grades elsewhere along the driveway shall not apply.*

4. *The driveway shall include a minimum of 20 feet in length between the garage door and the back of sidewalk, or, if no sidewalk is proposed, to the paved portion of the right-of-way.*

Comment: The homes on Parcels 2 and 3 will exceed 150 feet in distance from Skyline Drive. The minimum 10 foot unobstructed horizontal clearance standard will be met. The grade of the private drive will be under 15 percent. The driveways comply with the 20 foot minimum length between the garage and the sidewalk.

C. When any portion of one or more homes is more than 150 feet from the adjacent right-of-way, the provisions of subsection B of this section shall apply in addition to the following provisions.

1. *A turnaround may be required as prescribed by the Fire Chief.*

2. *Minimum vertical clearance for the driveway shall be 13 feet, six inches.*

3. *A minimum centerline turning radius of 45 feet is required unless waived by the Fire Chief.*

4. *There shall be sufficient horizontal clearance on either side of the driveway so that the total horizontal clearance is 20 feet.*

Comment: The applicant will coordinate with the Fire Chief to determine whether a turnaround or other mitigating measures, such as sprinklers, are warranted for Parcels 2 and 3. Compliance with other requirements of this section will be demonstrated at the time of building permit application.

D. Access to five or more single-family homes shall be by a street built to full construction code standards. All streets shall be public. This full street provision may only be waived by variance.

Comment: Not applicable. The proposed access will not serve five or more vehicles.

E. Access and/or service drives for multi-family dwellings shall be fully improved with hard surface pavement:

Comment: Not applicable. No multi-family development is proposed.

F. Where on-site maneuvering and/or access drives are necessary to accommodate required parking, in no case shall said maneuvering and/or access drives be less than that required in Chapters 46 and 48 CDC.

Comment: The proposed access drive complies with these standards

G. The number of driveways or curb cuts shall be minimized on arterials or collectors. Consolidation or joint use of existing driveways shall be required when feasible.

Comment: Not applicable. The access to all three parcels will be via the shared private driveway, thereby minimizing the number of driveways onto Skyline Drive.

H. In order to facilitate through traffic and improve neighborhood connections, it may be necessary to construct a public street through a multi-family site.

Comment: Not applicable. The site is not a multi-family site and there is no opportunity for a street connection due to development patterns to the north.

I. Gated accessways to residential development other than a single-family home are prohibited. (Ord. 1408, 1998; Ord. 1463, 2000; Ord. 1513, 2005; Ord. 1584, 2008; Ord. 1590 § 1, 2009; Ord. 1636 § 34, 2014)

Comment: No gated accessways are proposed.

48.040 MINIMUM VEHICLE REQUIREMENTS FOR NON-RESIDENTIAL USES

Comment: No non-residential uses are proposed so this section does not apply.

48.050 ONE-WAY VEHICULAR ACCESS POINTS

Where a proposed parking facility plan indicates only one-way traffic flow on the site, it shall be accommodated by a specific driveway serving the facility, and the entrance drive shall be situated closest to oncoming traffic, and the exit drive shall be situated farthest from oncoming traffic.

Comment: No one-way traffic flow patterns are proposed.

48.060 WIDTH AND LOCATION OF CURB CUTS AND ACCESS SEPARATION REQUIREMENTS

A. Minimum curb cut width shall be 16 feet.

Comment: The curb cut for the proposed access drive will comply with this minimum.

B. Maximum curb cut width shall be 36 feet, except along Highway 43 in which case the maximum curb cut shall be 40 feet. For emergency service providers, including fire stations, the maximum shall be 50 feet.

Comment: The proposed curb cut will not exceed 36 feet, as shown on the site plan.

C. No curb cuts shall be allowed any closer to an intersecting street right-of-way line than the following:

1. On an arterial when intersected by another arterial, 150 feet.

2. On an arterial when intersected by a collector, 100 feet.

3. On an arterial when intersected by a local street, 100 feet.

4. On a collector when intersecting an arterial street, 100 feet.

5. On a collector when intersected by another collector or local street, 35 feet.

6. On a local street when intersecting any other street, 35 feet.

Comment: Figure 17 in the Transportation System Plan designates Skyline Dr. as a collector street. The closest intersection is Firwood Drive, a local street, approximately 240 feet to the west. This standard is met.

D. There shall be a minimum distance between any two adjacent curb cuts on the same side of a public street, except for one-way entrances and exits, as follows:

1. On an arterial street, 150 feet.

2. On a collector street, 75 feet.

3. Between any two curb cuts on the same lot or parcel on a local street, 30 feet.

Comment: The applicant has a new home that will be commencing on the adjacent Tax Lot 9901 to the west of the subject property. It has a temporary curb cut off of Skyline

that is closer than 75 feet from the subject property. That driveway will be changed to come off of the shared private drive upon construction of the new access.

E. A rolled curb may be installed in lieu of curb cuts and access separation requirements.

Comment: Not proposed.

F. Curb cuts shall be kept to the minimum, particularly on Highway 43. Consolidation of driveways is preferred. The standard on Highway 43 is one curb cut per business if consolidation of driveways is not possible.

Comment: The proposed plan makes use of the single accessway and curb cut to service the three parcels within this partition as well as TL 9901, consistent with this provision.

G. Adequate line of sight pursuant to engineering standards should be afforded at each driveway or accessway.

Comment: There are no obstructions to sight distance at the driveway location.

CHAPTER 55 DESIGN REVIEW

55.100 APPROVAL STANDARDS – CLASS II DESIGN REVIEW

Design Review is only applicable to significant trees as cross referenced by CDC 85.200(J) (9).

B. Relationship to the natural and physical environment.

1 The buildings and other site elements shall be designed and located so that all heritage trees, as defined in the municipal code, shall be saved. Diseased heritage trees, as determined by the City Arborist, may be removed at his/her direction.

2. All heritage trees, as defined in the municipal code, all trees and clusters of trees ("cluster" is defined as three or more trees with overlapping driplines; however, native oaks need not have an overlapping dripline) that are considered significant by the City Arborist, either individually or in consultation with certified arborists or similarly qualified professionals, based on accepted arboricultural standards including consideration of their size, type, location, health, long term survivability, and/or numbers, shall be protected pursuant to the criteria of subsections (B)(2)(a) through (f) of this section. (….)

Comment: The tree survey information was reviewed by the City's Arborist. He determined that the entire stand of trees is significant. There are no heritage trees on the subject property. Please refer to the Tree Plan and Tree Inventory submitted with this application.

a. *Non-residential and residential projects on Type I and II lands shall protect all heritage trees and all significant trees and tree clusters by limiting development in the protected area. The protected area includes the protected tree, its dripline, and an additional 10 feet beyond the dripline, as depicted in the figure below. Development of Type I and II lands shall require the careful layout of streets, driveways, building pads, lots, and utilities to avoid heritage trees and significant trees and tree clusters, and other natural resources pursuant to this code. The method for delineating the protected trees or tree clusters ("dripline plus 10 feet") is explained in subsection (B)(2)(b) of this section. Exemptions of subsections (B)(2)(c), (e), and (f) of this section shall apply.*

Comment: Only a small area in the northwest corner of Parcel 3 contains Type II lands. No trees are located in that area so this section does not apply.

b. *Non-residential and residential projects on non-Type I and II lands shall set aside up to 20 percent of the protected areas for significant trees and tree clusters, plus any heritage trees. Therefore, in the event that the City Arborist determines that a significant tree cluster exists at a development site, then up to 20 percent of the non-Type I and II lands shall be devoted to the protection of those trees by limiting development in the protected areas. The exact percentage is determined by establishing the driplines of the trees or tree clusters that are to be protected. In order to protect the roots which typically extend further, an additional 10-foot measurement beyond the dripline shall be added. The square footage of the area inside this "dripline plus 10 feet" measurement shall be the basis for calculating the percentage (see figure below). The City Arborist will identify which tree(s) are to be protected. Development of non-Type I and II lands shall also require the careful layout of streets, driveways, building pads, lots, and utilities to avoid significant trees, tree clusters, heritage trees, and other natural resources pursuant to this code. Exemptions of subsections (B)(2)(c), (e), and (f) of this section shall apply. Please note that in the event that more than 20 percent of the non-Type I and II lands comprise significant trees or tree clusters, the developer shall not be required to save the excess trees, but is encouraged to do so.*

Comment: The grove of trees takes up nearly all of Parcel 3. In order to develop a home on that parcel, plus the sewer line needed to service all three Parcels, nearly all of the trees will need to be removed, as shown on the Tree Plan submitted with this application. An area in the southwest corner of Parcel 3 will be preserved out to the dripline plus 10 feet. This area measures 809 sq. ft. in area (2.4% of the total 32,569 sq. ft. site area). This falls within the allowable 1 to 20% set aside requirement.

Chapter 92, required improvements

92.010 PUBLIC IMPROVEMENTS FOR ALL DEVELOPMENT

The following improvements shall be installed at the expense of the developer and meet all City codes and standards:

E. Surface drainage and storm sewer system. A registered civil engineer shall prepare a plan and statement which shall be supported by factual data and comply with the standards for the improvement of public and private drainage systems located in the West Linn Public Works Design Standards. (….)

Comment: The applicant proposes to provide a rain garden on all parcels to accommodate runoff from the new home. Skyline Drive is fully improved to City standards so no new impervious surface will be added there. A drainage report prepared by Theta Engineering for the adjacent Tax Lot 9901 is attached and demonstrates that the soils in this area are suitable for rain gardens.

Skyline Drive Partition

Compliance with Grading Criteria of CDC 85.200E

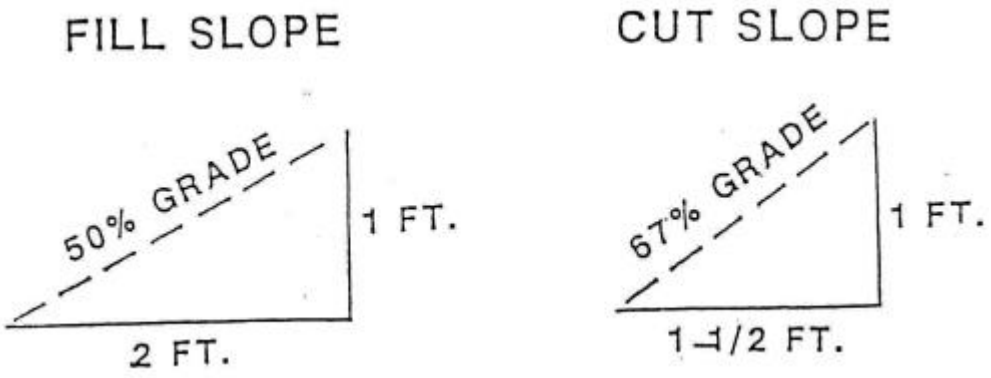
E. Grading. Grading of building sites shall conform to the following standards unless physical conditions demonstrate the propriety of other standards:

1. All cuts and fills shall comply with the excavation and grading provisions of the Uniform Building Code and the following:

a. Cut slopes shall not exceed one and one-half feet horizontally to one foot vertically (i.e., 67 percent grade).

Comment: As shown on the Grading Plan submitted with this application, all cut slopes will not exceed the 1.5:1 ratio.

b. Fill slopes shall not exceed two feet horizontally to one foot vertically (i.e., 50 percent grade). Please see the following illustration.



Comment: The Grading Plan illustrates the cut and fill slopes proposed in conjunction with the development of this property. All slopes proposed comply with these standards.

2. The character of soil for fill and the characteristics of lot and parcels made usable by fill shall be suitable for the purpose intended.

Comment: Major portions of Parcels 1 and 2 and a small area of Parcel 3 contain non-engineered fill materials associated with the development of the water reservoir on the property to the east. Per the recommendations

of the GeoPacific Engineering geotechnical report for this site, these materials will either be excavated to native soil level and replaced with engineered fill or footings will be excavated to be placed on native soil. Soils imported for replacement will be installed as an engineered fill. The final grading plan for each lot will be submitted for review with the building permit application.

3. If areas are to be graded (more than any four-foot cut or fill), compliance with CDC 85.170(C) is required.

Comment: The depth of the proposed cuts and fills are four feet or less, as shown on the Grading Plan.

4. The proposed grading shall be the minimum grading necessary to meet roadway standards, and to create appropriate building sites, considering maximum allowed driveway grades.

Comment: As shown on the Grading Plan, the grading for the proposed private driveway is minimal with no more than a foot of cut or fill. The grading proposed for the building pads conforms as closely to native grade as possible. All fills are less than four feet in depth.

5. Type I lands shall require a report submitted by an engineering geologist, and Type I and Type II lands shall require a geologic hazard report.

Comment: No Type I lands exist on the subject property. There is a small area of Type II land in the northwest corner of Parcel 3, but this area will not be developed.

6. Per the submittals required by CDC 85.170(C)(3), the applicant must demonstrate that the proposed methods of rendering known or potential hazard sites safe for development, including proposed geotechnical remediation, are feasible and adequate to prevent landslides or other damage to property and safety. The review authority may impose conditions, including limits on type or intensity of land use, which it

determines are necessary to mitigate known risks of landslides or property damage.

Comment: The applicant relies upon the geotechnical analysis prepared by GRI for the Bolton Reservoir site adjacent to the subject property for the broader geotechnical issues affecting the site area. The GRI report notes that there are faults in the area, notably the Bolton Fault, and that this site is located in an ancient (15,000-20,000 years old) landslide area. The report notes, "reconnaissances by GRI as part of this study and during our 2012 study did not disclose indications of recent landslide movement. A reconnaissance recently completed by Cornforth Consultants (December 2014) also did not identify signs of active movement. It is our opinion the risk of significant future movement of the large, ancient landslide is low."

Regarding seismic considerations, the report notes, "Based on preliminary evaluations, there is some risk of seismically induced soil strength loss in relatively thin zones in the decomposed basalt that have weathered to the consistency of soft soil that were encountered locally between depths of about 25 to 40 ft below the existing ground surface. In our opinion, the risk of significant post-earthquake settlement due to soil strength loss in these isolated layers is low."

Given this information, we conclude that there are no broad general geologic hazards that require geologic mitigation. The site does have non-engineered fill materials in the area of Parcels 1 and 2. The applicant has retained GeoPacific Engineering, Inc. to provide an analysis of these fill materials and make recommendations as to how to properly deal with them. Please refer to pages 8 to 10 of that report for more detail.

7. On land with slopes in excess of 12 percent, cuts and fills shall be regulated as follows:

a. Toes of cuts and fills shall be set back from the boundaries of separate private ownerships at least three feet, plus one-fifth of the vertical height of the cut or fill. Where an exception is required from that requirement, slope easements shall be provided.

b. Cuts shall not remove the toe of any slope where a severe landslide or erosion hazard exists.

c. Any structural fill shall be designed by a registered engineer in a manner consistent with the intent of this code and standard engineering practices, and certified by that engineer that the fill was constructed as designed.

d. Retaining walls shall be constructed pursuant to Section 2308(b) of the Oregon State Structural Specialty Code.

e. Roads shall be the minimum width necessary to provide safe vehicle access, minimize cut and fill, and provide positive drainage control.

Comment: As shown on the Grading Plan, all cuts and fills are set back at least three feet from adjacent properties. Only minimal grading is proposed and no cuts would impact landslide potential. Any structural fills will be designed by a registered engineer and will be designed so as to meet the intent and requirements of the code and standard engineering practices. No retaining walls are proposed or required. The proposed shared private drive has been designed to conform to City code and to provide clearances required for safe vehicular access. As shown on the Grading Plan, the driveway grading is minimal, conforming within approximately one foot of native grade. Positive drainage is provided. Storm water from the driveway drains to a lined rain garden on Parcel 3, with an overflow to the storm detention facility at the water reservoir site. As discussed in the storm report dated 4/30/2019, there is adequate capacity at that facility to accommodate the runoff from the subject property.

8. Land over 50 percent slope shall be developed only where density transfer is not feasible. The development will provide that:

a. At least 70 percent of the site will remain free of structures or impervious surfaces.

b. Emergency access can be provided.

c. Design and construction of the project will not cause erosion or land slippage.

d. Grading, stripping of vegetation, and changes in terrain are the minimum necessary to construct the development in accordance with subsection J of this section.

Comment: No land over 50 percent slope is proposed for development in this application.

6123 Skyline Dr. West Linn

tree is 12-inch diameter.
Pac. Dog, madrone, Garry oak is 6-inch diameter.

Tag	Species	Diameter	Rating	Condition
10	Douglas fir	31	2	viable
11	Douglas fir	39	2	viable; 4 large root flares
12	Douglas fir	31	2	viable; 4 large root flares; offsite
13	Douglas fir	36	2	viable; offsite
14	Douglas fir	38	2	viable; added to map; offsite
15	English walnut	11	1	trunk decay; added to map; on or near property line
16	English walnut	12	2	viable
17	Douglas fir	31	2	viable; dead branches
18	Douglas fir	43	2	viable; dead branches; larg root flares
19	western redcedar	15	2	viable; on or near property line behind wood fence; added to map
20	western redcedar	14	2	viable; on or near property line behind wood fence; added to map
21	western redcedar	12	2	viable; on or near property line behind wood fence; added to map
22	Douglas fir	28	2	viable; dead branches
23	Douglas fir	24	2	viable; 20% LCR
24	Douglas fir	19	2	viable; 20% LCR; poor trunk taper
25	Douglas fir	20	0	red-ring rot; hollow
26	Douglas fir	12	2	suppressed
27	Douglas fir	24	2	viable; dead branches
28	Douglas fir	12	2	suppressed
29	Douglas fir	25	2	viable; dead branches; added to map
30	Douglas fir	20	2	added to map; offsite
31	Douglas fir	22	2	added to map; offsite
32	Douglas fir	21	2	one re-grown top @ 70'; 30 LCR; poor trunk taper

Ryan Neumann PN-5539A

Fieldwork done on 9/12/2018

0/dead or hazard 1/decline 2/average 3/excellent

tree is 12-inch diameter.
Pac. Dog, madrone, Garry oak is 6-inch diameter.

33	Douglas fir	29	2	dead branches
34	Douglas fir	31	2	viable
35	Douglas fir	36	2	viable
36	Douglas fir	24	2	viable; large dead branches; offsite approx. 8'
37	Port-Orford cedar	10	2	viable; undersize
38	bigleaf maple	9	2	viable
39	Port-Orford cedar	11	0	dead; undersize
40	Douglas fir	38	2	viable; dead branches; nice tree
41	Douglas fir	36	2	viable; offsite approx. 12'; in neighbor's front yard

Arborist Notes

The owner of Parcels 2 and 3 is proposing to remove the stand of Douglas-fir trees there and the owner of Tax Lot 5300 questions the effect of increased exposure on the continuation of the stand on Tax Lot 5300. I went to the site on 3/22/2019 to evaluate the situation and the proposal.

Looking north into the east portion of Tax Lot 5300 with Tree 18 (T18) on Parcel 2 in the foreground. T18 will be removed. Trees beyond the fence (Tax Lot 5300) constitute an open stand, a continuation of the subject tree on Parcel 2. Most of Parcel 2 and all of Parcel 1 are devoid of trees, providing little or no wind break for Tax Lot 5300.

T18 on Parcel 2.

Looking north from T24 on Parcel 3 into Tax Lot 5300. Here on the west side of Tax Lot 5300 the open stand continues as mixed hardwoods and conifers. The trees on Parcel 3 do provide a wind break for Tax Lot 5300 from southerly winds. The trees on the west side of Tax Lot 5300 are fully exposed to winds from the east, north and west. The elevation is lower that Parcel 2 and this reduces wind exposure from the south.

A bigleaf maple (typical) on the west side of Tax Lot 5300. This is a very open stand of trees.

Conifers in an open stand have exposure to sun and wind. This enables them to develop a strong trunk taper and a robust live crown ratio (LCR). Tree taper is a measure of the decrease in trunk diameter as a function of height above ground. Trees with a high degree of taper are more wind firm and resistant to trunk snap caused by wind. LCR is the ratio of a tree's total height that has foliage vs. bare trunk. A conifer with high LCR is less likely to snap or uproot in a wind storm.

This Douglas-fir tree in Tax Lot 5300 has a high LCR and strong trunk taper. It is characteristic of the conifers on Tax Lot 5300 and on Parcels 2 and 3. These traits indicate an open stand vs. a closed stand. In a closed stand, tree trunks grow with no noticeable taper and with live branches only at the top. Such trees are typical of plantations and the pole-like quality is desirable in the logging industry. They rarely snap or blow down because they all support each other, and because there are edge trees that develop strong taper and LCR by virtue of growing at the edge of the closed stand, exposed to the elements. When edge trees are removed, the interior poles tend to snap and blow down in wind events. None of the subject trees here are interior trees.

A view from Skyline Drive nearby. The mature conifers seen here are doing well as stand-alone trees. This landscape is typical of the neighborhood.

Tree removal on Parcels 2 and 3 will allow increased wind velocity from the south on the trees in Tax Lot 5300. In my opinion the adverse effect will be negligible, and I do not anticipate trunk snap or blow down to result.

Portland Tree Consulting PO Box 19042 Portland, OR 97280
503.421.3883 info@pdxtreeconsulting.com CCB 154349

1. Client warrants any legal description provided to the Consultant is correct and titles and ownerships to property are good and marketable. Consultant shall not be responsible for incorrect information provided by Client.

2. Consultant can neither guarantee nor be responsible for the accuracy of information provided by others.

3. The Consultant shall not be required to give testimony or attend court or hearings unless subsequent contractual arrangements are made, including additional fees.

4. The report and any values expressed therein represent the opinion of the Consultant, and the Consultant's fee is in no way contingent upon the reporting of a specified value, a stipulated result, the occurrence of a subsequent event, nor upon any finding to be reported.

5. Sketches, drawings and photographs in the report are intended as visual aids and may not be to scale. The reproduction of information generated by others will be for coordination and ease of reference. Inclusion of such information does not warrant the sufficiency or accuracy of the information by the Consultant.

6. Unless expressed otherwise, information in the report covers only items that were examined and reflects the condition at the time of inspection. The inspection is limited to visual examination of accessible items without laboratory analysis, dissection, excavation, probing, or coring, unless otherwise stated.

7. There is no warranty or guarantee, expressed or implied, that problems or deficiencies of the plants or property in question may not arise in the future.

8. The report is the completed work product. Any additional work, including production of a site plan, addenda and revisions, construction of tree protection measures, tree work, or inspection of tree protection measures, for example, must be contracted separately.

9. Loss or alteration of any part of the report invalidates the entire report.

Peter Torres

Master of Forestry ASCA RCA 372 ISA Certified Arborist PN-0650B TRAQ Qualified

Preliminary Drainage Report

Skyline Partition

Address: 6123 Skyline, West Linn, Oregon

Date: April 29, 2019

NARRATIVE:

This is a vacant property that is proposed to be divided into 3-lots by partition. This tract that slopes easterly away from Skyline Drive. There isn't a storm sewer system in Skyline Drive and no access to a public storm to the North. The USDA Web Soil Survey reports the soils as being 13B Cascade silt loam and 92F Xerochrepts and Haploxerools. Cascade has is a hydrologic group C and Xerochrepts is hydrologic group B. Two geotechnical reports have been prepared for this Bolton reservoir site which includes this property. Evidence has been provided that an ancient and currently inactive landslide condition exists on the property. The Bolton reservoir site includes both water quality and quantity storm water facilities for the new reservoir site. This residential site was mapped by Centerline Concepts to include illustrating 1-foot contours. This was compared with the West Linn GIS contour map and found to be substantially the same. It is clear the almost 8000 SF of the residential property naturally drains to the Bolton Reservoir site. It does not appear that this area was included in the storm management report for the reservoir.
The GeoPacific report recommends that any storm water facility does not use on-site infiltration as a solution for storm water disposal. Based on this report all rain gardens are to be lined.

ASSUMPTION:

Above ground facilities: flow through lined raingardens
2500 SF roof areas = 0.057acres for individual parcels
7500 SF impervious area total = 0.17 acres
25-year event = 3.9 inches/hour

REFERENCE:

Murray Smith & Associates – Storm water Management Report, September 2015

GRI – Geotechnical Investigation report # 5338, September 10, 2015

GeoPacific – Soils investigation Report # 19-5206, April 29, 2019

City of Portland Storm Water Management Manual

The King County Department of Public Works, Hydrographic Program, ver 4.21B

CALCULATIONS:

STORM OPTIONS:
1 – S.C.S. TYPE -1A
2- 7-DAY DESIGN STPRM
3 – STORM DATA FILE
SPECIFY STORM OPTION:
1
S.C.S. TYPE 1-A RAINFALL DISTRIBUTION
ENTER:
FREQ(YEAR), DURATION(HOUR), PRECIP(INCHES)
25,24.3.9
Xxxxxxxxxxxxxxxxxxxxxx S.C.S.TYPE-1A DISTRIBUTION xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx
XXXXXXXXXXXX 25-YEAR 24-HOUR STORM xxxx 3.90" TOTAL PRECIP. Xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx

---

ENTER: A(PERV),CN(PERV),A(IMPERV),CN(IMPERV),TC FOR BASIN NO. 1
0.0,86,0.057,98,5
DATA PRINT OUT:

AREA(ACRES)	PERVIOUS		IMPERVIOUS		TC(MINUTES)
	A	CN	A	CN	
.1	.1	86.0	.0	98.0	5.0
PEAK-Q(CFS)	T-PEAK(HRS)		VOL(CU-FT)		
.06	7.67		758		

ENTER [d:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:
C:sky
SPECIFY: C - CONTINUE, N - NEWSTORM, P - PRINT, S – STOP
C

ENTER: A(PERV),CN(PERV),A(IMPERV),CN(IMPERV),TC FOR BASIN NO. 1
0.057,86,0.0,98,5
DATA PRINT OUT:

AREA(ACRES)	PERVIOUS		IMPERVIOUS		TC(MINUTES)
	A	CN	A	CN	
.1	.0	86.0	.1	98.0	5.0
PEAK-Q(CFS)	T-PEAK(HRS)		VOL(CU-FT)		
.04	7.67		508		

ENTER [d:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:
C:sky25
SPECIFY: C - CONTINUE, N - NEWSTORM, P - PRINT, S – STOP
C

ENTER: A(PERV),CN(PERV),A(IMPERV),CN(IMPERV),TC FOR BASIN NO. 1

```
0.0,86,0.17,98,5
DATA PRINT OUT:
AREA(ACRES)          PERVIOUS              IMPERVIOUS            TC(MINUTES)
                     A       CN            A       CN
     .2              .0      86.0          .2      98.0          5.0
PEAK-Q(CFS)          T-PEAK(HRS)           VOL(CU-FT)
    .17                 7.67                  2261
ENTER [d:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:
C:ALL25
SPECIFY:  C - CONTINUE,  N - NEWSTORM,  P - PRINT,  S – STOP
S
10
R/D FACILITY DESIGN ROUTINE
SPECIFY TYPE OF R/D FACILITY:
1 – POND                  4 – INFILTRATION POND
2 – TANK                  5 – INFILTRATION TANK
3 – VAULT                 6 – GRAVEL TRENCH/BED
1
ENTER: POND SIDE SLOPE (HORIZ. COMPONENT)
3
ENTER: EFFECTIVE STORAGE DEPTH (ft) BEFORE OVERFLOW
1
ENTER [d:][pathfilename[.ext} OF PRIMARY DESIGN INFLOW HYDROGRAPH
C: SKY
PRIMARY DESIGN INFLOW PEAK =      .06 CFS
ENTER PRIMARY DESIGN RELEASE RATE (cfs)
0.04
ENTER NUMBER OF INFLOW HYDROGRAPHS TO BE TESTED FOR PERFORMANCE (5 MAXIMUM):
0
ENTER: NUMBER OF ORIFICES,  RISER-HEAD(ft),  RISER-DIAMETER(in)
0,1,6
RISER OVERFLOW DEPTH FOR PRIMARY PEAK INFLOW =  0.05
SPECIFY ITERATION DISPLAY:  Y – YES,  N – NO
N
SPECIFY:  R – REVIEW/REVISE INPUT,  C – CONTINUE
C
INITIAL STORAGE VALUE FOR ITERATION PURPOSES:      285 CU-FT
SINGLE ORIFICE RESTRICTOR:  DIA= 1.21 "
PERFORMANCE:   INFLOW   TARGET-OUTFLOW   ACTUAL-OUTFLOW   PK-STAGE   STORAGE
   DESIGN HYD:    .06          .04            .04          1.00         22
```

SIZING:

A raingarden with outside dimensions of 10 X 15feet and 3:1 slopes with 1-foot of surface storage, 1.5-feet of medium, and 1-foot of drain rock has a storage capacity of approximately 248 CF, and 150 SF of surface area. Pursuant to the City of Portland Storm Water Management

Manual, using the simplified approach water quality is 6% of the impervious area or 150 SF. An orifice placed in each flow through rain garden will control the outflow to the pre-developed condition for the 25-year event with an orifice of 1.21". Limiting flow for other storm events would be difficult because the orifices would become too small to maintain.
The Murray Smith report for the Bolton reservoir reports a detention pond with a top elevation of 435 and with the emergency overflow set at elevation 434. There are three orifices at the facility (1.5" @ 432, 1.0" @ 430, 1.0" @ 425.5) The 2,5,10,25, & 100 year storm events were calculated and indicate a corresponding detention pond elevation of 428.87 (2yr), 429.82 (5yr), 430.62 (10yr), 432.34 (25yr), and 432.04 (100yr) thus showing additional capacity within the detention point of 434-432.34 = 1.7 feet.

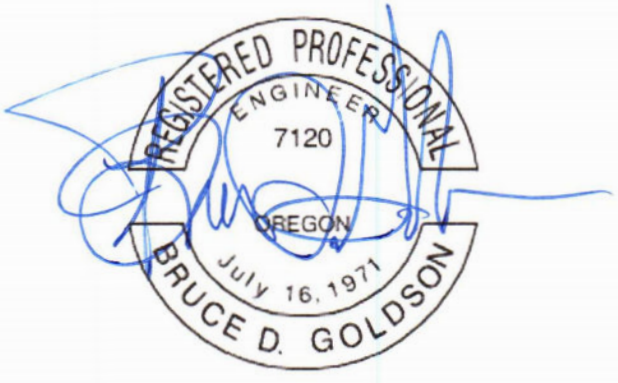
CONCLUSION:

The site specific soils report recommends that infiltration not be used to dispose of storm water generated from the new impervious surfaces. Calculations show that it is feasible to provide water quality and quantity flow through lined rain gardens for each parcel and direct the flow to the Bolton reservoir facility. A portion of this residential site appears to have been unaccounted for in the Murry Smith report and in comparing the GSI contours and the current field contours it evident that some of the property does slope to the reservoir site. A review of the Murry Smith report finds additional capacity available in the detention pond and it is practical to convey storm water from the new impervious surfaces to the detention pond.

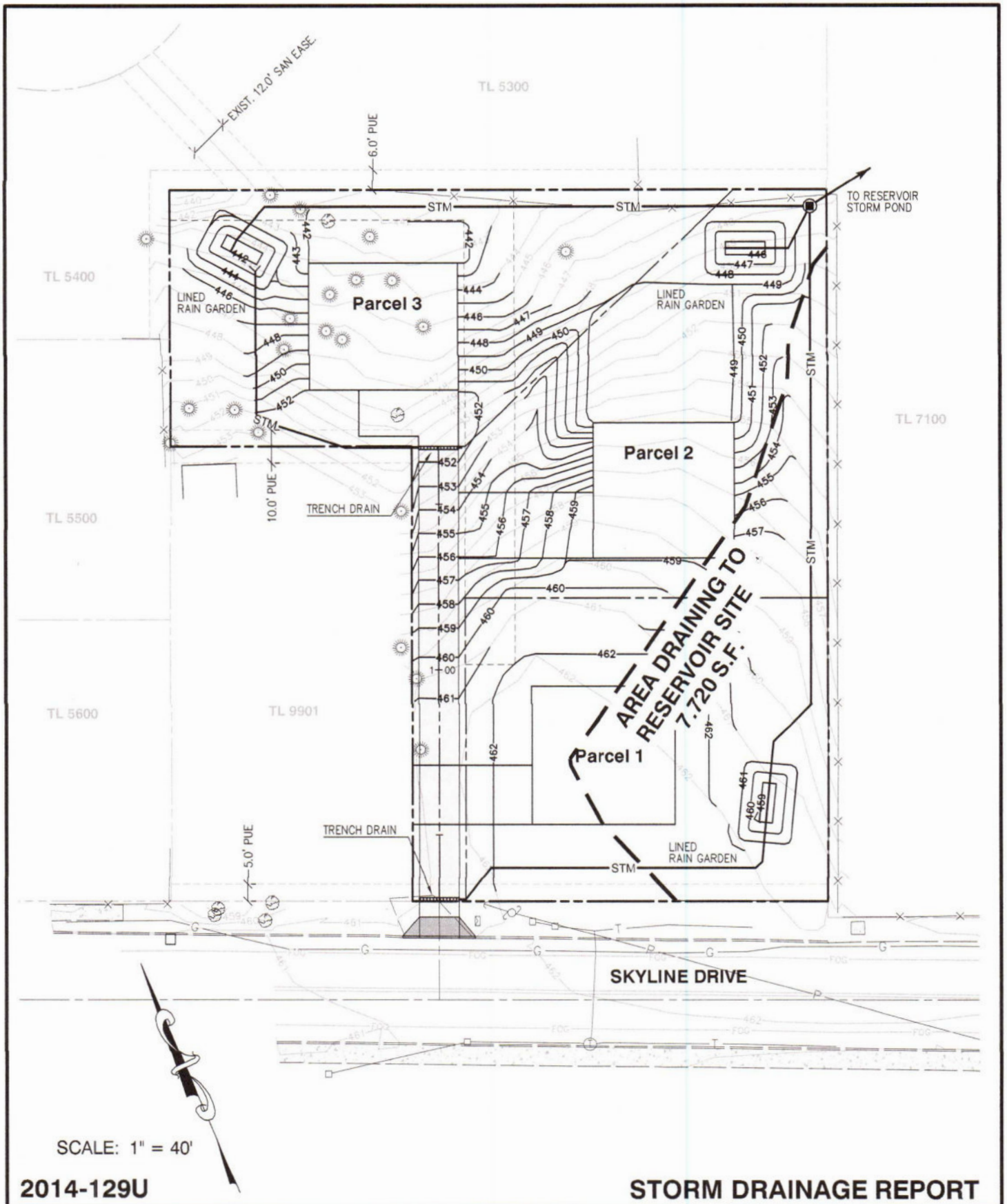
Prepared by:

Bruce D. Goldson, PE

Theta,LLC
PO Box 1345
Lake Oswego, Oregon 97035



EXPIRES: 06/30/2019
SIGNATURE DATE: 4/30/19



2014-129U

STORM DRAINAGE REPORT

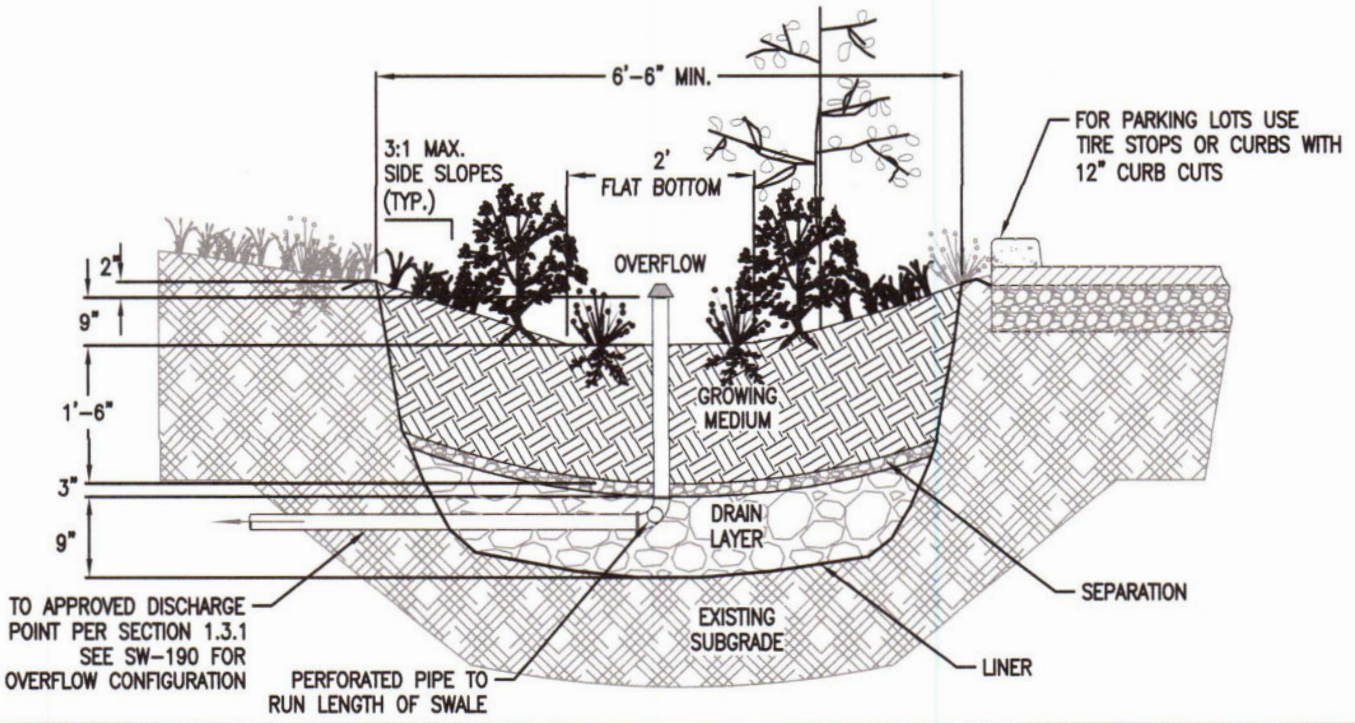
Theta, llc

ENGINEERING - SURVEYING - PLANNING

PO Box 1345
Lake Oswego, Oregon 97035

503-481-8822
email: thetaeng@comcast.net

Partition Site Plan
6123 Skyline Drive
West Linn, Oregon



1. Dimensions:
   Width of swale: 6'-6" minimum
   Depth of swale (from top of growing medium to overflow elevation): 9".
   Longitudinal slope of swale: 6.0% or less.
   Flat bottom width: 2' recommended.
   Side slopes of swale: 3:1 maximum.

2. Overflow: Swales must connect to approved discharge point according to SWMM Section 1.3.1 and detail SW-190.
   Inlet elevation must allow for 2" of freeboard, minimum.
   Protect from debris and sediment with strainer or grate.

3. Piping must be cast iron, ABS or PVC. 3" pipe required for facilities draining up to 1500 s.f., otherwise 4" minimum pipe. Uniform Plumbing Code also applies.

4. Drain Layer:
   3/4" – 1 ½" washed round rock. Depth: 9".
   Separation between drain rock and growing medium:
   Pea gravel lens, 2 to 3 inches deep.

5. Growing Medium:
   18" minimum depth. Use sand/loam/compost 3-way mix, or approved mix that will support healthy plants.
   24" minimum depth is required if the lined facility is also meeting BDS landscape requirements.

6. Vegetation: Follow landscape plans otherwise refer to plant list in SWMM section 2.4.1. Minimum container size is #1. # of plantings per 100sf of facility area:
   Zone A (wet): 80 herbaceous plants OR 72 herbaceous plants and 4 small shrubs.
   Zone B (moderate to dry): 7 large or small shrubs AND 70 groundcover plants.
   The delineation between Zone A and B must be either at the outlet elevation or the check dam elevation, whichever is lowest.
   If project area is over 200sf consider adding a tree.

7. Check Dams: Must be placed every 10' where slope exceeds 4% and be equal to the width of the planter.

8. Waterproof Liner: 30 mil EPDM, HDPE or approved equivalent.

9. Splash Block: Install 4-6" washed river rock or splash pad for erosion control at inlets and downspout.

10. Inspections: Call BDS MR Inspection Line, (503) 823-7000, request 487. 3 inspections required.

- DRAWING NOT TO SCALE -

STORMWATER MANAGEMENT TYPICAL DETAILS

– Simplified Design Approach –

Swale – lined

Bureau of Environmental Services

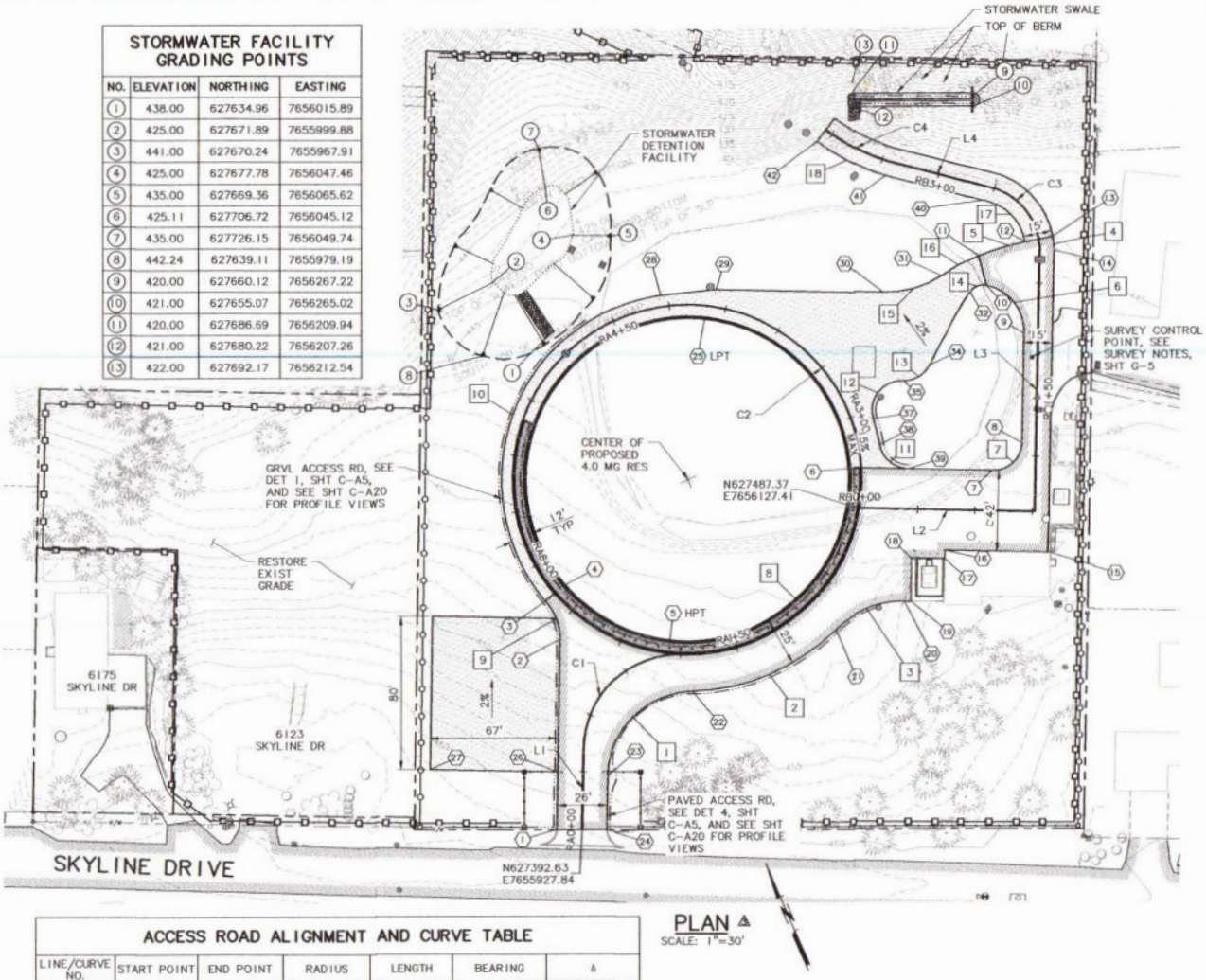
NUMBER

SW-130

7-1-16

STORMWATER FACILITY GRADING POINTS

NO.	ELEVATION	NORTHING	EASTING
1	438.00	627634.96	7656015.89
2	425.00	627671.89	7655999.88
3	441.00	627670.24	7655967.91
4	425.00	627677.78	7656047.46
5	435.00	627669.36	7656065.62
6	425.11	627706.72	7656045.12
7	435.00	627726.15	7656049.74
8	442.24	627639.11	7655979.19
9	420.00	627660.12	7656267.22
10	421.00	627655.07	7656265.02
11	420.00	627686.69	7656209.94
12	421.00	627680.22	7656207.26
13	422.00	627692.17	7656212.54



PLAN
SCALE: 1"=30'

ACCESS ROAD ALIGNMENT AND CURVE TABLE

LINE/CURVE NO.	START POINT	END POINT	RADIUS	LENGTH	BEARING	Δ
L1	STA RA0+00	STA RA0+42	N/A	42.00	N24°34'27"E	N/A
C1	STA RA0+42	STA RA1+21	50.00	79.30	N/A	90°52'03"
C2	STA RA1+21	STA RA6+93	91.00	571.75	N/A	360°00'00"
L2	STA RB0+00	STA RB0+91	N/A	91.25	S65°11'02"E	N/A
L3	STA RB0+91	STA RB2+27	N/A	136.22	N24°48'58"E	N/A
C3	STA RB2+27	STA RB2+74	34.93	46.96	N/A	77°01'58"
L4	STA RB2+74	STA RB3+27	N/A	52.11	N52°13'59"W	N/A
C4	STA RB3+27	STA RB3+64	119.48	37.25	N/A	17°51'46"

EDGE OF PAVEMENT ELEVATION TABLE

POINT	ELEVATION	POINT	ELEVATION
1	460.68	14	440.00
2	451.00	15	448.25
3	448.17	16	447.35
4	448.89	17	447.43
5	451.00	18	447.29
6	451.00	19	448.05
7	446.16	20	448.03
8	446.95	21	448.32
9	440.05	22	451.50
10	439.95	23	457.86
11	439.95	24	460.43
12	439.90	25	440.30
13	439.85		

EDGE OF GRAVEL ELEVATION TABLE

POINT	ELEVATION	POINT	ELEVATION
26	457.75	35	442.65
27	456.99	36	NOT USED
28	440.67	37	441.74
29	439.01	38	444.50
30	439.59	39	444.95
31	439.80	40	435.00
32	439.95	41	429.10
33	NOT USED	42	425.50
34	441.97		

EDGE OF PAVEMENT CURVE TABLE

NO.	PC	PT	DELTA	RADIUS	LENGTH
1	N627435.61 E7656009.40	N627412.72 E7655951.33	87°49'53"	45.00'	68.98'
2	N627498.15 E7656154.13	N627435.61 E7656009.40	91°33'20"	110.00'	175.77'
3	N627433.70 E7656129.17	N627434.67 E7656090.89	45°02'00"	50.00'	39.30'
4	N627570.23 E7656274.48	N627574.29 E7655275.83	5°47'03"	42.43'	4.28'
5	N627579.57 E7656261.69	N627582.57 E7656236.74	11°47'45"	122.29'	25.18'
6	N627536.19 E7656242.24	N627566.90 E7656234.25	78°47'54"	25.00'	34.38'
7	N627477.58 E7656198.62	N627484.90 E7656218.53	90°00'00"	15.00'	23.56'
8	N627512.58 E7655970.01	N627508.68 E7656131.36	143°24'20"	85.00'	212.75'
9	N627487.88 E7655957.11	N627509.08 E7655958.53	41°29'17"	30.00'	21.72'

EDGE OF GRAVEL CURVE TABLE

NO.	PC	PT	DELTA	RADIUS	LENGTH
10	N627630.98 E7656076.52	N627509.08 E7655958.53	121°57'51"	97.00'	206.48'
11	N627513.19 E7656150.44	N627496.03 E7656158.72	78°51'30"	15.00'	20.65'
12	N627538.91 E7656174.26	N627526.94 E7656152.80	110°00'57"	15.00'	28.80'
13	N627538.91 E7656174.26	N627540.78 E7656190.34	64°58'19"	15.07'	17.09'
14	N627566.90 E7656234.25	N627566.97 E7656224.95	71°08'30"	8.00'	9.93'
15	N627585.18 E7656183.84	N627580.95 E7656214.81	34°16'51"	53.04'	31.73'
16	N627580.95 E7656214.81	N627582.57 E7656236.74	10°18'51"	122.29'	22.01'
17	N627579.57 E7656261.69	N627608.29 E7656250.37	68°29'49"	27.43'	32.79'
18	N627640.26 E7656209.11	N627671.31 E7656183.69	18°30'00"	124.83'	40.31'

NO.	DATE	REVISION	BY
A	2/20/15	SITE ACCESS REVISIONS	ANB
A	7/17/15	SITE GRADING REVISIONS	ANB
A		RECORD DRAWING	ANB

DESIGNED: MLM
DRAWN: DKH
CHECKED: TPB
APPROVED: TPB

SHEET C-A4
20 of 189

RECORD DRAWING
SEE DISCLAIMER, SHEET 1.
VERSION 4.1
12-9-97

VERT. AS SHOWN
HORZ. AS SHOWN
SCALE

NOTICE
THIS BAR DOES NOT MEASURE 1" ON THIS DRAWING IS NOT TO SCALE

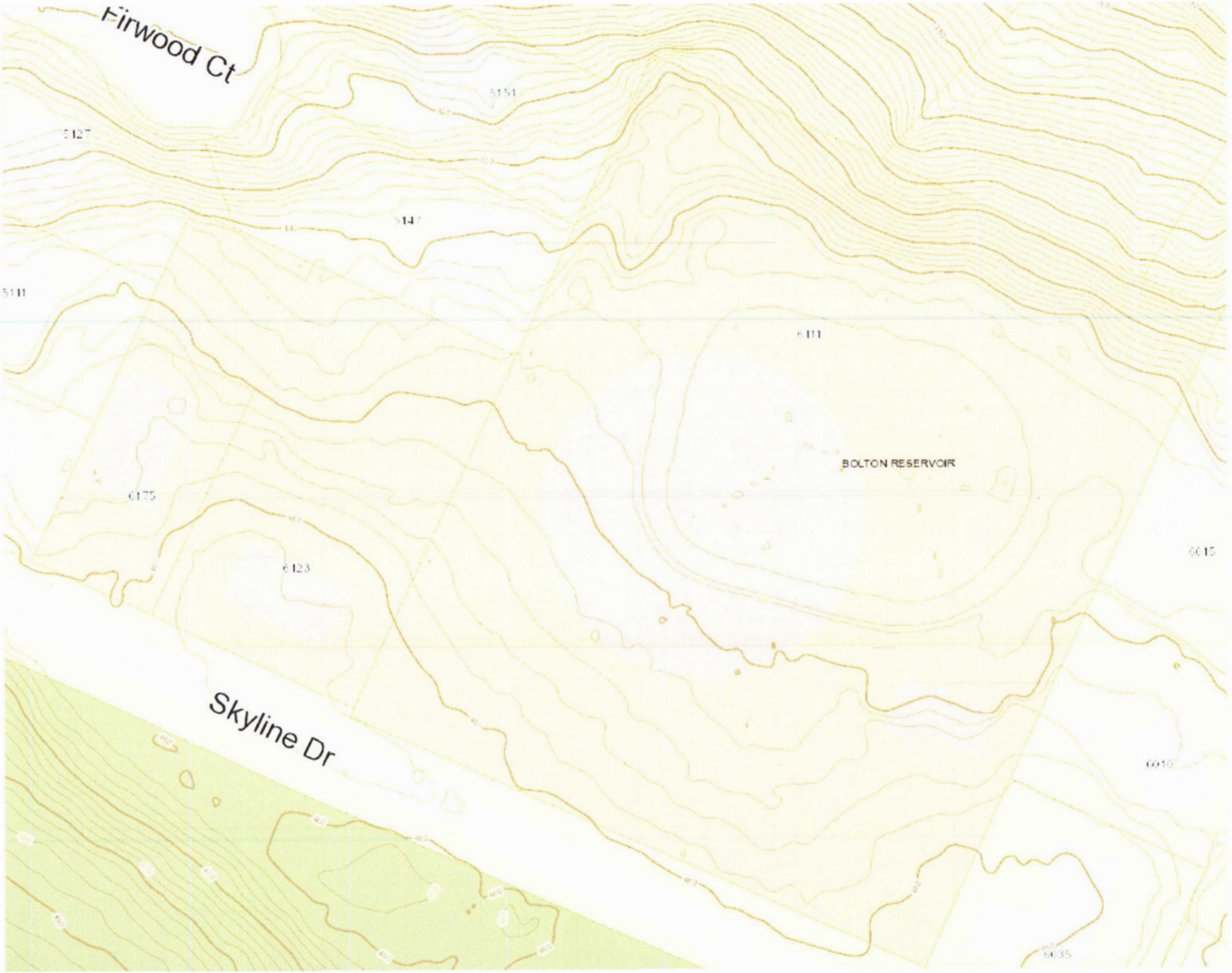
PROJECT NAME: CITY OF WEST LINN, OREGON
BOLTON RESERVOIR REPLACEMENT
PROJECT NO. PW 14-06

SHEET TITLE: **RESERVOIR SITE GRADING AND ACCESS ROAD PAVING PLAN**

MSA Murray, Smith & Associates, Inc. Engineers/Planners
121 SW Salmon, Suite 1100 Portland, Oregon 97204
PHONE 503-225-9010 FAX 503-225-9022

MSA PROJECT 14-1586 DATE: SEPTEMBER 2015

G:\PDX_Projects\14\1586 - Bolton Reservoir Replacement\CAD\Record Drawings\Sheets\SCHED A\14-1586-OR-A-C1V-R.dwg C-A4 6/8/2018 3:21 PM NICK MCFADDIN 21.0s (LMS Tech)


Firwood Ct
5151
5127
5147
5111
6111
BOLTON RESERVOIR
6175
6123
6015
Skyline Dr
6010
6035

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

Soils Investigation Report

6123 Skyline Drive Partition
West Linn, Oregon 97068

GeoPacific Engineering, Inc. Project No. 19-5206
April 29, 2019

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel (503) 598-8445
Fax (503) 941-9281

TABLE OF CONTENTS

List of Appendices

Figures
Exploration Logs
Site Research
Photographic Log

List of Figures

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

April 29, 2019
Project No. 19-5206

Mr. Darren Gusdord
ICON Construction
1980 Willamette Falls Drive, #200
West Linn, Oregon 97068
Phone: (503) 657-0406
Email: darren@iconconstruction.net

SUBJECT: **GEOTECHNICAL ENGINEERING REPORT**
6123 SKYLINE DRIVE PARTITION
WEST LINN, OREGON 97068

1.0 PROJECT INFORMATION

This report presents the results of a geotechnical engineering study conducted by GeoPacific Engineering, Inc. (GeoPacific) for the above-referenced project. The purpose of our investigation was to evaluate subsurface conditions at the site, and to provide geotechnical recommendations for site development. This geotechnical study was performed in accordance with GeoPacific Proposal No. P-6958, dated April 15, 2019, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

Site Location:	6123 Skyline Drive West Linn, Oregon 97068 Clackamas County Parcel No. 00377666 (see Figures 1 through 3)
Developer:	ICON Construction 1980 Willamette Falls Drive, #200 West Linn, Oregon 97068 Phone: (503) 657-0406
Jurisdictional Agency:	City of West Linn, Oregon
Geotechnical Engineer:	GeoPacific Engineering, Inc 14835 SW 72nd Avenue Portland, Oregon 97224 Tel (503) 598-8445 Fax (503) 941-9281

2.0 SITE AND PROJECT DESCRIPTION

As indicated on Figures 1 through 3, the subject site is located at 6123 Skyline Drive in West Linn, Oregon. The site consists of Clackamas County Parcel No. 00377666, totaling approximately 0.75-acres in size. The site latitude and longitude are 45.368147, -122.625978, and the legal description is the SE ¼ NE ¼ of Section 25, T2S, R1E, Willamette Meridian. The site is bordered by Skyline Drive to the south, by existing residential properties to the north and west, and by the City of West Linn Bolton Reservoir to the east. Historically the property contained a residential home which was located in the southern portion of the property adjacent Skyline Drive. The remainder of the property was primarily surfaced with lawn and landscaping. A garden was present in the approximate center of the site. The northern and western portion of the property contained large trees. The home was removed in 2016 during re-construction of Bolton Reservoir to the east. The property was utilized as a construction staging area which included placement of soil, gravel, and various equipment. A gravel pad was constructed extending from Skyline Drive to the approximate center of the property and a rectangular shaped working pad area was created encompassing the central and southern portion of the property. Following completion of the reservoir reconstruction the site was cleared, leveled, and revegetated with grass. Currently vegetation at the site primarily consists of grasses, weeds, and other brush, with trees still present in the northwestern portion of the site. Topography at the site is relatively level to gently sloping to the north with site elevations ranging from approximately 439 to 462 feet above mean sea level (amsl). Beyond the property line to the north/northwest topography becomes moderately to steeply sloping to the north and northwest, extending to Firwood Court below. Firwood Court is at an elevation of approximately 410 feet amsl.

Based upon communication with the client and review of a preliminary site and grading plan prepared by Theta LLC, GeoPacific understands that the proposed development at the site will consist of a three-lot property partition to support construction of new residential homes, construction of a private access drive extending from Skyline Drive to the lots, construction of individual lot stormwater swales, and installation of associated underground utilities. The site plan indicates the approximate locations of the proposed building footprints (see Figure 3). We anticipate that the homes will be two-stories, constructed with typical spread foundations and wood framing, with maximum structural loading on column footings and continuous strip footings on the order of 10 to 35 kips, and 2 to 6 kips respectively. Based on review of the grading plan we understand that cuts and fills on the order of three feet or less have been proposed.

2.1 State of Oregon Landslide Hazard Mapping

We have reviewed the State of Oregon Department of Geology and Mineral Industries (DOGAMI) landslide hazard and inventory mapping, and SLIDO LiDAR imagery which indicates that the site is located within a large ancient landslide area mapped and identified as landslide No. Canby 133. The DOGAMI mapping indicates that the landslide consists of a rock slide or translational slide with an average slope of 15 percent, and a failure depth of 38.6 feet. The direction of failure is reported to be approximately N45°E. Many homes, roads, and public infrastructure are built across the slide area including the Bolton Reservoir.

Detailed geotechnical evaluation of the Canby 133 landslide is beyond the scope of this study, however we have reviewed available public literature regarding the slide. We reviewed a technical

memorandum prepared for the City of West Linn regarding reservoir siting alternatives for the Bolton Reservoir, prepared by Murray, Smith & Associates, Inc., of Portland, Oregon, dated September 24, 2014. The memorandum and subsequent information posted to the public on City of West Linn websites indicates that the city was aware of the DOGAMI landslide mapping during planning phase of the reservoir re-construction and considered alternative sites due to potential risk of future sliding at the existing site. We understand that after over ninety other sites were assessed, it was determined that the existing site was the most suitable for re-construction of the reservoir. Measures were apparently implemented to reduce risks associated with future sliding which included achieving greater slope setbacks from localized sloping areas to the north and constructing deep foundation ground improvements consisting of 812 rammed aggregate piers to depths of approximately 27 feet bgs. In addition, we understand that soil was removed from steep slopes at the site and extensive drainage was installed around and beneath the reservoir structure to allow ground and surface water seepage to flow through. The image below indicates the DOGAMI landslide mapping and the location of the subject site.

Image: SLIDO Statewide Landslide Information Layer for Oregon, DOGAMI

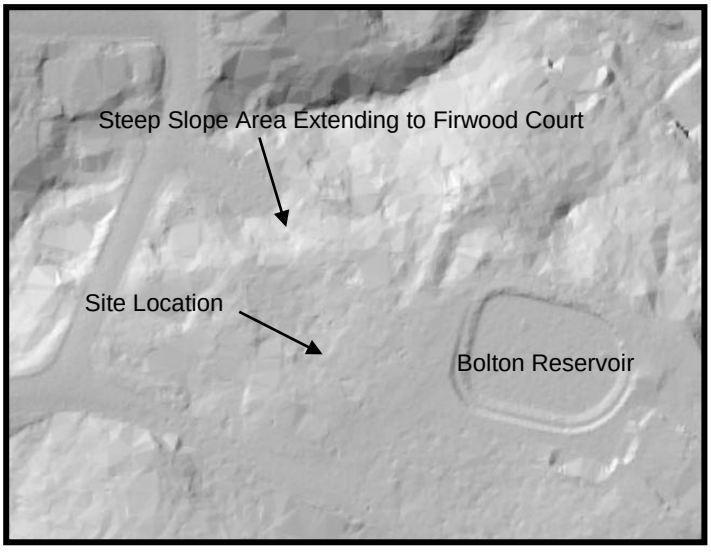


Image: LiDAR HAZVU Statewide Landslide Information Layer for Oregon, DOGAMI

3.0 REGIONAL GEOLOGIC SETTING

Regionally, the subject site lies within the Willamette Valley/Puget Sound lowland, a broad structural depression situated between the Coast Range on the west and the Cascade Range on the east. A series of discontinuous faults subdivide the Willamette Valley into a mosaic of fault-bounded, structural blocks (Yeats et al., 1996). Uplifted structural blocks form bedrock highlands, while down-warped structural blocks form sedimentary basins.

The *Generalized Geologic Map of the Willamette Lowland, Marshall W. Gannett and Rodney R. Caldwell, (U.S. Department of the Interior, U.S. Geological Survey, 1998)*, indicates that the site is underlain by Miocene-aged (approximately 23 to 11 million years ago) Columbia River basalt flows, which consist of phyric basalt and basaltic-andesite flows erupted eastern Oregon, Washington, and Idaho, (Tcr). The basalts are generally composed of dense, finely crystalline rock that is commonly fractured along blocky and columnar vertical joints.

The *Web Soil Survey (United States Department of Agriculture, Natural Resource Conservation Service (USDA NRCS 2019 Website)*, indicates that near-surface soils consist of the Cascade Silt Loam soil series. Cascade series soils generally consist of moderately deep to a fragipan, poorly drained soils that formed in silty materials.

4.0 REGIONAL SEISMIC SETTING

At least three major fault zones capable of generating damaging earthquakes are thought to exist in the vicinity of the subject site. These include the Portland Hills Fault Zone, the Gales Creek-Newberg-Mt. Angel Structural Zone, and the Cascadia Subduction Zone.

4.1 Portland Hills Fault Zone

The Portland Hills Fault Zone is a series of NW-trending faults that include the central Portland Hills Fault, the western Oatfield Fault, and the eastern East Bank Fault. These faults occur in a northwest-trending zone that varies in width between 3.5 and 5.0 miles. The combined three faults reportedly vertically displace the Columbia River Basalt by 1,130 feet and appear to control thickness changes in late Pleistocene (approx. 780,000 years) sediment (Madin, 1990). The Portland Hills Fault occurs along the Willamette River at the base of the Portland Hills and is located approximately 3 miles northeast of the site. The Oatfield Fault occurs along the western side of the Portland Hills and is located approximately 2.3 miles northeast of the site. The East Bank Fault occurs along the eastern margin of the Willamette River, and is located approximately 7.3 miles southeast of the site. The accuracy of the fault mapping is stated to be within 500 meters (Wong, et al., 2000).

According to the USGS Earthquake Hazards Program, the fault was originally mapped as a down-to-the-northeast normal fault but has also been mapped as part of a regional-scale zone of right-lateral, oblique slip faults, and as a steep escarpment caused by asymmetrical folding above a south-west dipping, blind thrust fault. The Portland Hills fault offsets Miocene Columbia River Basalts, and Miocene to Pliocene sedimentary rocks of the Troutdale Formation. No fault scarps on surficial Quaternary deposits have been described along the fault trace, and the fault is mapped as buried by the Pleistocene aged Missoula flood deposits. No historical seismicity is correlated with the mapped portion of the Portland Hills Fault Zone, but in 1991 a M3.5 earthquake occurred

on a NW-trending shear plane located 1.3 miles east of the fault (Yelin, 1992). Although there is no definitive evidence of recent activity, the Portland Hills Fault Zone is assumed to be potentially active (Geomatrix Consultants, 1995).

4.2 Gales Creek-Newberg-Mt. Angel Structural Zone

The Gales Creek-Newberg-Mt. Angel Structural Zone is a 50-mile-long zone of discontinuous, NW-trending faults that lies about 17.7 miles southwest of the subject site. These faults are recognized in the subsurface by vertical separation of the Columbia River Basalt and offset seismic reflectors in the overlying basin sediment (Yeats et al., 1996; Werner et al., 1992). A geologic reconnaissance and photogeologic analysis study conducted for the Scoggins Dam site in the Tualatin Basin revealed no evidence of deformed geomorphic surfaces along the structural zone (Unruh et al., 1994). No seismicity has been recorded on the Gales Creek Fault or Newberg Fault (the fault closest to the subject site); however, these faults are considered to be potentially active because they may connect with the seismically active Mount Angel Fault and the rupture plane of the 1993 M5.6 Scotts Mills earthquake (Werner et al. 1992; Geomatrix Consultants, 1995).

According to the USGS Earthquake Hazards Program, the Mount Angel fault is mapped as a high-angle, reverse-oblique fault, which offsets Miocene rocks of the Columbia River Basalts, and Miocene and Pliocene sedimentary rocks. The fault appears to have controlled emplacement of the Frenchman Spring Member of the Wanapum Basalts, and thus must have a history that predates the Miocene age of these rocks. No unequivocal evidence of deformation of Quaternary deposits has been described, but a thick sequence of sediments deposited by the Missoula floods covers much of the southern part of the fault trace.

4.3 Cascadia Subduction Zone

The Cascadia Subduction Zone is a 680-mile-long zone of active tectonic convergence where oceanic crust of the Juan de Fuca Plate is subducting beneath the North American continent at a rate of 4 cm per year (Goldfinger et al., 1996). A growing body of geologic evidence suggests that prehistoric subduction zone earthquakes have occurred (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). This evidence includes: (1) buried tidal marshes recording episodic, sudden subsidence along the coast of northern California, Oregon, and Washington, (2) burial of subsided tidal marshes by tsunami wave deposits, (3) paleoliquefaction features, and (4) geodetic uplift patterns on the Oregon coast. Radiocarbon dates on buried tidal marshes indicate a recurrence interval for major subduction zone earthquakes of 250 to 650 years with the last event occurring 300 years ago (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). The inferred seismogenic portion of the plate interface lies approximately along the Oregon Coast at depths of between 20 and 40 kilometers below the surface.

5.0 FIELD EXPLORATION AND SUBSURFACE CONDITIONS

Our subsurface explorations for this report were conducted on April 19, 2019. Four exploratory test pits (TP-1 through TP-4) were excavated at the site using a track-mounted excavator provided by the client to a maximum depth of approximately 11 feet bgs. Explorations were conducted under the full-time observation of a GeoPacific engineer. The primary purpose of the explorations was to determine depths and soil consistency of undocumented fill soils known to be present on

the parcel by the developer who had previously conducted several excavator test pits at the property and identified up to 9 feet of undocumented fill soils. It appears that the undocumented fill soils were likely placed at the site during re-construction of the Bolton Reservoir.

During our explorations, pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and groundwater occurrence were recorded. Soils were classified in accordance with the Unified Soil Classification System (USCS). Soil samples obtained from the explorations were placed in relatively air-tight plastic bags. The test pits were loosely backfilled with onsite soils. The approximate locations of the explorations are indicated on Figures 2 and 3. It should be noted that exploration locations were located in the field by pacing or taping distances from apparent property corners and other site features shown on the plans provided. As such, the locations of the explorations should be considered approximate. Summary exploration logs are attached. The stratigraphic contacts shown on the individual test pit logs represent the approximate boundaries between soil types. The actual transitions may be more gradual. The soil and groundwater conditions depicted are only for the specific dates and locations reported, and therefore, are not necessarily representative of other locations and times. Soil and groundwater conditions encountered in the explorations are summarized below.

5.1 Soil Descriptions

Topsoil: At the locations of our test pits, the ground surface was generally vegetated by grass and weeds. The top soil horizon was primarily observed to consist of dark brown, very moist, organic SILT (OL-ML), with roots extending to approximately 6 to 8 inches bgs.

Undocumented Fill: At the locations of our test pits, the grassy topsoil layers were found to be underlain by approximately 1 to 9 feet of undocumented fill soils consisting of a range of materials.

- At the location of Parcel 1, undocumented fill soils were observed to be present on the order of 1 to 3 feet thick consisting of stiff, reddish brown, moist, Clayey SILT (ML), containing trace concrete debris.
- At the location of Parcel 2, undocumented fill soils were observed to be present on the order of 9 feet thick, consisting of medium stiff, dark brown, moist, Clayey SILT (ML), containing subrounded cobble sized rock, wood debris, and a buried drain field.
- At the location of Parcel 3, undocumented fill soils were observed to be present on the order of 2 to 2.5 feet thick, consisting of loose, dark brown, moist, SILT (ML), containing roots.

SILT (Loess): Underlying the undocumented fill soils at the site, apparent native soils were encountered consisting of very stiff, brown to light brown, moist, micaceous, SILT (ML). The soil type appears to represent wind-blown loess which likely blanketed clayey residual soils and bedrock mapped as being present at the site. The soil type extended to the maximum depth of exploration within our test pits. Review of available well logs indicates that the loess has been found to range in thickness from 10 to 40 feet in other drilling explorations conducted on Skyline Drive (see Site Research Appendix).

5.2 Shrink-Swell Potential

Fine-grained SILT displaying low-plasticity characteristics was encountered within our subsurface explorations. The shrink-swell potential of near surface soils are considered to be low and is not anticipated to require special design measures where structures are proposed.

5.3 Groundwater and Soil Moisture

On April 19, 2019, the observed soil moisture conditions were generally moist. Groundwater seepage was not encountered within our explorations which extended to a maximum depth of 11 feet bgs. Based upon review of available well logs obtained from the State of Oregon Water Resources Department Well Log Query Report, static groundwater is commonly encountered at depths of 20 to 40 feet bgs in the vicinity of the subject site. Perched groundwater may be encountered in localized areas. Seeps and springs may exist in areas not explored and may become evident during site grading.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Our site investigation indicates that the proposed development appears to be geotechnically feasible, provided that the recommendations of this report are incorporated into the design and construction phases of the project. The primary geotechnical concerns associated with development at this site are:

1. The presence of 1 to 9 feet of variable undocumented fill soils at the site. It appears that the fill soils are present up to 3 feet thick on Parcels 1 and 3, and up to 9 feet thick on Parcel 2. The existing undocumented fill soils are not considered to be suitable to provide adequate bearing support for construction of foundations. Differential settlement is a concern due to variable soil conditions. At building lots where the undocumented fill soils are shallow it may be feasible to remove, scarify, sort, and replace the soils in the upper 2 to 3 feet as engineered fill. Alternatively, foundations may extend through the fill soils to bear directly on competent native soils. Where the fill soils were observed to be present to depths greater than 3 or 4 feet, either full removal of the fill and replacement as engineered fill should be conducted, or deep foundations such as rammed aggregate piers may be considered. See Section 6.1, Site Preparation Recommendations, and Section 6.6, Spread Foundations, for more detail.

2. The site is located on a large ancient landslide identified by DOGAMI as Canby 133. Extensive development is present across the landslide. Detailed evaluation of the Canby 133 landslide and the affect it may have on the proposed development is beyond the scope of this study. It appears that with consideration to the degree of surrounding development, the understanding that the landside is considered to be ancient, and the long history of residential homes on the property, the overall landslide mass may be relatively stable. However, given the noted information and mapping, if additional study is determined to be needed by the client or local building official, we would recommend conducting soil borings and a quantitative slope stability of the sloping areas on the north end of the property to determine if additional stabilization measures may be desired or needed for the homesites.

3. A moderate to steep slope is present beyond the northwestern property line that extends to Firwood Court below. Parcel 3 will be located above the slope as shown on Figure 3. The hillside is heavily vegetated and showed no signs of recent erosion or instability at the time of our study. Based on review of the site plan and the indicated building envelope for Parcel 3 it appears that 20 to 30 feet of setback is proposed from the top of the slope which is approximately 30 feet high. Recommendations regarding adequate footing-to-slope setback distances for foundations are presented below in Section 6.6, Spread Foundations.

4. Due to the site being located on a large ancient landslide, and the presence of a steep slope area to the north of the property, we recommend that the grading plan be adjusted to eliminate any raising of grades or additional soil surcharges on the building lots. Grade adjustments within the proposed drive and driveways appear to be feasible.

5. Proposed stormwater ponds near the tops of steep slope areas should be lined with an appropriate liner so that the ponds are impermeable. In no case shall stormwater be directed or allowed to flow freely over the slope faces.

6.1 Site Preparation Recommendations

Areas of proposed construction and areas to receive fill should be cleared of any organic and inorganic debris, and loose stockpiled soils. Inorganic debris and organic materials from clearing should be removed from the site or spread back over the lots as topsoil. Organic-rich soils and root zones should then be stripped from construction areas of the site or where engineered fill is to be placed. Depth of stripping of existing grassy organic topsoil is estimated to be approximately 6 to 8 inches across the majority of the site, however depth of organic soil layers may increase to 24 to 30 inches in areas where trees and vegetation are present.

As previously noted, undocumented fill soils were encountered within our subsurface explorations:

- At the location of Parcel 1, undocumented fill soils were observed to be present on the order of 1 to 3 feet thick consisting of stiff, reddish brown, moist, Clayey SILT (ML), containing trace concrete debris. The fill soils are underlain by stiff native silts. We recommend either: 1) excavating and recompacting the undocumented fill soils to achieve 95 percent compaction relative to ASTM D698; or 2) leave the fill material in place and over-excavate and extend foundation elements through the fill soils to bear directly on firm native soil.

- At the location of Parcel 2, undocumented fill soils were observed to be present on the order of 9 feet thick, consisting of medium stiff, dark brown, moist, Clayey SILT (ML), containing subrounded cobble sized rock, wood debris, and a buried drain field. The fill soils are underlain by stiff native silts. We recommend either: 1) excavating and recompacting the undocumented fill soils to achieve 95 percent compaction relative to ASTM D698; or 2) leave the fill material in place and install rammed aggregate piers to support the proposed home foundation.

- At the location of Parcel 3, undocumented fill soils were observed to be present on the order of 2 to 2.5 feet thick, consisting of loose, dark brown, moist, SILT (ML), containing roots. The fill soils are underlain by stiff native silts. We recommend either: 1) excavating

and recompacting the undocumented fill soils to achieve 95 percent compaction relative to ASTM D698; or 2) leave the fill material in place and over-excavate and extend foundation elements through the fill soils to bear directly on firm native soil.

The final depth of soil removal should be determined by the geotechnical engineer or designated representative during site inspection while stripping/excavation is being performed. Stripped topsoil should be removed from areas proposed for placement of engineered fill. Any remaining topsoil should be stockpiled only in designated areas and stripping operations should be observed and documented by the geotechnical engineer or his representative.

Where/if encountered, undocumented fills and any subsurface structures (dry wells, basements, driveway and landscaping fill, old utility lines, septic leach fields, etc.) should be completely removed and the excavations backfilled with engineered fill. Understanding of the extent and types of undocumented fill is based on the observed conditions within our subsurface explorations. Experience has shown that soil conditions can change greatly over short distances. It is possible fill exists in areas and extents other than those identified in our subsurface explorations.

Site earthwork may be impacted by wet weather conditions. Stabilization of subgrade soils may require aeration and recompaction. If subgrade soils are found to be difficult to stabilize, over-excavation, placement of granular soils, or cement treatment of subgrade soils may be feasible options. GeoPacific should be onsite to observe preparation of subgrade soil conditions prior to placement of engineered fill.

6.2 Engineered Fill

Due to the site being located on a large ancient landslide, and the presence of a steep slope area to the north of the property, we recommend that the grading plan be adjusted to eliminate any raising of grades or additional soil surcharges on the building lots. Grade adjustments within the proposed drive and driveways appear to be feasible. Engineered fill recommendations below are specific to removal and replacement of the existing undocumented fill back to existing grades.

All grading for the proposed construction should be performed as engineered grading in accordance with the applicable building code at the time of construction with the exceptions and additions noted herein. Site grading should be conducted in accordance with the requirements outlined in the 2015 International Building Code (IBC), Chapter 18 and Appendix J. Areas proposed for fill placement should be prepared as described in Section 6.1, Site Preparation Recommendations. Surface soils should then be scarified and recompacted prior to placement of structural fill. Site preparation, soil stripping, and grading activities should be observed and documented by a geotechnical engineer or his representative. Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill.

Onsite native soils consisting of Clayey SILT/SILT (ML), appear to be suitable for use as engineered fill assuming any inorganic or organic debris is removed. Soils containing greater than 3 percent organic content should not be used as structural fill. Imported fill material must be approved by the geotechnical engineer prior to being imported to the site. Oversize material

greater than 6 inches in size should not be used within 3 feet of foundation footings, and material greater than 12 inches in diameter should not be used in engineered fill.

Engineered fill should be compacted in horizontal lifts not exceeding 12 inches using standard compaction equipment. We recommend that engineered fill be compacted to at least 95 percent of the maximum dry density determined by ASTM D698 (Standard Proctor) or equivalent. Field density testing should conform to ASTM D2922 and D3017, or D1556. All engineered fill should be observed and tested by the project geotechnical engineer or his representative. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 500 yd^3, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that the earthwork contractor be held contractually responsible for test scheduling and frequency.

Site earthwork may be impacted by shallow groundwater, soil moisture and wet weather conditions. Earthwork in wet weather would likely require extensive use of additional crushed aggregate, cement or lime treatment, or other special measures, at considerable additional cost compared to earthwork performed under dry-weather conditions.

6.3 Excavating Conditions and Utility Trench Backfill

We anticipate that onsite soils can generally be excavated using conventional heavy equipment. Bedrock was not encountered within our subsurface explorations which extended to a maximum depth of 11 feet bgs, however we encountered cobble-sized rock. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. Actual slope inclinations at the time of construction should be determined based on safety requirements and actual soil and groundwater conditions. All temporary cuts in excess of 4 feet in height should be sloped in accordance with U.S. Occupational Safety and Health Administration (OSHA) regulations (29 CFR Part 1926) or be shored. The existing native soils classify as Type B Soil and temporary excavation side slope inclinations as steep as 1H:1V may be assumed for planning purposes. These cut slope inclinations are applicable to excavations above the water table only.

Shallow, perched groundwater may be encountered at the site and should be anticipated in excavations and utility trenches. Vibrations created by traffic and construction equipment may cause some caving and raveling of excavation walls. In such an event, lateral support for the excavation walls should be provided by the contractor to prevent loss of ground support and possible distress to existing or previously constructed structural improvements.

Underground utility pipes should be installed in accordance with the procedures specified in ASTM D2321 and City of West Linn standards. We recommend that structural trench backfill be compacted to at least 95 percent of the maximum dry density obtained by the Modified Proctor (ASTM D1557) or equivalent. Initial backfill lift thicknesses for a ¾"-0 crushed aggregate base may need to be as great as 4 feet to reduce the risk of flattening underlying flexible pipe. Subsequent lift thickness should not exceed 1 foot. If imported granular fill material is used, then the lifts for large vibrating plate-compaction equipment (e.g. hoe compactor attachments) may be up to 2 feet, provided that proper compaction is being achieved and each lift is tested. Use of large vibrating compaction equipment should be carefully monitored near existing structures and improvements due to the potential for vibration-induced damage.

Adequate density testing should be performed during construction to verify that the recommended relative compaction is achieved. Typically, at least one density test is taken for every 4 vertical feet of backfill on each 100-lineal-foot section of trench.

6.4 Erosion Control Considerations

During our field exploration program, we did not observe soil conditions that may be considered highly susceptible to erosion. In our opinion, the primary concern regarding erosion potential will occur during construction in areas that have been stripped of vegetation. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of straw waddles, fiber rolls, and silt fences. If used, these erosion control devices should remain in place throughout site preparation and construction.

Erosion and sedimentation of exposed soils can also be minimized by quickly re-vegetating exposed areas of soil, and by staging construction such that large areas of the project site are not denuded and exposed at the same time. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with either mulch or erosion control netting/blankets. Areas of exposed soil requiring permanent stabilization should be seeded with an approved grass seed mixture, or hydroseeded with an approved seed-mulch-fertilizer mixture.

6.5 Wet Weather Earthwork

Soils underlying the site are likely to be moisture sensitive and will be difficult to handle or traverse with construction equipment during periods of wet weather. Earthwork is typically most economical when performed under dry weather conditions. Earthwork performed during the wet-weather season will require expensive measures such as cement treatment or imported granular material to compact areas where fill may be proposed to the recommended engineering specifications. If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when soil moisture content is difficult to control, the following recommendations should be incorporated into the contract specifications.

- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation or the removal of unsuitable soils should be followed promptly by the placement and compaction of clean engineered fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance. Under some circumstances, it may be necessary to excavate soils with a backhoe to minimize subgrade disturbance caused by equipment traffic;
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water;
- Material used as engineered fill should consist of clean, granular soil containing less than 5 percent passing the No. 200 sieve. The fines should be non-plastic. Alternatively, cement treatment of on-site soils may be performed to facilitate wet weather placement;
- The ground surface within the construction area should be sealed by a smooth drum vibratory roller, or equivalent, and under no circumstances should be left uncompacted and exposed to moisture. Soils which become too wet for compaction should be removed and replaced with clean granular materials;

- Excavation and placement of fill should be observed by the geotechnical engineer to verify that all unsuitable materials are removed and suitable compaction and site drainage is achieved; and
- Geotextile silt fences, straw waddles, and fiber rolls should be strategically located to control erosion.

If cement or lime treatment is used to facilitate wet weather construction, GeoPacific should be contacted to provide additional recommendations and field monitoring.

6.6 Spread Foundations

Based upon communication with the client and review of a preliminary site and grading plan prepared by Theta LLC, GeoPacific understands that the proposed development at the site will consist of a three-lot property partition to support construction of new residential homes. The site plan indicates the approximate building footprints of the proposed homes. We anticipate that the homes will be two-stories, constructed with typical spread foundations and wood framing, with maximum structural loading on column footings and continuous strip footings on the order of 10 to 35 kips, and 2 to 6 kips respectively.

6.6.1 Recommendations Regarding Undocumented Fill Soil

- At the location of Parcel 1, undocumented fill soils were observed to be present on the order of 1 to 3 feet thick, consisting of stiff, reddish brown, moist, Clayey SILT (ML), containing trace concrete debris. The existing undocumented fill soils are not considered to be suitable to provide adequate bearing support for construction of foundations. Differential settlement is a concern due to variable soil conditions. The fill soils are underlain by stiff native silts. We recommend either: 1) excavating and recompacting the undocumented fill soils to achieve 95 percent compaction relative to ASTM D698; or 2) leave the fill material in place and over-excavate and extend foundation elements through the fill soils to bear directly on firm native soil.

- At the location of Parcel 2, undocumented fill soils were observed to be present on the order of 9 feet thick, consisting of medium stiff, dark brown, moist, Clayey SILT (ML), containing subrounded cobble sized rock, wood debris, and a buried drain field. The fill soils are underlain by stiff native silts. We recommend either: 1) excavating and recompacting the undocumented fill soils to achieve 95 percent compaction relative to ASTM D698; or 2) leave the fill material in place and install rammed aggregate piers to support the foundation.

- At the location of Parcel 3, undocumented fill soils were observed to be present on the order of 2 to 2.5 feet thick, consisting of loose, dark brown, moist, SILT (ML), containing tree roots. The fill soils are underlain by stiff native silts. We recommend either: 1) excavating and recompacting the undocumented fill soils to achieve 95 percent compaction relative to ASTM D698; or 2) leave the fill material in place and over-excavate and extend foundation elements through the fill soils to bear directly on firm native soil.

6.6.2 Recommended Footing-to-Slope Setbacks

As described above, a moderate to steep slope is present beyond the northwestern property line that extends to Firwood Court below. Parcel 3 will be located above the slope as shown on Figure 3. The overall slope height is approximately 30 feet. The hillside is heavily vegetated and showed no clear signs of recent erosion or instability at the time of our study. Based on review of the site plan and the indicated building envelope for Parcel 3 it appears that 20 to 30 feet of setback is proposed from the top of the slope to the home foundation. The noted setback distance appears to be adequate, however we recommend that a minimum footing-to-slope setback distance of at least 20 feet be maintained for foundations, engineered fill, and any structures or slabs. Reductions in setback distance should not be conducted without supporting soil boring explorations, and detailed quantitative slope stability assessment and calculations. Based on our review of the proposed locations of foundation envelopes of Parcels 1 and 2 it appears that each will be located at least 80 to 100 feet from steeply sloping areas.

Foundation design, construction, and setback requirements should conform to the applicable building code at the time of construction. For maximization of bearing strength and protection against frost heave, spread footings should be embedded at a minimum depth of 12 inches below exterior grade. Foundations should be designed by a licensed structural engineer.

The anticipated allowable soil bearing pressure is 1,500 lbs/ft^2 for footings bearing on competent, native soil and/or engineered fill, adequately prepared as described above. If over-excavation is needed, it should be conducted under the direction and supervision of the geotechnical engineer or designated representative. The recommended maximum allowable bearing pressure may be increased by 1/3 for short-term transient conditions such as wind and seismic loading. For heavier loads, the geotechnical engineer should be consulted. The coefficient of friction between on-site soil and poured-in-place concrete may be taken as 0.42, which includes no factor of safety. The maximum anticipated total and differential footing movements (generally from soil expansion and/or settlement) are 1 inch and ¾ inch over a span of 20 feet, respectively. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied. Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings.

Footing excavations should penetrate through topsoil and any disturbed soil to competent subgrade that is suitable for bearing support. All footing excavations should be trimmed neat, and all loose or softened soil should be removed from the excavation bottom prior to placing reinforcing steel bars. Due to the moisture sensitivity of on-site native soils, foundations constructed during the wet weather season may require over-excavation of footings and backfill with compacted, crushed aggregate.

Our recommendations are for residential construction incorporating raised wood floors and conventional spread footing foundations. After site development, a Final Soil Engineer's Report should either confirm or modify the above recommendations.

6.7 Concrete Slabs-on-Grade

Preparation of areas beneath concrete slab-on-grade floors should be performed as described in Section 6.1, Site Preparation Recommendations and Section 6.6, Spread Foundations. Care should be taken during excavation for foundations and floor slabs, to avoid disturbing subgrade soils. If subgrade soils have been adversely impacted by wet weather or otherwise disturbed, the surficial soils should be scarified to a minimum depth of 8 inches, moisture conditioned to within about 3 percent of optimum moisture content and compacted to engineered fill specifications. Alternatively, disturbed soils may be removed and the removal zone backfilled with additional crushed rock.

For evaluation of the concrete slab-on-grade floors using the beam on elastic foundation method, a modulus of subgrade reaction of 150 kcf (87 pci) should be assumed for the medium dense, fine to coarse-grained soils anticipated to be present at foundation subgrade elevation following adequate site preparation as described above. This value assumes the concrete slab system is designed and constructed as recommended herein, with a minimum thickness of 8 inches of 1½"-0 crushed aggregate beneath the slab. The total thickness of crushed aggregate will be dependent on the subgrade conditions at the time of construction and should be verified visually by proof-rolling. Under-slab aggregate should be compacted to at least 95 percent of its maximum dry density as determined by ASTM D1557 (Modified Proctor) or equivalent.

In areas where moisture will be detrimental to floor coverings or equipment inside the proposed structure, appropriate vapor barrier and damp-proofing measures should be implemented. A commonly applied vapor barrier system consists of a 10-mil polyethylene vapor barrier placed directly over the capillary break material. Other damp/vapor barrier systems may also be feasible. Appropriate design professionals should be consulted regarding vapor barrier and damp proofing systems, ventilation, building material selection and mold prevention issues, which are outside GeoPacific's area of expertise.

6.8 Footing and Roof Drains

Construction should include typical measures for controlling subsurface water beneath the structures, including positive crawlspace drainage to an adequate low-point drain exiting the foundation, visqueen covering the exposed ground in the crawlspace, and crawlspace ventilation (foundation vents). The client should be informed and educated that some slow flowing water in the crawlspaces is considered normal and not necessarily detrimental to the structures given these other design elements incorporated into construction. Appropriate design professionals should be consulted regarding crawlspace ventilation, building material selection and mold prevention issues, which are outside GeoPacific's area of expertise.

Down spouts and roof drains should collect roof water in a system separate from the footing drains to reduce the potential for clogging. Roof drain water should be directed to an appropriate discharge point and storm system well away from structural foundations. Grades should be sloped downward and away from buildings to reduce the potential for ponded water near structures.

Perimeter footing drains are considered necessary for this building. Perimeter footing drains should consist of 3 or 4-inch diameter, perforated plastic pipe embedded in a minimum of 1 ft^3 per

lineal foot of clean, free-draining drain rock. The drain pipe and surrounding drain rock should be wrapped in non-woven geotextile (Mirafi 140N, or approved equivalent) to minimize the potential for clogging and/or ground loss due to piping. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. Figure 4 presents a typical perimeter footing drain detail. In our opinion, footing drains may outlet at the curb, or on the back sides of lots where sufficient fall is not available to allow drainage to meet the street. In no case shall collected stormwater be allowed to flow freely over slope faces.

6.9 Permanent Below-Grade Walls

Lateral earth pressures against below-grade retaining walls will depend upon the inclination of any adjacent slopes, type of backfill, degree of wall restraint, method of backfill placement, degree of backfill compaction, drainage provisions, and magnitude and location of any adjacent surcharge loads. At-rest soil pressure is exerted on a retaining wall when it is restrained against rotation. In contrast, active soil pressure will be exerted on a wall if its top is allowed to rotate or yield a distance of roughly 0.001 times its height or greater.

If the subject retaining walls will be free to rotate at the top, they should be designed for an active earth pressure equivalent to that generated by a fluid weighing 35 pcf for level backfill against the wall. For restrained wall, an at-rest equivalent fluid pressure of 55 pcf should be used in design, again assuming level backfill against the wall. These values assume that the recommended drainage provisions are incorporated, and hydrostatic pressures are not allowed to develop against the wall.

During a seismic event, lateral earth pressures acting on below-grade structural walls will increase by an incremental amount that corresponds to the earthquake loading. Based on the Mononobe-Okabe equation and peak horizontal accelerations appropriate for the site location, seismic loading should be modeled using the active or at-rest earth pressures recommended above, plus an incremental rectangular-shaped seismic load of magnitude 6.5H, where H is the total height of the wall.

We assume relatively level ground surface below the base of the walls. As such, we recommend passive earth pressure of 320 pcf for use in design, assuming wall footings are cast against competent native soils or engineered fill. If the ground surface slopes down and away from the base of any of the walls, a lower passive earth pressure should be used and GeoPacific should be contacted for additional recommendations.

A coefficient of friction of 0.42 may be assumed along the interface between the base of the wall footing and subgrade soils. The recommended coefficient of friction and passive earth pressure values do not include a safety factor, and an appropriate safety factor should be included in design. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

The above recommendations for lateral earth pressures assume that the backfill behind the subsurface walls will consist of properly compacted structural fill, and no adjacent surcharge loading. If the walls will be subjected to the influence of surcharge loading within a horizontal distance equal to or less than the height of the wall, the walls should be designed for the additional

horizontal pressure. For uniform surcharge pressures, a uniformly distributed lateral pressure of 0.3 times the surcharge pressure should be added. Traffic surcharges may be estimated using an additional vertical load of 250 psf (2 feet of additional fill), in accordance with local practice.

The recommended equivalent fluid densities assume a free-draining condition behind the walls so that hydrostatic pressures do not build-up. This can be accomplished by placing a 12 to 18-inch wide zone of sand and gravel containing less than 5 percent passing the No. 200 sieve against the walls. A 3-inch minimum diameter perforated, plastic drain pipe should be installed at the base of the walls and connected to a suitable discharge point to remove water in this zone of sand and gravel. The drain pipe should be wrapped in filter fabric (Mirafi 140N or other as approved by the geotechnical engineer) to minimize clogging.

Wall drains are recommended to prevent detrimental effects of surface water runoff on foundations – not to dewater groundwater. Drains should not be expected to eliminate all potential sources of water entering a basement or beneath a slab-on-grade. An adequate grade to a low point outlet drain in the crawlspace is required by code. Underslab drains are sometimes added beneath the slab when placed over soils of low permeability and shallow, perched groundwater.

Water collected from the wall drains should be directed into the local storm drain system or other suitable outlet. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. Down spouts and roof drains should not be connected to the wall drains in order to reduce the potential for clogging. The drains should include clean-outs to allow periodic maintenance and inspection. Grades around the proposed structure should be sloped such that surface water drains away from the building.

GeoPacific should be contacted during construction to verify subgrade strength in wall keyway excavations, to verify that backslope soils are in accordance with our assumptions, and to take density tests on the wall backfill materials.

Structures should be located a horizontal distance of at least 1.5H away from the back of the retaining wall, where H is the total height of the wall. GeoPacific should be contacted for additional foundation recommendations where structures are located closer than 1.5H to the top of any wall.

6.10 Proposed Stormwater Ponds

Stormwater detention ponds are proposed near the top of steep slope areas for Parcels 2 and 3. We recommend that the stormwater ponds are constructed to be impermeable. A typical liner should be placed in the pond. In no case shall stormwater be directed or allowed to flow freely over the slope faces.

7.0 SEISMIC DESIGN

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2019 Statewide GeoHazards Viewer indicates that the site is in an area where *very strong* ground shaking is anticipated during an earthquake. Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2015 International Building Code (IBC) with applicable Oregon Structural Specialty Code (OSSC) revisions (current 2014). We recommend Site Class D be used for design per the OSSC, Table 1613.5.2 and as defined in ASCE 7-10, Chapter 20, Table 20.3-1. Design values determined for the site using the ATC Hazards by Location 2019 Seismic Design Maps Summary Report are summarized in Table 1 and are based upon observed existing soil conditions.

Table 1: Recommended Earthquake Ground Motion Parameters (USGS 2019)

Parameter	Value
Location (Lat, Long), degrees	45.368, -122.626
Probabilistic Ground Motion Values, 2% Probability of Exceedance in 50 yrs	
Peak Ground Acceleration PGA_M	0.448 g
Short Period, S_s	0.948 g
1.0 Sec Period, S_1	0.408 g
Soil Factors for Site Class D:	
F_a	1.121
F_v	1.592
$SD_s = 2/3 \times F_a \times S_s$	0.708 g
$SD_1 = 2/3 \times F_v \times S_1$	0.433 g
Seismic Design Category	D

7.1 Soil Liquefaction

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2019 Statewide GeoHazards Viewer indicates that the site is in an area considered to be at *low* risk for soil liquefaction during an earthquake. Soil liquefaction is a phenomenon wherein saturated soil deposits temporarily lose strength and behave as a liquid in response to ground shaking caused by strong earthquakes. Soil liquefaction is generally limited to loose, sands and granular soils located below the water table, and fine-grained soils with a plasticity index less than 15. According to our review of geologic mapping the site is underlain by fine-grained soil deposits underlain by basaltic bedrock. Review of available well logs indicates static groundwater is commonly encountered at depths of 20 to 40 feet bgs in the vicinity of the subject site. Based upon the results of our study, it is our opinion that the risk of soil liquefaction at the site during a seismic event at the subject site should be considered to be low.

8.0 UNCERTAINTIES AND LIMITATIONS

We have prepared this report for the owner and their consultants for use in design of this project only. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, GeoPacific should be notified for review of the recommendations of this report, and revision of such if necessary.

Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. The checklist attached to this report outlines recommended geotechnical observations and testing for the project. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications.

Within the limitations of scope, schedule and budget, GeoPacific attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology at the time the report was prepared. No warranty, expressed or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

We appreciate this opportunity to be of service.

Sincerely,

GEOPACIFIC ENGINEERING, INC.

REGISTERED OREGON BENJAMIN L COOK 2245 GEOLOGIST EXP 12/31/19

REGISTERED PROFESSIONAL ENGINEER 14743PE OREGON JAN. 23, 1990 JAMES D. IMBRIE

EXPIRES: 06/30/2019

Benjamin L. Cook, R.G.
Senior Geologist

James D. Imbrie, P.E.
Principal Geotechnical Engineer

REFERENCES

ATC Hazards by Location, (https://hazards.atcouncil.org).

Atwater, B.F., 1992, Geologic evidence for earthquakes during the past 2,000 years along the Copalis River, southern coastal Washington: Journal of Geophysical Research, v. 97, p. 1901-1919.

Carver, G.A., 1992, Late Cenozoic tectonics of coastal northern California: American Association of Petroleum Geologists-SEPM Field Trip Guidebook, 1992.

Columbia County Web Maps (http://webmap.co.columbia.or.us)

Gannet, Marshall W., and Caldwell, Rodney R., Generalized Geologic Map of the Willamette Lowland, U.S. Department of the interior, U.S. Geological Survey, 1998.

Goldfinger, C., Kulm, L.D., Yeats, R.S., Appelgate, B, MacKay, M.E., and Cochrane, G.R., 1996, Active strike-slip faulting and folding of the Cascadia Subduction-Zone plate boundary and forearc in central and northern Oregon: in Assessing earthquake hazards and reducing risk in the Pacific Northwest, v. 1: U.S. Geological Survey Professional Paper 1560, P. 223-256.

Ma, L., Madin, I.P., Duplantis, S., and Williams, K.J., 2012, Lidar-based Surficial Geologic Map and Database of the Greater Portland, Oregon, Area, Clackamas, Columbia, Marion, Multnomah, Washington, and Yamhill Counties, Oregon, and Clark County, Washington, DOGAMI Open-File Report O-12-02

Mabey, M.A., Madin, I.P., and Black G.L., 1996, Relative Earthquake Hazard Map of the Lake Oswego Quadrangle, Clackamas, Multnomah and Washington Counties, Oregon: Oregon Department of Geology and Mineral Industries

Madin, I.P., 1990, Earthquake hazard geology maps of the Portland metropolitan area, Oregon: Oregon Department of Geology and Mineral Industries Open-File Report 0-90-2, scale 1:24,000, 22 p.

Oregon Department of Geology and Mineral Industries, Statewide Geohazards Viewer, www.oregongeology.org/hazvu.

Oregon Department of Geology and Mineral Industries, Madin, Ian P., Ma, Lina, and Niewendorp, Clark A., *Open-File Report 0-08-06, Preliminary Geologic Map of the Linnton 7.5' Quadrangle, Multnomah and Washington Counties*, Oregon, 2008.

Peterson, C.D., Darioenzo, M.E., Burns, S.F., and Burris, W.K., 1993, Field trip guide to Cascadia paleoseismic evidence along the northern California coast: evidence of subduction zone seismicity in the central Cascadia margin: Oregon Geology, v. 55, p. 99-144.

United States Geological Survey, USGS Earthquake Hazards Program Website (earthquake.usgs.gov).

Unruh, J.R., Wong, I.G., Bott, J.D., Silva, W.J., and Lettis, W.R., 1994, Seismotectonic evaluation: Scoggins Dam, Tualatin Project, Northwest Oregon: unpublished report by William Lettis and Associates and Woodward Clyde Federal Services, Oakland, CA, for U. S. Bureau of Reclamation, Denver CO (in Geomatrix Consultants, 1995).

Web Soil Survey, Natural Resources Conservation Service, United States Department of Agriculture *2015 website.*

(http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm.).

Werner, K.S., Nabelek, J., Yeats, R.S., Malone, S., 1992, The Mount Angel fault: implications of seismic-reflection data and the Woodburn, Oregon, earthquake sequence of August, 1990: Oregon Geology, v. 54, p. 112-117.

Wong, I. Silva, W., Bott, J., Wright, D., Thomas, P., Gregor, N., Li., S., Mabey, M., Sojourner, A., and Wang, Y., 2000, Earthquake Scenario and Probabilistic Ground Shaking Maps for the Portland, Oregon, Metropolitan Area; State of Oregon Department of Geology and Mineral Industries; Interpretative Map Series IMS-16

Yeats, R.S., Graven, E.P., Werner, K.S., Goldfinger, C., and Popowski, T., 1996, Tectonics of the Willamette Valley, Oregon: in Assessing earthquake hazards and reducing risk in the Pacific Northwest, v. 1: U.S. Geological Survey Professional Paper 1560, P. 183-222, 5 plates, scale 1:100,000.

Yelin, T.S., 1992, An earthquake swarm in the north Portland Hills (Oregon): More speculations on the seismotectonics of the Portland Basin: Geological Society of America, Programs with Abstracts, v. 24, no. 5, p. 92.

CHECKLIST OF RECOMMENDED GEOTECHNICAL TESTING AND OBSERVATION

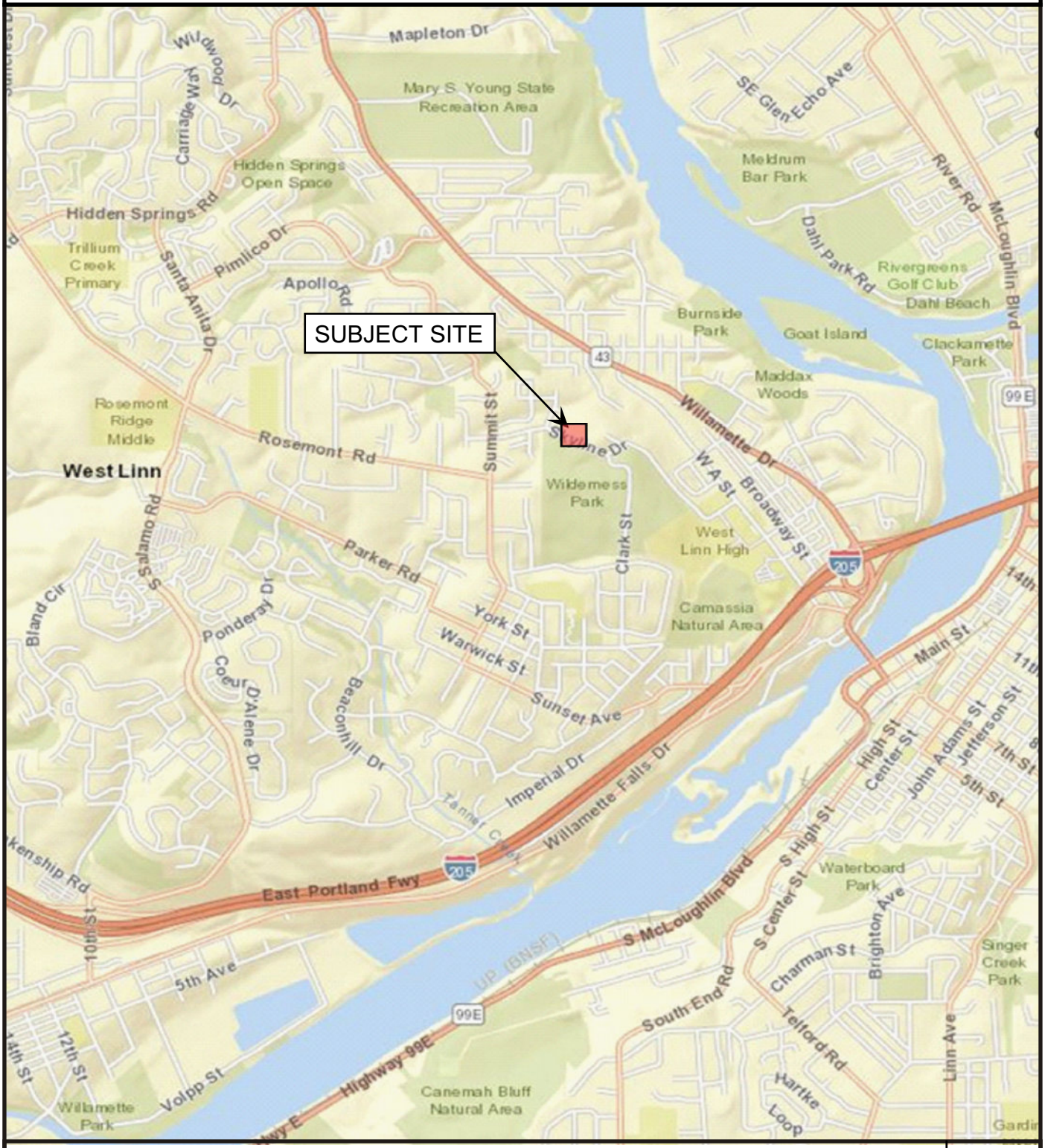
Item No.	Procedure	Timing	By Whom	Done
1	Preconstruction meeting	Prior to beginning site work	Contractor, Developer, Civil and Geotechnical Engineers	
2	Fill removal from site or sorting and stockpiling	Prior to mass stripping	Soil Technician/ Geotechnical Engineer	
3	Stripping, aeration, and root-picking operations	During stripping	Soil Technician	
4	Compaction testing of engineered fill (95% of Standard Proctor)	During filling, tested every 2 vertical feet	Soil Technician	
5	Foundation Subgrade Compaction (95% of Modified Proctor)	During Foundation Preparation, Prior to Placement of Reinforcing Steel	Soil Technician/ Geotechnical Engineer	
6	Compaction testing of trench backfill (95% of Modified Proctor)	During backfilling, tested every 4 vertical feet for every 200 linear feet	Soil Technician	
7	Street Subgrade Inspection (95% of Standard Proctor)	Prior to placing base course	Soil Technician	
8	Base course compaction (95% of Modified Proctor)	Prior to paving, tested every 200 linear feet	Soil Technician	
9	Asphalt Compaction (92% Rice Value)	During paving, tested every 100 linear feet	Soil Technician	
10	Final Geotechnical Engineer's Report	Completion of project	Geotechnical Engineer	

FIGURES

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel (503) 598-8445
Fax (503) 941-9281

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

SITE VICINITY MAP



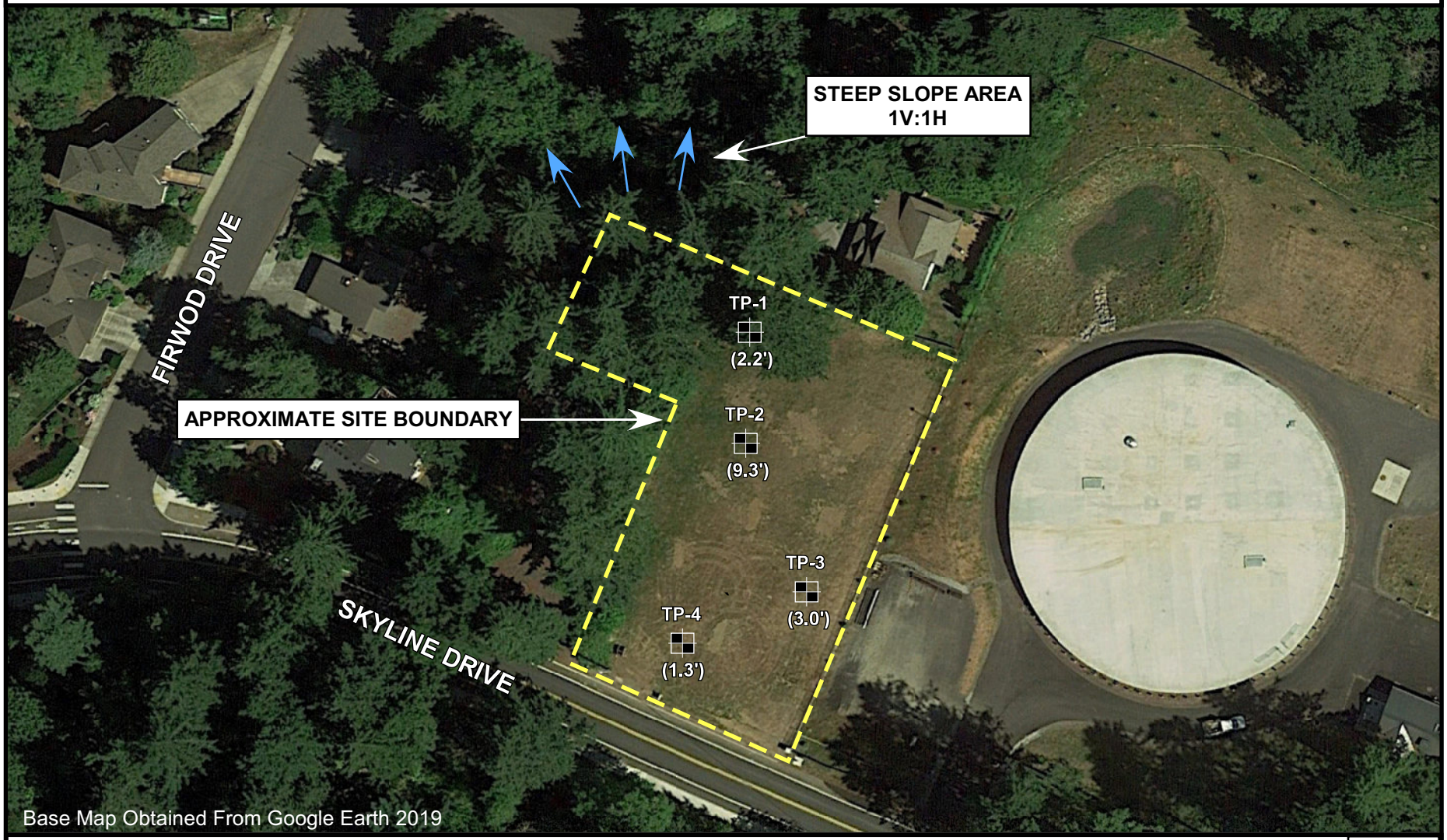
Base Map: DOGAMI HAZVU STREET VIEW, 2019
Date: 4/25/2019
Drawn by: BLC

North

Project: 6123 Skyline Drive Partition West Linn, Oregon 97068	Project No. 19-5167	FIGURE 1

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

SITE AERIAL AND EXPLORATION LOCATIONS



Legend:

TP-1
Test Pit Designation and Approximate Location
(2.0') Depth of Undocumented Fill, Feet

APPROXIMATE SCALE (FEET)
40 0 40 80

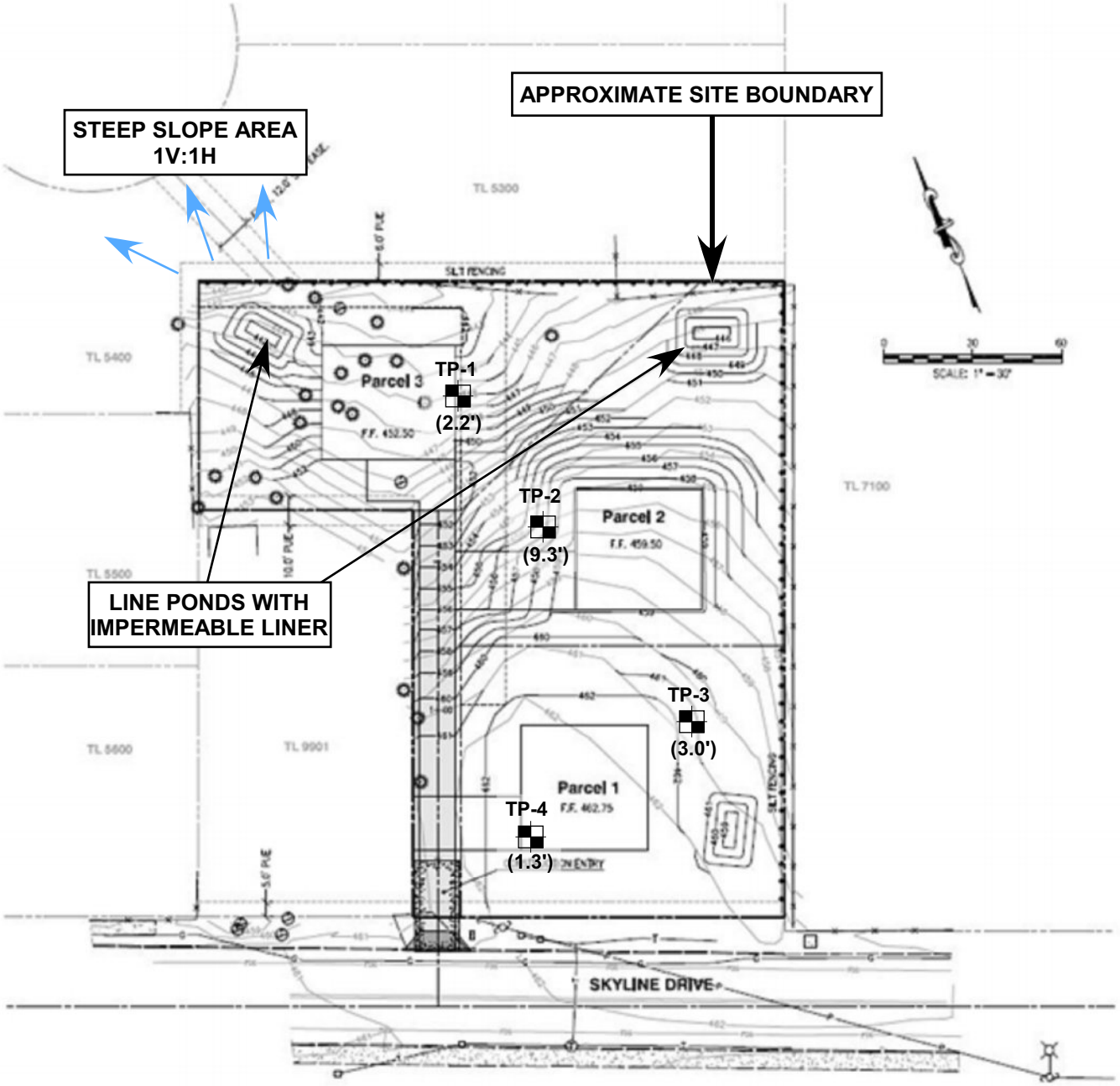
Drawn by: TJT
Date: 04/22/2019

North

Project: 6123 Skyline Drive Partition West Linn, Oregon 97068	Project No. 19-5206	FIGURE 2

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

SITE PLAN AND EXPLORATION LOCATIONS



Legend: **TP-1** Test Pit Designation and Approximate Location

(2.0') Depth of Undocumented Fill, Feet

Base Map: Preliminary Grading Plan, 6123 Skyline Drive Partition, Theta, LLC Sheet 4/4

APPROXIMATE SCALE (AS NOTED)

Date: 4/22/19

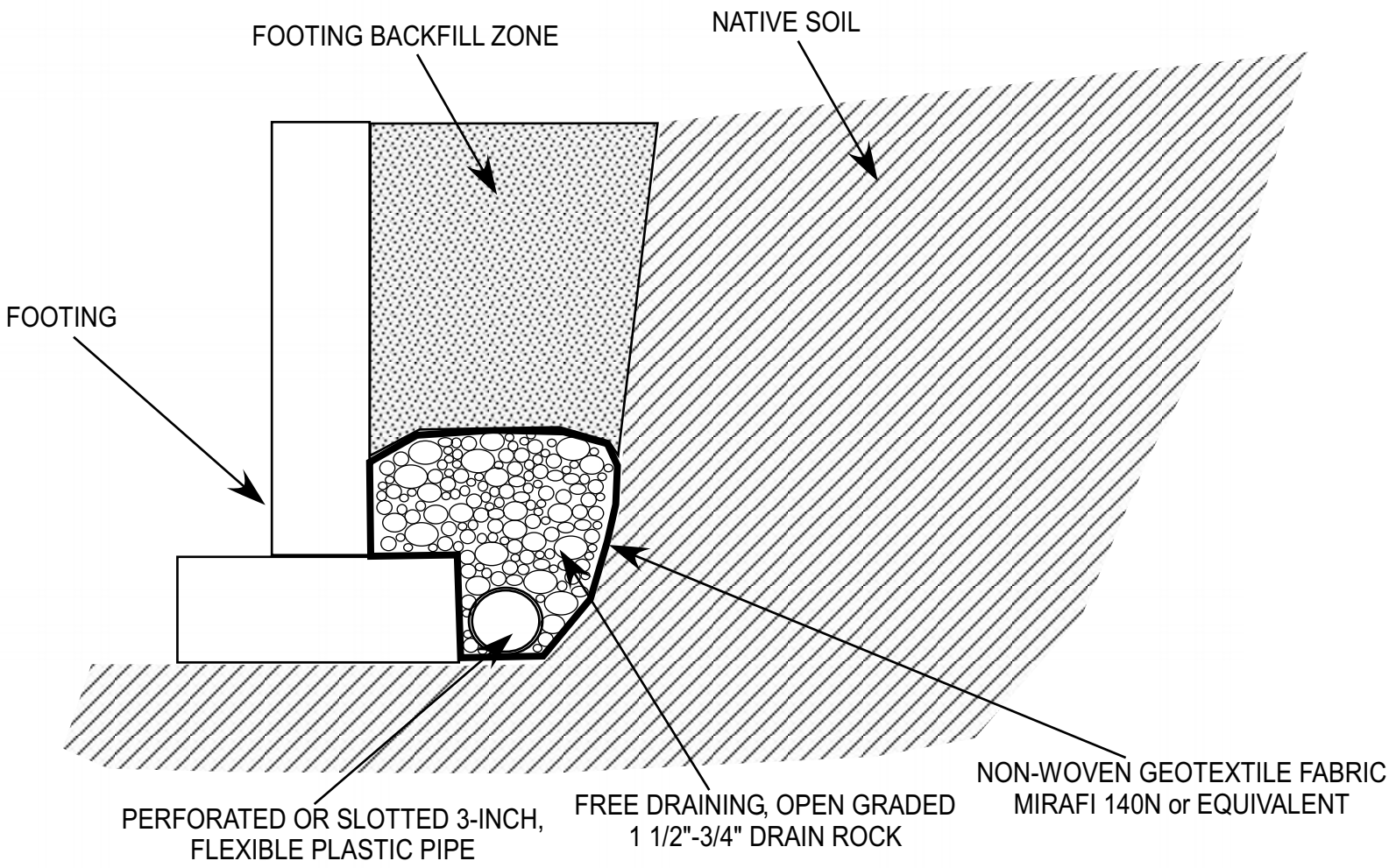
Drawn by: TJT

North

Project: 6123 Skyline Drive Partition West Linn, Oregon 97068	Project No. 19-5206	FIGURE 3

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TYPICAL PERIMETER FOOTING DRAIN DETAIL



Notes:
1) Drain rock should contain no more than 5 percent fines passing the U.S. No. 200 Sieve.
2) Trench bottom and drain pipe should be sloped to drain to approved discharge location.

Date: 4/26/2019
Drawn by: BLC

Project: 6123 Skyline Drive West Linn, Oregon 97068	Project No. 19-5142	FIGURE 4

EXPLORATION LOGS

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel (503) 598-8445
Fax (503) 941-9281

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: 6123 Skyline Drive Partition
West Linn, Oregon 97068

Project No. 19-5206

Test Pit No. **TP- 1**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
1	1.0					FILL. SILT (ML), loose, dark brown, with fine and medium roots throughout, 6 inch thick layer of moderately organic topsoil developed on the surface, with trace angular gravel and coarse sand, moist.
2	0.75					
3	4.0					Apparent Native Soil. SILT (ML), very stiff, brown to light brown, with trace black staining, low plasticity, micaceous, moist. (Loess).
4	3.75					
5						Test Pit Terminated at 4.5 feet bgs. No seepage or groundwater encountered.
6						
7						
8						
9						
10						
11						
12						

LEGEND

Bag Sample

Bucket Sample


Shelby Tube Sample


Seepage


Water Bearing Zone


Water Level at Abandonment

Date Excavated: 04/19/2019
Logged By: T. Torkelson
Surface Elevation: 444 feet amsl

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: 6123 Skyline Drive Partition
West Linn, Oregon 97068

Project No. 19-5206

Test Pit No. **TP- 2**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
1	2.5					FILL. Clayey SILT (ML), stiff, reddish brown with dark brown mottling, 4 inch thick layer of moderately organic topsoil developed on the ground surface, disturbed texture, moist.
2	4.0					Grades to very stiff at 2 feet bgs.
3	3.0					
4	2.0					Grades to medium stiff and dark brown below 3.5 feet bgs. Grades to with subrounded, cobble sized rock and trace wood debris below 4 feet bgs.
5		100 to 1,000 g				
6						
7						
8		100 to 1,000 g				
9						Drain field consisting of drain rock encountered at 9 feet bgs.
10						Apparent Native Soil. Clayey SILT (ML), very stiff, brown, low plasticity, micaceous, moist. (Loess).
		100 to 1,000 g				
11						Test Pit Terminated at 11 feet bgs. No seepage or groundwater encountered.
12						

LEGEND

100 to 1,000 g — Bag Sample
5 Gal. Bucket — Bucket Sample
Shelby Tube Sample
Seepage
Water Bearing Zone
Water Level at Abandonment

Date Excavated: 04/19/2019
Logged By: T. Torkelson
Surface Elevation: 456 feet amsl

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: 6123 Skyline Drive Partition
West Linn, Oregon 97068

Project No. 19-5206

Test Pit No. **TP- 3**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
1	4.5+					FILL. Clayey SILT (ML), very stiff, reddish brown with trace black staining, 4 inch thick layer of moderately organic topsoil developed on the ground surface, with trace angular gravel, moist.
2	4.5+					
3	4.5+					
4	3.5	100 to 1,000 g				Apparent Native Soil. SILT (ML), very stiff, brown, very low plasticity, micaceous, very moist. (Loess).
5						Test Pit Terminated at 4 feet bgs. No seepage or groundwater encountered.
6						
7						
8						
9						
10						
11						
12						

LEGEND

Bag Sample

Bucket Sample


Shelby Tube Sample


Seepage


Water Bearing Zone


Water Level at Abandonment

Date Excavated: 04/19/2019
Logged By: T. Torkelson
Surface Elevation: 460 feet amsl

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: 6123 Skyline Drive Partition
West Linn, Oregon 97068

Project No. 19-5206

Test Pit No. **TP- 4**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
1	3.0					FILL. Clayey SILT (ML), loose, dark brown, 6 inch thick layer of moderately organic topsoil developed on the ground surface, contains trace gravel and concrete debris, very moist. Grades to stiff at 6 inches bgs.
2	3.5					Apparent Native Soil. SILT (ML), very stiff, brown, very low plasticity, with trace black staining, micaceous, very moist. (Loess).
3	4.0 4.5					
4						Test Pit Terminated at 3.5 feet bgs. No seepage or groundwater encountered.
5						
6						
7						
8						
9						
10						
11						
12						

LEGEND

Bag Sample | Bucket Sample | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 04/19/2019
Logged By: T. Torkelson
Surface Elevation: 462 feet amsl

SOIL DESCRIPTION AND CLASSIFICATION GUIDELINES

Particle-Size Classification

COMPONENT	ASTM/USCS		AASHTO	
	size range	sieve size range	size range	sieve size range
Cobbles	> 75 mm	greater than 3 inches	> 75 mm	greater than 3 inches
Gravel	75 mm – 4.75 mm	3 inches to No. 4 sieve	75 mm – 2.00 mm	3 inches to No. 10 sieve
Coarse	75 mm – 19.0 mm	3 inches to 3/4-inch sieve	-	-
Fine	19.0 mm – 4.75 mm	3/4-inch to No. 4 sieve	-	-
Sand	4.75 mm – 0.075 mm	No. 4 to No. 200 sieve	2.00 mm – 0.075 mm	No. 10 to No. 200 sieve
Coarse	4.75 mm – 2.00 mm	No. 4 to No. 10 sieve	2.00 mm – 0.425 mm	No. 10 to No. 40 sieve
Medium	2.00 mm – 0.425 mm	No. 10 to No. 40 sieve	-	-
Fine	0.425 mm – 0.075 mm	No. 40 to No. 200 sieve	0.425 mm – 0.075 mm	No. 40 to No. 200 sieve
Fines (Silt and Clay)	< 0.075 mm	Passing No. 200 sieve	< 0.075 mm	Passing No. 200 sieve

Consistency for Cohesive Soil

CONSISTENCY	SPT N-VALUE (BLOWS PER FOOT)	POCKET PENETROMETER (UNCONFINED COMPRESSIVE STRENGTH, tsf)
Very Soft	2	less than 0.25
Soft	2 to 4	0.25 to 0.50
Medium Stiff	4 to 8	0.50 to 1.0
Stiff	8 to 15	1.0 to 2.0
Very Stiff	15 to 30	2.0 to 4.0
Hard	30 to 60	greater than 4.0
Very Hard	greater than 60	-

Relative Density for Granular Soil

RELATIVE DENSITY	SPT N-VALUE (BLOWS PER FOOT)
Very Loose	0 to 4
Loose	4 to 10
Medium Dense	10 to 30
Dense	30 to 50
Very Dense	more than 50

Moisture Designations

TERM	FIELD IDENTIFICATION
Dry	No moisture. Dusty or dry.
Damp	Some moisture. Cohesive soils are usually below plastic limit and are moldable.
Moist	Grains appear darkened, but no visible water is present. Cohesive soils will clump. Sand will bulk. Soils are often at or near plastic limit.
Wet	Visible water on larger grains. Sand and silt exhibit dilatancy. Cohesive soil can be readily remolded. Soil leaves wetness on the hand when squeezed. Soil is much wetter than optimum moisture content and is above plastic limit.

AASHTO SOIL CLASSIFICATION SYSTEM

TABLE 1. Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35 Percent or Less Passing .075 mm)				Silt-Clay Materials (More than 35 Percent Passing 0.075)		
Group Classification	A-1	A-3	A-2	A-4	A-5	A-6	A-7
Sieve analysis, percent passing:							
2.00 mm (No. 10)	-	-	-				
0.425 mm (No. 40)	50 max	51 min	-	-	-	-	-
0.075 mm (No. 200)	25 max	10 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)							
Liquid limit				40 max	41 min	40 max	41 min
Plasticity index	6 max	N.P.		10 max	10 max	11 min	11 min
General rating as subgrade	Excellent to good				Fair to poor		

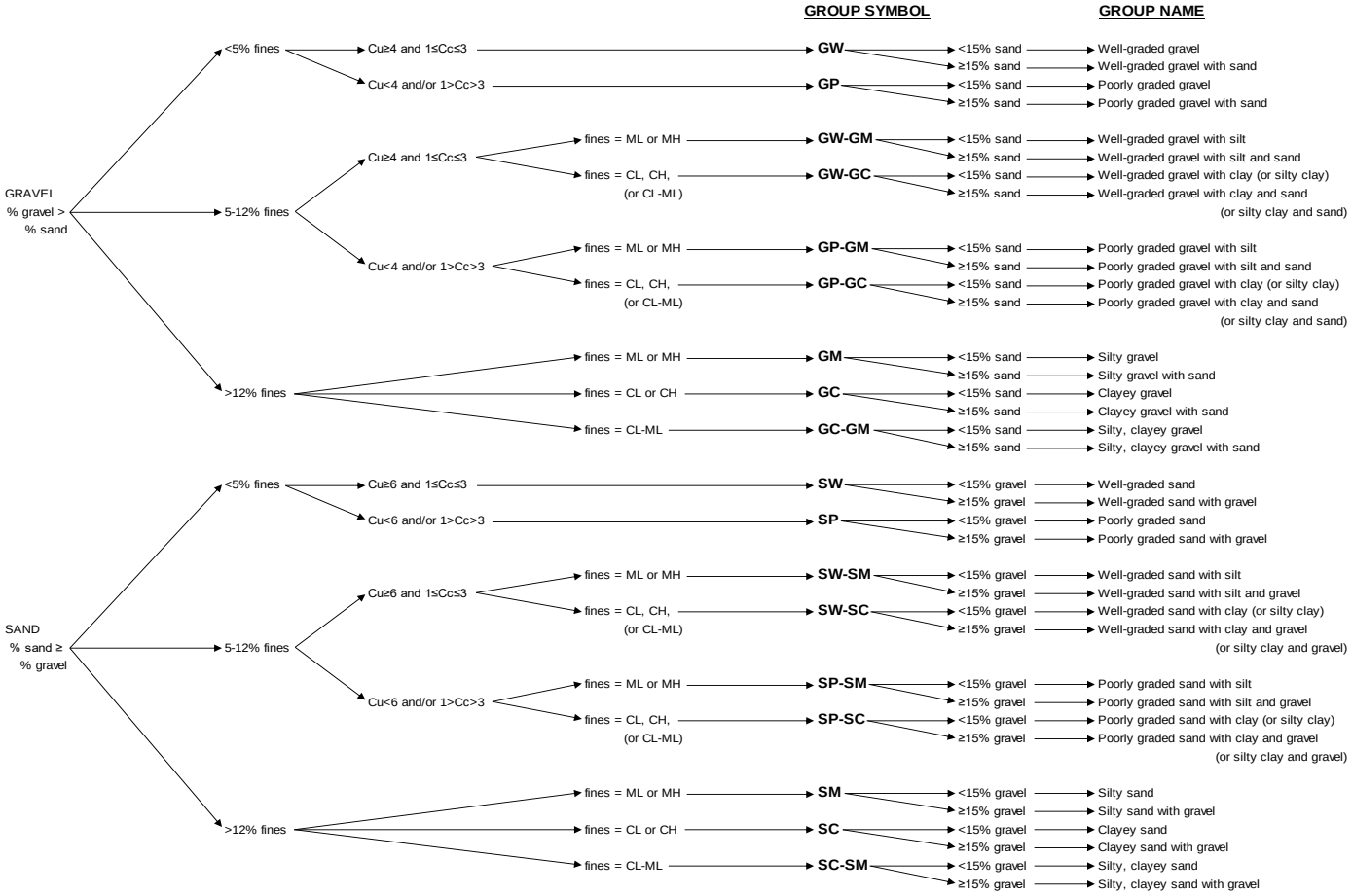
Note: The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.

TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

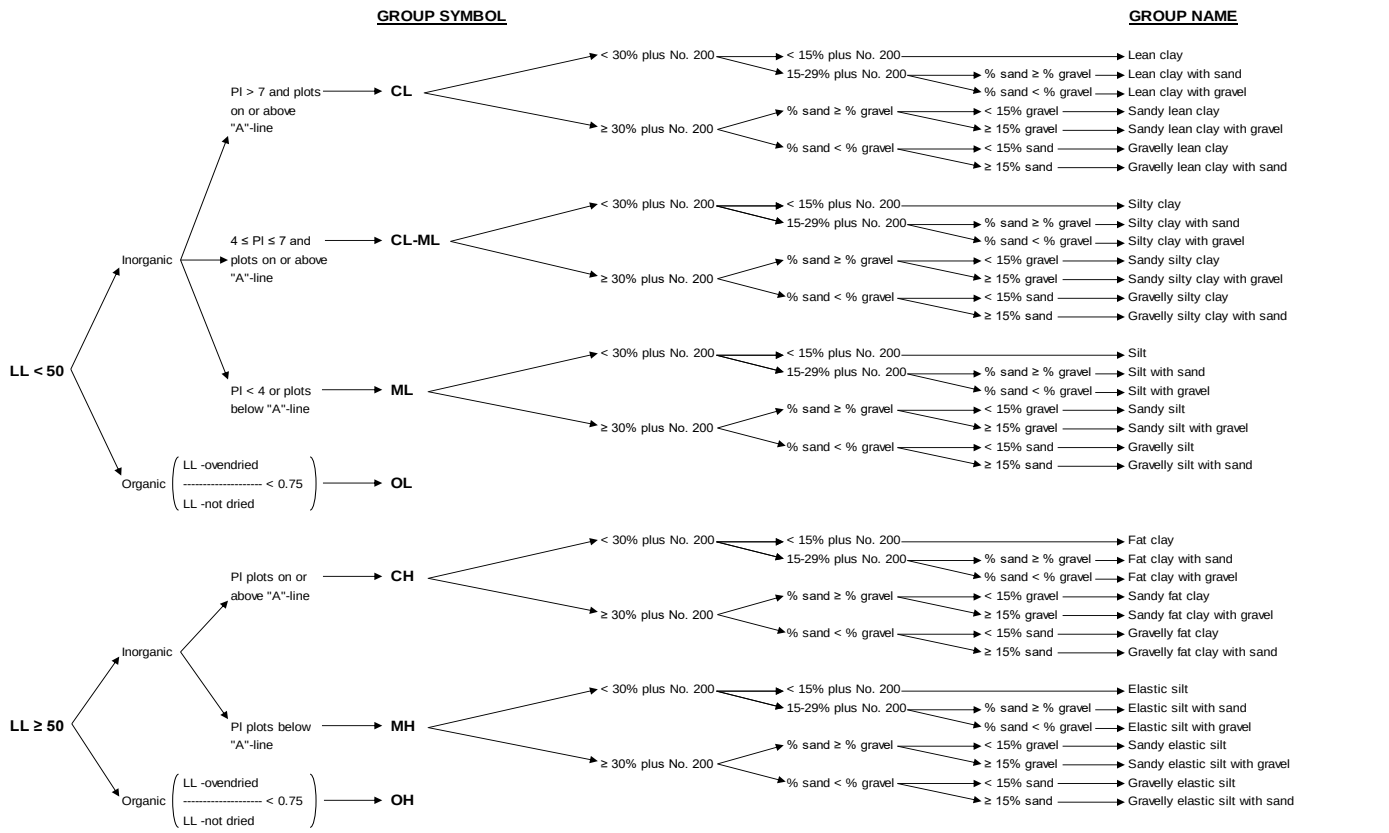
General Classification	Granular Materials (35 Percent or Less Passing 0.075 mm)							Silt-Clay Materials (More than 35 Percent Passing 0.075 mm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing:											
2.00 mm (No. 10)	50 max	-	-	-	-	-	-	-	-	-	-
0.425 mm (No. 40)	30 max	50 max	51 min	-	-	-	-	-	-	-	-
0.075 mm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11min
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General ratings as subgrade	Excellent to Good							Fair to poor			

Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

AASHTO = American Association of State Highway and Transportation Officials



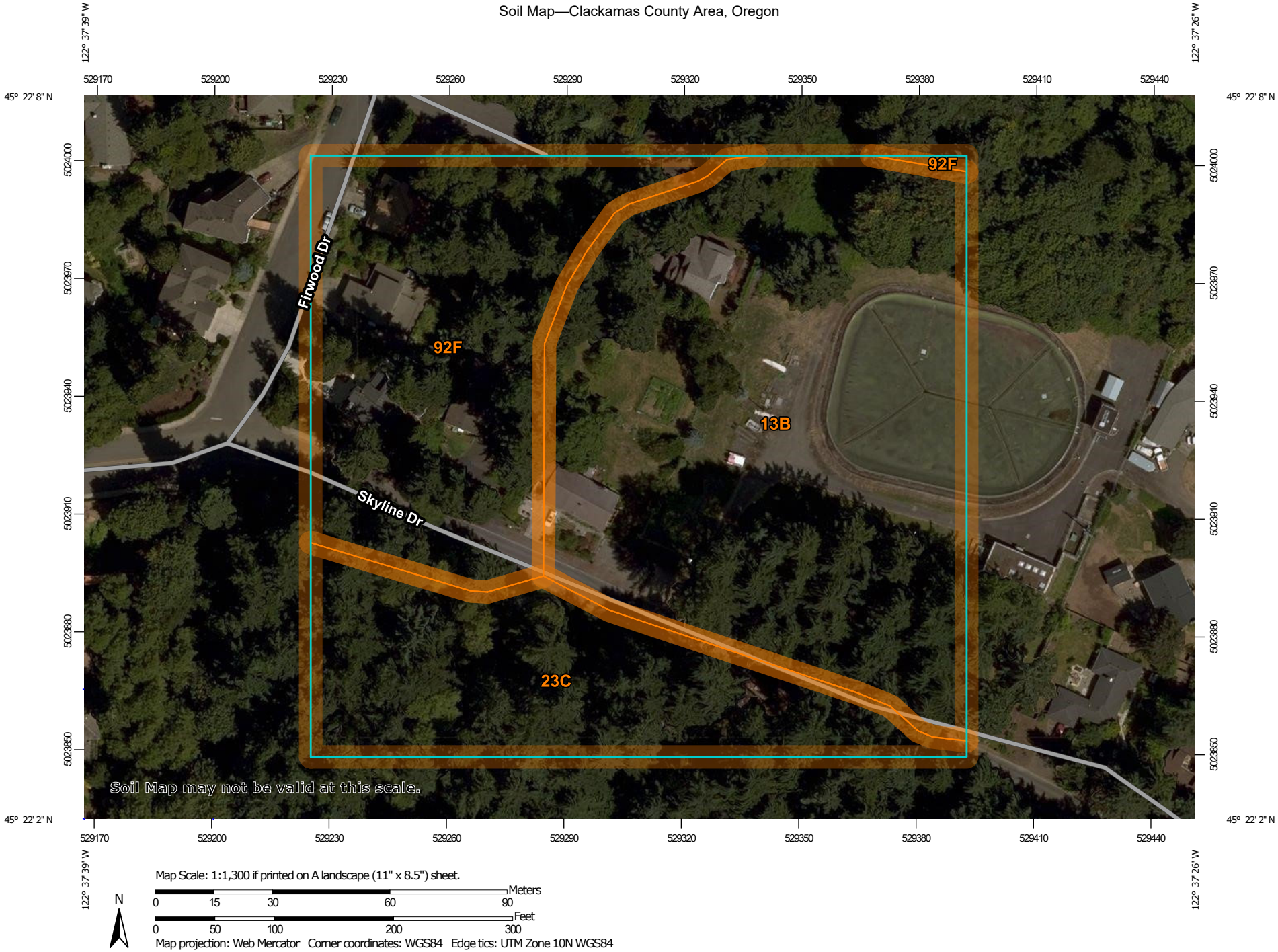
Flow Chart for Classifying Coarse-Grained Soils (More Than 50% Retained on No. 200 Sieve)



Flow Chart for Classifying Fine-Grained Soil (50% or More Passes No. 200 Sieve)

SITE RESEARCH

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel (503) 598-8445
Fax (503) 941-9281



Map Scale: 1:1,300 if printed on A landscape (11" x 8.5") sheet.

N

Meters
0 15 30 60 90

Feet
0 50 100 200 300

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Clackamas County Area, Oregon
Survey Area Data: Version 14, Sep 18, 2018

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Jul 26, 2014—Sep 5, 2014

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
13B	Cascade silt loam, 3 to 8 percent slopes	3.2	50.9%
23C	Cornelius silt loam, 8 to 15 percent slopes	1.4	21.2%
92F	Xerochrepts and Haploxerolls, very steep	1.8	27.9%
Totals for Area of Interest		**6.4**	**100.0%**

Hazards by Location

Search Information

Coordinates: 45.368199, -122.625767

Elevation: 460 ft

Timestamp: 2019-04-25T19:52:55.550Z

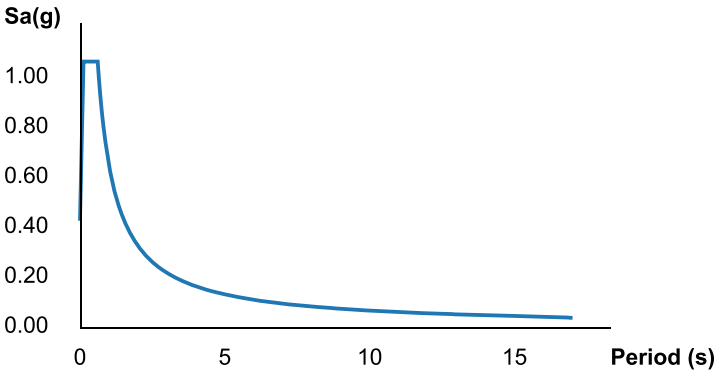
Hazard Type: Seismic

Reference Document: ASCE7-10

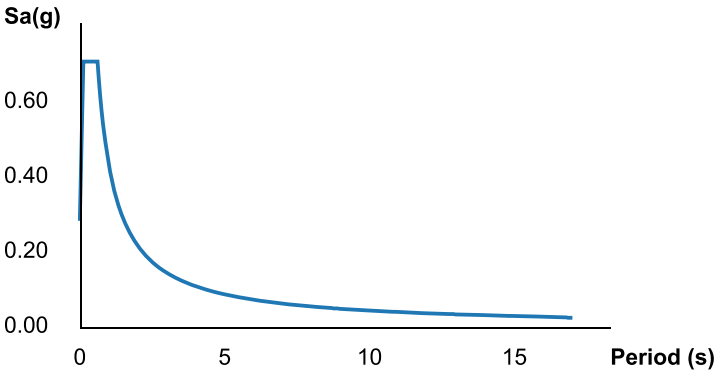
Risk Category: III

Site Class: D

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.948	MCE_R ground motion (period=0.2s)
S_1	0.408	MCE_R ground motion (period=1.0s)
S_{MS}	1.063	Site-modified spectral acceleration value
S_{M1}	0.649	Site-modified spectral acceleration value
S_{DS}	0.708	Numeric seismic design value at 0.2s SA
S_{D1}	0.433	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	D	Seismic design category
F_a	1.121	Site amplification factor at 0.2s
F_v	1.592	Site amplification factor at 1.0s
CR_S	0.906	Coefficient of risk (0.2s)

CR_1	0.874	Coefficient of risk (1.0s)
PGA	0.411	MCE_G peak ground acceleration
F_{PGA}	1.089	Site amplification factor at PGA
PGA_M	0.448	Site modified peak ground acceleration
T_L	16	Long-period transition period (s)
SsRT	0.948	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.047	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	3.037	Factored deterministic acceleration value (0.2s)
S1RT	0.408	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.467	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.034	Factored deterministic acceleration value (1.0s)
PGAd	1.159	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

While the information presented on this website is believed to be correct, ATC and its sponsors and contributors assume no responsibility or liability for its accuracy. The material presented in the report should not be used or relied upon for any specific application without competent examination and verification of its accuracy, suitability and applicability by engineers or other licensed professionals. ATC does not intend that the use of this information replace the sound judgment of such competent professionals, having experience and knowledge in the field of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the results of the report provided by this website. Users of the information from this website assume all liability arising from such use. Use of the output of this website does not imply approval by the governing building code bodies responsible for building code approval and interpretation for the building site described by latitude/longitude location in the report.

STATE OF OREGON

WATER WELL REPORT

(as required by ORS 537.765)

RECEIVED
APR 30 1986
WATER RESOURCES DEPT
SALEM, OREGON

CLAC 03378

25/1E-25dc

(1) OWNER:

Owner's Well Number: 03

Name: Daniel S. Lee

Address: 3726 S.E. Tenine

City: Portland State: Oregon Zip: 97202

(2) TYPE OF WORK:

☒ New Well ☐ Deepen ☐ Recondition ☐ Abandon

(3) DRILL METHOD:

☒ Rotary Air ☐ Rotary Mud ☐ Cable ☐ Other

(4) PROPOSED USE:

☒ Domestic ☐ Community ☐ Industrial ☐ Irrigation

☐ Thermal ☐ Injection ☐ Other

(5) BORE HOLE CONSTRUCTION:

Depth of Completed Well: 104 ft.

Special Standards date of approval:

HOLE Diameter	From	To	SEAL Material	From	To	Amount sacks or pounds
10	0	18	GrBen	0	18	9 Sacks

How was seal placed? Method ☐ A ☐ B ☐ C ☐ D ☐ E

☒ Other: Granuler Bentonite placed dry

Backfill placed from ft. to ft. Material

Gravel placed from ft. to ft. Size of gravel

(6) CASING/LINER:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	6"	+1	19	.250	☒	☐	☒	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Liner:					☐	☐	☐	☐
					☐	☐	☐	☐

Final location of shoe(s)

(7) PERFORATIONS/SCREENS:

☐ Perforations Method

☐ Screens Type Material

From	To	Slot size	Number	Diameter	Tele/pipe size	Casing	Liner
						☐	☐
						☐	☐
						☐	☐
						☐	☐
						☐	☐
						☐	☐

(8) WELL TESTS: Minimum testing time is 1 hour

☐ Pump ☐ Bailer ☒ Air ☐ Flowing Artesian

Yield gal/min	Pumping level	Drill stem at	Time
80		100'	½ hr
25		42'	1 hr

Temperature of water Depth Artesian Flow Found

Was a water analysis done? ☐ Yes By whom

Did any strata contain water not suitable for intended use? ☐ Too little

☐ Salty ☐ Muddy ☐ Odor ☐ Colored ☐ Other

Depth of strata:

(9) LOCATION OF WELL by legal description:

County Clack. Latitude ′ ″ Longitude ′ ″

Township 2 South N or S, Range 1 East E or W, WM.

Section 25 SE ¼ SW ¼

Tax Lot 3900 Lot Block Subdivision

Street Address of Well (or nearest address) 2929 Lancaster St. West Linn, OR 97068

(10) STATIC WATER LEVEL:

27 ft. below land surface. Date 4-21-86

Artesian pressure lb. per square inch. Date

(11) WELL LOG:

Ground elevation

Material	From	To	WB?	SWL
Clay brown	0	6		
Basalt brown	6	12		
Basalt grey	12	42		
Basalt black	42	68		
Basalt grey fractured	68	74		
Basalt brown	74	82	X	27
Basalt brown fractured	82	104	X	27

Date started 4-21-86 Completed 4-21-86

(unbonded) Water Well Constructor Certification:

I constructed this well in compliance with Oregon well construction standards. Materials used and information reported above are true to my best knowledge and belief.

Signed Date

(bonded) Water Well Constructor Certification:

I accept responsibility for construction of this well and its compliance with all Oregon water well standards. This report is true to the best of my knowledge and belief.

Signed Marvin A Skyles Date 22 Apr 1986

Company SKYLES DRILLING, INC Co. Job No. 1486

9809C 10/85

STATE OF OREGON

WATER WELL REPORT

(as required by ORS 537.765)

RECEIVED
FEB 6 - 1986
WATER RESOURCES DEPT
SALEM, OREGON

CLAC 03379

25/1E-25dc

Owner's Well Number

(1) OWNER:

Name Daniel S. Lee

Address 2929 Lancaster

City West Linn State Oregon Zip 97068

(2) TYPE OF WORK:

☒ New Well ☐ Deepen ☐ Recondition ☐ Abandon

(3) DRILL METHOD:

☒ Rotary Air ☐ Rotary Mud ☐ Cable ☐ Other

(4) PROPOSED USE:

☒ Domestic ☐ Community ☐ Industrial ☐ Irrigation

☐ Thermal ☐ Injection ☐ Other

BORE HOLE CONSTRUCTION:

Depth of Completed Well 104 ft.

Special Standards date of approval

HOLE Diameter	From	To	SEAL Material	From	To	Amount sacks or pounds
10"	0	18	Cem.	0	18'	5 Sacks
6"	18	104				

How was seal placed? Method ☐ A ☐ B ☒ C ☐ D ☐ E

☐ Other

Backfill placed from ft. to ft. Material

Gravel placed from ft. to ft. Size of gravel

(6) CASING/LINER:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	6"	+1	19	.250	☒	☐	☒	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Liner:					☐	☐	☐	☐
					☐	☐	☐	☐

Final location of shoe(s)

PERFORATIONS/SCREENS:

☐ Perforations Method

☐ Screens Type Material

From	To	Slot size	Number	Diameter	Tele/pipe size	Casing	Liner
						☐	☐
						☐	☐
						☐	☐
						☐	☐
						☐	☐
						☐	☐

(8) WELL TESTS: Minimum testing time is 1 hour

☐ Pump ☐ Bailer ☒ Air ☐ Flowing Artesian

Yield gal/min	Pumping level	Drill stem at	Time
80		100'	½ hr
25		42'	1 hr

Temperature of water Depth Artesian Flow Found

Was a water analysis done? ☐ Yes By whom

Did any strata contain water not suitable for intended use? ☐ Too little

☐ Salty ☐ Muddy ☐ Odor ☐ Colored ☐ Other

Depth of strata:

(9) LOCATION OF WELL by legal description:

County Clack. Latitude ' " Longitude ' "

Township 2 South N or S, Range 1 East E or W, WM.

Section 25 SE ¼ SW ¼

Tax Lot Lot Block Subdivision

Street Address of Well (or nearest address) 2929 Lancaster West Linn, OR

(10) STATIC WATER LEVEL:

30 ft. below land surface. Date 1-21-86

Artesian pressure lb. per square inch. Date

(11) WELL LOG:

Ground elevation

Material	From	To	WB?	SWL
Clay brown	0	6		
Basalt brown	6	13		
Basalt grey	13	38		
Basalt grey & brown	38	42		
Basalt grey (softer)	42	50	X	
Basalt black	50	62		
Basalt brown fractured	62	67		
Basalt grey fractured	67	71		
Basalt brown	71	82		
Basalt brown fractured	82	96	X	
Basalt grey porous	96	104	X	30

Date started 01-21-86 Completed 01-21-86

(unbonded) Water Well Constructor Certification:

I constructed this well in compliance with Oregon well construction standards. Materials used and information reported above are true to my best knowledge and belief.

Signed Date

(bonded) Water Well Constructor Certification:

I accept responsibility for construction of this well and its compliance with all Oregon water well standards. This report is true to the best of my knowledge and belief.

Signed Date 01-21-86

Company SKYLES DRILLING, INC. Co. Job No. 0286

9809C 10/85

RECEIVED
DEC 17 1997
WATER RESOURCES DEPT.
SALEM, OREGON

CLAC 52908

STATE OF OREGON
GEOTECHNICAL HOLE REPORT
(as required by OAR 690-240-035)

(1) OWNER/PROJECT: Hole Number LT-1
Name City of West Linn
Address 2042 8th Ave.
City West Linn State OR Zip 97068

(2) TYPE OF WORK
☒ New ☐ Deepening ☐ Alteration (repair/recondition) ☐ Abandonment

(3) CONSTRUCTION:
☐ Rotary Air ☐ Hand Auger ☒ Hollow Stem Auger
☒ Rotary Mud ☐ Cable Tool ☐ Push Probe ☒ Other HQ Core

(4) TYPE OF HOLE:
☐ Uncased Temporary ☐ Cased Permanent
☐ Uncased Permanent ☒ Slope Stability ☒ Other Inclinometer

(5) USE OF HOLE: Geotechnical Data, soil descriptions & install inclinometer full depth of hole

(6) BORE HOLE CONSTRUCTION:
Special Construction approval ☐ Yes ☒ No Depth of Completed Hole 57½ ft.

HOLE Diameter	From	To	SEAL Material	From	To	Sacks or pounds
8"	0	54½	Bent. Grout	0	57½	Two 5# batches
4 7/8"	54½	57½				

Backfill placed from ____ ft. to ____ ft. Material ____
Filter Pack placed from ____ ft. to ____ ft. Size of pack ____

(7) CASING/SCREEN:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	2 7/8	0	57½		☐	☒	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Screen:					☐	☐	☐	☐
					☐	☐	☐	☐

Slot size N/A

(8) WELL TEST:
☐ Pump ☐ Bailer ☐ Air ☐ Flowing Artesian
Permeability ____ Yield ____ GPM ____
Conductivity ____ PH ____
Temperature of water ____ °F/C Depth artesian flow found ____ ft.
Was water analysis done? ☐ Yes ☐ No
By whom? ____
Depth of strata analyzed. From ____ ft. to ____ ft.
Remarks: ____

(9) LOCATION OF HOLE by legal description:
County Clakamas Latitude 45°22'7" Longitude 122°37'23"
Township 2 [redacted] or ☒ Range 1 ☐ E or W. WM.
Section 25 SE 1/4 NE 1/4
Tax Lot ____ Lot ____ Block ____ Subdivision ____
Street Address of Well (or nearest address) Skyline Dr West Linn

Map with location indentified must be attached

(10) STATIC WATER LEVEL:
N/A ft. below land surface. Date ____
Artesian pressure ____ lb. per square inch. Date ____

(11) SUBSURFACE LOG:
Ground Elevation ~430 ft.

Material Description	From	To	SWL
Clayey SILT	0	50	
decomp. BASALT	50	57½	

Date Started 12-5-97 Date Completed 12-5-97

(12) ABANDONMENT LOG:

Material Description	From	To	Sacks or Pounds

Date started ____ Date Completed ____

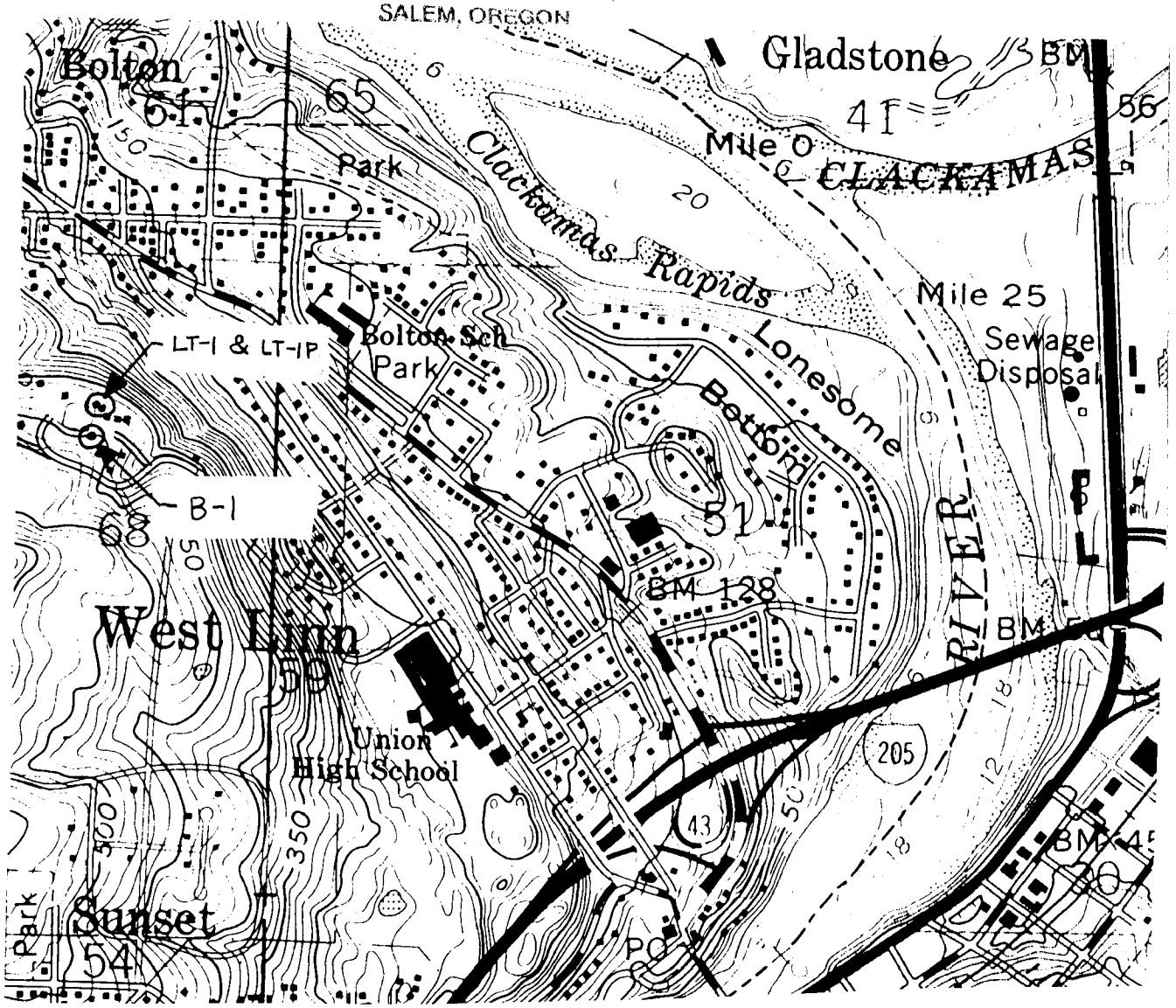
Professional Certification
(to be signed by a licensed water supply or monitoring well constructor, or registered geologist or civil engineer).

I accept responsibility for the construction, alteration, or abandonment work performed during the construction dates reported above. All work performed during this time is in compliance with Oregon's geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License or Registration Number PE 15,548
Signed D. A. Vessely Date 12/16/97
Affiliation Landslide Technology

THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK

ORIGINAL & FIRST COPY-WATER RESOURCES DEPARTMENT SECOND COPY-CONSTRUCTOR THIRD COPY-CUSTOMER



Cornforth Consultants, Inc. 10250 S.W. Greenburg Rd. Suite 111 Portland, OR 97223	TITLE LOCATION MAP	DATE 12-16-97
		JOB NO.
	JOB Bolton Pump Station, West Linn, OR	FIG. 1

RECEIVED

DEC 17 1997

WATER RESOURCES DEPT.
SALEM, OREGON

CLAC 52909

STATE OF OREGON
GEOTECHNICAL HOLE REPORT
(as required by OAR 690-240-035)

(1) OWNER/PROJECT: Hole Number LT-1P
Name City of West Linn
Address 2042 8th Ave.
City West Linn State OR Zip 97068

(2) TYPE OF WORK
☒ New ☐ Deepening ☐ Alteration (repair/recondition) ☐ Abandonment

(3) CONSTRUCTION:
☐ Rotary Air ☐ Hand Auger ☒ Hollow Stem Auger
☐ Rotary Mud ☐ Cable Tool ☐ Push Probe ☐ Other

(4) TYPE OF HOLE:
☐ Uncased Temporary ☐ Cased Permanent
☐ Uncased Permanent ☒ Slope Stability ☐ Other

(5) USE OF HOLE: Geotechnical Data Groundwater measurement.

(6) BORE HOLE CONSTRUCTION:
Special Construction approval ☐ Yes ☒ No Depth of Completed Hole ____ ft.

HOLE Diameter	From	To	SEAL Material	From	To	Sacks or pounds
8"	0	40'	Bent. chip	0	35'	11 sacks
			~~8-12 SAND~~			

Backfill placed from ____ ft. to ____ ft. Material 8-12 SAND
Filter Pack placed from 35' ft. to 40' ft. Size of pack ____

(7) CASING/SCREEN:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	1"	0	37.5		☐	☒	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Screen:	2"	37.5	39.5		☐	☒	☐	☐
					☐	☐	☐	☐

Slot size 10 slot

(8) WELL TEST:
☐ Pump ☐ Bailer ☐ Air ☐ Flowing Artesian
Permeability ____ Yield ____ GPM ____
Conductivity ____ PH ____
Temperature of water ____ °F/C Depth artesian flow found ____ ft.
Was water analysis done? ☐ Yes ☐ No
By whom? ____
Depth of strata analyzed. From ____ ft. to ____ ft.
Remarks: ____

(9) LOCATION OF HOLE by legal description:
County Clakamas Latitude 45° 22' 7" Longitude 122° 37' 23"
Township 2 N or (S) Range 1 (E) or W. WM.
Section 25 SE 1/4 NE 1/4
Tax Lot ____ Lot ____ Block ____ Subdivision ____
Street Address of Well (or nearest address) Skyline Dr. West Linn

Map with location indentified must be attached

(10) STATIC WATER LEVEL:
20 ft. below land surface. Date 12-10-97
Artesian pressure ____ lb. per square inch. Date ____

(11) SUBSURFACE LOG:
Ground Elevation ____

Material Description	From	To	SWL
Clayey Silt	0'	40'	

Date Started 12-8-97 Date Completed 12-8-97

(12) ABANDONMENT LOG:

Material Description	From	To	Sacks or Pounds

Date started ____ Date Completed ____

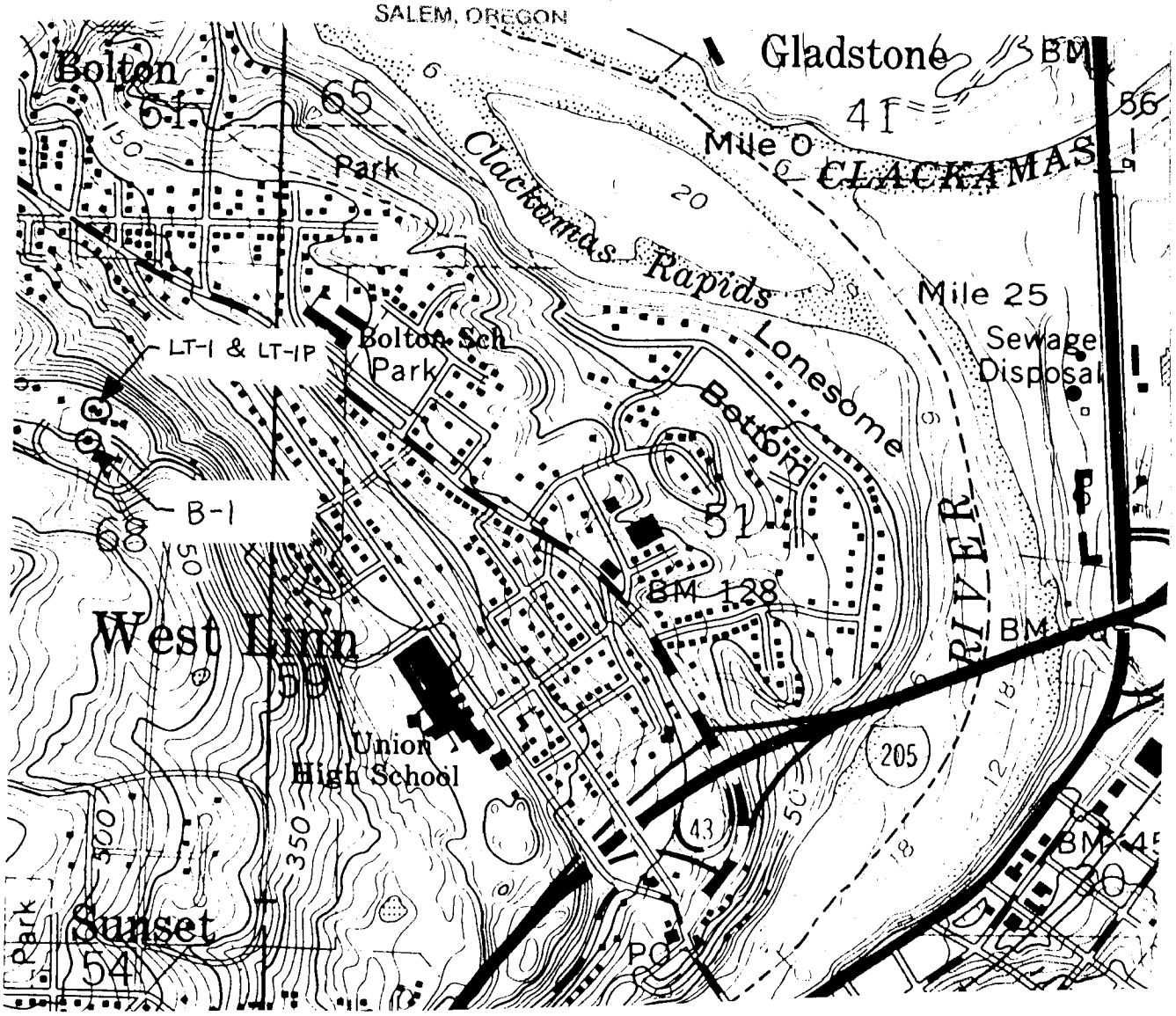
Professional Certification
(to be signed by a licensed water supply or monitoring well constructor, or registered geologist or civil engineer).

I accept responsibility for the construction, alteration, or abandonment work performed during the construction dates reported above. All work performed during this time is in compliance with Oregon's geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License or Registration Number OE 15548
Signed D. A. Vessely Date 12/16/97
Affiliation Landslide Technology

THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK

ORIGINAL & FIRST COPY-WATER RESOURCES DEPARTMENT SECOND COPY-CONSTRUCTOR THIRD COPY-CUSTOMER



NORTH

SCALE IN FEET
0 1000 2000

Cornforth Consultants, Inc. 10250 S.W. Greenburg Rd. Suite 111 Portland, OR 97223	TITLE LOCATION MAP	DATE 12-16-97
		JOB NO.
	JOB Bolton Pump Station, West Linn, OR	FIG. 1

GeoPacific Engineering, Inc.
Real-World Geotechnical Solutions
Investigation • Design • Construction Support

PHOTOGRAPHIC LOG

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel (503) 598-8445
Fax (503) 941-9281

6123 SKYLINE DRIVE
GEOTECHNICAL SITE INVESTIGATION
PHOTOGRAPHIC LOG

View of Property Facing North

Test Pit TP-1, View of Property Facing South

6123 SKYLINE DRIVE
GEOTECHNICAL SITE INVESTIGATION
PHOTOGRAPHIC LOG

View of Site Facing North

Undocumented Fill Soil Containing Organic Material

6123 SKYLINE DRIVE
GEOTECHNICAL SITE INVESTIGATION
PHOTOGRAPHIC LOG

Native Soils Below the Undocumented Fill

Undocumented Fill in Test Pit TP-2

6123 SKYLINE DRIVE
GEOTECHNICAL SITE INVESTIGATION
PHOTOGRAPHIC LOG

Undocumented Fill and Exposed Native Soils in Test Pit TP-2

South Side of Property Where House was Demolished

GRI
9750 SW Nimbus Avenue
Beaverton, OR 97008-7172
p | 503-641-3478 f | 503-644-8034

August 10, 2015

5338-A GEOTECHNICAL RPT
(REVISED 09-10-15)

Murray, Smith & Associates, Inc.
121 SW Salmon Street, Suite 900
Portland, OR 97204

Attention: Tom Boland, PE

SUBJECT: Geotechnical Investigation and Site-Specific Seismic Hazard Study
4-MG Bolton Reservoir
West Linn, Oregon

At your request, GRI has conducted a geotechnical investigation and site-specific seismic hazard study for the above-referenced project in West Linn, Oregon. The general location of the site is shown on the Vicinity Map, Figure 1. The purpose of this investigation was to evaluate subsurface materials and conditions at the site and develop geotechnical recommendations for use in design and construction of the reservoir. The investigation included a review of available geotechnical information for the site and vicinity, subsurface explorations, laboratory testing, and engineering and seismic analyses. This report describes the work accomplished and provides our conclusions and recommendations for design and construction of the proposed reservoir.

Because the reservoir is considered an essential facility in accordance with the 2014 Oregon Structural Specialty Code (OSSC), our investigation included a site-specific seismic hazard study.

GRI completed a preliminary geotechnical evaluation of the site to support the conceptual siting analysis. The results of our evaluation are summarized in our August 31, 2012, report to Murray, Smith & Associates, Inc. (MSA) titled, "Preliminary Geotechnical Evaluation for Conceptual Siting Analysis, 4-MG Bolton Reservoir, West Linn, Oregon."

PROJECT DESCRIPTION

As currently proposed, the existing 2.5-million gallon (MG) concrete reservoir will be replaced with a partially embedded 4-MG concrete reservoir established in a cut up to 25 ft deep. The approximate location of the proposed tank with respect to the existing reservoir and site topography is shown on the Site Plan, Figure 2. The new reservoir will consist of a partially embedded, American Water Works Association (AWWA) D110-13 Type I wire-wound, circular, pre-stressed concrete tank with an inside diameter of about 168 ft and wall height of 30 ft. Based on information from MSA, the finished floor of the new reservoir will be established at approximate elevation 429 ft (NAVD 88) and have an overflow at approximate elevation 457 ft with 2 ft of freeboard. The normal operating level of the reservoir will be at approximate elevation 454 ft (i.e. 25 ft depth of water). The reservoir foundation will consist of a 24-in.-thick reinforced concrete mat slab. The 9-in.-thick reinforced concrete roof will be supported by a 12-in.-thick core wall and 24-in. diameter columns located on approximately 20-ft center-to-center spacing. The

tank will be backfilled to elevation 440 and 450 ft on the north and south side, respectively, and will support a 15-ft-wide gravel service road.

As shown on Figure 2, the new reservoir will be established toward the southwest portion of the site, and the northern side of the reservoir will be about 50 ft farther south than the existing reservoir to reduce the risk of potential local slope instability along the north side of the site. The top of the slope along the north side of the site will be flattened by removing soil to improve the overall stability of the outer slope surface.

Based on our experience with similar projects, the amount of differential settlement that can be tolerated across the footprint of a concrete reservoir is small, and limiting differential settlement will be critical to the performance of the reservoir. Possibly poor quality fill and localized zones of soft, compressible soil in the upper, highly weathered portion of the basalt have been disclosed by recent exploration. To reduce the risk of undesirable settlement beneath the reservoir, ground improvement, such as aggregate piers overlain with several feet of compacted crushed rock, is planned to limit settlement. Ground improvement will also improve the factor of safety for the seismic slope stability.

The excavation necessary for construction of the new reservoir is anticipated to extend to approximately 30 ft below existing grades. As currently planned, the side slopes of the excavation will be sloped at up to 1H:1V where space allows. However, we anticipate a shoring system constructed from the top-down, such as a tied-back soldier pile wall or possibly a soil-nail wall, may be necessary to retain the temporary excavation next to the existing pump station to the southeast and along the west side of the reservoir footprint near the properly line. We anticipate the shoring walls could have a total retained height of up to 30 ft.

The project will include replacement of the existing 18-inch diameter reservoir water piping with 24-inch diameter inlet, outlet and pump station suction piping. The existing 8-inch diameter PVC water distribution main north of the reservoir will be replaced with a 12-inch diameter ductile iron main. The existing overflow and drain piping will be replaced with an 18-inch diameter overflow pipe that discharges to a dechlorination manhole north of the reservoir. The new reservoir foundation and leak detection drains will be routed to a monitoring manhole. All reservoir drainage, emergency overflows, and site drainage will be routed ultimately to a new terminal drainage manhole.

The existing 6-inch diameter cast iron drain pipe that discharges to the receiving creek will be replaced with a 12-inch diameter HDPE drain line from the new terminal drainage manhole. New impervious area on the site will be routed to a stormwater detention pond with a depth of about 10 to 12 ft located about 15 ft northwest of the reservoir and then to a subsequent water quality facility northeast of the reservoir that will discharge directly to the terminal drainage manhole. Both stormwater facilities will be underlain with an impervious liner to prevent stormwater from entering the slope and will also be equipped with leak detection systems routed to the monitoring manhole for observation.

SITE DESCRIPTION

Topography and Surface Conditions

As shown on the Site Plan, Figure 2, and the Site Map, Figure 3, the reservoir site is located northeast of Skyline Drive on a relatively flat bench at about elevation 445 to 450 ft (NAVD 88). Land use in the area surrounding the existing reservoir consists of forested undeveloped land to the south and residential to the

west, north, and east. The ground surface north of the reservoir slopes downward at about 25° to the northeast to residences along Caufield Street and is vegetated with mature trees and brush.

GEOLOGY

Geologic Setting

The site is located on the eastern flank of the Tualatin Mountains, a topographic upland that separates the Portland Basin to the northeast from the Tualatin Basin to the west and the Willamette Valley to the south. Geologic mapping completed for the area indicates the site is located in the vicinity of the contact between the Miocene Wanapum Basalt and the Grande Ronde Basalt units of the Columbia River Basalt Group (Madin, 2009). Where fresh and unweathered, these basalt units are typically a light to dark gray, dense volcanic rock. However, the Wanapum-Grande Ronde boundary is characterized in places by an erosional unconformity or an interbed that varies from non-marine sediments to a thick relic soil, and is referred to as the Vantage Horizon (Beeson et al., 1985). The Vantage Horizon originated during a period of erosion and soil development that occurred between volcanic flow events. Large-scale landslides are known to occur where the Vantage Horizon daylights at or near the ground surface. The reservoir site and other areas of the Tualatin Mountain upland are capped by deposits of fine-grained, windblown silt, referred to as Portland Hills Silt. Quaternary alluvial deposits associated with the Willamette River and the Ice Age Missoula Floods (about 15,000 to 20,000 years ago) are present northeast of the site, north of Highway 43. A geologic map and cross section of the project area are provided on Figure 4.

Faults

General. Several geologic faults are located in the project area. Two northeast-trending unnamed normal faults are mapped near the site (Yeats et al., 1991). These faults, which are bedrock faults in the Columbia River Basalt, do not have historic seismicity and are not considered by U.S. Geological Survey (USGS) to contribute to the seismic hazard at the site. The surface trace of the Bolton Fault is located about 900 ft northeast of the site, the Oatfield Fault is about 2.5 miles northeast of the site, and the Portland Hills Fault is about 3 miles northeast of the site (Schlicker and Finlayson, 1979; Personius et al., 2002). These faults do not have historic seismicity, but the USGS considers each of these faults to contribute to the overall seismic hazard at the site.

Bolton Fault. The northwest-trending Bolton Fault is responsible for the straight, abrupt front of the hills west of Highway 43 between Lake Oswego and West Linn. The Bolton Fault does not appear to have moved since the time of the Missoula Floods, about 15,000 to 20,000 thousand years ago (DOGAMI, 2009). This fault is located about 900 ft northeast of the site. USGS considers the structure a southwest-dipping reverse fault with down-to-the-northeast separation of up to 200 m (600 ft) in Miocene volcanic rocks (Personius et al., 2002). No fault scarps in surficial deposits or other unequivocal evidence of Quaternary displacement has been described in the literature. The USGS classifies the fault as Class B until further studies are conducted (Personius et al., 2002). Class A faults generally have a slip rate greater than 5 mm/yr and well constrained paleoseismic data. Class B faults include all other faults lacking paleoseismic data necessary to constrain the recurrence intervals of large events (Petersen et al., 1996).

An online Department of Geology and Mineral Industries (DOGAMI) mapping viewer (DOGAMI HazVue, accessed January 8, 2015) places the closest point of the surface trace of the Bolton Fault about 900 ft northeast of the existing reservoir (distance measured from northeast corner of existing reservoir to the

trace mapped at intersection of Highway 43 and Buck Street). Other published DOGAMI maps show the surface trace of the Bolton Fault generally coincident with the relatively linear eastern slope toe of the Tualatin Mountains upland, or about 900 ft northeast of the existing reservoir (Schlicker and Finlayson, 1979, scale 1:24,000; Burns et al., 1997, scale 1:100,000). However, it should be noted that the available geologic resolution and confidence to locate the Bolton Fault with about 500 ft at scales of 1:24,000 and 1:100,000 is low. Yeats et al. (1991) and Madin (2009) map two strands of the Bolton Fault near the site, see Figure 4. Their mapping shows one strand along the abrupt topographic escarpment, and another buried strand is concealed beneath Quaternary alluvial deposits near Highway 43.

Canby 133 Ancient Landslide

DOGAMI is the state agency responsible for geologic hazard mapping in Oregon. DOGAMI has indicated in its statewide landslide hazard database that Bolton Reservoir is located on a prehistoric (>150 yrs), deep-seated (>15 ft deep), translational rock landslide, referred to as Canby 133. Figure 5 shows the limits of the landslide from the state database. Mapping of landslide deposits are based, in part, on light detection and ranging (lidar) derived elevation data and interpretation of surface topography typical of landslide features. Canby 133 was mapped using lidar and a method protocol outlined by DOGAMI (2009) with a "moderate" level of confidence. The confidence ranking (low, moderate, and high) is based on desktop analysis. Bill Burns with DOGAMI was contacted regarding this feature and recalls they did a vehicle-based reconnaissance from public roads to map this feature, but he was not aware of other data (i.e., reports, borings, or anecdotal stories of ground movement) about the feature. Mr. Burns indicated unpublished DOGAMI field mapping from 2004 also indicates the area is a landslide. This information suggests the Bolton Reservoir site is located on a very large, old or "ancient" landslide.

As part of the Murray, Smith & Associates, Inc. (MSA) team, Cornforth Consultants, Inc. (2014) completed a seismic landslide evaluation for the planned reservoir. The evaluation was performed to identify any signs of landslide activity near the reservoir and to provide opinions on potential impacts of seismic landslide displacements on proposed improvements at the site. Their geotechnical reconnaissance of the ancient landslide around Bolton Reservoir did not identify signs of active movement, especially along the margins, where differential movement would be greatest. They also concluded the ancient landslide is likely to move feet rather than tens of feet during a large earthquake.

The mapped northeast boundary of the Canby 133 landslide near the site is essentially coincident with the prominent straight and abrupt topographic escarpment associated with the Bolton Fault. In our opinion, this indicates the Bolton Fault cross-cuts the toe of the Canby 133 landslide. Therefore, the Canby 133 landslide is likely on the order of at least 15,000 to 20,000 years old.

SLOPE STABILITY

Previous Reports

Three geotechnical engineering reports prepared for the Bolton Reservoir site in 1972, 1988, and 1998, were provided to GRI. The first report was prepared by Northwest Testing Laboratories (NTL) for the City of West Linn (City) in 1972 (NTL, 1972). The report provided the results of a soil and foundation investigation and recommendations for enlarging the reservoir. The report concluded the slope east of the site could accommodate the additional load of the reservoir.

L.R. Squier Associates, Inc. prepared a geologic reconnaissance report for the City in 1988 (L.R. Squier, 1988). The purpose of the report was to evaluate the slope northeast of the reservoir for a planned residential development, where there were concerns of slope stability. The report concluded that steep slopes, weak and locally thin soils, soil creep, and groundwater seepage from springs suggested a high risk for slope instability, and a comprehensive geotechnical investigation was recommended.

In the 1970s, a small earth flow landslide occurred along the steeply sloping wooded area northeast of the reservoir. Large ground cracks occurred north of the reservoir in 1996 following heavy rainfall. Landslide Technology conducted an investigation into the stability of the steep slope area in 1997 (Landslide Technology, 1998). The investigation included a reconnaissance, subsurface explorations, laboratory testing, installation of an open-pipe piezometer and inclinometer casing. Based on the results of the investigation, the report provided an approach for repair of the small earth flow failure.

Site Reconnaissance

A reconnaissance of the site and surrounding area was conducted by a registered geologist and a certified engineering geologist from GRI in June 2012 and January 2015. The following description of the site is a summary of the observations made during the reconnaissance activities. Private properties located immediately northwest and southeast of the site were not accessed, but observed from the public right of way for features of significance. To the northeast, the ground slopes downward at approximately 25° toward Caufield Street. The slope northeast of the reservoir site is wooded with predominantly deciduous trees and occasional conifer tree, and springs. The ground surface is generally covered by English ivy, ferns, and blackberries. Several springs and flowing water were also observed along Caufield Street and originated from the slope above. A concrete manhole and pipe valve were observed along the slope near the northern property boundary. The valve appeared to be rusted through and was leaking water. No indications of recent slope instability were observed along the northeast slope during the site reconnaissance. The surrounding neighborhood was also examined from the public right of way for indications of slope movement (cracked and separated sidewalks or curbs). The reconnaissance did not disclose obvious indications of relatively recent movement, such as cracked streets, sidewalks, or curbs. Limited interviews with City maintenance personnel did not disclose reports of broken or sheared underground utilities.

The slope failure that occurred along the northeast side of the existing reservoir in 1996 and investigated by Landslide Technologies has not been repaired and is covered with vegetation as observed during our January 2015 reconnaissance. Most of the remainder of the slope along the north side of the reservoir has the same general appearance and inclination of the slope adjacent to the landslide. The existing reservoir was fully covered with a liner and could not be examined. However, cracking is present along portions of the north side of the reservoir flatwork and ring wall, particularly in the northwest corner. As with previous observations in 2012, whether the flatwork and ring wall cracking is due to slope movement or fill settlement could not be ascertained.

Inclinometer

In June 2012 and November 2014, GRI monitored the inclinometer that was installed by Landslide Technologies in 1997 at the approximate location shown on Figure 2 during their evaluation of local instability at the northeast corner of the site. An inclinometer casing consists of a plastic pipe with a pair of

orthogonal slots, or grooves, that permit a calibrated instrument to be lowered to the bottom of the casing. When the ground surrounding the casing moves, the casing distorts above the zone of movement, and the orientation of the casing changes. The orientation of the casing is measured by lowering the calibrated instrument to the bottom of the casing and reading the instrument at 2-ft intervals as it is withdrawn. The zone and rate of movement can be determined by comparing the results of successive sets of readings. The inclinometer was installed east of the proposed tank footprint to provide long-term monitoring of the site with respect to potential slope movement.

GRI obtained the baseline measurements collected by Landslide Technologies in 1997 and compared those data with measurements obtained from the inclinometer in June 2012 and November 2014. The readings indicate very small creep-type slope movements have occurred since the inclinometer casing was installed in 1997. The measurements indicate cumulative horizontal movement of 1 and 1.25 in. at the ground surface between the 1997 base line reading and the readings by GRI in June 2012 and November 2014, respectively. The majority of the movement occurred in the upper approximately 10 to 12 ft of the soil profile and was less than about 0.25 in. below this depth. The movement detected in the inclinometer gradually decreases with depth, to no obvious movement at a depth of about 40 ft. Indications of obvious movement at the ground surface, such as ground cracks or settlement, have not been observed during our recent visits to the site.

In our opinion, information provided in the report by Landslide Technology and monitoring of the inclinometer indicate the slope instability that occurred in 1996 is likely related in part to the presence of fill soil placed along the northern slope during the original construction of the reservoir. As part of the reservoir replacement project, soil will be removed from the top of the slope to improve local stability, which may impact the existing inclinometer and piezometer installed by Landslide Technology. We recommend preserving the slope inclinometer and piezometer for future monitoring. In this regard, the upper portion of the inclinometer and piezometer may need to be removed followed by a new inclinometer baseline reading. GRI should participate closely with any field modifications to the inclinometer and piezometer casing.

SUBSURFACE CONDITIONS

General

Subsurface materials and conditions at the site were evaluated by GRI on June 15, 2012, with one boring, designated B-1, and on October 27 through 29, 2014, with two borings, designated B-2 and B-3. The locations of the borings are shown on Figure 2. The borings were advanced to depths of about 76 to 90 ft. The field and laboratory programs completed for this study are discussed in detail in Appendix A. Logs of the borings are provided on Figures 1A through 3A. The terms and symbols used to describe the soil and rock encountered in the borings are defined in Tables 1A and 2A and the attached legend.

In addition to the borings completed by GRI, Landslide Technology (1998) and Northwest Testing Laboratories (1972) completed borings at the locations shown on Figure 2. Logs of the previously drilled borings are provided in Appendix B.

The explorations indicate the reservoir site is mantled with a variable thickness of silty and clayey manmade fill, underlain by native silty and clayey soils, which are in turn underlain by basalt of the

Columbia River Basalt Group. The relative consistency of the silty and clayey fill and native soil is generally medium stiff to stiff. The native soil is underlain by extremely soft (R0), predominantly decomposed to decomposed basalt (Wanapum Basalt). The basalt has generally weathered to the consistency of medium stiff to hard soil. Localized zones in the decomposed basalt have weathered to the consistency of soft, silty and clayey soil. The soft soil-like zones were encountered locally between depths of about 20 and 40 ft below the ground surface. The basalt transitions to generally fresh to moderately weathered, medium hard to hard (R3 to R4) basalt at depths of about 55 to 60 ft below the ground surface. The Wanapum Basalt transitions to the Vantage Horizon of the Grande Ronde Basalt at a depth of about 79 and 71 ft below the ground surface in GRI borings B-2 and B-3, respectively. The zone between the two basalt formations is called the Vantage Horizon and consists of moderately weathered, very soft to medium hard (R1 to R3) basalt. Boring B-1 did not encounter the Vantage Horizon. GRI borings B-2 and B-3 did not disclose indications of soft soil and/or shear zones within the Vantage Horizon. The transition from soil-like weathered basalt to relatively intact medium hard to hard basalt at a depth of about 55 to 60 ft is interpreted to be the lower boundary of material within the mass of the very large, presently inactive, ancient/prehistoric, deep-seated landslide.

Groundwater

An observation standpipe piezometer was installed in GRI borings B-2 and B-3 to a depth of 90 and 48 ft, respectively, to monitor groundwater levels at the site. As discussed previously, Landslide Technology installed a standpipe piezometer to a depth of 40 ft in a boring at the northeast corner of the site. On November 18, 2014, groundwater levels in standpipe piezometers installed GRI borings B-2 and B-3, and Landslide Technology boring LT-1P were measured at depths of about 23, 42, and 19 ft, respectively, below the ground surface. On January 7, 2015, the groundwater level in borings B-2, B-3 and LT-1P was about 23, 41, and 19 ft, respectively, below the ground surface. We anticipate the regional groundwater level is significantly deeper, and the groundwater levels measured in the standpipes are perched within the soil and rock. It is expected that perched groundwater in the soil could approach the ground surface locally during periods of prolonged or intense precipitation that are common during the wet, fall through spring months and will likely drop to depths greater than 20 ft during typical dry, summer and early fall months.

CONCLUSIONS AND RECOMMENDATIONS

General

The new reservoir will be constructed toward the southwest portion of the site in a cut up to 30 ft deep and will have a finished floor at about elevation 429 ft and an overflow at elevation 457 ft with 2 ft of freeboard. The sides of the new reservoir will be backfilled to within about 20 and 10 ft of the top of the reservoir on the north and south sides, respectively. Ground improvement will be completed beneath the new tank to increase seismic slope stability for the new reservoir and reduce differential static settlements. Soil will be removed along the crest of the slope along the north side of the site to improve the local slope stability. Drainage will be installed around and beneath the reservoir to manage subsurface water, and new inlet/outlet and overflow piping will be installed.

The reservoir site is mantled with a variable thickness of relatively stiff, silty and clayey manmade fill that is underlain by relatively stiff, native silty and clayey soils, which are in turn underlain by basalt. The basalt has generally weathered to the consistency of medium stiff to hard soil to depths of about 55 to 60 ft.

However, localized zones in the decomposed basalt between depths of about 20 to 40 ft have weathered to the consistency of soft, silty and clayey soil. Soft to hard (R2 to R4) basalt underlies the decomposed basalt at depths of 55 to 60 ft. The groundwater level at the site may approach the ground surface during periods of prolonged or intense precipitation that are common during the wet, fall through spring months.

As previously discussed, the reservoir site is located on a very large, ancient landslide. However, reconnaissances by GRI as part of this study and during our 2012 study did not disclose indications of recent landslide movement. A reconnaissance recently completed by Cornforth Consultants (December 2014) also did not identify signs of active movement. It is our opinion the risk of significant future movement of the large, ancient landslide is low. It is expected that the greatest risk of significant movement of the large landslide would be during and/or following a large seismic event. Because the reservoir site is located within the middle of this large translational landslide mass and away from the margins, the risk of significant differential movement within the footprint of the new reservoir following the design-level earthquake is expected to be low. The planned ground improvement beneath the reservoir, removal of soil at the top of the slope along the north side of the site, and the gravel pad and subdrainage system around and beneath the reservoir will improve local factors of safety as they relate to potential reservoir instability. In our opinion, the new reservoir, as planned, will not materially affect the existing site slope stability. Slope stability analyses and discussion are provided in the Slope Stability Analyses section in this report.

In our opinion, the proposed reservoir can be supported on spread footings and a reinforced floor slab system underlain by a granular base course section underlain by improved ground. We anticipate overall site grading can be accomplished with conventional construction equipment. The major geotechnical considerations with construction of the planned reservoir are the moisture-sensitive nature of the soil and decomposed basalt and potential for shallow, perched groundwater. The following sections of this report provide our conclusions and recommendations for design and construction of the reservoir.

Seismic Considerations

We anticipate the new reservoir will be designed in accordance with the American Water Works Association (AWWA) D110-13 standard entitled, *Wire- and Strand-Wound, Circular, Prestressed Concrete Water Tanks*, and the 2012 International Building Code (IBC) with 2014 Oregon Structural Specialty Code (OSSC) modifications. The 2012 IBC evaluates seismic loading in accordance with the American Society of Civil Engineers (ASCE) 7-10 document entitled, *Minimum Design Loads for Buildings and Other Structure*. We anticipate seismic design of the new reservoir will be completed in accordance with the 2012 IBC and ASCE 7-10 documents.

The reservoir is considered an essential facility by Oregon Revised Statute (ORS) 455.447, and GRI has completed a site-specific seismic hazard study in accordance with the 2012 IBC with 2014 OSSC modifications. The results of this study are provided in Appendix B and indicate IBC Site Class D, or a stiff soil site, is appropriate for design of the new reservoir. The IBC design methodology uses two spectral response coefficients, S_S and S_1, corresponding to periods of 0.2 and 1.0 second, to develop the MCE_R earthquake spectrum. The S_S and S_1 coefficients for the site located at the approximate latitude/longitude coordinates of 45.3684°N and 122.6247°W are 0.95 and 0.41 g, respectively. We recommend using the code-based F_a and F_v factors of 1.12 and 1.59, respectively, for Site Class D conditions to estimate the

ground surface response spectrum. The design spectrum is based on a damping ratio of 5%. To evaluate sloshing at a damping ratio of 0.5%, the design spectrum for Site Class D can be multiplied by a factor of 1.5in accordance with the AWWA D110-13 standard.

Based on preliminary evaluations, there is some risk of seismically induced soil strength loss in relatively thin zones in the decomposed basalt that have weathered to the consistency of soft soil that were encountered locally between depths of about 25 to 40 ft below the existing ground surface. In our opinion, the risk of significant post-earthquake settlement due to soil strength loss in these isolated layers is low. However, the presence of these layers presents a risk of seismic slope instability. A discussion of slope stability and alternatives to reduce the risk of local instability are provided below.

The risk of damage by tsunami and/or seiche at the site is absent due to the elevation of the site. In our opinion, the risk of liquefaction-induced lateral spreading and ground deformation at the site is very low. As previously discussed, the surface trace of the Bolton Fault is about 900 ft northeast of the site. Unless occurring on a previously unmapped or unknown fault, it is our opinion the risk of ground rupture at the site is low. In our opinion, there is a risk of seismically induced localized slope instability at the site; however, we anticipate the proposed ground improvement program discussed in the following sections will be completed to reduce the risk of local seismic slope instability. Additional discussion of local faults and other seismic considerations is provided in Appendix C.

Slope Stability Analyses

As discussed previously, the silty and clayey soil that mantles the site is relatively stiff, and the underlying decomposed basalt typically has a consistency comparable to medium stiff to hard soil. However, localized zones in the decomposed basalt have weathered to soft, silty and clayey soil between depths of about 20 to 40 ft below the ground surface. It is possible that these soft zones in the decomposed basalt could extend laterally beneath the site and present a potential risk for localized slope instability, particularly during the design-level earthquake.

Slope stability analyses were completed to evaluate the potential risk of local slope instability affecting the new reservoir. The location of the assumed critical cross section used to develop the slope stability models is shown on Figure 2 and is oriented in a general south-north direction through the center of the planned reservoir, where the side of the reservoir is closest to the slope along the north side of the site. Models were developed to evaluate slope stability for the proposed reservoir (without and with ground improvement) and the existing reservoir. The stability models developed are shown on the Slope Stability Models, Figures 6 through 9. The slope stability models were analyzed with the aid of the computer software SLOPE/W developed by GeoSlope International of Calgary, Alberta, Canada. The groundwater level and locations/boundaries of soil and rock units and associated physical properties used in the models are provided on the aforementioned figures. The new reservoir was assumed to have a reinforced-concrete bottom thickness of 24 in. underlain by a 3-ft-thick crushed rock base course/drainage section. A horizontal pseudo-static coefficient of 0.22 (k_h) for the design-level earthquake, which is equal to about half of the design-level PGA (required by the 2014 OSSC), was used to evaluate the seismic factor of safety values. A residual (large strain) internal angle of friction of 21° and 0 psi cohesion were used to model potential soft zones that may be present in the decomposed basalt layer, based on torsional ring shear

residual strength testing of a sample of soft, clayey silt obtained from within the decomposed basalt at a depth of about 35 ft in boring B-2. The results of this testing are provided in Appendix A.

For the configurations and assumptions described above, and as shown on Figures 6 through 9, a factor of safety against local slope instability for seismic conditions was first computed for potential failure surfaces that could extend laterally beneath the new and existing reservoir. The computed factor of safety against instability is defined as the ratio of the forces (or moments) tending to resist failure to the forces (or moments) tending to cause failure. Computed factors of safety less than 1.0 represent potentially unstable conditions. Based on site geometry and subsurface conditions, it is assumed the most likely mode of failure will consist of translational block-type failures. As shown on Figure 6, the results of the modeling indicate a local seismic factor of safety of 1.0 for a potential slip surface that extends through potential soft zones in the silt and decomposed basalt beneath the new reservoir. To improve the local seismic factor of safety, ground improvement and subdrainage was assumed to be installed and completed beneath the reservoir. The ground improvement was assumed to extend to a depth of about 20 ft below the base of the reservoir and through potential soft zones observed in the borings to the top of the harder decomposed basalt. A French Drain was assumed to be installed along the south side of the reservoir and south to north beneath the center of the reservoir, as shown on Figure 2. The bottom of the drain is assumed to be located at about elevation 416 ft to maintain groundwater about 10 ft below the reservoir. For the purpose of analysis, it is assumed the ground improvement will likely consist of rammed aggregate piers (Geopiers or similar) with an average 10% replacement ratio. The replacement ratio is the area of improved ground (aggregate piers) relative to the total area. It is further assumed the aggregate piers will have an effective stress internal angle of friction of at least 49°, resulting in the improved zone having a composite average effective stress internal angle of friction of about 24.5° based on an assumed existing (untreated) soil mass residual effective stress friction angle of 21 degrees. As shown on Figures 7 and 8, the ground improvement zone in the model was assumed to extend 10 and 20 ft horizontally beyond the south and north side of the reservoir, respectively. As shown on Figure 7, a minimum seismic factor of safety of 1.05 against instability was computed for slip surfaces extending from south to north under the reservoir, assuming completion of ground improvement and installation of the French drain. As shown on Figure 8, the seismic factors of safety for potential slip surfaces on the sloping ground along the north side of the site that could potentially extend under the reservoir are at least 1.1, assuming the ground improvement and French drain installation is completed. In addition, the static factors of safety for potential slip surfaces extending under the reservoir are at least 1.5 for all cases. For comparison purposes, a slope stability model for the existing reservoir was also developed and is shown on Figure 9. The minimum seismic factor of safety against instability computed for a potential slip surface extending south to north under the existing reservoir is about 0.7 and is notably lower than for the planned reservoir constructed either without or with ground improvement. The primary reasons the new reservoir has a greater factor of safety than the existing reservoir, even without ground improvement, are the new reservoir will be set back a greater distance from the slope along the north side of the site, the drainage layers beneath and around the new reservoir will maintain a lower local groundwater level, and there will be an overall net decrease in gravity loads since the new reservoir will replace a significant amount of heavier excavated soil.

A discussion of recommended ground improvement is provided in the next section. Additionally, the top of the slope along the north side of the site should be flattened as much as practical by removal of soil. The planned flattening of the top of the slope along the north side of the site will reduce the soil loads and

improve the overall stability of the sloping ground north of the reservoir and, consequently, will reduce the risk of relatively shallow failures like those that occurred at the northeast corner of the site in the 1970s and in 1996. We recommend the subsurface drains under and around the reservoir, and surface drainage, be collected and discharged to an appropriate off-site location.

Our analysis indicates the measures discussed above will provide for a seismic factor of safety against local instability that could affect the new reservoir of about 1.05 for potential south to north slip surfaces extending under the entire reservoir (Figure 7) and at least 1.1 for potential slip surface extending upward under the reservoir along the sloping north side of the site (Figure 8). However, the planned improvements will not mitigate potential movements of the underlying ancient large landslide mass. Due to the large size of the landslide and potential deep failure surfaces, mitigation measures to improve the stability of the large ancient landslide mass are likely not practical or cost effective. As discussed previously, obvious indications of recent movement of the large landslide mass were not observed during site reconnaissances completed by GRI and Cornforth Consultants, nor have there been reports of potential movements of the large landslide. Based on the available information, the risk of significant movement of the large landslide within the design life of the reservoir is expected to be low and would most likely occur during/following a large seismic event. The seismic movement of the landslide has been estimated to be on the order of feet rather than tens of feet (Cornforth Consultants, 2014). It is expected that if movement of the large landslide mass occurs, the ground supporting the reservoir will tend to "raft" along with the greater landslide mass and the risk of significant differential movements beneath the reservoir will be reduced. In addition, the proposed ground improvement will strengthen the ground beneath the reservoir, which will further reduce the risk of significant differential movements.

Ground Improvement

As discussed in the previous section, ground improvement will be installed beneath the new reservoir to improve local seismic slope stability and limit static differential settlement. We anticipate the ground improvement will need to extend to depths of about 20 ft beneath the base of the new reservoir and through potential soft zones in the decomposed basalt to the top of harder basalt. We recommend the ground improvement be installed to a minimum elevation of 405 ft (NAVD 88). Based on the subsurface conditions, site constraints, and cost, we anticipate rammed aggregate piers (RAP) or similar ground improvement methods would be a practical alternative for this project. The RAPs provide a dense/stiff vertical element with significant shearing resistance and will effectively increase the shear resistance within the zone that is being treated. RAPs also attract vertical loads from the overlying structure and distribute the load to the denser and stiffer layers beneath, thereby reducing total and differential settlement, which is an important consideration for large concrete water reservoirs. RAPs are typically constructed by augering a shaft, typically 30 in. in diameter, to the bottom of the zone requiring improvement and backfilling the shaft with aggregate (crushed rock) that is compacted with a tamping ram in approximate 1-ft-thick lifts. RAPs are typically constructed using large hydraulic excavators equipped with augers and tampers. Augered RAP installation is generally limited to depths of 20 to 25 ft. An alternative method for RAP construction is installation using a hollow mandrel that is vibrated to the required depth instead of augered. Following insertion to the required depth, the mandrel is retracted as aggregate is placed in the bottom of the hole through the center of the mandrel. The mandrel is typically raised about 3 ft as the aggregate is placed and then driven back down about 2 ft to form a 1-ft-thick layer of compacted aggregate. Vibrated RAP methods can be used to construct RAPs to depths of up to 40 ft if conditions are

favorable. Advantages of the vibratory RAP method are reduced spoils generation and it can be used in soft or loose soils below groundwater that may cave without casing. The borings completed for the project indicate the presence of layers and zones of soft soil that are located below groundwater. There is a potential these soft layers and zones could run or cave into the hole if not supported. In this regard, if RAPs are installed using auger methods we recommend casing be readily available in the event of caving or running soils. We recommend the need for casing be evaluated by GRI during construction based on actual conditions at the time of installation.

To achieve the local seismic factor of safety values discussed previously, we recommend a composite effective stress friction angle of the aggregate pier treated soil mass of at least 24.5° based on an assumed existing (untreated) soil mass residual effective stress friction angle of 21°. We anticipate this will result in a minimum replacement ratio of about 10% (the ratio is the area of aggregate piers relative to the total area) using RAPs or comparable methods of ground improvement. We recommend the ground improvement footprint to be essentially square and extend at least 10 ft beyond the south, west, and east sides of the reservoir and 20 ft beyond the north side of the reservoir. The north side of the square treatment area should be parallel to the face of the slope north of the reservoir, which may require greater amounts of excavation than needed to construct the reservoir. It may be possible to limit the amount of excavation in the corner areas of the treatment area by using vibratory RAPs installed at or near existing grade. To provide adequate support for the RAP installation equipment and minimize the risk of subgrade disturbance, we recommend placing a minimum 18-in.-thick working blanket of compacted crushed rock over the reservoir subgrade. A greater thickness of crushed rock may be required if the subgrade is particularly soft. In this regard, the subgrade conditions should be evaluated by GRI before placing the working blanket. It is expected the working blanket will remain as part of the base course section beneath the reservoir. Recommendations for base course are discussed in the Foundation Support, Settlement, and Subdrainage sections of this report.

As discussed above, construction of the RAPs using either a tamping foot or a vibrating mandrel to compact the aggregate backfill will result in ground vibrations. Based on our experience with similar projects that included RAP installation, vibrations from construction of RAPs typically decrease significantly over relatively short distances. Based on previous experience we do not anticipate adjacent residences will be subjected to vibrations in excess of currently acceptable construction levels. However, in our opinion, it would be prudent to install vibration instrumentation along the property lines of the site to monitor potential vibrations from construction equipment. Modifications can be made to construction procedures to reduce excessive vibrations, if necessary. Pre- and post-surveys of adjacent structures/residences should also be completed as part of the vibration monitoring program.

Site Preparation

Vegetation, roots, and other deleterious materials will not be suitable for use as structural fill; therefore, it will be necessary to remove surface organics prior to excavating soils that will be used later for structural fill. The ground surface in areas to receive new fills should also be stripped. Strippings may be used for landscaping purposes or should be removed from the site. We anticipate stripping to a depth of about 3 to 4 in. will be required in areas of lawn. Deeper stripping and grubbing will be required to remove brush and tree stumps where present. With the exception of backfilling around the new reservoir, we anticipate most soil that is excavated to complete the project will be removed from the site. However, stripped areas

to receive structural fill should be evaluated by a qualified geotechnical engineer. Excavation spoils should not be stockpiled during construction within 75 ft of the slope along the north side of the site. The planned locations of soil stockpiles should be evaluated by GRI.

All concrete, piping, and other structural elements associated with the existing reservoir should be removed within the footprint of the new reservoir. Soft, loose, or otherwise unsuitable materials beneath the existing reservoir and within the footprint of the new reservoir should also be removed.

The fine-grained soils and decomposed basalt that mantle the site are sensitive to moisture content and are easily disturbed and softened by construction activity during wet conditions. In Addition, groundwater and site drainage, which are important for maintaining satisfactory slope stability during construction, will be more straightforward to manage during dry conditions. Therefore, we recommend as much site preparation and earthwork as practical be accomplished during the dry, summer months. It has been our experience that the moisture content of the upper approximate 2 to 3 ft of the silt will decrease during warm, dry weather. However, the moisture content of the soil below this depth tends to remain relatively unchanged and well above the optimum moisture content for compaction. As a result, the contractor must employ working procedures that prevent disturbance and softening of the subgrade soils. For this reason, excavation within the final 2 to 3 ft of subgrades should be accomplished with a trackhoe equipped with a smooth-edge bucket. It may be necessary to construct granular haul roads and work pads to provide access during wet conditions to minimize subgrade disturbance during construction. In general, a minimum 18- to 24-in. thickness of relatively clean, fragmental rock having a nominal maximum size of 4 to 6 in. would be required to support heavy construction traffic and protect the silt subgrade during wet ground conditions. If the subgrade is particularly soft, it may be prudent to place a geotextile fabric (AMOCO 2002, or equivalent) on the subgrade as a separation membrane prior to placing and compacting the granular work pad.

Excavation

General. Construction of the new reservoir will require an excavation of about 30 ft below existing site grades. The finished floor of the reservoir will be at about elevation 429 ft, and the bottom of the excavation will be at least 3 ft lower to accommodate the granular base course and subdrainage section. Additionally, the French drain will require excavating a trench to depths of about 7 to 8 ft below the bottom of the granular base course section at the location shown on Figure 2. We anticipate the soils within the zones of excavation can be readily excavated with conventional excavation equipment, such as a large hydraulic trackhoe. The finished subgrade should be completed with a smooth-edge bucket as previously discussed. We anticipate significant portions of the reservoir will be established in the underlying predominantly decomposed to decomposed basalt. The borings made for this investigation indicate the basalt within the planned depth of excavation has a relative consistency comparable to medium stiff to stiff, fine-grained soil. Although not encountered in the borings, it is possible that zones of harder basalt and/or cobble- to boulder-size pieces of relatively hard basalt could be present within the depth of the excavation. The contractor should have means and methods available to accommodate excavation of potentially harder rock.

Cut Slopes. We recommend the temporary cut slopes made to construct the reservoir be no steeper than 1H:1V. However, flatter slopes maybe necessary to maintain an acceptable level of stability depending on

the actual conditions exposed during construction, particularly in locations of groundwater seepage, if encountered in excavations. In this regard, temporary excavation slopes should be evaluated by a qualified geotechnical engineer at the time of construction.

Temporary slopes should be covered with plastic sheeting to reduce erosion during wet weather. In addition, excavation spoils and construction materials should not be stockpiled within 15 ft of the top of the temporary cut slope. The temporary excavation slopes should be evaluated on a daily basis by a knowledgeable person for obvious indications of slope instability such as sloughing, slumping, or ground cracks. Any indications of instability should be reported promptly to GRI for our evaluation. To minimize the risk of instability of temporary cut slopes, we recommend backfilling the reservoir excavation as soon as practical.

Depending on the time of year, perched groundwater may be present within the depth of excavation required to construct the reservoir. We anticipate that seepage, if encountered, can be controlled by pumping from sumps. A ditch should be installed at the top of the cut slopes to direct surface runoff away from the excavation. Water removed from the excavation should not be discharged on or near the top of the slope on the north site.

If temporary excavation slopes extend below the groundwater table or perched groundwater, a 6- to 12-in.-thick layer of relatively clean, well-graded crushed rock placed on the slopes may be required to reduce the risk of running soil conditions.

Permanent cut slopes following final grading, if present, should be no steeper than 2H:1V. Flatter cut slopes may be required if soft and/or wet ground conditions are encountered, which may also require installation of drainage. Permanent excavation slopes should be evaluated by a qualified geotechnical engineer at the time of construction so modifications can be made if necessary.

Temporary Shoring

As discussed previously, the side slopes of the excavation for the reservoir will be sloped at up to 1H:1V where space allows. However, we anticipate a shoring system constructed top-down, such as a tied-back soldier pile wall or possibly a soil-nail wall, may be necessary to retain the sides of the temporary excavation next to the existing pump station southeast of the planned reservoir and along the west side of the reservoir footprint near the properly line. The shoring could have a retained height of up to 30 ft. GRI can provide more detailed design and construction criteria for practical types of top-down shoring once detailed grading plans become available.

Structural Fill

As currently planned, backfill will be placed to within about 20 and 10 ft of the top of the reservoir on the north and south sides, respectively. It is anticipated the backfill will consist of soil and/or decomposed basalt removed from excavations made during construction. With the exception of the tank backfill, no other significant fills are planned.

Excluding the surface strippings, excavation spoils approved by the geotechnical engineer may be used to backfill the reservoir. However, the fine-grained and decomposed basalt excavation spoils will be sensitive to moisture content and can only be placed and compacted during dry weather. Our

investigation indicates the natural moisture content of the excavated materials will typically be in the range from 35 to 50%. In this regard, we anticipate the excavation spoils will require significant moisture conditioning and frequent field evaluations to confirm the material is being adequately compacted. If wet conditions prevent proper moisture conditioning of the excavation spoils, material used to construct structural backfills should consist of relatively clean, granular materials, such as sand, sandy gravel, or crushed rock. The maximum particle size of granular material placed against structures should be limited to not more than 1½ in. in diameter unless approved by the designer. A drainage blanket should be placed between common backfill and the side of embedded structures as discussed in the Lateral and Vertical Earth Pressures section of this report.

The structural backfill should be placed in horizontal lifts and compacted to at least 95% of the maximum dry density as determined by ASTM D 698 (standard Proctor). Fill placed within 5 ft of the reservoir should be compacted to 93 to 95% of the maximum dry density as determined by ASTM D 698 (standard Proctor) with small, light-weight compactors to avoid overcompaction and prevent the development of excessive lateral pressures. Appropriate lift thickness will depend on the type of compaction equipment used and the type of material being placed. For hand-operated or small compactors, we recommend a maximum loose lift thickness of 8 in. For moderate- to heavy-weight compactors, we recommend a maximum loose lift thickness of 12 in.

Finished fill slopes can be slightly overbuilt and then trimmed back to final grade using a trackhoe with a smooth-edge bucket. A qualified geotechnical engineer should review the proposed placement of any fill and evaluate the subgrade prior to fill placement. The proposed compaction equipment should be reviewed by the design team prior to fill placement to evaluate loads on embedded walls.

Landscape fill should be compacted to at least 90% of the maximum dry density as determined by ASTM D 698. The moisture content of soils placed in landscaped areas is generally not critical, provided construction equipment can effectively handle the material. Landscape fill should be no steeper than 3H:1V.

Foundation Support, Settlement, and Subdrainage

Based on information provided by Peterson Structural Engineers (PSE), the new reservoir foundation will consist of a 24-in.-thick, reinforced mat slab. In our opinion, a mat slab is a suitable foundation system for accommodating potential deformations that may occur as a result of the design-level seismic event. The reservoir was preliminary designed to consist of a 9-in.-thick roof slab supported by a 24-in.-diameter, reinforced concrete interior columns placed on a 20.5-ft center-to-center spacing that are cast directly into the mat slab (i.e., no spread footings on the top of the mat slab). The 12-in.-thick reservoir wall will also be cast directly into the mat slab. The maximum service (unfactored) loads are 90 kips for columns and 5.1 kips/ft for the wall, which do not include the weight of the water. Based on the information provided by PSE, static real soil bearing pressures will be in the range of about 2,000 to 2,500 beneath most of the mat slab increasing to a maximum of about 3,100 psf at the outer edge of the slab. Maximum seismic real soil bearing pressures will be about 4,100 psf at the outer edge of the slab.

To provide adequate support for the mat slab and assumed loading, we recommend the mat slab be underlain by a minimum 3-ft thickness of compacted crushed rock placed directly over the RAPs. The minimum 18-in.-thick working blanket placed for support of the RAP installation equipment can be

considered part of the required base course section. However, it should be expected that the upper portion of the working blanket will be contaminated with soil and need to be removed. The amount of removal should be evaluated by the geotechnical engineer following RAP construction. Following removal, we recommend placing a subgrade geotextile prior to placing of remaining general granular base course and/or the assumed 2-ft-thick granular drainage layer discussed below.

General granular base course placed beneath the reservoir, including the RAP working blanket up to the bottom of the drainage layer, should consist of well-graded crushed rock with a maximum particle size of up to 1 1/2-in. meeting the requirements for Dense-Graded Aggregate as specified in Section 02630.10 of the Oregon Department of Transportation (ODOT) 2008 Standard Specifications for Highway Construction. The well-graded crushed rock should only be placed on firm, undisturbed subgrade that has been evaluated by a qualified geotechnical engineer. Soft or otherwise unsuitable materials that are identified at subgrade elevation should be overexcavated and replaced with granular structural fill. Other types of general granular material proposed by the contractor may be used with the approval of the design team. Materials used to construct drainage blankets should consist of open-graded, angular crushed rock with a maximum size of up to 1 1/2 in., with not more than about 2% passing the No. 200 sieve (washed analysis). Crushed rock of 3/4- to 1 1/2-in. gradation (drain rock) is commonly available and is suitable for this purpose. Open-graded rock (drain rock) placed on silty soil (where present) should be separated by a non-woven geotextile, such as Mirafi 140N or similar. All crushed rock placed beneath the reservoir should be compacted as structural fill using vibratory compaction equipment. The relative density of the well-graded compacted crushed rock should be at least 95% of the maximum dry density as determined by ASTM D 698 (standard Proctor). To protect the native subgrade soil, the initial lift of crushed rock base should be at least 12 in. thick. The drain rock cannot be density tested, but should be compacted until well keyed. The base course section (general granular base course plus drainage layer) should extend horizontally at least one-half the total thickness of the crushed rock section beyond the limits of the perimeter footing, or 1 1/2 ft for a 3-ft thickness of crushed rock.

RAP systems are typically designed by the RAP contractor to meet performance criteria developed by the reservoir designer. Based on similar reservoir projects with similar subsurface conditions, we recommend RAPs installed to to a minimum elevation of 405 ft below the reservoir be designed to limit total settlements (static condition) of the reservoir to about 3/4 to 1 1/4 in. when full of water and about one-half to two-thirds this amount near the edge of the reservoir, depending somewhat on the amount of fill placed on the sides of the reservoir. Further, we recommend designing the RAPs to limit differential settlements occurring between the edges of footings to a point on the floor slab halfway between any adjacent footings to a range of about 1/4 to 1/2 in. We do not anticipate significant deformations will occur in the RAP-treated zone following the design-level earthquake.

For a subgrade prepared as discussed above and with the RAP-treated zone beneath the reservoir, we anticipate the mat slab for the reservoir can be designed to impose an allowable soil bearing pressure of up to 3,500 psf to limit settlements to the range of values discussed previously. We recommend the 3,500 psf allowable bearing pressure be used as performance criteria for the RAPs. This value applies to the total of dead load plus frequently and/or permanently applied live loads and can be increased by one-third for the total of all loads; dead, live, and wind or seismic. The allowable bearing pressure(s) and estimated settlements will need to be verified during design by the RAP designer

To address the actual deformation of the floor slab, we recommend analyzing the floor slab as a plate on an elastic foundation using a coefficient of subgrade reaction, k, of 75 and 125 pci for long-term and short-term loading conditions. The RAP designer should confirm the ground improvement system meets the coefficient of subgrade values. These values assume the floor slab will be underlain by the aforementioned base course section above the RAP zone.

As discussed previously, the sides of the reservoir will be backfilled. Figure 2 indicates the backfill will extend up to about elevation 440 and 450 ft (13 to 23 ft thick) on the north and south side of the reservoir, respectively. We estimate these fills could induce up to 3/4 to 1 in. of settlement around the perimeter of the reservoir and should occur relatively quickly as the fill is placed. In our opinion, placement of the fill around the reservoir will not induce significant downdrag loads on the walls of the reservoir or settlement under the edge of the reservoir, assuming RAPs are installed beyond the edge of the reservoir as discussed previously.

Lateral loads (seismic, soil, etc.) can be resisted partially or completely by frictional forces developed between the base of the mat foundation and underlying crushed rock. The total frictional resistance between the mat slab and the underlying material is the normal force times the coefficient of friction between the crushed rock and the base of the reservoir. We recommend a value of 0.45 for the coefficient of friction between mass concrete cast directly on angular, granular structural fill. If a synthetic membrane, such as HDPE, is placed between the concrete and the underlying crushed rock, we recommend using a coefficient of friction of 0.30. If additional lateral resistance is required, passive earth pressures against embedded foundations and the reservoir walls can be computed on the basis of an equivalent fluid having a unit weight of 225 pcf for limiting lateral deflections to 1/4 to 1/2 in. and 300 pcf for larger deflections. These design passive earth pressures values would be applicable only if the backfill for the foundations or walls is placed as compacted structural fill where the backfill is horizontal. In areas where the backfill is sloped downward at 2H:1V these values should be reduced to about half. The coefficient of friction values provided above are also applicable for the frictional interaction of backfill soils against walls.

Although a French drain will be installed behind (south) and beneath the center of the reservoir to maintain groundwater levels below the bottom of the reservoir, we recommend installing a subdrainage layer beneath the floor slab of the new reservoir. The subdrainage layer will provide drainage in the event of leakage through the reservoir floor and minimize the risk of hydrostatic pressures from groundwater rise if the French drain becomes blocked or otherwise nonfunctional. We anticipate the reservoir will be underlain by a minimum 2-ft-thick layer of aforementioned open-graded crushed rock (drain rock) that will include 6-in.-diameter PVC drain pipes installed radially from the center of the reservoir in the lower part of the drainage layer outward to collection pipes at the perimeter of the reservoir. We recommend the radial drain pipes be spaced no greater than about 40 ft apart at the perimeter of the reservoir. The subdrainage section can be considered part of the recommended minimum 3-ft thickness of compacted crushed rock base course beneath the reservoir. The top 2 to 3 in. of the open-graded rock can be substituted with relatively clean 3/4-in.-minus crushed rock to facilitate leveling and placement of concrete.

French Drain

As discussed in the Slope Stability section of this report, a French drain will be installed to manage groundwater levels beneath the reservoir. The recommended location of the French drain is shown on

Figure 2. We recommend the drain consist of a minimum 2-ft-wide trench backfilled with open-graded crushed rock (drain rock) and a minimum 6-in.-diameter perforated drain pipe installed in the crushed rock near the bottom of the trench. Crushed rock with a gradation of 1/4- to 3/4-in. or 3/4- to 1 1/2-in., and containing less than 2% passing the No. 200 sieve (washed analysis) is commonly used for this purpose. The drain rock should be completely enveloped in a non-woven filter fabric, such as Mirafi 140N or equivalent. To minimize the risk of clogging, the drain pipe should not be wrapped in a filter fabric "sock".

To intercept groundwater flowing toward the reservoir from the south, we recommend installing a French drain to a depth of about elevation 416 to 417 ft below existing grades in an east-west orientation along the south side of the reservoir at the base of the temporary excavation slope as shown on Figure 2. The drain should be constructed and the trench fully backfilled in relatively short segments, on the order of 20 to 25 ft long, to minimize the risk of instability in the excavation cut slope to the south. To manage groundwater beneath the reservoir, a French drain should extend along a north-south alignment through the center of the reservoir footprint to about the north edge of the RAP treated area. Water collected in the perforated drain should be hard-piped to discharge into the stormwater system downslope to the north. A concrete dam should be constructed in the lower 3 to 4 ft of the trench around the pipe to prevent water collected in the drain rock from flowing into the slope. The backfill north of the RAP treatment area above the pipe zone should consist of either well-graded crushed rock or on-site fine-grained soil compacted to at least 92% of ASTM D 698. To avoid damage to the drain from RAP construction we recommend the drains be constructed following RAP installation.

Lateral Earth Pressures for Reservoir and Vaults

As discussed previously, the walls of the reservoir will be backfilled to within about 20 and 10 ft of the top of the reservoir on the north and south sides, respectively. In addition, a valve vault embedded about 10 ft below site grades will also be constructed northeast of the reservoir to service the new reservoir. Drainage will be provided on the sides and bottom of the reservoir to limit the risk of hydrostatic conditions from developing. We anticipate drainage will also be provided around valve vault. Lateral earth pressure and drainage recommendations for design of the reservoir and vault are provided below.

Design lateral earth pressures on embedded walls depend on the backfill geometry, drainage condition behind the wall, and the ability of the wall to yield by either translation or rotation away from the backfill. The two possible conditions regarding the ability of a wall to yield include the at-rest and the active earth pressure cases. The at-rest earth pressure case is applicable to a wall that is considered to be relatively rigid and unable to yield. The active earth pressure case is applicable to a wall that is capable of yielding slightly away from the backfill by either sliding or rotating about its base. A conventional cantilevered retaining wall is an example of a wall that develops the active earth pressure case by yielding. The walls of the new reservoir and valve vault will be braced at the top and bottom by the roof and floor and should be considered to be non-yielding. Yielding and non-yielding walls can be designed on the basis of a hydrostatic pressure based on an equivalent fluid having a unit weight of 35 and 55 pcf, respectively. In addition, it is assumed the backfill is fully drained and the surface of backfill is flat behind the wall.

We recommend using a distribution of 15 pcf to account for seismic earth pressures, with the resultant applied at 1/3H from the base of the structure, where H is the overall height of the soil retained. The

seismic pressure should be added to the static earth pressures. Horizontal pressures due to surcharge loads, such as wheel loads associated with traffic on the backfill behind the walls, can be estimated using the guidelines provided on Figure 10. Transient surcharge loads, such as wheel loads, do not need to be included in the seismic loading case.

The backfill behind embedded walls must be fully drained for use of the aforementioned equivalent fluid values. The drainage system should consist of a minimum 2-ft-wide zone of free-draining granular fill adjacent to the embedded walls. The granular material used for the drainage layer behind embedded walls should conform to our previous recommendations for free-draining structural fill material. A 4- to 6-in.-diameter, rigid, perforated drain pipe should be provided near the bottom of the embedded wall. A non-woven geotextile, such as Mirafi 140N (or similar), is recommended between the free-draining backfill and the general wall backfill to reduce the risk of contamination of the wall drain system. Recommendations regarding placement of backfill behind embedded walls are provided in the Structural Fill section of this report.

Utilities

As discussed in the Project Description portion of this report, numerous new water and drain lines will be constructed as part of the project. We anticipate the depth of trenches for installation of the piping will generally be about 4 to 6 ft below the finished ground surface except at connections to the new reservoir. Also, the drain line from the French drain north of the reservoir could be as deep as 15 to 20 ft locally. Depending on the time of year, groundwater seepage could be encountered in utility excavations, which could create the potential for running soil conditions and unstable trench sidewalls. All excavation sidewalls should be properly sloped or shored to conform to applicable local, state, or federal regulations. Some overexcavation of the trench bottom may also be necessary to permit installation of stabilization/drainage material if wet ground conditions are encountered. To provide a relatively dry working base and facilitate dewatering, a drainage/stabilization layer consisting of a 12- to 18-in. thickness of open-graded crushed rock (drain rock) containing less than 2% passing the No. 200 sieve (washed analysis) may be appropriate. However, the need for a stabilization layer should be evaluated based on actual conditions. We anticipate that seepage, where encountered, can be controlled by pumping from sumps in the trench excavation.

Utility trenches beneath or near pavement, the reservoir foundation, sidewalks, slabs, other structures, should be backfilled with well-graded crushed rock with a maximum particle size of up to 1 1/2-in. and meeting the requirements for Dense-Graded Aggregate as specified in Section 02630.10 of the ODOT 2008 Standard Specifications for Highway Construction. The crushed rock backfill should be compacted to at least 95% of the maximum dry density as determined by ASTM D 698 in the upper 4 ft of the trench and at least 92% of this density below this depth. The use of trackhoe-mounted vibratory plate compactors is usually most efficient for compaction of trench backfill. Lift thicknesses should be evaluated on the basis of field density tests; however, particular care should be taken when operating hoe-mounted compactors to prevent damage to the newly placed utilities. Flooding or jetting to compact the trench backfill should not be permitted.

Due to slope stability considerations, the backfill placed in utility trenches on the sloping ground north of the reservoir should be compacted to at least 92% maximum dry density as determined by ASTM D 698. In addition, it would also be prudent to install a 4-in.-diameter perforated drain pipe in the granular pipe

bedding to collect any groundwater that may be intercepted during wet conditions. The perforated drain pipes should be discharged into a stormwater system and not discharge directly onto the slope.

Utility pipes should be underlain by a minimum 6-in. thickness of good-quality bedding material. We recommend the bedding material and any pipe zone backfill consist of relatively clean, granular material such as 3/4- or 1-in.-minus crushed rock. Material conforming to ODOT specifications for dense-graded aggregate would be suitable for this purpose. The bottom of the excavation should be thoroughly cleaned to remove loose materials before installing the bedding material.

Design Review and Construction Services

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI 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. In addition, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, we are of the opinion that all construction operations dealing with earthwork and foundations should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in this report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

LIMITATIONS

This report has been prepared to aid the project team in the planning and design of this project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of the proposed reservoir.

The conclusions and recommendations submitted in this report are based on the data obtained from the explorations made at the locations indicated on Figure 2 and from other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil and rock conditions may exist between exploration locations. This report does not reflect any variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions different from those encountered in the explorations are observed or encountered, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Submitted for GRI,

Renews 12/2016

Renews 1/2016

A. Wesley Spang, PhD, PE, GE
Principal

Keith S. Martin, PE, GE
Senior Engineer

George Freitag, CEG
Associate

References

Beeson, M.H., Fecht, K.R., Reidel, S.P., Tolan, T.L., August 1985, Regional correlations within the Frenchman Springs Member of the Columbia River Basalt Group - new insights into the middle Miocene tectonics of northwestern Oregon: Oregon Geology, vol. 47, no. 8.

Burns, S., Growney, L., Brodersen, B., Yeats, R. S., and Popowski, T. A., 1997, Map showing faults, bedrock geology, and sediment thickness of the western half of the Oregon City 1:100,000 quadrangle, Washington, Multnomah, and Marion Counties, Oregon: Oregon Department of Geology and Mineral Industries Interpretive Map Series IMS-4.

Cornforth Consultants, Inc., December 2, 2014, (Draft) Geotechnical engineering services, Bolton reservoir seismic landslide evaluation, West Linn, Oregon; prepared for Murray, Smith & Associates, Inc.

L. R. Squier Associates, Inc., 1988, Geologic Reconnaissance, Property Downslope of 2.5-MG Skyline (Bolton) Reservoir, West Linn, Oregon; prepared for City of West Linn (missing page 5)

Landslide Technology (LT), June 19, 1998, Revised report, phases I and II geotechnical investigation, Bolton Reservoir landslide, West Linn, Oregon; prepared for Montgomery Watson.

Madin, I. P, 2009, Geologic map of the Oregon City 7.5' quadrangle, Clackamas County, Oregon: Oregon Department of Geology and Mineral Industries GMS-119.

Northwest Testing Laboratories, October 11, 1972, Soils and foundation investigation, city of West Linn, c/o John Cunningham and Associates, City Reservoir, West Linn, Oregon; prepared for City of West Linn.

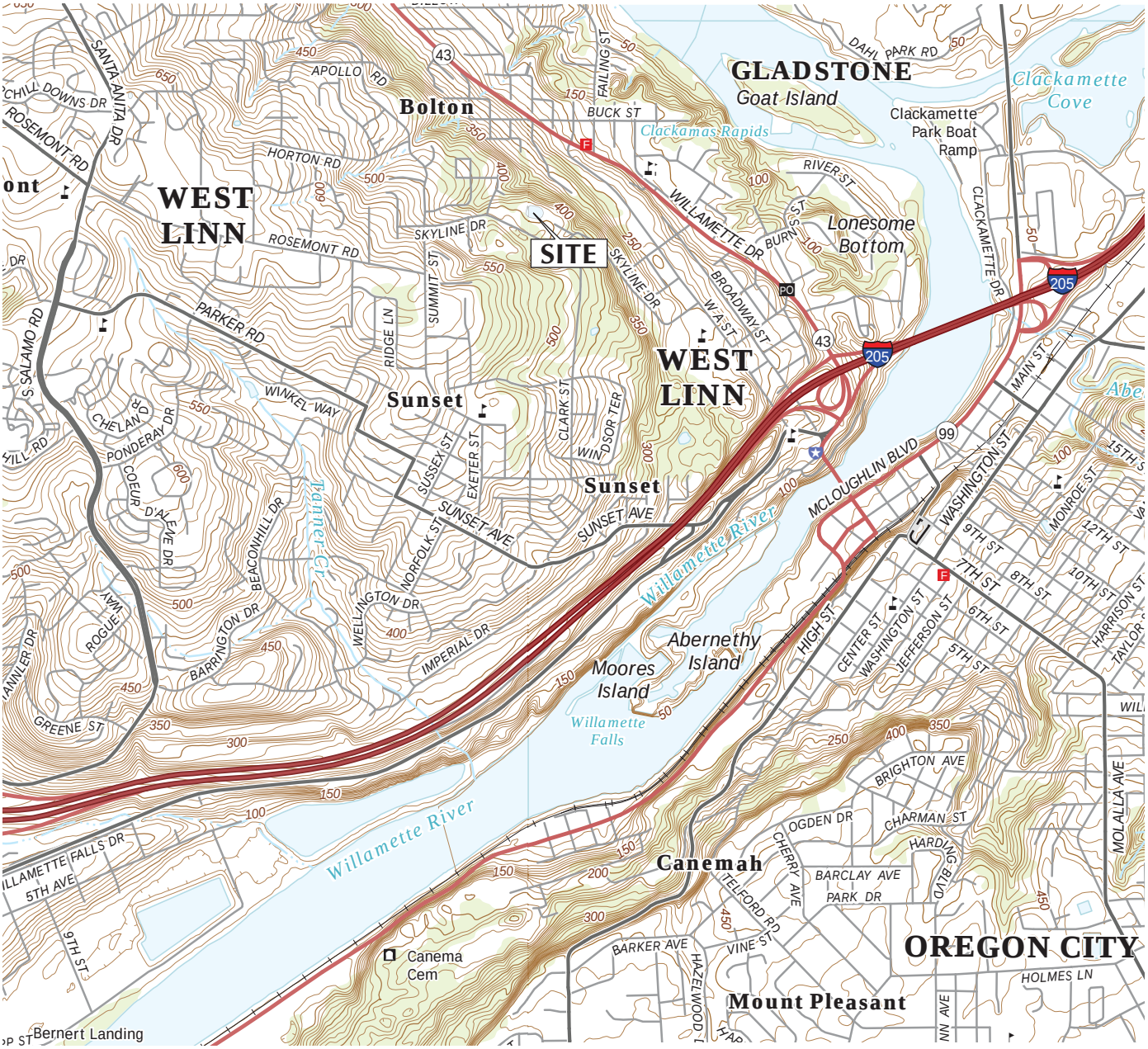
Oregon Department of Geology and Mineral Industries (DOGAMI), September 29, 2009, News release regarding GMS 119.

Personius, S.F., compiler, 2002, Fault numbers 874, 875, 877, in Quaternary fault and fold database of the United States: U.S. Geological Survey, website http://earthquakes.usgs.gov/regional/qfaults, accessed August 3, 2012.

Petersen, M. D., Bryant, W. A, Cramer, C. H., Tiaquing, C., Reichle, M. S., Frankel, A. D, , Lienkaemper, J. J., McCrory, P. A., and Schwartz, D. P., 1996, Probabilistic seismic hazard assessment for the state of California: U.S. Geological Survey Open-File Report 96-706

Schlicker, H.G., and Finlayson, C.T., 1979, Geology and geologic hazards of northwestern Clackamas County, Oregon: Oregon Department of Geology and Mineral Industries Bulletin 99, 79 p.

Yeats, R.S., Graven, E.P., Werner, K.S., Goldfinger, C., and Popowski, T., 1991, Tectonics of the Willamette Valley, Oregon: U.S. Geological Survey Open-File Report 91-441-P, 47 p.



USGS TOPOGRAPHIC MAP
OREGON CITY, OREG. (2014)

North

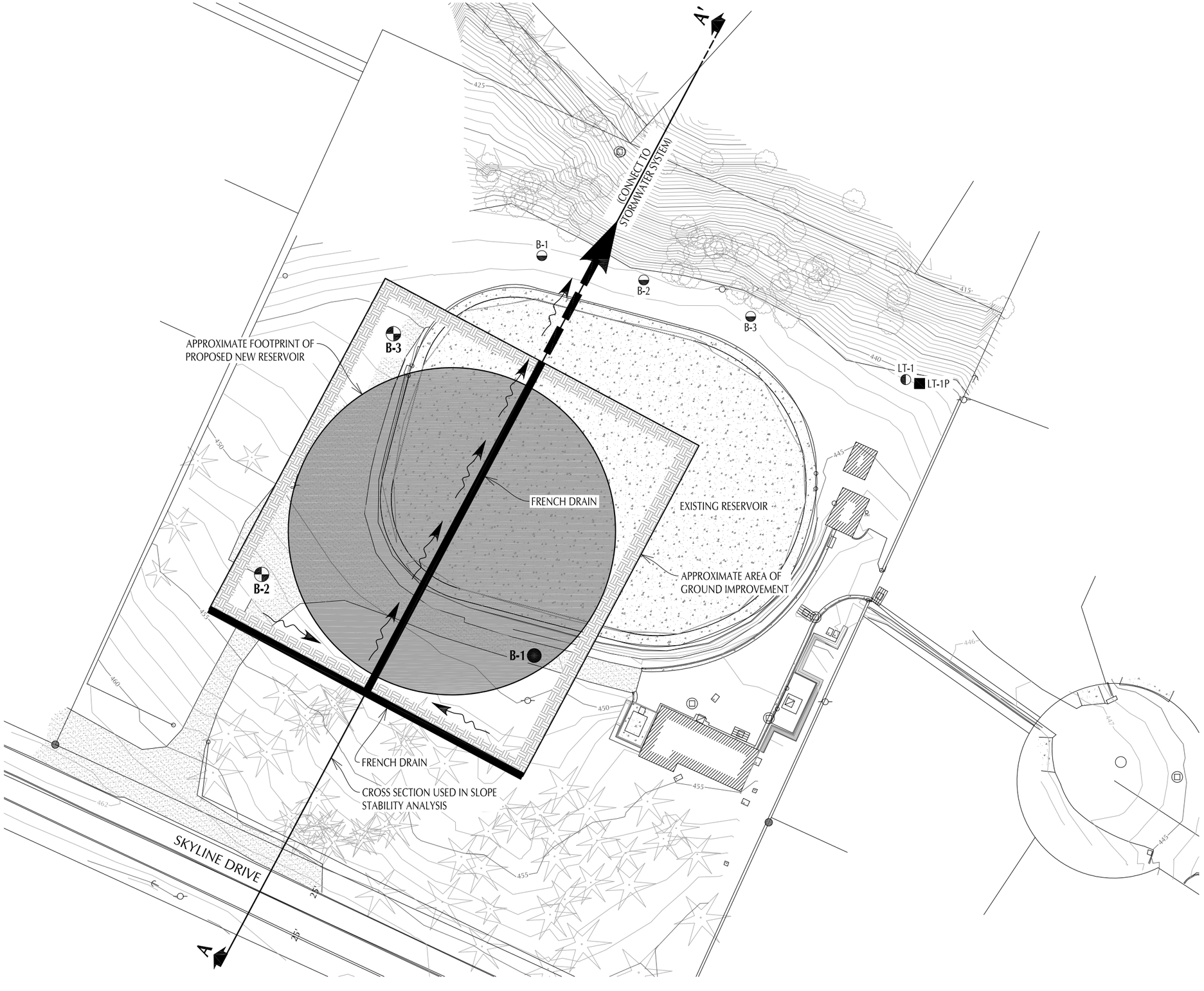
0 1/2 1 MILE

GRI

MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

VICINITY MAP

AUG. 2015 JOB NO. 5338-A FIG. 1



BORING MADE BY GRI (NOVEMBER 27 - 29, 2014)

BORING MADE BY GRI (JUNE 15, 2012)

BORING AND INCLINOMETER MADE / INSTALLED BY LANDSLIDE TECHNOLOGY (1997)

STANDPIPE INSTALLED BY LANDSLIDE TECHNOLOGY (1997)

BORING MADE BY NORTHWEST TESTING LABORATORIES (1972)

ELEVATION DATUM NAVD 88

SITE PLAN FROM FILE BY MURRAY, SMITH & ASSOCIATES, INC.

North

0 50 100 FT

GRI

MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

SITE PLAN

AUG. 2015 JOB NO. 5338-A FIG. 2

SITE MAP FROM AERIAL PHOTO BY BING IMAGE (UNDATED)

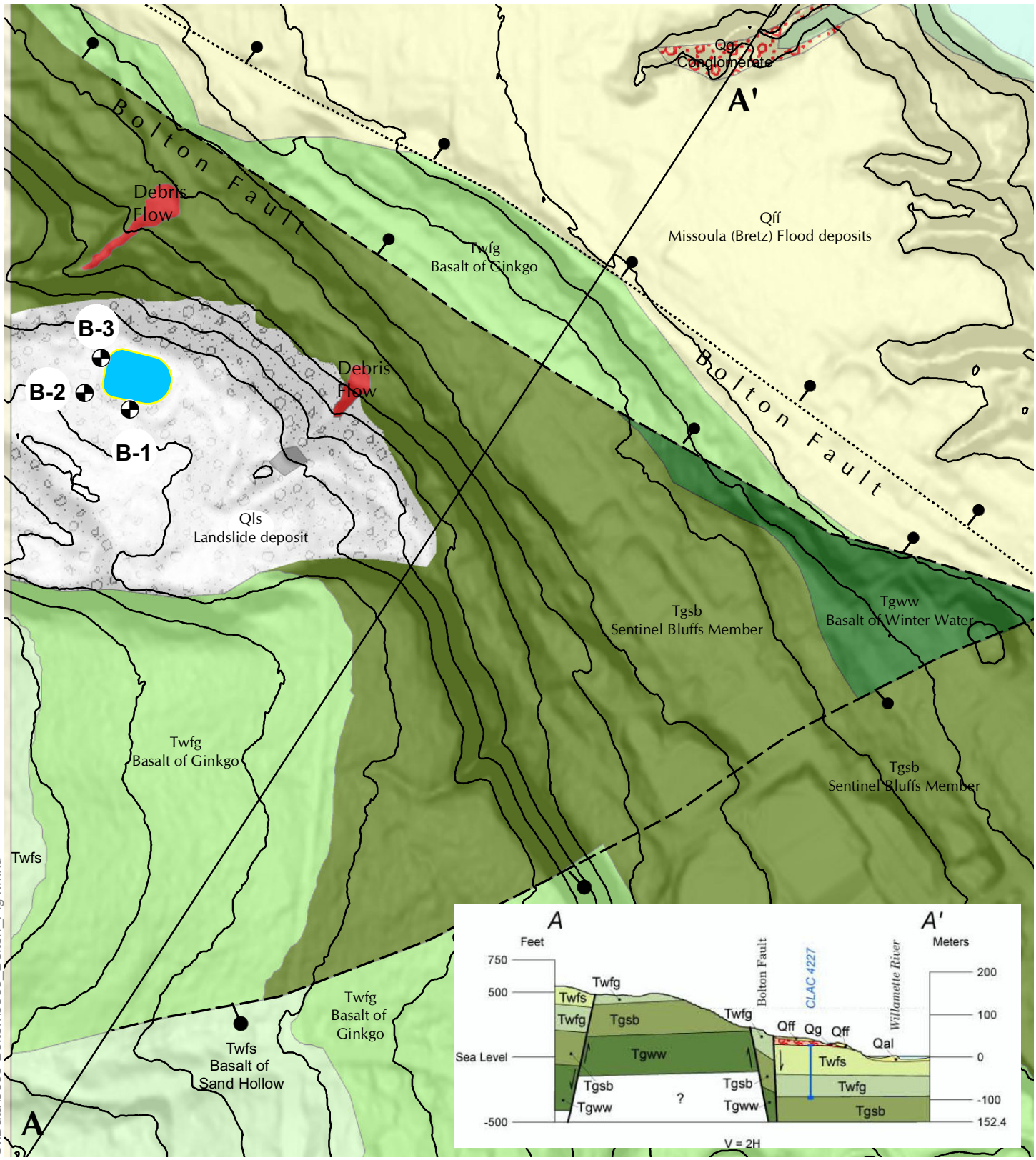
North

0 250 500 FT

GRI MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

SITE MAP

AUG. 2015 JOB NO. 5338-A FIG. 3



C:\Data\5338-Bolton\5338_Bolton_Fig4.mxd

Source Madin (2009)

Borings by GRI (2012, 2014)

Bolton Reservoir

North

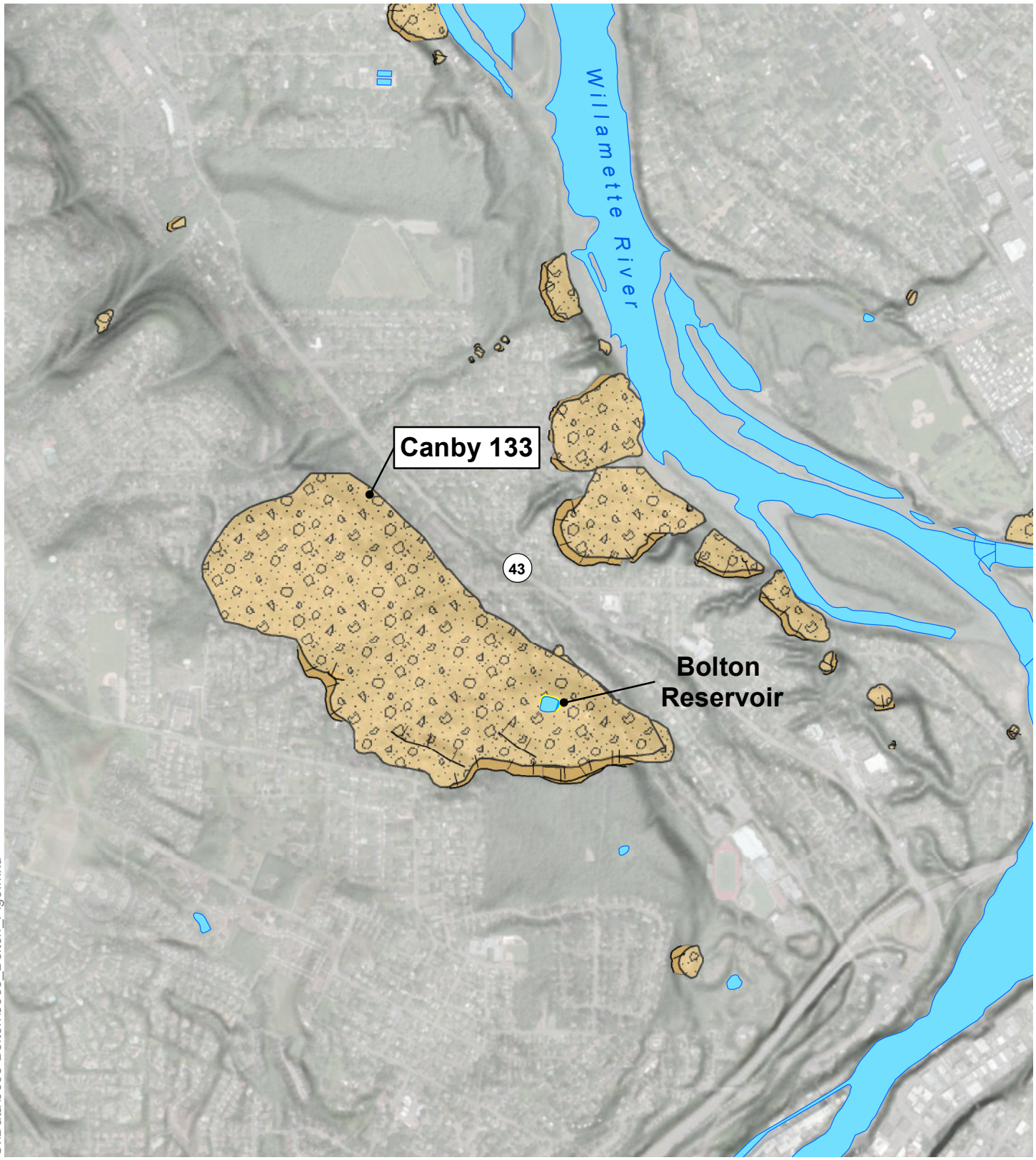
0 300 600 Ft

GRI

MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

GEOLOGIC MAP AND CROSS SECTION

AUG. 2015 JOB NO. 5338-A FIG.4



C:\Data\5338-Bolton\5338_Bolton_Fig5.mxd

Bolton Reservoir
Landslide Headscarp
Landslide Deposit
Landslide Scarp Flank

North

0 1,000 2,000 Ft

GRI

MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

STATEWIDE LANDSLIDE INFORMATION DATABASE OF OREGON VERSION 3 (SLIDO 3.2) 2014

AUG. 2015 JOB NO. 5338-A FIG.5

Material Properties:

Crushed Rock Base Course
Total Unit Wt. = 130 pcf
phi' = 40 degrees
c' = 0

Structural Backfill
Total Unit Wt. = 120 pcf
phi' = 32 degrees
c' = 0

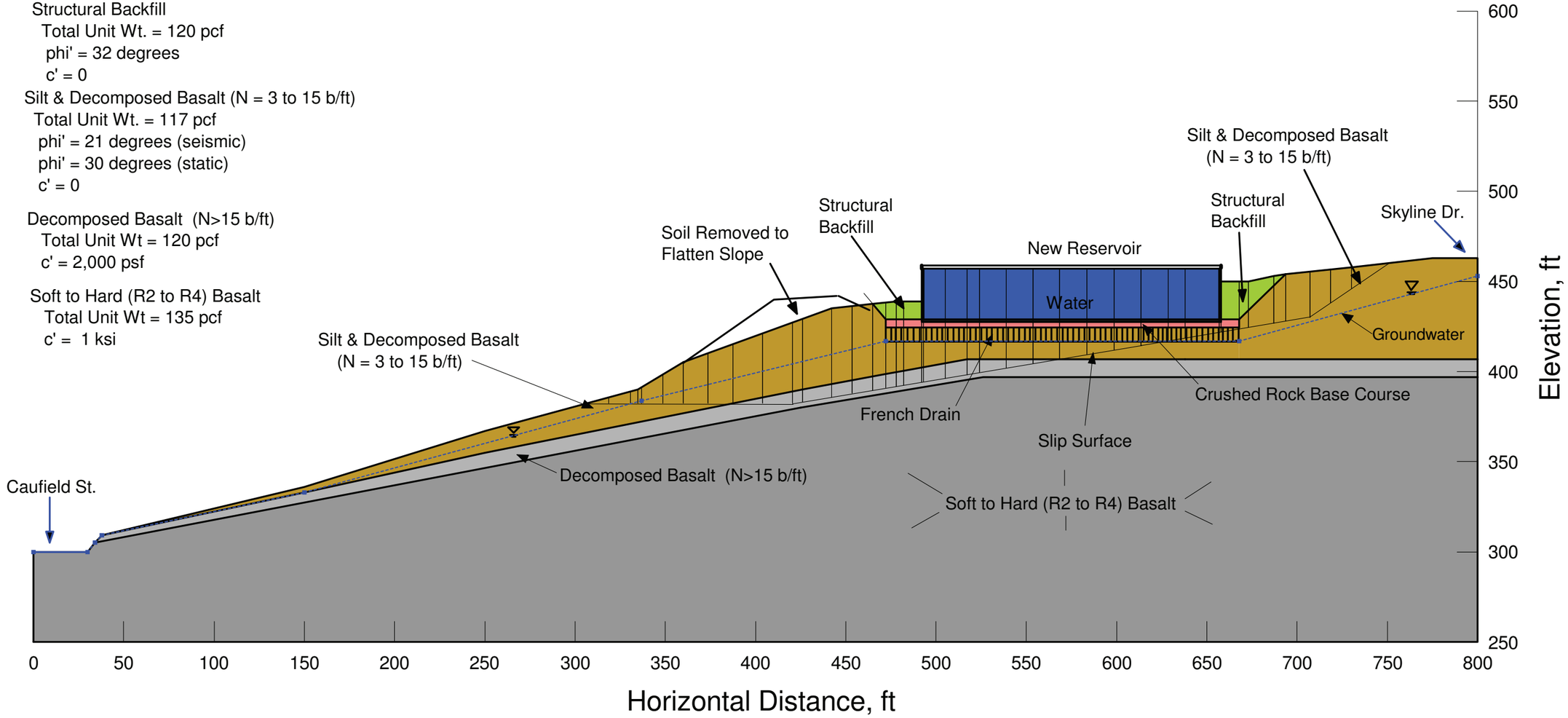
Silt & Decomposed Basalt (N = 3 to 15 b/ft)
Total Unit Wt. = 117 pcf
phi' = 21 degrees (seismic)
phi' = 30 degrees (static)
c' = 0

Decomposed Basalt (N>15 b/ft)
Total Unit Wt = 120 pcf
c' = 2,000 psf

Soft to Hard (R2 to R4) Basalt
Total Unit Wt = 135 pcf
c' = 1 ksi

Minimum Factor of Safety Values:
Seismic = 1.0
Static > 1.5

Seismic kh = 0.22 (1/2 PGA)



GRI MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

SLOPE STABILITY MODEL
(NEW RESERVOIR WITHOUT GROUND IMPROVEMENT)

SEP. 2015 JOB NO. 5338-A FIG. 6

Material Properties:

Crushed Rock Base Course
Total Unit Wt. = 130 pcf
phi' = 40 degrees
c' = 0

Structural Backfill
Total Unit Wt. = 120 pcf
phi' = 32 degrees
c' = 0

Aggregate Pier Zone
Total Unit Wt. = 119 pcf
phi' = 24.5 degrees
c' = 0

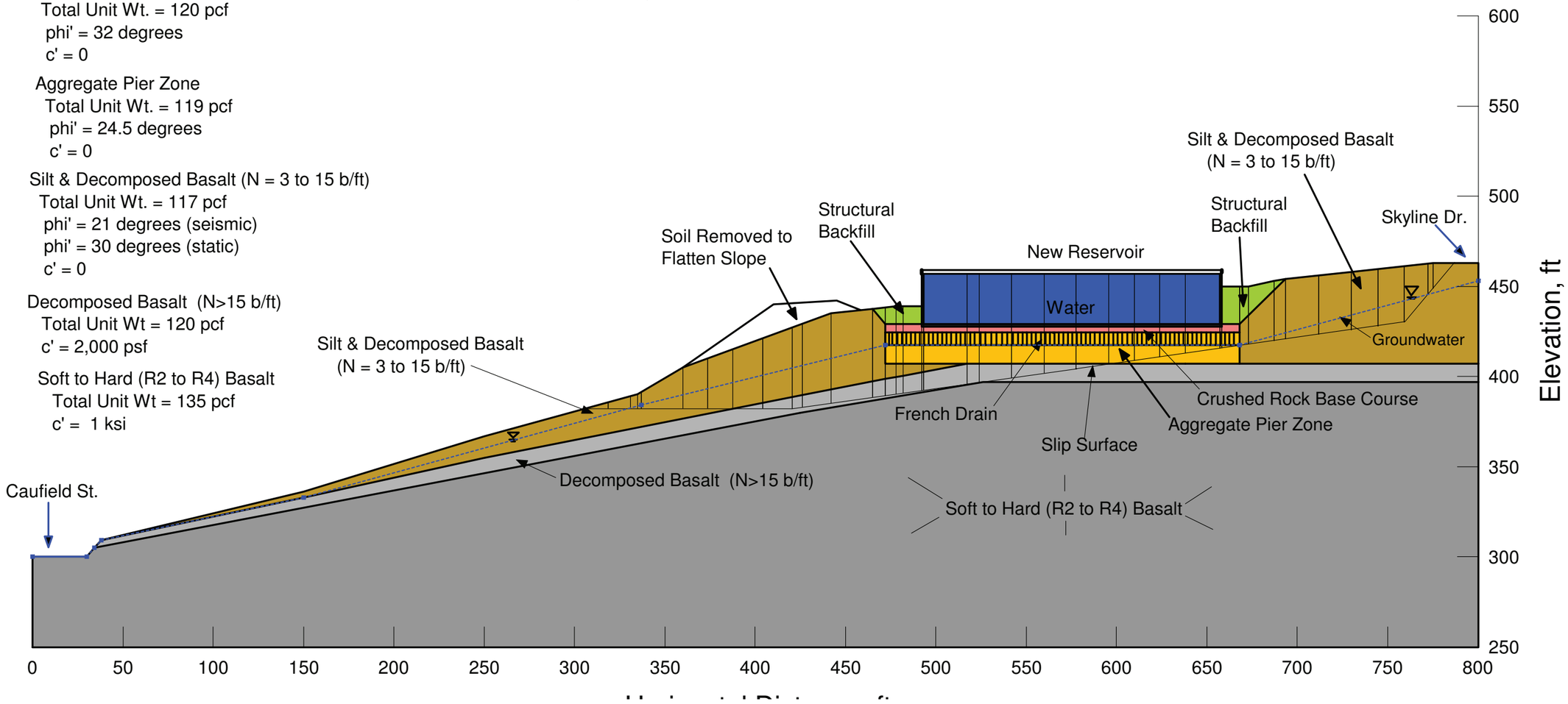
Silt & Decomposed Basalt (N = 3 to 15 b/ft)
Total Unit Wt. = 117 pcf
phi' = 21 degrees (seismic)
phi' = 30 degrees (static)
c' = 0

Decomposed Basalt (N>15 b/ft)
Total Unit Wt = 120 pcf
c' = 2,000 psf

Soft to Hard (R2 to R4) Basalt
Total Unit Wt = 135 pcf
c' = 1 ksi

Minimum Factor of Safety Values:
Seismic = 1.05
Static > 1.5

Seismic kh = 0.22 (1/2 PGA)



GRI MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

SLOPE STABILITY MODEL
(NEW RESERVOIR WITH GROUND IMPROVEMENT)

SEP. 2015 JOB NO. 5338-A FIG. 7

Material Properties:

Crushed Rock Base Course
Total Unit Wt. = 130 pcf
phi' = 40 degrees
c' = 0

Structural Backfill
Total Unit Wt. = 120 pcf
phi' = 32 degrees
c' = 0

Silt & Decomposed Basalt (N = 3 to 15 b/ft)
Total Unit Wt. = 117 pcf
phi' = 21 degrees (seismic)
phi' = 24.5 degrees (static)
c' = 0

Aggregate Pier Zone
Total Unit Wt. = 119 pcf
phi' = 24.5 degrees
c' = 0

Decomposed Basalt (N>15 b/ft)
Total Unit Wt = 120 pcf
c' = 2,000 psf

Soft to Hard (R2 to R4) Basalt
Total Unit Wt = 135 pcf
c' = 1 ksi

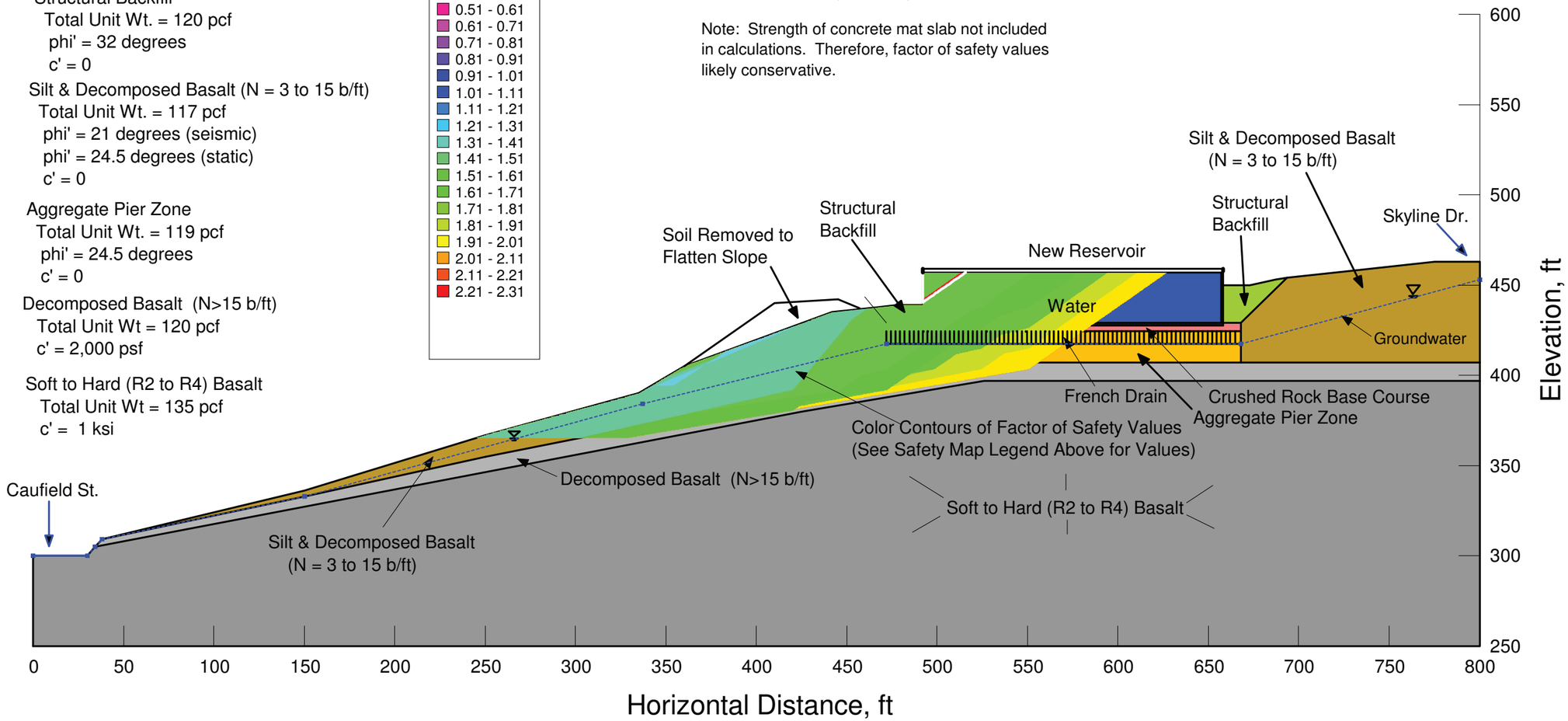
Static Factor of Safety Map

- 0.31 - 0.41
- 0.41 - 0.51
- 0.51 - 0.61
- 0.61 - 0.71
- 0.71 - 0.81
- 0.81 - 0.91
- 0.91 - 1.01
- 1.01 - 1.11
- 1.11 - 1.21
- 1.21 - 1.31
- 1.31 - 1.41
- 1.41 - 1.51
- 1.51 - 1.61
- 1.61 - 1.71
- 1.71 - 1.81
- 1.81 - 1.91
- 1.91 - 2.01
- 2.01 - 2.11
- 2.11 - 2.21
- 2.21 - 2.31

Minimum Factor of Safety Values:
Seismic => 1.1
Static => 1.5

Seismic kh = 0.22 (1/2 PGA)

Note: Strength of concrete mat slab not included in calculations. Therefore, factor of safety values likely conservative.



GRI MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

SLOPE STABILITY MODEL

(NEW RESERVOIR WITH GROUND IMPROVEMENT, NORTH SLOPE)

SEP. 2015 JOB NO. 5338-A FIG. 8

Material Properties:

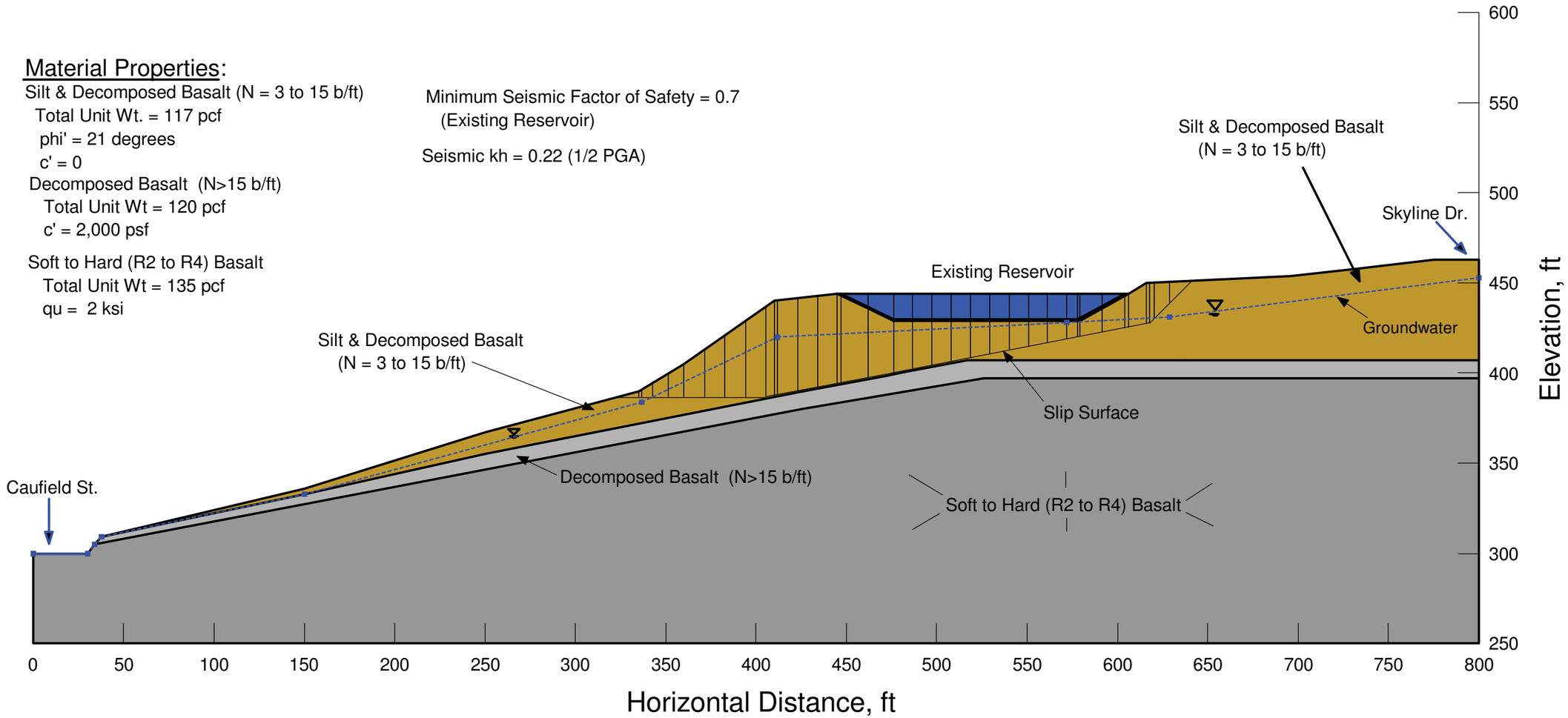
Silt & Decomposed Basalt (N = 3 to 15 b/ft)
Total Unit Wt. = 117 pcf
phi' = 21 degrees
c' = 0

Decomposed Basalt (N>15 b/ft)
Total Unit Wt = 120 pcf
c' = 2,000 psf

Soft to Hard (R2 to R4) Basalt
Total Unit Wt = 135 pcf
qu = 2 ksi

Minimum Seismic Factor of Safety = 0.7
(Existing Reservoir)

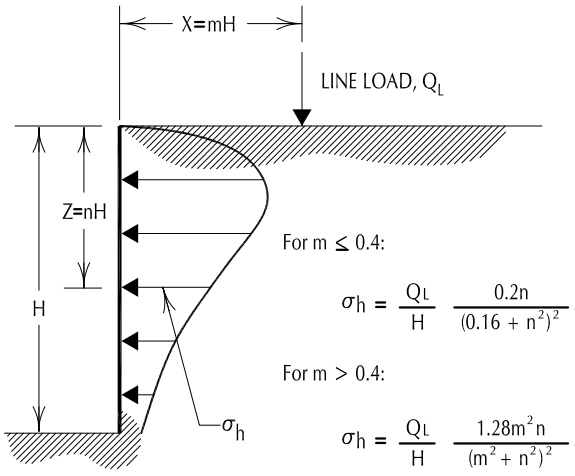
Seismic kh = 0.22 (1/2 PGA)



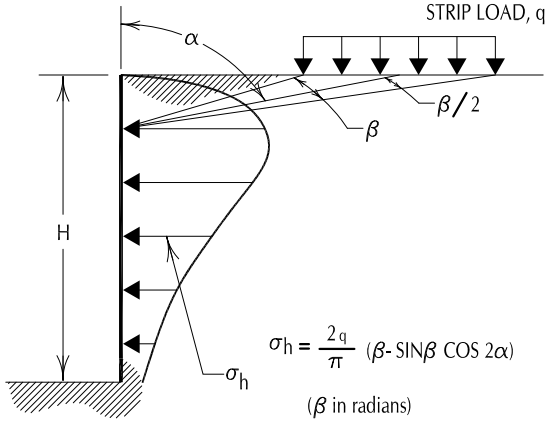
GRI MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

SLOPE STABILITY MODEL
(EXISTING RESERVOIR)

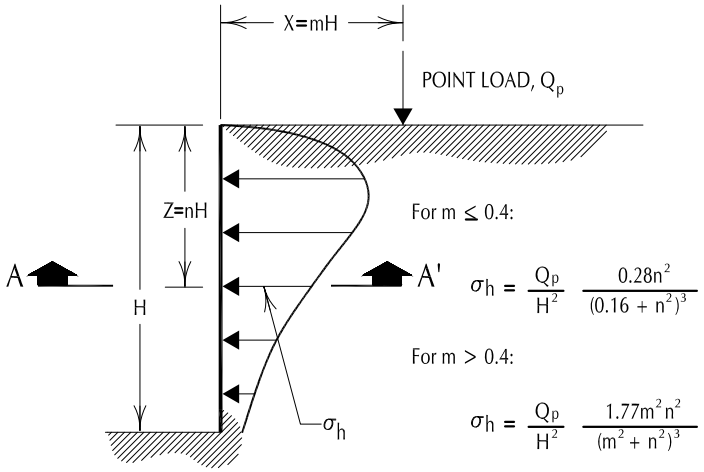
SEP. 2015 JOB NO. 5338-A FIG. 9

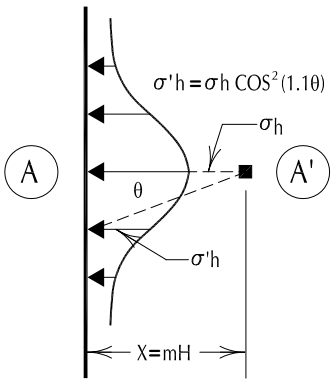


LINE LOAD PARALLEL TO WALL



STRIP LOAD PARALLEL TO WALL





DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.

2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

GRI

MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

SURCHARGE-INDUCED LATERAL PRESSURE

AUG. 2015 JOB NO. 5338-A FIG. 10

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

General

Subsurface materials and conditions at the site were evaluated by GRI on June 15, 2012, with one boring designated B-1, and on October 27 through 29, 2014, with two borings, designated B-2 and B-3. The locations of the borings are shown on Figure 2. All explorations were observed by a certified engineering geologist from GRI.

The borings were advanced to depths ranging from 76 to 90 ft with mud-rotary drilling methods using CME 75 track- and truck-mounted drill rigs provided and operated by Western States Soil Conservation, Inc., of Hubbard, Oregon. Disturbed and undisturbed samples were obtained from the borings at about 2.5-to 5-ft intervals of depth. Disturbed samples were obtained using a standard split-spoon sampler. At the time of sampling, the Standard Penetration Test was conducted. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is known as the Standard Penetration Resistance, or N-value. The N-values provide a measure of the relative density of granular soils and the relative consistency of cohesive soils. The soil and rock samples obtained in the split-spoon sampler were carefully examined in the field, and representative portions were saved in airtight jars for further examination and physical testing in our laboratory. In addition, relatively undisturbed Shelby tube samples of soil and decomposed rock were collected and returned to our laboratory for further evaluation and testing. Below a depth of about 64and 60 ft in boring B-1 and B-2, respectively, and 55 ft in boring B-3 wireline coring methods were used to obtain continuous samples of rock. The rock cores were placed in core boxes and returned to our laboratory for further evaluation.

Logs of the borings are provided on Figures 1A through 3A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depth where the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, N-values are shown graphically, along with the natural moisture contents, Torvane shear strength values, Atterberg limits, and percentage of material passing the No. 200 sieve. The terms and symbols used to describe the soil and rock encountered in the borings are defined in Tables 1A and 2A and the attached legend.

Observation Standpipe

An observation standpipe piezometer was installed in boring B-2 and B-3 to depths of about 90 and 48 ft, respectively. The standpipes consist of a 1-in.-I.D. plastic pipe slotted below a depth of 60 and 17 ft in boring B-2 and B-3, respectively. Each boring was flushed with clean water prior to installing the pipe, and the annular space around the pipe was backfilled with Colorado Sand to about 1 ft above the slotted zone. The remaining portion of the hole was backfilled with a seal consisting of bentonite. The top of the standpipe is protected with a flush-mounted monument. Groundwater enters through the slots and rises to a static level, which is measured with an electrical probe lowered inside the pipe.

LABORATORY TESTING

General

The samples obtained from the borings were examined in our laboratory, where the physical characteristics of the samples were noted, and the field classifications were modified where necessary. At the time of classification, the natural moisture content of each sample was determined. Additional tests included determinations of Torvane shear strengths, undisturbed unit weights, one-dimensional consolidation testing, washed sieve analysis, Atterberg limits, drained residual torsional shear strength, and grain-size analysis.

Natural Moisture Contents

Natural moisture content determinations were made in conformance with ASTM D 2216. The results are summarized on the Boring Logs, Figures 1A through 3A.

Torvane Shear Strength

The approximate undrained shear strength of the fine-grained soils obtained in the Shelby tubes was measured using the Torvane shear device. The Torvane is a hand-held apparatus with vanes that are inserted into the soil. The torque required to fail the soil in undrained shear around the vanes is measured using a calibrated spring. The torque measurements have been correlated to the undrained shear strength of various fine-grained soils. The results of the Torvane shear strength testing are shown on Figures 1A through 3A.

Undisturbed Unit Weight

The dry unit weight, or dry density, of undisturbed soil samples was determined in the laboratory in substantial conformance with ASTM D 2937. The unit weight determinations are summarized below.

SUMMARY OF UNIT WEIGHT DETERMINATIONS

Boring	Sample	Approximate Depth, ft	Soil Type	Moisture Content, %	Dry Unit Weight, pcf
B-1	S-2	8.2	Clayey SILT, some fine- to medium-grained sand, brown, stiff (Landslide Debris)	40	81.7
	S-5	16.2	Clayey SILT, some fine- to medium-grained sand, brown, stiff (Landslide Debris)	31	94.3
	S-10	35.7	Clayey SILT, trace sand- to gravel-size fragments of extremely soft (R0), predominantly decomposed basalt, stiff to very stiff (Landslide Debris)	37	88.0
B-2	S-4	11.3	SILT, some clay to clayey, trace to some fine-grained sand, red-brown, black manganese staining, medium stiff (Landslide Debris)	35	87.8
	S-8	21.2	Clayey SILT, trace to some fine-grained sand, brown to red-brown, stiff (Landslide Debris)	27	101.5
	S-11	31.3	BASALT, gray-brown, decomposed, extremely soft (R0), manganese oxide mineralization, relic rock structure, consistency of medium stiff soil (Wanapum Basalt; Landslide Debris)	44	80.0
	S-14	37.8	BASALT, gray-brown to red-brown, decomposed, extremely soft (R0), manganese oxide mineralization, relic rock structure, consistency of soft to hard soil (Wanapum Basalt; Landslide Debris)	43	76.0

Boring	Sample	Approximate Depth, ft	Soil Type	Moisture Content, %	Dry Unit Weight, pcf
B-2	S-16	46.8	BASALT, gray-brown to red-brown, decomposed, extremely soft (R0), manganese oxide mineralization, relic rock structure, consistency of soft to hard soil (Wanapum Basalt; Landslide Debris)	39	84.0
B-3	S-6	15.8	BASALT, gray-brown to red-brown, decomposed, extremely soft (R0), secondary mineralization, relic rock structure, consistency of soft to hard soil (Wanapum Basalt; Landslide Debris)	49	76.0
	S-10	26.0	BASALT, gray-brown to red-brown, decomposed, extremely soft (R0), secondary mineralization, relic rock structure, consistency of soft to hard soil (Wanapum Basalt; Landslide Debris)	52	68.0

One-Dimensional Consolidation Testing

Two, one-dimensional consolidation test was performed in conformance with ASTM D 2435 on relatively undisturbed samples from borings B-1 and B-2 at a depth of about 16.5 and 37.3 ft, respectively. The test provides data on the compressibility of the underlying fine-grained soils and decomposed rock, necessary for settlement studies. The test results are summarized on Figures 4A and 5A in the form of a curve showing percent strain versus applied effective stress. The initial dry unit weight and moisture content of the samples are also shown on the figures.

Washed-Sieve Analysis

Washed sieve analyses were performed using selected soil samples to assist in classification of the soils. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the sieve is oven-dried and weighed. The percentage of material passing the No. 200 sieve is then calculated. The results are tabulated below and shown on Figures 2A and 3A.

SUMMARY OF WASHED SIEVE ANALYSES

Boring	Sample	Depth, ft	Percent Passing No. 200 Sieve	Description
B-2	S-6	15.0	90	Clayey SILT, some fine-grained sand, brown to reddish-brown, stiff (Landslide Debris)
	S-7	17.5	90	Clayey SILT, some fine-grained sand, brown to reddish-brown, stiff (Landslide Debris
	S-9	22.0	85	Clayey SILT, some fine-grained sand, brown to reddish-brown, stiff (Landslide Debris)
B-3	S-4	10.0	82	Clayey SILT, some fine-grained sand, brown (Landslide Debris)

Atterberg Limits

Atterberg limits determinations were performed by GRI on representative samples in conformance with ASTM D 4318. The results of the tests completed by GRI are summarized on Figure 6A Atterberg limits testing were also performed by Cooper Testing Laboratory of Palo Alto, California, on a representative sample of decomposed basalt from a depth of 35 ft in boring B-2 that was used to perform the drained residual torsional shear strength test discussed below. The results of the Atterberg limit test by Cooper Testing Laboratory are shown on Figure 7A.

Drained Residual Torsional Shear Strength

The drained residual torsional shear strength test of a representative sample of decomposed basalt from a depth of 35 ft in boring B-2 was completed in conformance with ASTM D 6467 by Cooper Testing Laboratory. The results of the test are summarized on Figure 8A.

Grain Size Analysis

Grain size analysis was completed by Cooper Testing Laboratory of Palo Alto, California on representative sample decomposed basalt from a depth of 35 ft in boring B-2 that was used to perform the drained residual torsional shear strength test discussed above in conformance with ASTM D 422. The results of the test are shown on Figure 9A.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per foot
very loose	0 - 4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values) blows per foot	Torvane or Undrained Shear Strength, tsf
very soft	0 - 2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Grain-Size Classification	Modifier for Subclassification		
Boulders:		Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
> 12 in.	Adjective	Percentage of Other Material (by weight)	
Cobbles:			
3 - 12 in.	trace:	5 - 15 (sand, gravel)	5 - 15 (sand, gravel)
Gravel:	some:	15 - 30 (sand, gravel)	15 - 30 (sand, gravel)
1/4 - 3/4 in. (fine)	sandy, gravelly:	30 - 50 (sand, gravel)	30 - 50 (sand, gravel)
3/4 - 3 in. (coarse)			
Sand:	trace:	< 5 (silt, clay)	*Relationship of clay and silt determined by plasticity index test*
No. 200 - No. 40 sieve (fine)	some:	5 - 12 (silt, clay)	
No. 40 - No. 10 sieve (medium)	silty, clayey:	12 - 50 (silt, clay)	
No. 10 - No. 4 sieve (coarse)			
Silt/Clay:			
pass No. 200 sieve			

GRI

Table 2A: GUIDELINES FOR CLASSIFICATION OF ROCK

RELATIVE ROCK WEATHERING SCALE

Term	Field Identification
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 in. into rock.
Moderately Weathered	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Predominantly Decomposed	Rock mass is more than 50% decomposed. Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock "fabric" may be evident. May be reduced to soil with hand pressure.

RELATIVE ROCK HARDNESS SCALE

Term	Hardness Designation	Field Identification	Approximate Unconfined Compressive Strength
Extremely Soft	R0	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.	< 100 psi
Very Soft	R1	Crumbles under firm blows with point of a geology pick. Can be peeled by a pocket knife and scratched with fingernail.	100 - 1,000 psi
Soft	R2	Can be peeled by a pocket knife with difficulty. Cannot be scratched with fingernail. Shallow indentation made by firm blow of geology pick.	1,000 - 4,000 psi
Medium Hard	R3	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.	4,000 - 8,000 psi
Hard	R4	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.	8,000 - 16,000 psi
Very Hard	R5	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.	> 16,000 psi

RQD AND ROCK QUALITY

Relation of RQD and Rock Quality

RQD (Rock Quality Designation), %	Description of Rock Quality
0 - 25	Very Poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

Terminology for Planar Surface

Bedding	Joints and Fractures	Spacing
Laminated	Very Close	< 2 in.
Thin	Close	2 in. – 12 in.
Medium	Moderately Close	12 in. – 36 in.
Thick	Wide	36 in. – 10 ft
Massive	Very Wide	> 10 ft

BORING AND TEST PIT LOG LEGEND

SOIL SYMBOLS

Symbol	Typical Description
	LANDSCAPE MATERIALS
	FILL
	GRAVEL; clean to some silt, clay, and sand
	Sandy GRAVEL; clean to some silt and clay
	Silty GRAVEL; up to some clay and sand
	Clayey GRAVEL; up to some silt and sand
	SAND; clean to some silt, clay, and gravel
	Gravelly SAND; clean to some silt and clay
	Silty SAND; up to some clay and gravel
	Clayey SAND; up to some silt and gravel
	SILT; up to some clay, sand, and gravel
	Gravelly SILT; up to some clay and sand
	Sandy SILT; up to some clay and gravel
	Clayey SILT; up to some sand and gravel
	CLAY; up to some silt, sand, and gravel
	Gravelly CLAY; up to some silt and sand
	Sandy CLAY; up to some silt and gravel
	Silty CLAY; up to some sand and gravel
	PEAT

BEDROCK SYMBOLS

Symbol	Typical Description
	BASALT
	SILTSTONE
	SANDSTONE

SURFACE MATERIAL SYMBOLS

Symbol	Typical Description
	Asphaltic-concrete PAVEMENT
	Portland cement concrete PAVEMENT
	Crushed rock BASE COURSE

SAMPLER SYMBOLS

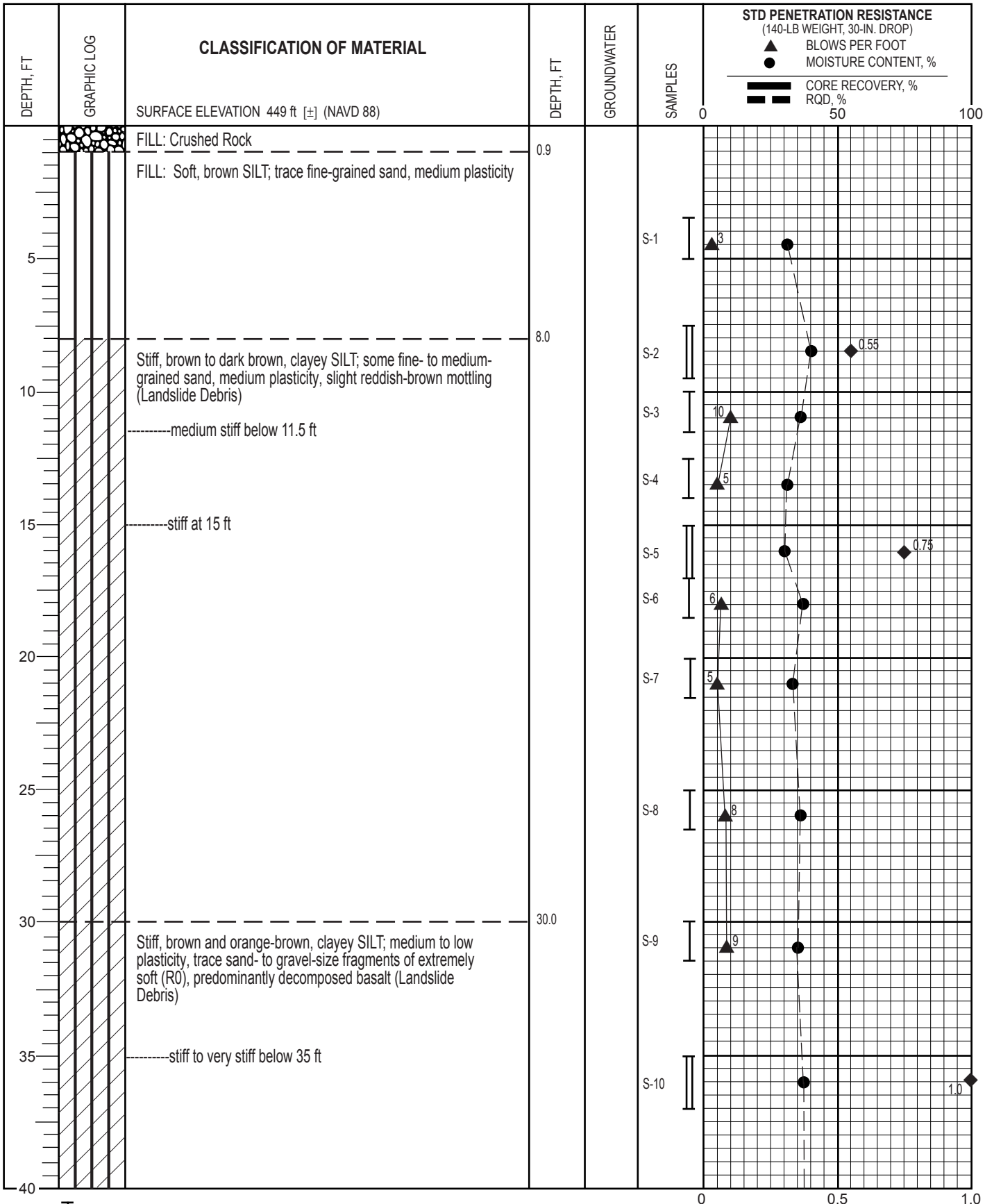
Symbol	Sampler Description
	2.0-in. O.D. split-spoon sampler and Standard Penetration Test with recovery (ASTM D1586)
	Shelby tube sampler with recovery (ASTM D1587)
	3.0-in. O.D. split-spoon sampler with recovery (ASTM D3550)
	Grab Sample
	Rock core sample interval
	Sonic core sample interval
	Geoprobe sample interval

INSTALLATION SYMBOLS

Symbol	Symbol Description
	Flush-mount monument set in concrete
	Concrete, well casing shown where applicable
	Bentonite seal, well casing shown where applicable
	Filter pack, machine-slotted well casing shown where applicable
	Grout, vibrating-wire transducer cable shown where applicable
	Vibrating-wire pressure transducer
	1-in.-diameter solid PVC
	1-in.-diameter hand-slotted PVC
	Grout, inclinometer casing shown where applicable

FIELD MEASUREMENTS

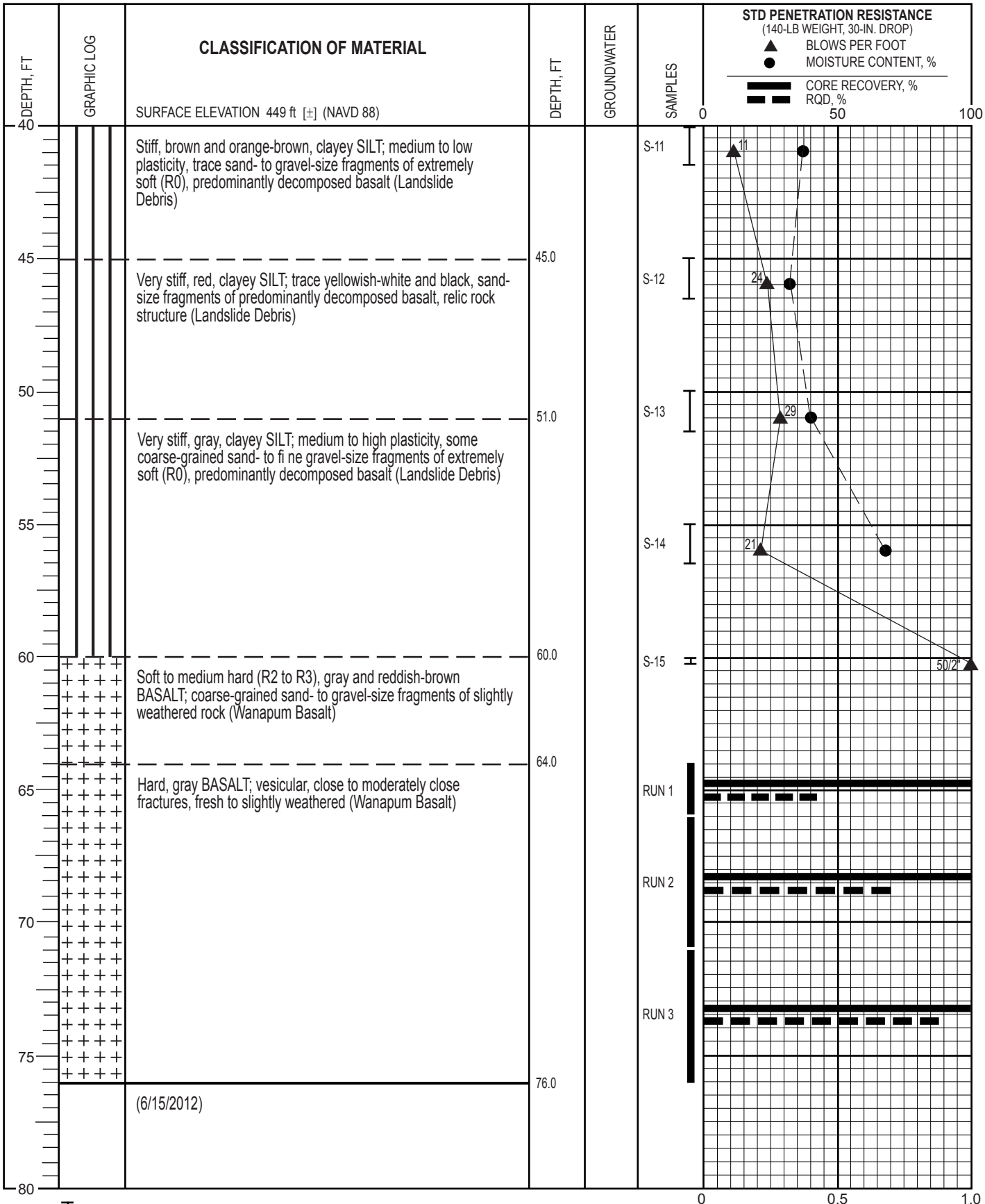
Symbol	Typical Description
	Groundwater level during drilling and date measured
	Groundwater level after drilling and date measured
	Rock core recovery
	Rock quality designation (RQD)



GRI

BORING B-1

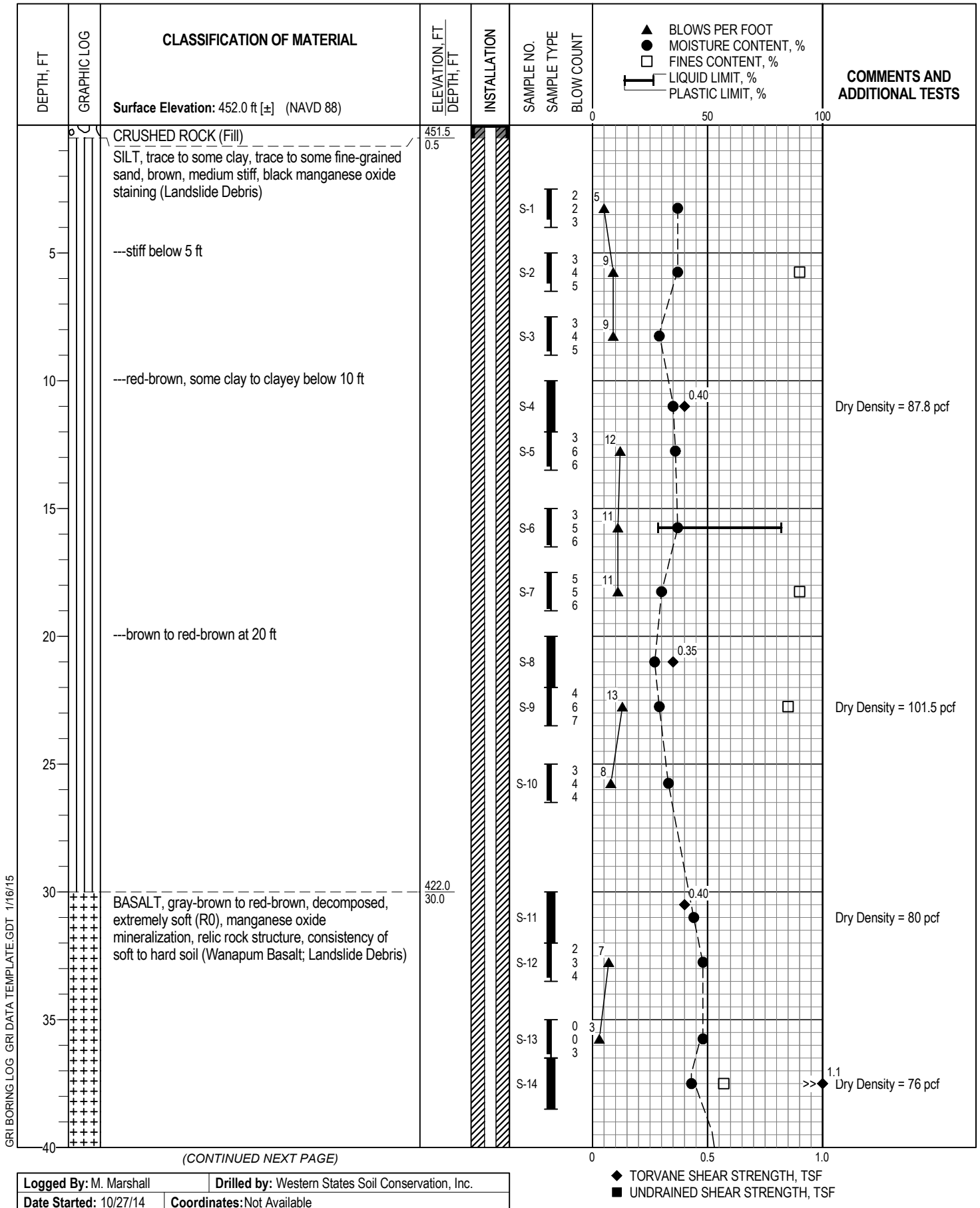
AUG. 2015 JOB. NO. 5338-A FIG. 1A



GRI

BORING B-1 (cont.)

AUG. 2015 JOB. NO. 5338-A FIG. 1A



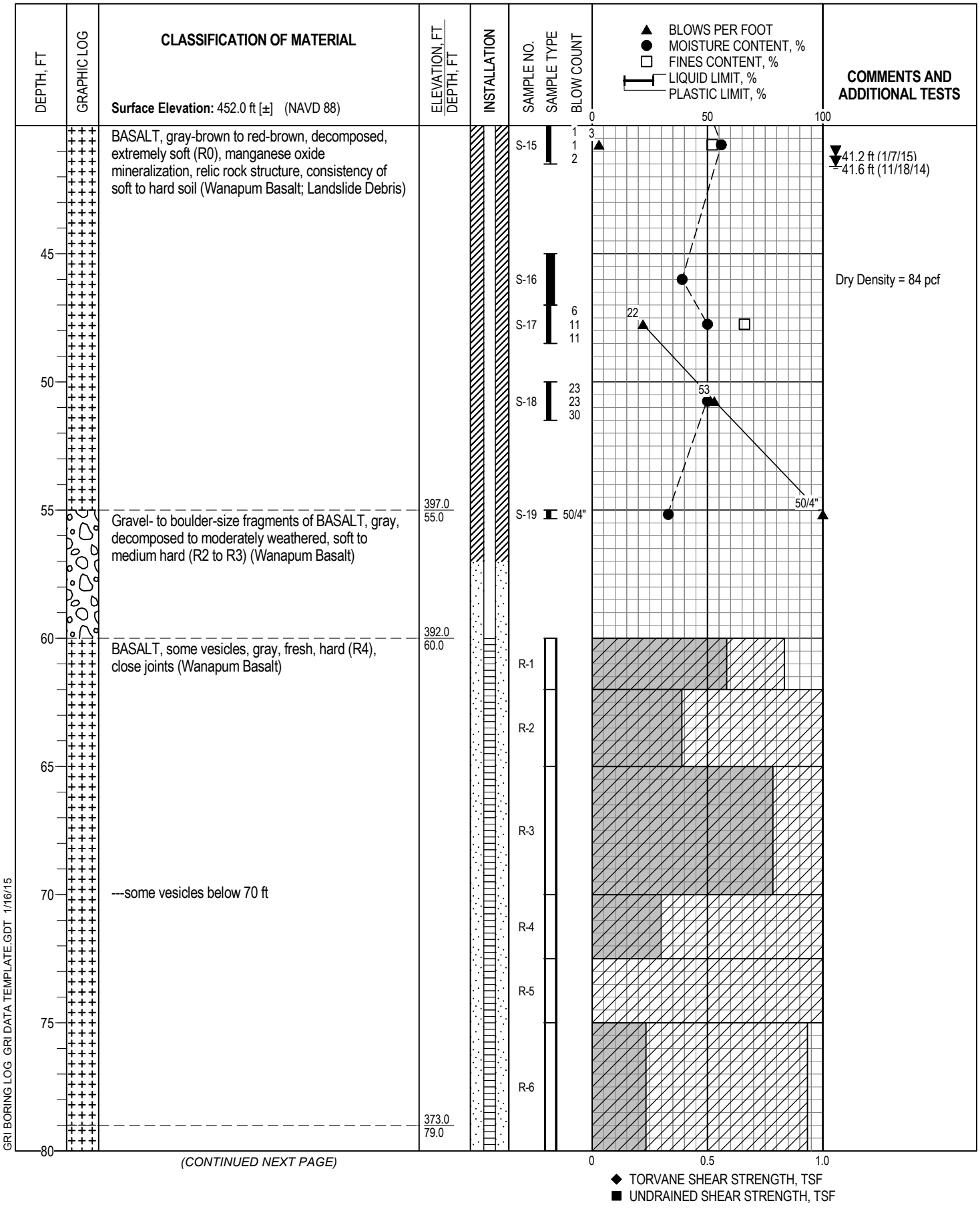
(CONTINUED NEXT PAGE)

GRI BORING LOG GRI DATA TEMPLATE.GDT 1/16/15

Logged By: M. Marshall	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 10/27/14	**Coordinates:** Not Available
Drilling Method: Mud Rotary **Equipment:** CME 75 HT Truck-Mounted Drill Rig **Hole Diameter:** 3.875 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 71.7 %
Note: See Legend for Explanation of Symbols	

BORING B-2

AUG. 2015 JOB NO. 5338-A FIG. 2A

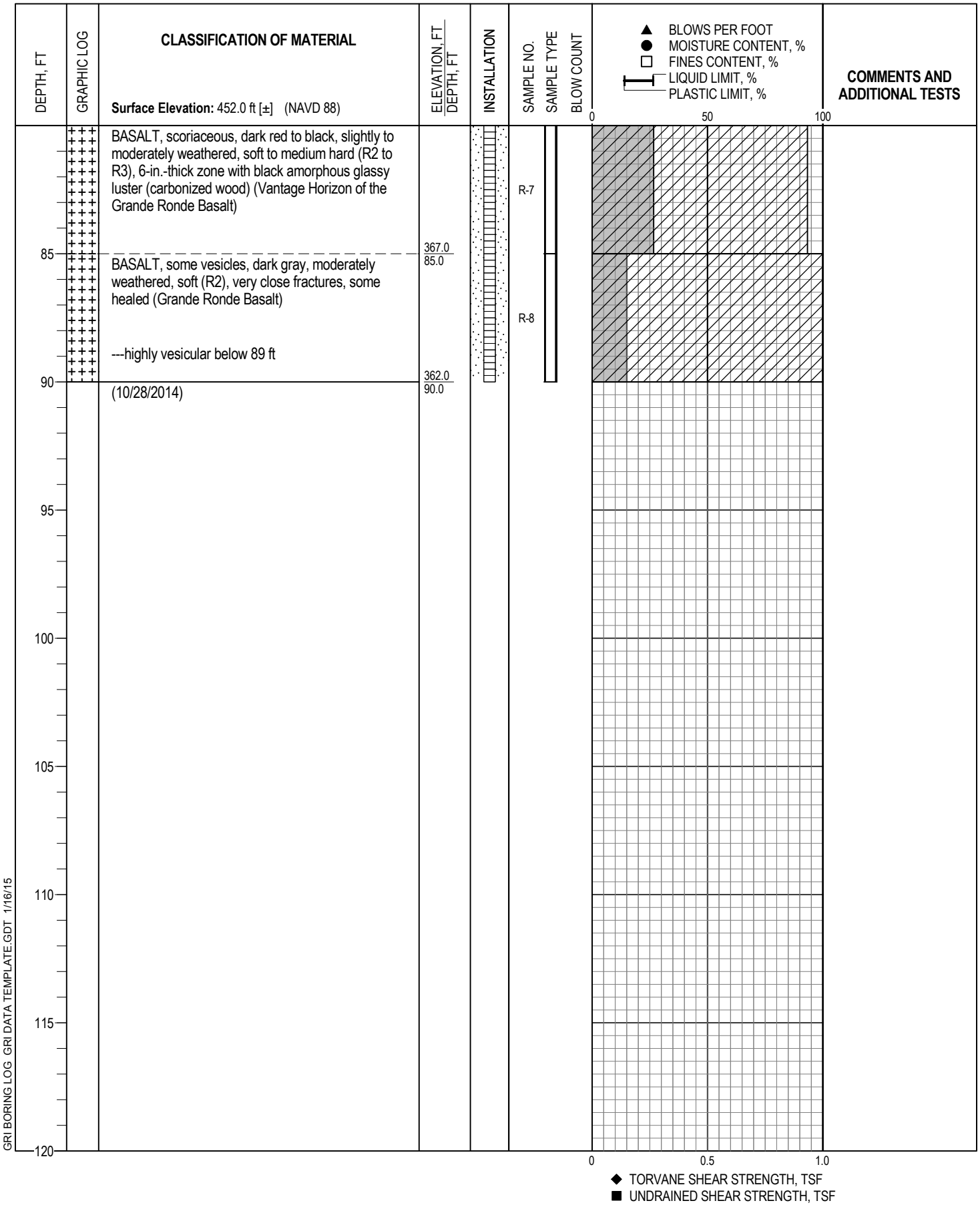


(CONTINUED NEXT PAGE)

GRI BORING LOG GRI DATA TEMPLATE.GDT 1/16/15

BORING B-2

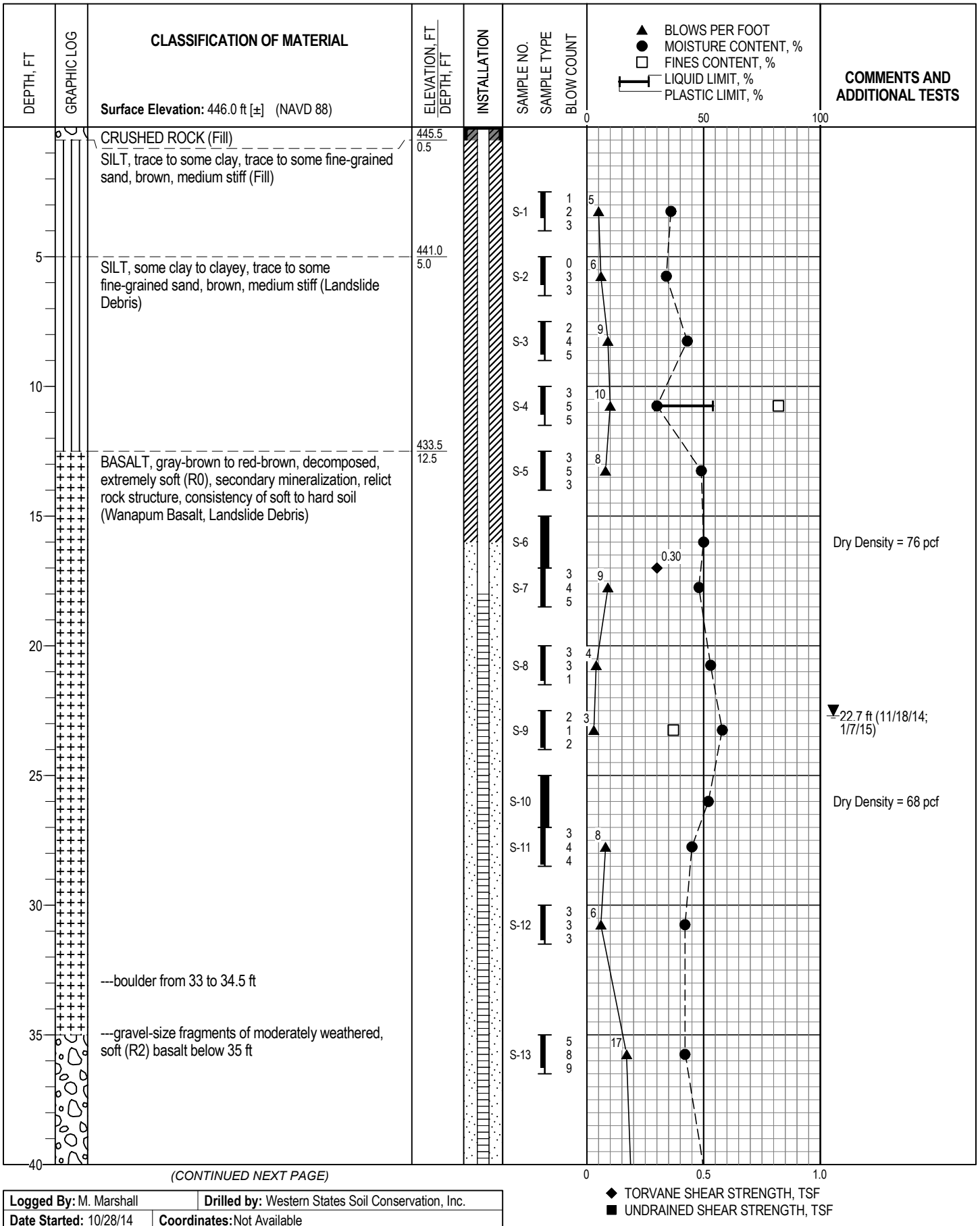
AUG. 2015 JOB NO. 5338-A FIG. 2A



GRI BORING LOG GRI DATA TEMPLATE.GDT 1/16/15

BORING B-2

AUG. 2015 JOB NO. 5338-A FIG. 2A



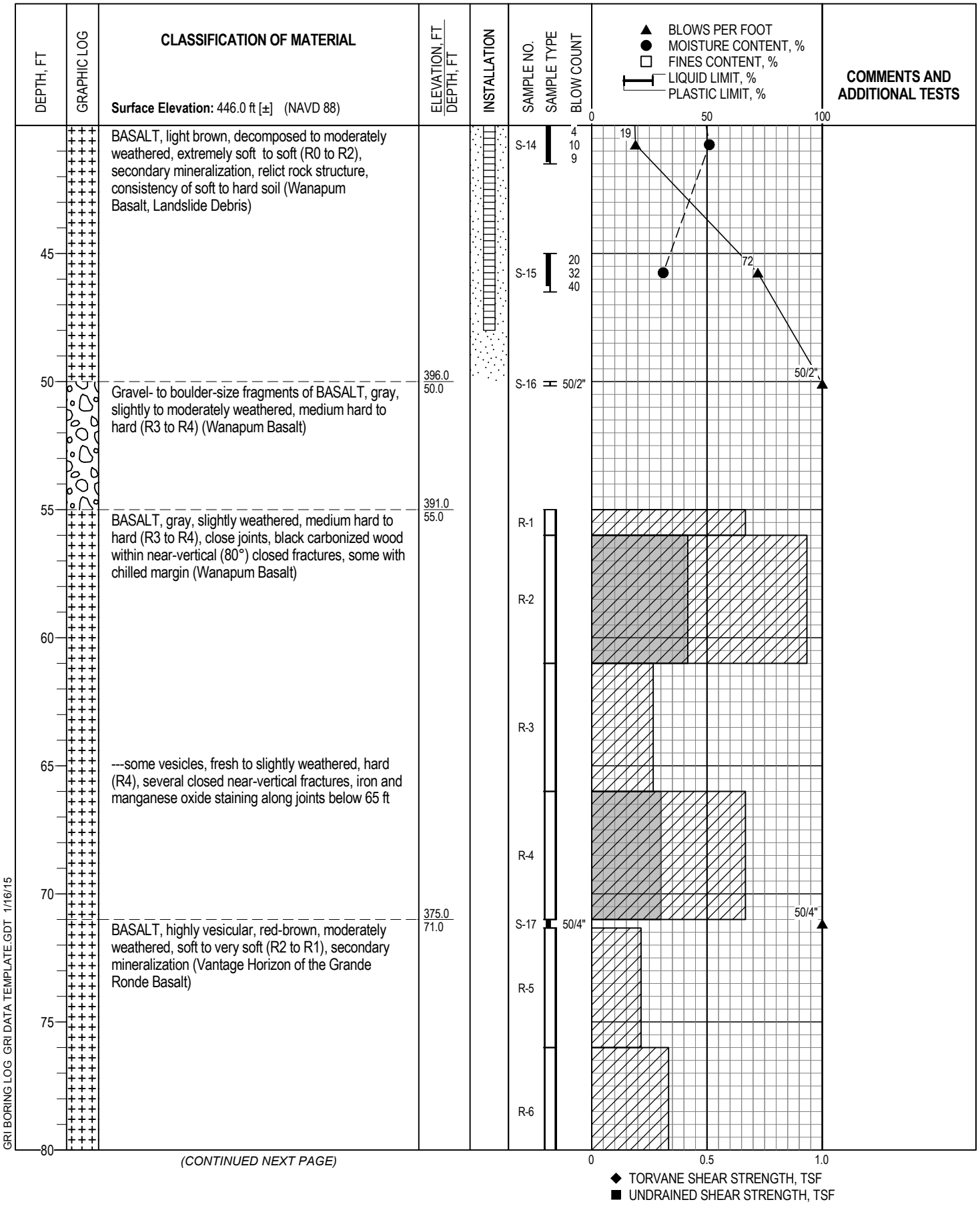
(CONTINUED NEXT PAGE)

Logged By: M. Marshall	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 10/28/14	**Coordinates:** Not Available
Drilling Method: Mud Rotary **Equipment:** CME 75 HT Truck-Mounted Drill Rig **Hole Diameter:** 3.875 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 71.7 %
Note: See Legend for Explanation of Symbols	

BORING B-3

AUG. 2015 JOB NO. 5338-A FIG. 3A

GRI BORING LOG GRI DATA TEMPLATE.GDT 1/16/15



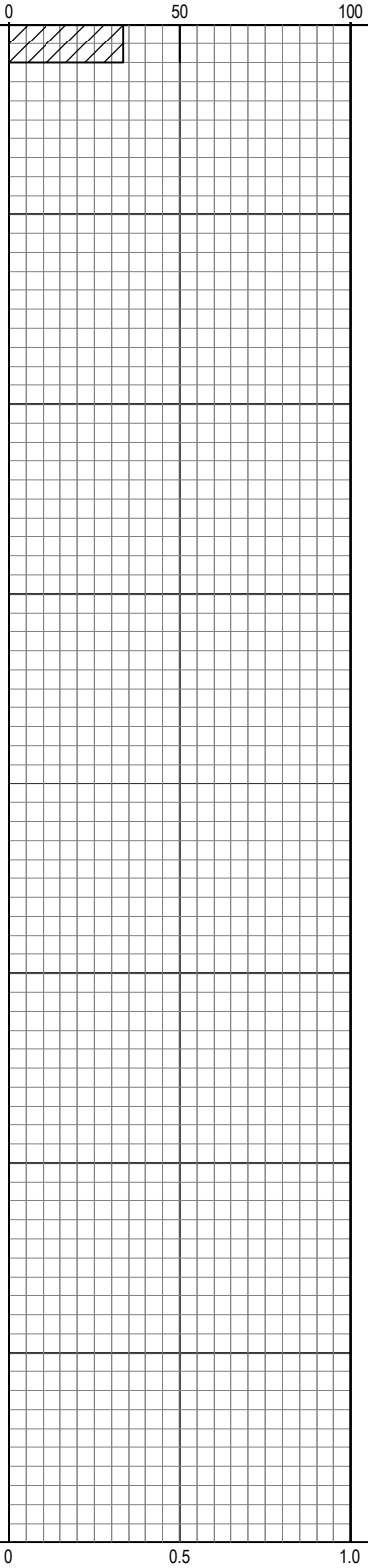
(CONTINUED NEXT PAGE)

GRI BORING LOG GRI DATA TEMPLATE.GDT 1/16/15

BORING B-3

AUG. 2015 JOB NO. 5338-A FIG. 3A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO. / SAMPLE TYPE / BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: 446.0 ft [±] (NAVD 88)				
		BASALT, highly vesicular, red-brown, moderately weathered, soft to very soft (R2 to R1), secondary mineralization (Vantage Horizon of the Grande Ronde Basalt) (10/29/2014)	365.0 / 81.0			

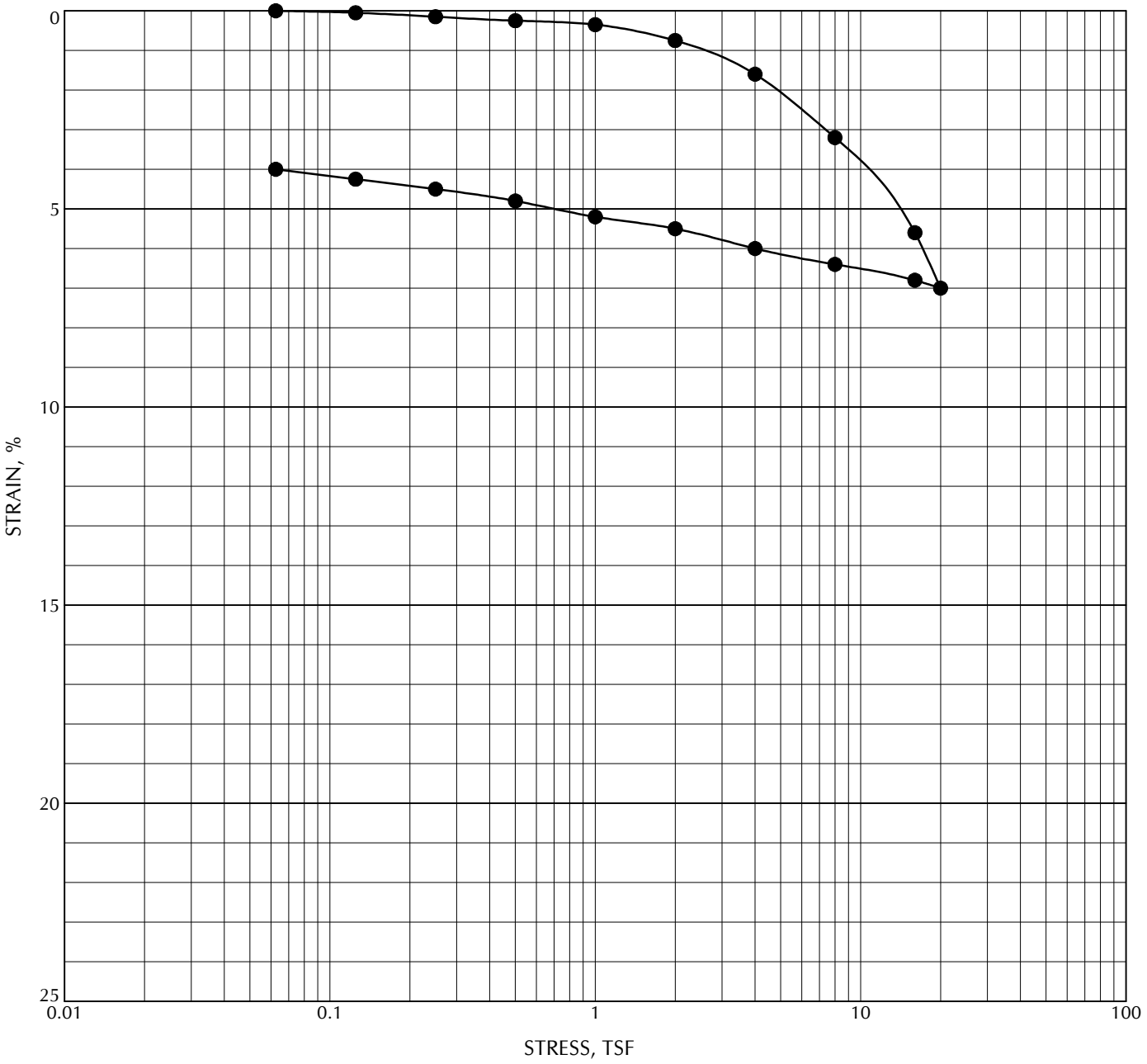


GRI BORING LOG GRI DATA TEMPLATE.GDT 1/16/15

GRI

BORING B-3

AUG. 2015 JOB NO. 5338-A FIG. 3A

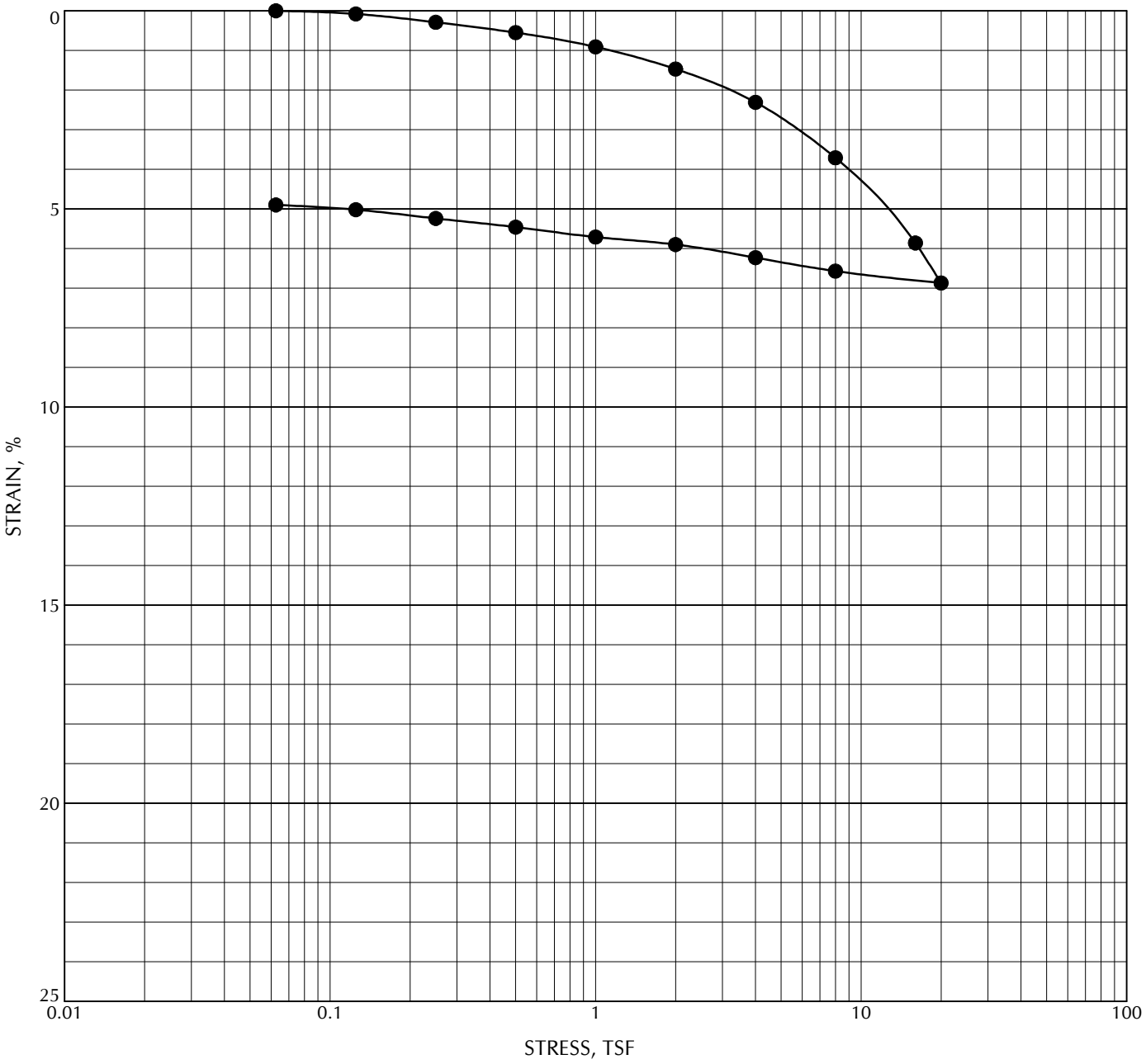


	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-1	S-5	16.5	Clayey SILT, some fine- to medium-grained sand, brown, medium stiff (Landslide Debris)	89	33

GRI

CONSOLIDATION TEST

AUG. 2015 JOB NO. 5338-A FIG. 4A



	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-2	S-14	37.3	BASALT, gray-brown to red-brown, decomposed, extremely soft (R0), manganese oxide mineralization, relic rock structure, consistency of soft to hard soil (Wanapum Basalt; Landslide Debris)	80	43

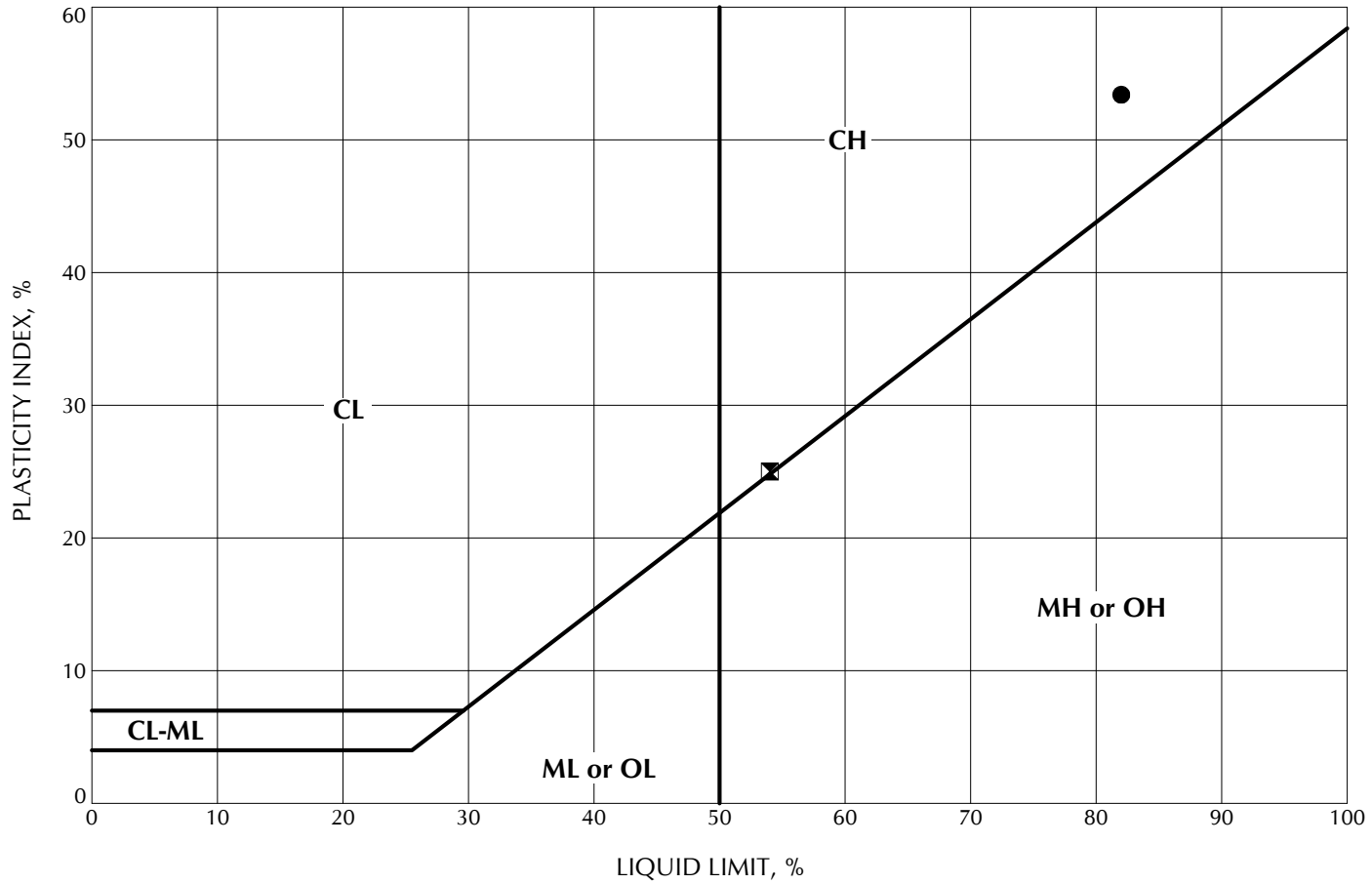
GRI

CONSOLIDATION TEST

AUG. 2015 JOB NO. 5338-A FIG. 5A

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-2	S-6	15.0	SILT, some clay to clayey, trace to some fine-grained sand, red-brown (Landslide Debris)	82	29	53	37
⊠	B-3	S-4	10.0	SILT, some clay to clayey, trace to some fine-grained sand, brown (Landslide Debris)	54	29	25	30

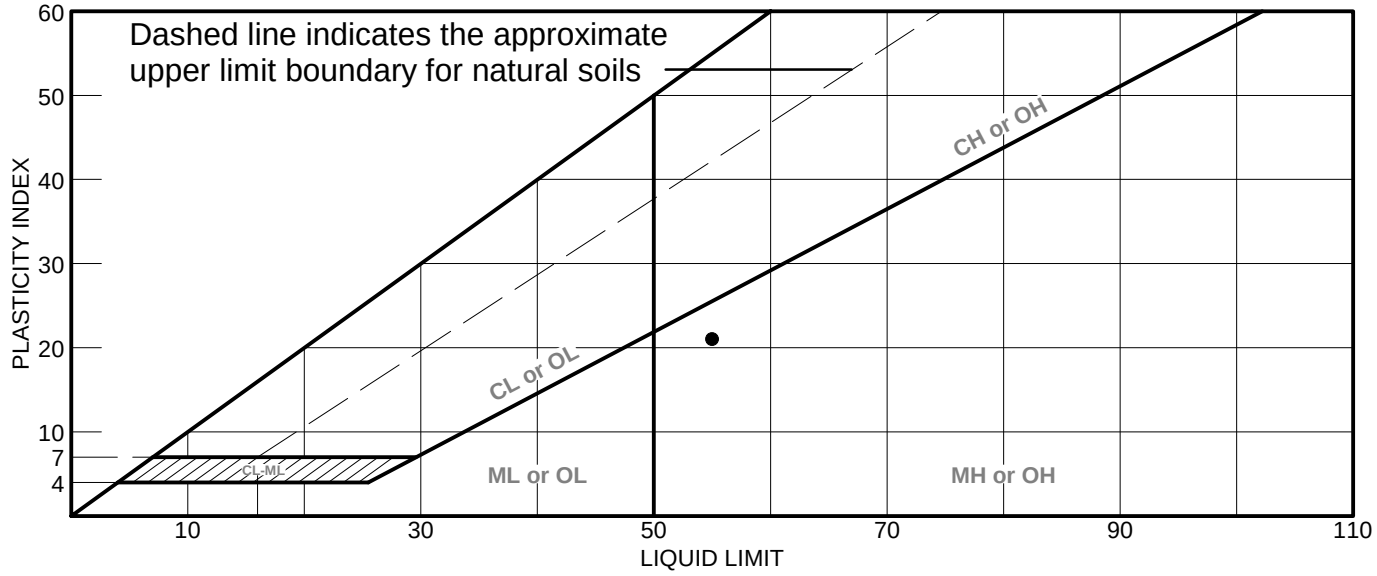
ATTERBERG-PLASTICITY 2 PER PAGE GRI DATA TEMPLATE.GDT 1/16/15

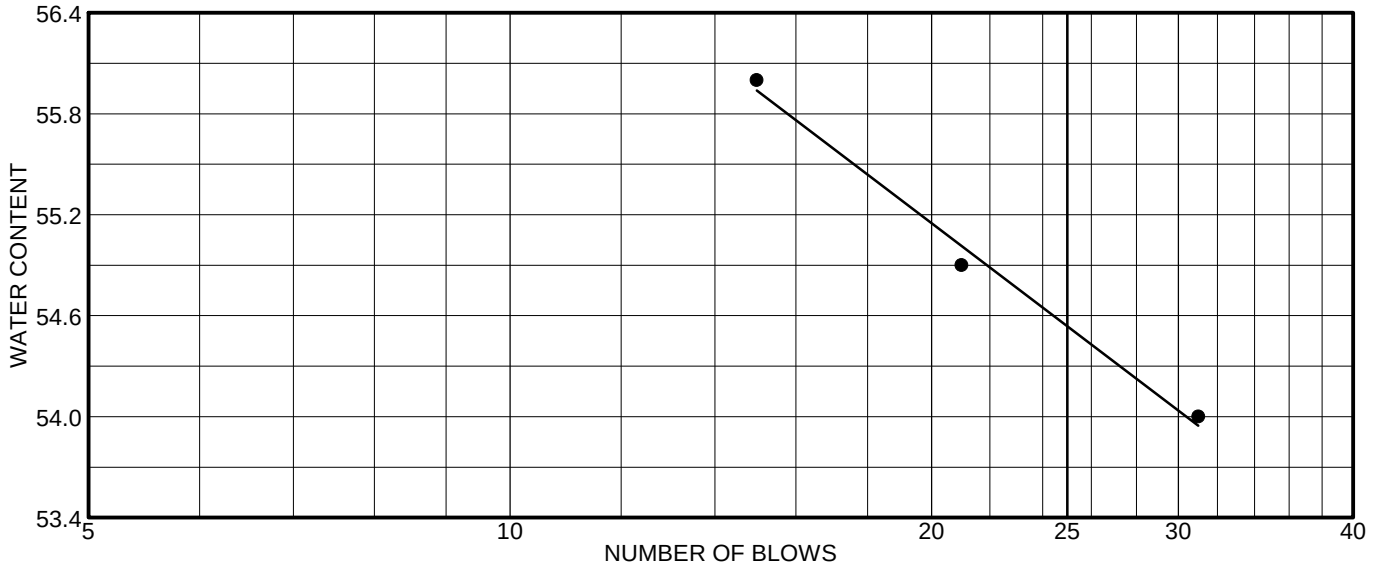
GRI

PLASTICITY CHART

AUG. 2015 JOB NO. 5338-A FIG. 6A

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Strong Brown Sandy Elastic SILT	55	34	21	87.2	60.1	MH

Project No. 823-005 **Client:** GRI

Project: Bolton Reservoir - 5338-A

● **Source:** B-2 **Sample No.:** S-13 **Elev./Depth:** 35'

Remarks:

●

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

AUG. 2015 JOB NO. 5338-A FIG. 7A

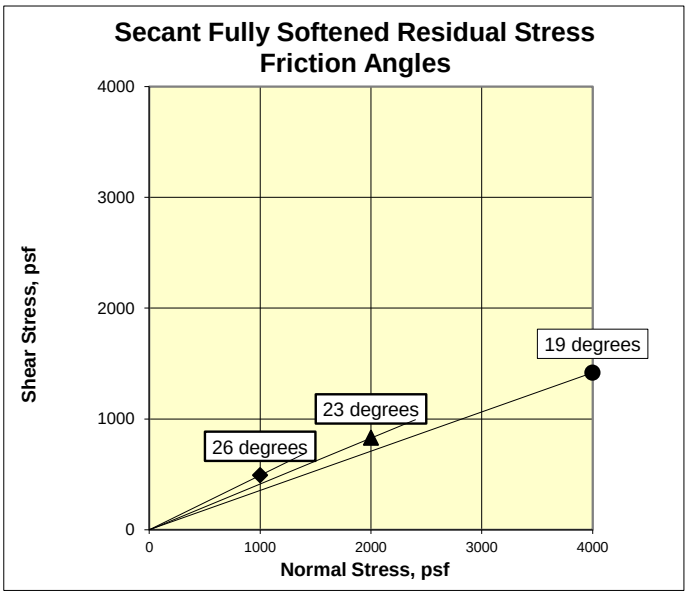
Drained Residual Torsional Shear Strength (ASTM D6467)

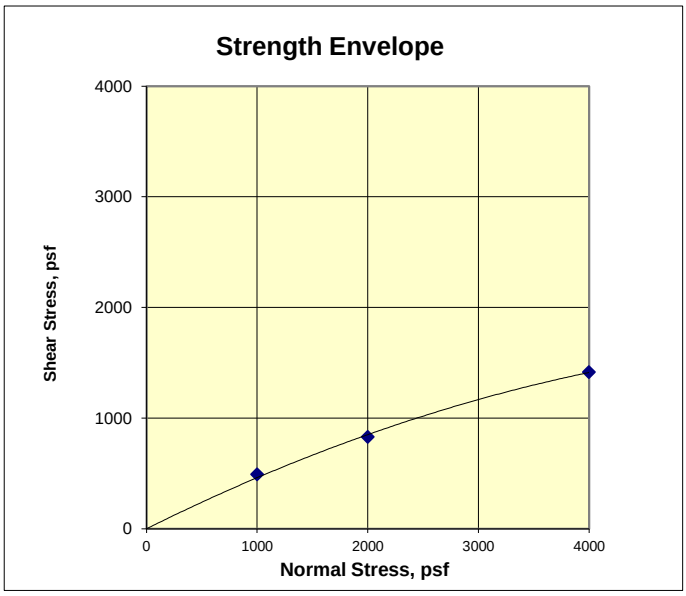
CTL Job No.:	823-005	**Boring:**	B-2	**Date:**	12/9/2014	**Clay, %:**	29.2
Client:	GRI	**Sample:**	S-13	**By:**	PJ	**LL:**	55
Project Name:	Bolton Reservoir	**Depth (ft):**	35	**Checked:**	DC	**PL:**	34
Project Number:	5338-A	**Test Type:**	Fully Softened Residual				

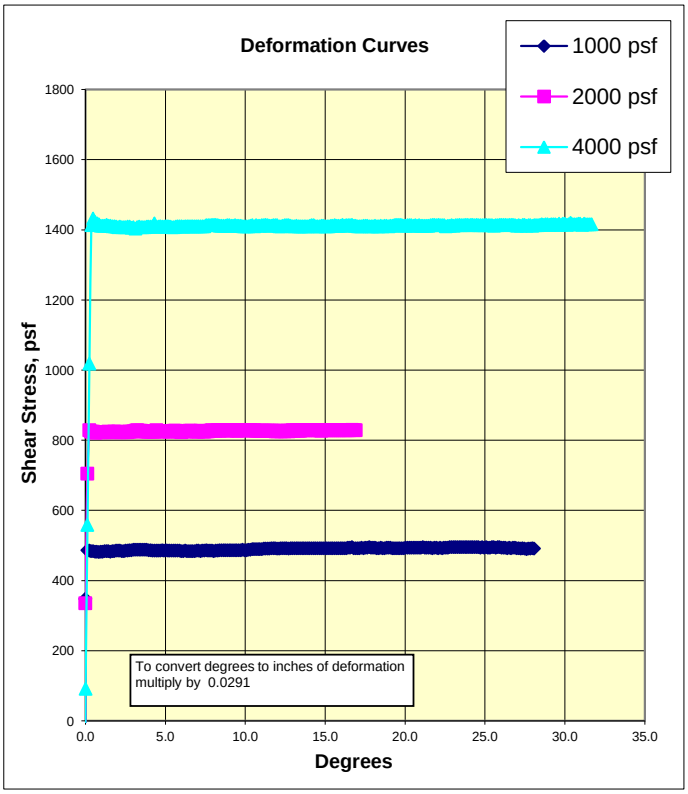
Soil Type: Strong Brown Sandy Elastic SILT

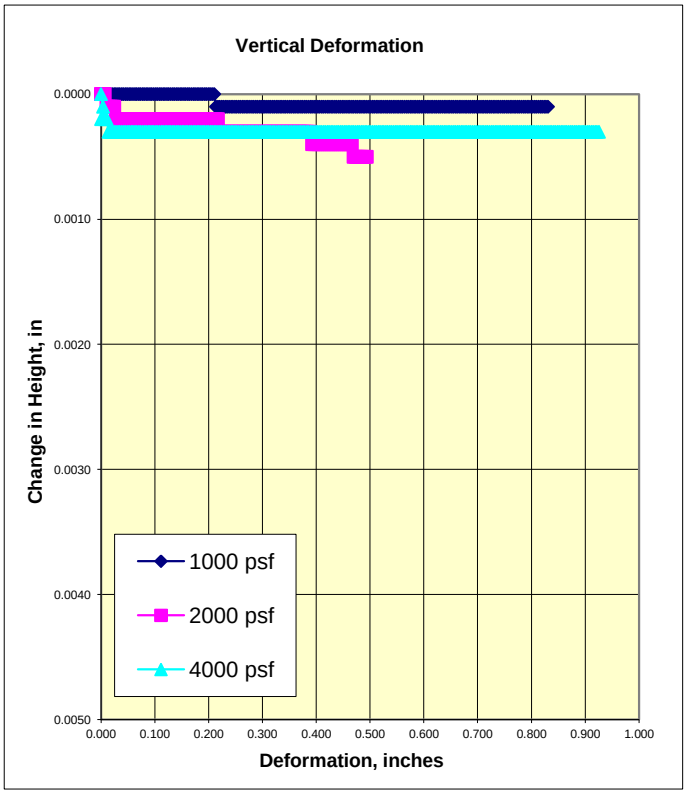
Normal Stress, psf:	1000	2000	4000
Secant Phi, deg.:	26	23	19

Remarks: This sample has an unusually highly curved strength envelope.

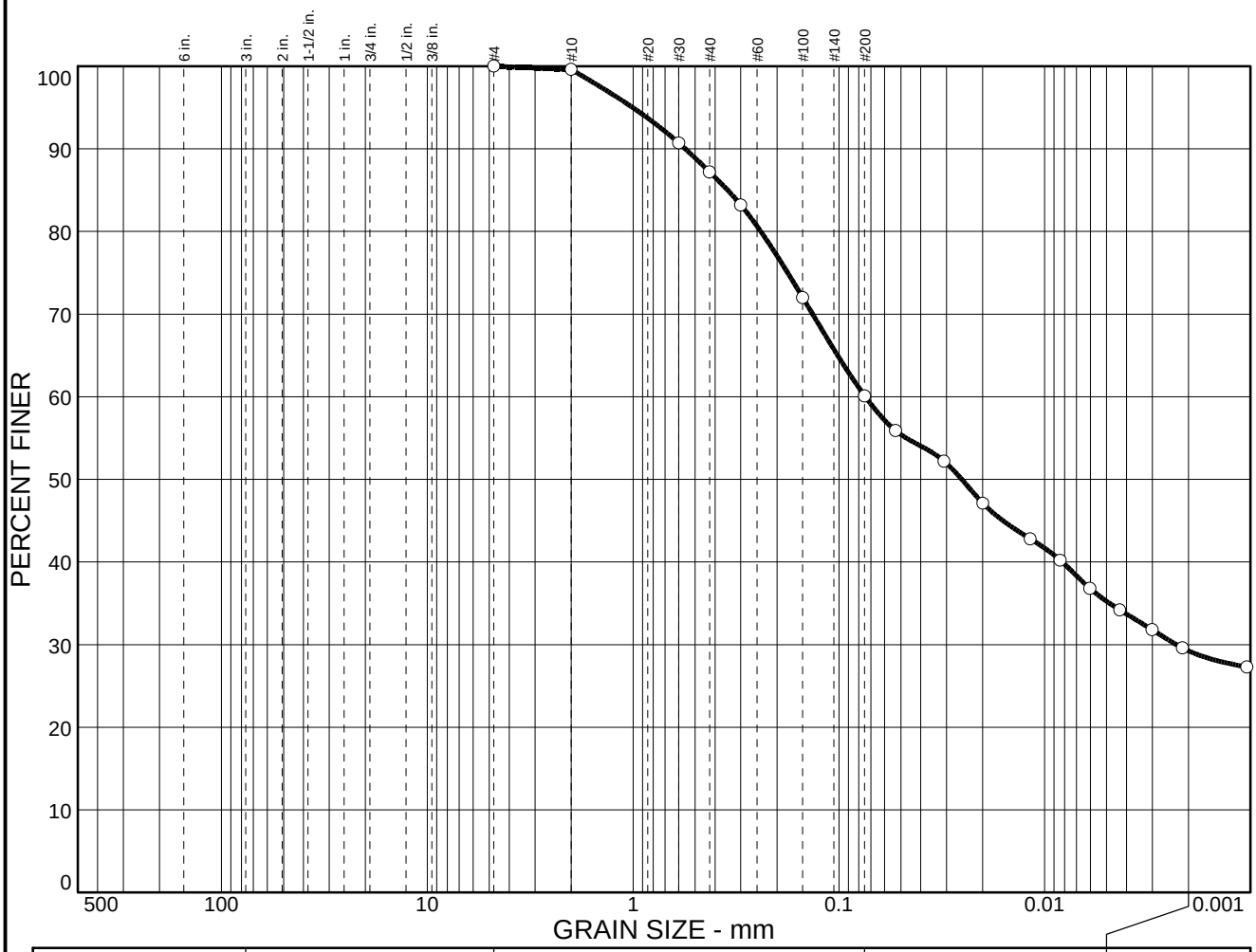








Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	39.9	30.9	29.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.6		
#30	90.7		
#40	87.2		
#50	83.2		
#100	72.0		
#200	60.1		
#270	55.9		
0.0308 mm.	52.2		
0.0200 mm.	47.1		
0.0118 mm.	42.8		
0.0084 mm.	40.2		
0.0060 mm.	36.8		
0.0043 mm.	34.2		
0.0030 mm.	31.8		
0.0021 mm.	29.6		
0.0010 mm.	27.3		

* (no specification provided)

Soil Description

Strong Brown Sandy Elastic SILT

Atterberg Limits

PL= 34 LL= 55 PI= 21

Coefficients

D_{85}= 0.348 D_{60}= 0.0745 D_{50}= 0.0253
D_{30}= 0.0023 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= MH AASHTO=

Remarks

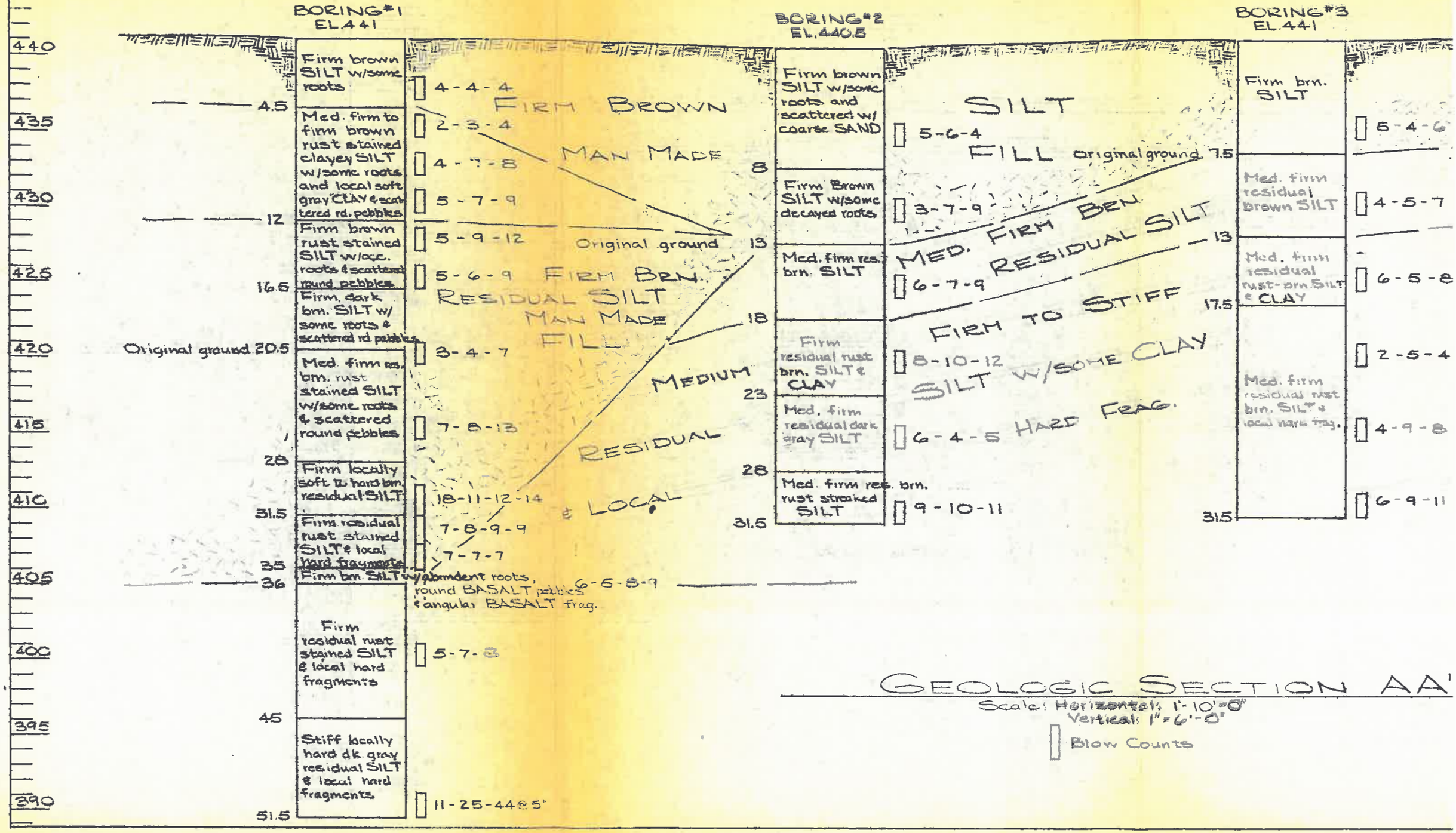
Sample No.: S-13 **Source of Sample:** B-2 **Date:** 12/3/14
Location: **Elev./Depth:** 35'

COOPER TESTING LABORATORY

Client: GRI
Project: Bolton Reservoir - 5338-A
Project No: 823-005 **Figure**

APPENDIX B

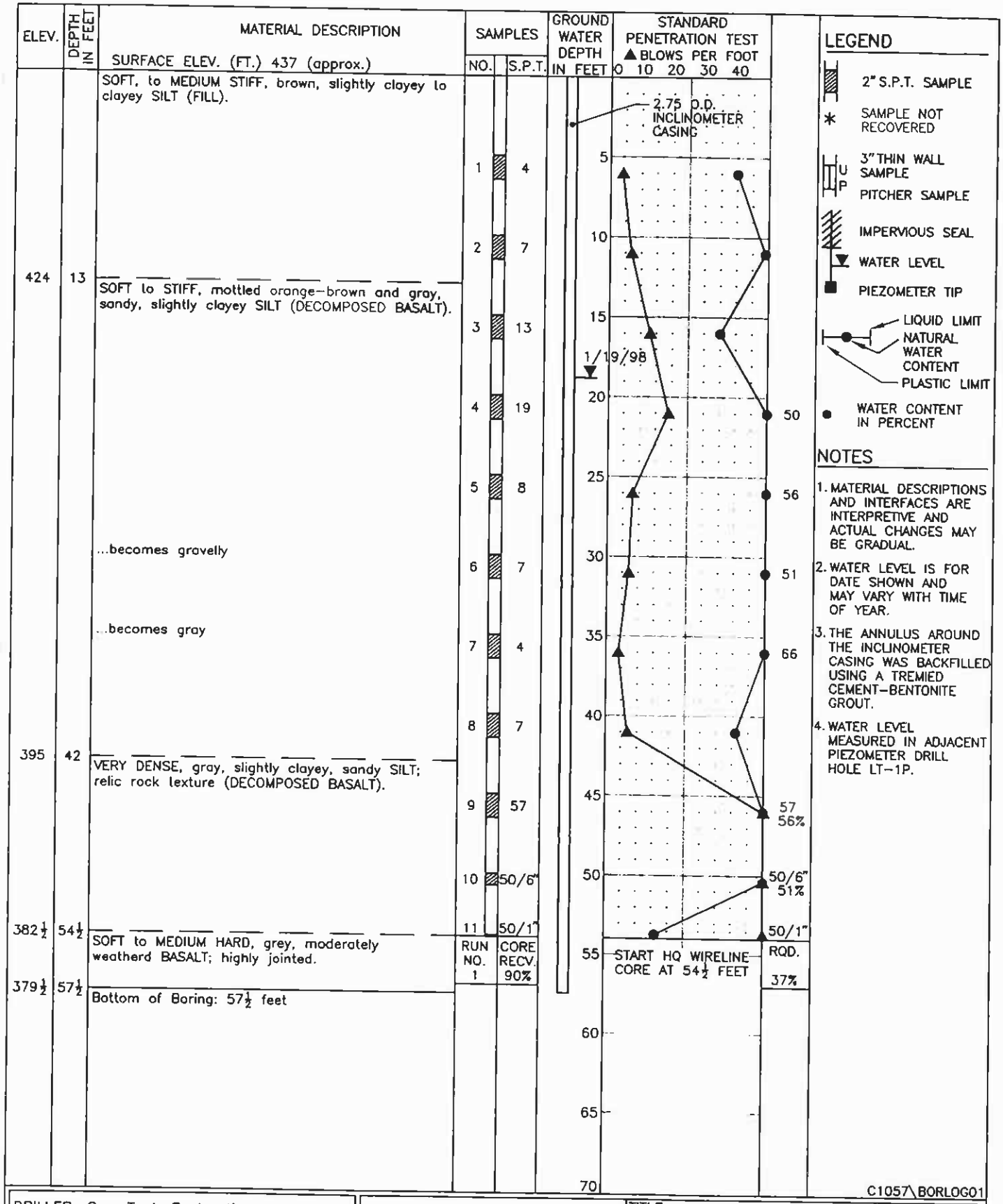
Logs of Subsurface Explorations by Others



GEOLOGIC SECTION AA'

Scale: Horizontal: 1"-10'-0"
Vertical: 1"=6'-0"

Blow Counts


ELEV.
DEPTH IN FEET
MATERIAL DESCRIPTION
SURFACE ELEV. (FT.) 437 (approx.)
SAMPLES
NO.
S.P.T.
GROUND WATER DEPTH IN FEET
STANDARD PENETRATION TEST
▲ BLOWS PER FOOT
0 10 20 30 40
SOFT, to MEDIUM STIFF, brown, slightly clayey to clayey SILT (FILL).
2.75 O.D. INCLINOMETER CASING
1 4
2 7
424 13
SOFT to STIFF, mottled orange-brown and gray, sandy, slightly clayey SILT (DECOMPOSED BASALT).
3 13
1/19/98
4 19
50
5 8
56
...becomes gravelly
6 7
51
...becomes gray
7 4
66
8 7
395 42
VERY DENSE, gray, slightly clayey, sandy SILT; relic rock texture (DECOMPOSED BASALT).
9 57
57
56%
10 50/6"
50/6"
51%
382½ 54½
11 50/1"
50/1"
SOFT to MEDIUM HARD, grey, moderately weatherd BASALT; highly jointed.
RUN NO.
CORE RECV.
RQD.
1 90%
37%
START HQ WIRELINE CORE AT 54½ FEET
379½ 57½
Bottom of Boring: 57½ feet
5
10
15
20
25
30
35
40
45
50
55
60
65
70
LEGEND
2" S.P.T. SAMPLE
SAMPLE NOT RECOVERED
3" THIN WALL SAMPLE
U
P
PITCHER SAMPLE
IMPERVIOUS SEAL
WATER LEVEL
PIEZOMETER TIP
LIQUID LIMIT
NATURAL WATER CONTENT
PLASTIC LIMIT
WATER CONTENT IN PERCENT
NOTES
1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR.
3. THE ANNULUS AROUND THE INCLINOMETER CASING WAS BACKFILLED USING A TREMIED CEMENT-BENTONITE GROUT.
4. WATER LEVEL MEASURED IN ADJACENT PIEZOMETER DRILL HOLE LT-1P.
C1057\BORLOG01
DRILLER Geo-Tech Explorations
DATE START 12/5/97 FINISH 12/5/97
DRILLING TECHNIQUE 4⅞-inch O.D. Tricone Mud Rotary, HSA 8-inch O.D., HQ Coring
Landslide Technology
10250 S.W. Greenburg Rd. Portland, OR 97223
TITLE
SUMMARY BORING LOG
LT-1
DATE
JAN 1998
JOB NO.
1057
JOB
BOLTON RESERVOIR LANDSLIDE
WEST LINN, OREGON
FIG. 2

APPENDIX C

Site-Specific Seismic Hazard Study

APPENDIX C

SITE-SPECIFIC SEISMIC HAZARD STUDY

General

GRI has completed a site-specific seismic hazard study for the proposed Bolton Reservoir in West Linn, Oregon. The purpose of the study was to evaluate potential seismic hazards associated with regional and local seismicity. The site-specific hazard study is intended to meet the requirements of the 2012 International Building Code (IBC), which was recently adopted by the 2014 Oregon Structural Specialty Code (OSSC). The 2012 IBC is based on the American Society of Civil Engineers (ASCE) 7-10 document *Minimum Design Loads for Buildings and Other Structures*. Our work was based on the potential for regional and local seismic activity, as described in the existing scientific literature, and on the subsurface conditions at the site, as disclosed by the subsurface explorations completed for this project. Specifically, our work included the following tasks:

1) A detailed review of available literature, including published papers, maps, open-file reports, seismic histories and catalogs, , and other sources of available information regarding the tectonic setting, regional and local geology, and historical seismic activity that might have a significant effect on the site.

2) Compilation and evaluation of subsurface data collected at and in the vicinity of the site, including classification and laboratory analysis of soil samples. This information was used to prepare a generalized subsurface profile for the site.

3) Identification of the potential seismic events (earthquakes) appropriate for the site and characterization of those events in terms of a generalized design event.

4) Office studies, based on the generalized subsurface profile and the generalized design earthquake, resulting in conclusions and recommendations concerning:

    a) specific seismic events that might have a significant effect on the site,

    b) the potential for seismic energy amplification and liquefaction or soil strength loss at the site, and

    c) site-specific acceleration response spectra for design of the proposed reservoir.

This appendix describes the work accomplished and summarizes our conclusions and recommendations.

Geologic Setting

On a regional scale, the site is located at the northern end of the Willamette Valley, a broad, gently deformed, north-south-trending topographic feature separating the Coast Range to the west from the Cascade Mountains to the east. The site is located approximately 100 km inland from the Cascadia Subduction Zone (CSZ), an active plate boundary along which remnants of the Farallon plate (the Gorda, Juan de Fuca, and Explorer plates) are being subducted beneath the western edge of the North American

plate. The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting slabs of the Gorda, Juan de Fuca, and Explorer plates and the over-riding North American plate as shown on the Tectonic Setting Summary, Figure 1C.

On a local scale, the site is located in the Portland Basin, a large, well-defined, northwest-trending structural basin bounded by high-angle, northwest-trending, right-lateral strike-slip faults considered to be seismogenic. The distribution of these faults relative to the site is shown on the Regional Geologic Map, Figure 2C. Additional faults in the project area that are considered potentially active by the U. S. Geological Survey (USGS) are shown on the Local Fault Map, Figure 3C. Information regarding the continuity and potential activity of these faults is lacking, due largely to the scale at which geologic mapping in the area has been conducted and the presence of thick, relatively young, basin-filling sediments that obscure underlying structural features. Other faults may be present within the basin, but clear stratigraphic and/or geophysical evidence regarding their location and extent is not presently available. Additional discussion regarding crustal faults is provided in the Local Crustal Event section below.

Because of the proximity of the site to the CSZ and its location within the Portland Basin, three distinctly different sources of seismic activity contribute to the potential for the occurrence of damaging earthquakes. Each of these sources is generally considered to be capable of producing damaging earthquakes. Two of these sources are associated with the deep-seated tectonic activity related to the subduction zone; the third is associated with movement on the local, relatively shallow structures within and adjacent to the Portland Basin.

The site is located on the eastern flank of the Tualatin Mountains, a topographic upland that separates the Portland Basin to the northeast from the Tualatin Basin to the west and the Willamette Valley to the south. Geologic mapping completed for the area indicates the site is located in the vicinity of the contact between the Miocene-age Wanapum Basalt and the Grande Ronde Basalt units of the Columbia River Basalt Group (Madin, 2009). The site and other areas of the Tualatin Mountain upland are capped by deposits of fine-grained, wind-blown silt, referred to as Portland Hills Silt. Quaternary alluvial deposits associated with the Willamette River and the Ice Age Missoula Floods (about 15,000 to 20,000 years ago) are present northeast of the site, north of Hwy 43.

Seismicity

General. The geologic and seismologic information available for identifying the potential seismicity at the site is incomplete, and large uncertainties are associated with estimates of the probable magnitude, location, and frequency of occurrence of earthquakes that might affect the site. The available information indicates the potential seismic sources that may affect the site can be grouped into three independent categories: *subduction zone events* related to sudden slip between the upper surface of the Juan de Fuca plate and the lower surface of the North American plate, *subcrustal events* related to deformation and volume changes within the subducted mass of the Juan de Fuca plate, and *local crustal events* associated with movement on shallow, local faults within and adjacent to the Portland Basin. Based on our review of currently available information, we have developed generalized design earthquakes for each of these categories in accordance with Section 1803 of the OSSC. The design earthquakes are characterized by three important properties: size, location relative to the subject site, and the peak horizontal bedrock accelerations produced by the event. In this study, earthquake size is expressed by the moment magnitude

(M); location is expressed as the closest distance to the fault rupture, measured in kilometers; and peak horizontal bedrock accelerations are expressed in units of gravity (1 g = 32.2 ft/sec^2 = 981 cm/sec^2).

Subduction Zone Event. The last interplate earthquake on the CSZ occurred in January 1700. Geological studies show that great megathrust earthquakes have occurred repeatedly in the past 7,000 years (Atwater et al., 1995; Clague, 1997; Goldfinger, 2003; and Kelsey et al., 2005), and geodetic studies (Hyndman and Wang, 1995; Savage et al., 2000) indicate rate of strain accumulation consistent with the assumption that the CSZ is locked beneath offshore northern California, Oregon, Washington, and southern British Columbia (Fluck et al., 1997; Wang et al., 2001). Numerous geological and geophysical studies suggest the CSZ may be segmented (Hughes and Carr, 1980; Weaver and Michaelson, 1985; Guffanti and Weaver, 1988; Goldfinger, 1994; Kelsey et al., 1994; Mitchell et al., 1994; Personius, 1995; Nelson and Personius, 1996; Witter, 1999), but the most recent studies suggest that for the last great earthquake in 1700, most of the subduction zone ruptured in a single M9.0 earthquake (Satake et al., 1996; Atwater and Hemphill-Haley, 1997; Clague et al., 2000). Published estimates of the probable maximum size of subduction zone events range from M8.3 to greater than M9.0. Numerous detailed studies of coastal subsidence, tsunamis, and turbidites yield a wide range of recurrence intervals, but the most complete records (>4,000 years) indicate average intervals of 350 to 600 years between great earthquakes on the CSZ (Adams, 1990; Atwater and Hemphill-Haley, 1997; Witter, 1999; Clague et al., 2000; Kelsey et al., 2002; Kelsey et al., 2005; Witter et al., 2003; Goldfinger et al, 2012). Tsunami inundation in buried marshes along the Washington and Oregon coast and stratigraphic evidence from the Cascadia margin support these recurrence intervals (Kelsey et al., 2005; Goldfinger, 2003).

The USGS probabilistic analysis assumes four potential locations for the location of the eastern edge of the earthquake rupture zone as shown on Figure 4C. The 2008 USGS mapping effort indicates two rupture scenarios are assumed to represent these megathrust events: 1) M9±0.2 events that rupture the entire CSZ every 500 years and 2) M8.3 to 8.7 events with rupture zones that occur on segments of the CSZ and occur over the entire length of the CSZ during a period of about 500 years (Petersen et al., 2008). The assumed distribution of earthquakes is shown on the Assumed Magnitude-Frequency Distribution, Figure 5C. This distribution assumes the larger M9.0 earthquake is the most likely single CSZ earthquake scenario, as also indicated by the USGS deaggregation for the site. Therefore, for our deterministic analysis, we have chosen to represent the subduction zone event by a design earthquake of M9.0 at a focal depth of 20 km and rupture distance of 100 km. This corresponds to a sudden rupture of the whole length of the Juan de Fuca-North American plate interface with an assumed rupture zone due west of the site. Based on an average of the attenuation relationships published by Youngs et al. (1997), Atkinson and Boore (2003), and Zhao et al. (2006), a subduction zone earthquake of this size and location would result in a peak horizontal bedrock acceleration of approximately 0.12 g at the site.

Subcrustal Event. There is no historic earthquake record of subcrustal, intraslab earthquakes in Oregon. Although both the Puget Sound and northern California region have experienced many of these earthquakes in historic times, Wong (2005) hypothesizes that due to subduction zone geometry, geophysical conditions, and local geology, Oregon may not be subject to intraslab earthquakes. In the Puget Sound area, these moderate to large earthquakes are deep (40 to 60 km) and over 200 km from the deformation front of the subduction zone. Offshore, along the northern California coast, the earthquakes are shallower (up to 40 km) and located along the deformation front. Estimates of the probable size, location, and frequency of subcrustal events in Oregon are generally based on comparisons of the CSZ

with active convergent plate margins in other parts of the world and on the historical seismic record for the region surrounding Puget Sound, where significant events known to have occurred within the subducting Juan de Fuca plate have been recorded. Published estimates of the probable maximum size of these events range from M7.0 to 7.5. The 1949, 1965, and 2001 documented subcrustal earthquakes in the Puget Sound area correspond to M7.1, 6.5, and 6.8, respectively. Published information regarding the location and geometry of the subducting zone indicates that a focal depth of 50 km is probable (Weaver and Shedlock, 1989). We have chosen to represent the subcrustal event by a design earthquake of M7.0 at a focal depth of 50 km and a rupture distance of 60 km. Based on the attenuation relationships published by Youngs et al. (1997) and Atkinson and Boore (2003), a subcrustal earthquake of this size and location would result in a peak horizontal bedrock acceleration of approximately 0.14 g at the site.

Local Crustal Event. Sudden crustal movements along relatively shallow, local faults in the project area, although rare, have been responsible for local crustal earthquakes. The precise relationship between specific earthquakes and individual faults is not well understood, since few of the faults in the area are expressed at the ground surface, and the foci of the observed earthquakes have not been located with precision. The history of local seismic activity is commonly used as a basis for determining the size and frequency to be expected of local crustal events. Although the historical record of local earthquakes is relatively short (the earliest reported seismic event in the area occurred in 1920), it can serve as a guide for estimating the potential for seismic activity in the area.

Based on fault mapping conducted by the USGS, the Bolton Fault is the closest mapped crustal fault identified as a hazard to the site (USGS, 2008). The surface trace of the Bolton Fault is located about 900 ft northeast of the site (Madin, 2009). The Bolton Fault has a characteristic earthquake magnitude of 6.2. A crustal earthquake of this size and location would result in a peak horizontal bedrock acceleration of approximately 0.45 g at the site based on an average of the NGA ground motion relations published by Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008).

Summary of Deterministic Earthquake Parameters

In summary, three distinctly different types of earthquakes affect seismicity in the project area. Deterministic evaluation of the earthquake sources using recently published attenuation ground motion relations provides estimates of ground response for each individual earthquake type. Unlike probabilistic estimates, these deterministic estimates are not associated with a relative hazard level or probability of occurrence and simply provide an estimate of the ground motion parameters for each type of fault at a given distance from the site. For each earthquake source, we have attempted to use attenuation relationships and weighting that are consistent with the development of the 2008 USGS seismic hazard maps. The basic parameters of each type of earthquake are as follows:

Earthquake Source	Attenuation Relationships for Target Spectra	Magnitude, M	Rupture Distance, km	Focal Depth, km	Peak Bedrock Acceleration, g	Average Peak Bedrock Acceleration, g
Subduction Zone	Youngs et al., 1997	9.0	100	20	0.14	0.12
	Atkinson and Boore, 2003	9.0	100	20	0.07	
	Zhao et al., 2006 [(1)]	9.0	100	20	0.14	
Subcrustal	Youngs et al., 1997	7.0	60	50	0.15	0.14
	Atkinson and Boore, 2003	7.0	60	50	0.13	

Earthquake Source	Attenuation Relationships for Target Spectra	Magnitude, M	Rupture Distance, km	Focal Depth, km	Peak Bedrock Acceleration, g	Average Peak Bedrock Acceleration, g
Local Crustal	Campbell and Bozorgnia, 2008	6.2	1	NA	0.43	0.45
	Chiou and Youngs, 2008	6.2	1	NA	0.52	
	Boore and Atkinson, 2008	6.2	1	NA	0.40	

[(1)] Relationship by Zhao et al. (2006) limited to magnitude 8.5.

Probabilistic Considerations

The probability of an earthquake of a specific magnitude occurring at a given location is commonly expressed by its return period, i.e., the average length of time between successive occurrences of an earthquake of that size or larger at that location. The return period of a design earthquake is calculated once a project design life and some measure of the acceptable risk that the design earthquake might occur or be exceeded are specified. These expected earthquake recurrences are expressed as a probability of exceedance during a given time period or design life. Historically, building codes have adopted an acceptable risk level by identifying ground acceleration values that meet or exceed a 10% probability of exceedance in 50 years, which corresponds to an earthquake with an expected recurrence interval of 475 years. Previous versions of the IBC developed response spectra based on ground motions associated with the Maximum Considered Earthquake (MCE), which is generally defined as a probabilistic earthquake with a 2% probability of exceedance in 50 years (return period of about 2,500 years) except where subject to deterministic limitations (Leyendecker et al., 2000).

The recent 2012 IBC develops response spectra using a Risk-Targeted Maximum Considered Earthquake (MCE_R), which is defined as the response spectrum that is expected to achieve a 1% probability of building collapse within a 50-year period. The design-level response spectrum is calculated as two-thirds of the MCE_R ground motions. Since the MCE_R earthquake ground motions were developed by the USGS to incorporate the targeted 1% in 50 years risk of structural collapse based upon a generic structural fragility, they are different than the ground motions associated with the traditional MCE. Although site response is evaluated based on the MCE_R, it should be noted that seismic hazards, such as liquefaction and soil strength loss, are evaluated using the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration (PGA), which is more consistent with the traditional MCE.

The 2012 IBC design methodology uses two mapped spectral acceleration parameters, S_S and S_1, corresponding to periods of 0.2 and 1.0 second, to develop the MCE_R earthquake. The S_S and S_1 coefficients for the site located at the approximate latitude and longitude coordinates of 45.3684°N and 122.6247°W are 0.95 and 0.41 g, respectively.

Estimated Site Response

The effect of a specific seismic event on the site is related to 1) the type and quantity of seismic energy delivered to the bedrock beneath the site by the earthquake and 2) the type and thickness of soil overlying the bedrock at the site. Ground motion hazard analysis was completed to estimate this site-specific behavior in accordance with Section 21.2 of ASCE 7-10. The ground motion hazard analysis consisted of three significant components: 1) estimation of ground surface response using recently developed attenuation relationships that are capable of modeling soil site conditions (deterministic evaluation), 2) estimation of ground surface response using code-based adjustment factors based on soil site class

(probabilistic evaluation), and 3) comparison of the deterministic and probabilistic ground surface response spectra to recommend a site-specific response spectrum for design. The following paragraphs describe the details of the ground motion hazard analysis.

To estimate the deterministic ground surface response spectrum, recently developed attenuation relationships were used to evaluate amplification and/or attenuation of bedrock ground motions through the soil column at the site. Based on our review of the USGS deaggregation for the site (USGS, 2014), an event on the CSZ and crustal seismicity represent the largest contributing sources to the seismic hazard at the site. Considering this, we have chosen to estimate the deterministic ground surface response using 84th percentile ground motions from the following two earthquake scenarios: 1) a M9.0 subduction zone earthquake at a distance of 100 km from the site, and 2) a M6.2 crustal earthquake at a distance of 1 km from the site. The attenuation relationship of Youngs et al. (1997) and the recently developed BC Hydro relationship of Abrahamson et al. (2012) were used to evaluate the subduction zone earthquake response. The NGA ground motion relations published by Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008) were used to evaluate the crustal earthquake response. One input parameter for the attenuation relationships is the average shear wave velocity in the upper 100 ft of the soil profile. Based on published correlations with standardized field data and our experience with similar subsurface conditions, we estimate the average shear wave velocity at the site is on the order of 1,100 ft/s. The resulting deterministic MCE_R ground surface response spectra are shown on Figure 6C. As required by Section 21.2.2 of ASCE 7-10, Figure 6C also shows the deterministic lower limit MCE_R spectrum. The deterministic MCE_R ground surface spectrum is taken as the larger of the 84th percentile ground motions and the deterministic lower limit. To estimate the probabilistic ground surface response spectrum, adjustment factors based on observed soil conditions are used to evaluate amplification and/or attenuation of bedrock ground motions through the soil column at the site. The site is classified as Site Class D, or a stiff soil site, based on the estimated average shear wave velocity in the upper 100 ft of the soil profile in accordance with Section 20.3 of ASCE 7-10. Corresponding short- and long-period adjustment factors F_a and F_v, of 1.12 and 1.59, respectively, were used to develop the probabilistic Site Class D MCE_R response spectrum shown on Figure 7C.

In accordance with Section 21.2.3 of ASCE 7-10, the site-specific ground surface MCE_R response spectrum is taken as the lesser of the probabilistic and deterministic MCE_R ground motions. Figure 7C shows a comparison of the deterministic and probabilistic MCE_R ground motions and indicates the code-based probabilistic Site Class D MCE_R response spectrum is appropriate for the site. The design-level response spectrum is calculated as two-thirds of the MCE_R response spectrum. We recommend using the Site Class D design response spectrum shown on Figure 8C for design of the reservoir.

Seismic Hazards

Liquefaction. Liquefaction is a process by which loose, saturated, granular materials, such as sand, and to a somewhat lesser degree soft, non-plastic silts, temporarily lose strength during and immediately after a seismic event. Liquefaction occurs as seismic shear stresses propagate through a saturated soil and distort the soil structure causing loosely packed groups of particles to contract or collapse. If drainage is impeded and cannot occur quickly, the collapsing soil structure increases the porewater pressure between the soil grains. If the porewater pressure increases to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid rather than a solid. As strength is lost, there is an

increased risk of settlement, lateral spread, and/or slope instability. Liquefaction-induced settlement occurs as the elevated porewater pressures dissipate and the soil consolidates after the earthquake.

Based on preliminary evaluations, there is some risk of seismically induced soil strength loss in isolated soft layer(s) within the decomposed basalt that were encountered in some of the explorations at depths of about 20 to 40 ft below the ground surface. In our opinion, the risk of significant settlement due to seismically induced soil strength loss in these isolated zones is low. However, there is some risk of seismic slope instability at the site, and the presence of these loose and soft soil zones may increase the risk of slope movement during and immediately following an earthquake. We anticipate a ground improvement program will be completed at the site to limit the risk of seismically induced soil strength loss and slope instability.

Other Hazards. The risk of damage by tsunami and/or seiche at the site is absent due to the elevation of the site. In our opinion, the risk of liquefaction-induced lateral spreading and ground deformation at the site is low. As previously discussed, the surface trace of the Bolton Fault is located about 900 ft northeast of the site. Unless occurring on a previously unmapped or unknown fault, it is our opinion the risk of ground rupture at the site is low.

Based on our slope stability analyses completed for the project, there is a risk of seismically induced local slope instability at the site associated with a relatively horizontal to shallow dip of soft layer(s) within the decomposed basalt. Soft layers were encountered locally in the borings between depths of about 20 and 40 ft below the ground surface. Our analyses indicate the potential seismic instability at the reservoir site would most likely consist of near-horizontal, translational block failures beneath the tank and on the sloping ground north of the tank. As currently planned, a ground improvement program will be completed beneath the tank footprint to reduce the risk of seismic movements beneath the tank from local movements. In addition, the top of the slope along the north side of the site will be flattened to decrease the risk of slope movement on the reservoir.

The reservoir site is located on a very large, ancient landslide. The ancient landslide is likely to move feet rather than tens of feet during a large earthquake (Cornforth Consultants, 2014).

Conclusions

The 2012 IBC design methodology uses two spectral response coefficients, S_S and S_1, corresponding to periods of 0.2 and 1.0 second, to develop the MCE_R response spectrum. The S_S and S_1 coefficients for the site are 0.95 and 0.41 g, respectively. The results of the ground motion hazard analysis indicate the 2012 IBC Site Class D spectrum provides an appropriate estimate of the spectral accelerations at the site. We recommend using the Site Class D design spectrum shown on Figure 8C for the project.

References

Abrahamson, N.A., Gregor N., and Addo, K., 2012, BC Hydro ground motion prediction equations for subduction earthquakes, submitted to Earthquake Spectra.

Adams, J., 1990, Paleoseismicity of the Cascadia subduction zone: Evidence from turbidites off the Oregon-Washington margin: Tectonics, v. 9, no. 4, p. 569-583.

Atkinson, G.M., and Boore, D.M., 2003, Empirical ground motion relations for subduction zone earthquakes and their application to Cascadia and other regions: Seismological Research Letters, v. 93, no. 4, pp. 1703-1729.

Atwater, B.F., and Hemphill-Haley, E., 1997, Recurrence intervals for great earthquakes of the past 3,500 years at northeastern Willapa Bay, Washington: U. S. Geological Survey Professional Paper 1576, 108 p.

Atwater, B.F., Nelson, A.R., Clague, J.J., Carver, G.A., Yamaguchi, D.K., Bobrowsky, P.T., Bourgeois J., Darienzo, M.E., Grant, W.C., Hemphill-Haley, E., Kelsey, H.M., Jacoby, G.C., Nishenko, S.P., Palmer, S.P., Peterson, C.D., and Reinhart, M.A., 1995, Summary of coastal geologic evidence for past great earthquakes at the Cascadia subduction zone. *Earthquake Spectra*, 11:1, 1-18.

Boore, D.M., and Atkinson, G.M., February 2008, Ground-motion prediction equations for the average horizontal component of PGA, PGV, and 5%-damped PSA at spectral periods between 0.01 s and 10.0 s: Earthquake Engineering Research Institute, Earthquake Spectra, vol. 24, no. 1, pp 99-138.

Campbell, K.W., and Bozorgnia, Y., February 2008, NGA ground motion model for the geometric mean horizontal component of PGA, PGV, and PGD and 5%-damped linear elastic response spectra for periods ranging from 0.01 to 10 s: Earthquake Engineering Research Institute, Earthquake Spectra, vol. 24, no. 1, pp 139-171.

Chiou, S.J., and Youngs, R.R., 2008, NGA Model for Average Horizontal Component of Peak Ground Motion and Response Spectra: Pacific Earthquake Engineering Research Center, PEER 2008/09, November 2008.

Clague, J.J., 1997, Evidence for large earthquakes at the Cascadia subduction zone: Reviews of Geophysics, v. 35, no. 4, p. 439-460.

Clague, J.J., Atwater, B.F., Wang, K., Wang, Y., and Wong, I., 2000, Penrose conference report–Great Cascadia earthquake tricentennial: GSA Today, v. 10, no. 11, p. 14-15.

Cornforth Consultants, Inc., December 2, 2014, (Draft) Geotechnical engineering services, Bolton reservoir seismic landslide evaluation, West Linn, Oregon; prepared for Murray, Smith & Associates, Inc.

Fluck, P., Hyndman, R.D., and Wang, K., 1997, Three-dimensional dislocation model for great earthquakes of the Cascadia subduction zone: Journal of Geophysical Research, v. 102, no. B9, p. 20,539-20,550.

Goldfinger, C., 1994, Active deformation of the Cascadia Forearc–Implications for great earthquake potential in Oregon and Washington: Oregon State University, unpublished Ph.D. dissertation, 246 p.

_____, 2003, Great earthquakes in Cascadia: a who dunnit success story: presentation at 2003 EERI national conference, Portland, Oregon.

Goldfinger, C., Nelson, C.H., Morey, A.E., Johnson, J.R., Patton, J., Karabanov, E., Gutierrez-Pastor, J., Eriksson, A.T., Gracia, E., Dunhill, G., Enkin, R.J., Dallimore, A., and Vallier, T., 2012, Turbidite event history—Methods and implications for Holocene paleoseismicity of the Cascadia subduction zone: US Geological Survey Professional Paper 1661–F, 170 p, 64 figures, available at http://pubs.usgs.gov/pp/pp1661.

Guffanti, M., and Weaver, C.S., 1988, Distribution of Late Cenozoic volcanic vents in the Cascade Range–Volcanic arc segmentation and regional tectonic considerations: Journal of Geophysical Research, v. 93, no. B6, p. 6513-6529.

Hughes, J.M., and Carr, M.J., 1980, Segmentation of the Cascade volcanic chain: Geology, v. 8, p. 15-17.

Hyndman, R.D., and Wang, K., 1995, The rupture zone of Cascadia great earthquakes from current deformation and the thermal regime: Journal of Geophysical Research, v. 100, no. B11, p. 22,133-22,154.

Kelsey, H.M., and Bockheim, J.G., 1994, Coastal landscape evolution as a function of eustasy and surface uplift rate, Cascadia margin, southern Oregon: Geological Society of America Bulletin, v. 106, p. 840-854.

Kelsey, H.M., Witter, R.C., and Hemphill-Haley, E, 2002, Pl.-boundary earthquakes and tsunamis of the past 5500 yr, Sixes River estuary, southern Oregon: Geological Society of America Bulletin, v. 114, no. 3, p. 298-314.

Kelsey, H.M., Nelson, A.R., Hemphill-Haley, E., Witter, R.C., 2005, Tsunami history of an Oregon coastal lake reveals a 4600 yr record of great earthquakes on the Cascadia subduction zone: GSA Bulletin, v. 117 p 1009-1032.

Leyendecker, E.V., and Frankel, A.D., February 2000, Development of maximum considered earthquake ground motion maps, in Earthquake Spectra, vol. 16, no. 1.

Mitchell, C.E., Vincent, P., Weldon, R.J., III, and Richards, M.A., 1994, Present-day vertical deformation of the Cascadia margin, Pacific Northwest, United States: Journal of Geophysical Research, v. 99, no. B6, p. 12,257-12,277.

Madin, I., 2009, Geologic map of the Oregon City 7.5-minute quadrangle, Clackamas County, Oregon: Oregon Department of Geology and Mineral Industries GMS-119.

Nelson, A.R., and Personius, S.F., 1996, Great-earthquake potential in Oregon and Washington–An overview of recent coastal geologic studies and their bearing on segmentation of Holocene ruptures, central Cascadia subduction zone, in Rogers, A.M., Walsh, T.J., Kockelman, W.J., and Priest, G.R., eds., Assessing earthquake hazards and reducing risk in the Pacific Northwest: U. S. Geological Survey Professional Paper 1560, v. 1, p. 91-114.

Personius, S.F., 1995, Late Quaternary stream incision and uplift in the forearc of the Cascadia subduction zone, western Oregon: Journal of Geophysical Research, v. 100, no. B10, p. 20,193-20,210.

Petersen, M. D., Frankel, A. D., Harmsen, S. C., Mueller, C. S., Haller, K. M., Wheeler, R. L., Wesson, R. L., Zeng, Y, Boyd, O. S., Perkins, D. M., Luco, N., Field, E. H., Wills, C. J., and Rukstales, K. S., 2008, Documentation for the 2008 update of the United States National Seismic Hazard Maps: U. S. Geological Survey Open-File Report 2008-1128.

Satake, K., Shimazaki, K., Tsuji, Y., and Ueda, K., 1996, Time and size of a giant earthquake in Cascadia inferred from Japanese tsunami records of January 1700: Nature, V.379, pp. 246-249, 18 January.

Savage, J.C., Svarc, J.L., Prescott, W.H., and Murray, M.H., 2000, Deformation across the forearc of the Cascadia subduction zone at Cape Blanco, Oregon: Journal of Geophysical Research, v. 105, no. B2, p. 3095-3102.

U. S. Geological Survey, 2008 Interactive Deaggregations, accessed 11/14/14, from USGS website: https://geohazards.usgs.gov/deaggint/2008/

_____,, 2014 National Seismic Hazard Maps – Source Parameters, accessed 11/14/14, from USGS website: http://geohazards.usgs.gov/cfusion/hazfaults_2014_search/query_main.cfm.

Wang, Y., He, J., Dragert, H., and James, T.S., 2001, Three-dimensional viscoelastic interseismic deformation model for the Cascadia subduction zone: Earth, Planets and Space, v. 53, p. 295-306.

Weaver, C.S., and Shedlock, K.M., 1989, Potential subduction, probable intraplate and known crustal earthquake source areas in the Cascadia Subduction Zone: U. S. Geological Survey Open-File Report 89-465, p. 11-26.

Weaver, C.S., and Michaelson, C.A., 1985, Seismicity and volcanism in the Pacific Northwest–Evidence for the segmentation of the Juan de Fuca Pl.: Geophysical Research Letters, v. 12, no. 4, p. 215-218.

Witter, R.C., 1999, Late Holocene Paleoseismicity, tsunamis and relative sea-level changes along the south-central Cascadia subduction zone, southern Oregon: University of Oregon, unpublished Ph.D. dissertation, 178 p.

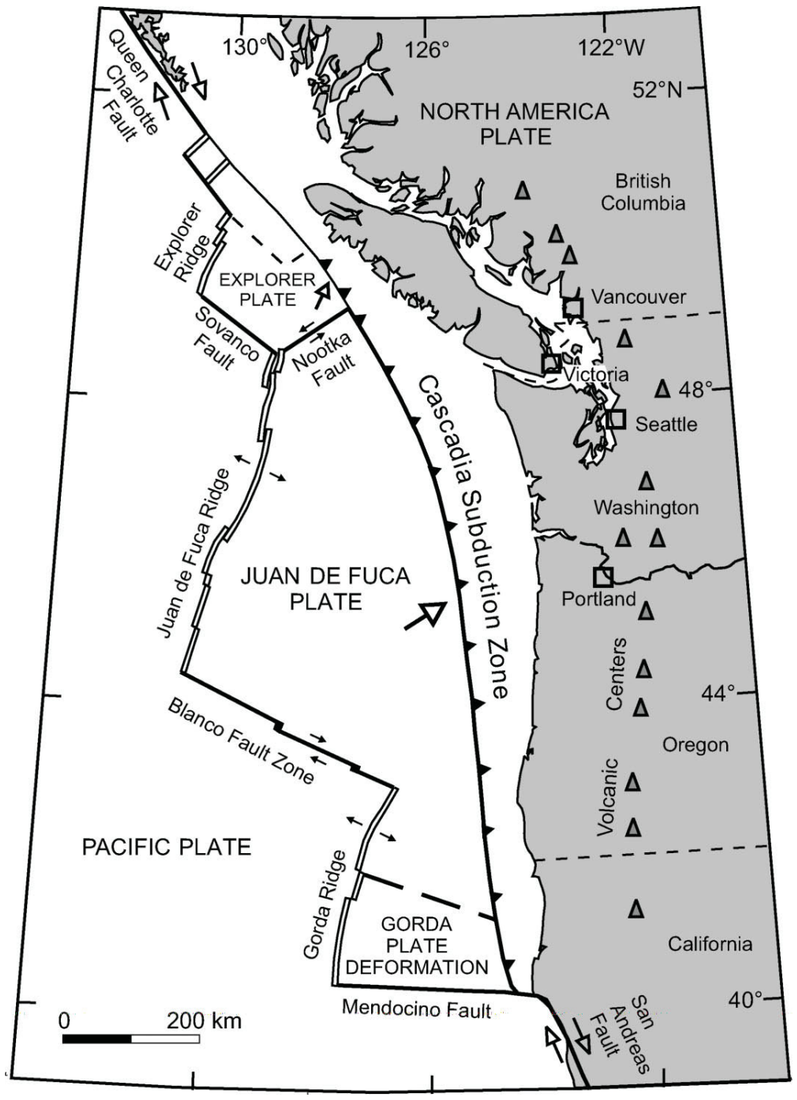
Witter, R.C., Kelsey, H.M., Hemphill-Haley, E., 2003, Great Cascadia earthquakes and tsunamis of the past 6700 years, Coquille River estuary, southern coastal Oregon. Geological Society of America Bulletin 115, 1289–1306.

Wong, I., 2005, Low potential for large intraslab earthquakes in the central Cascadia Subduction Zone: Bulletin of the Seismological Society of America, vol. 95, no. 5.

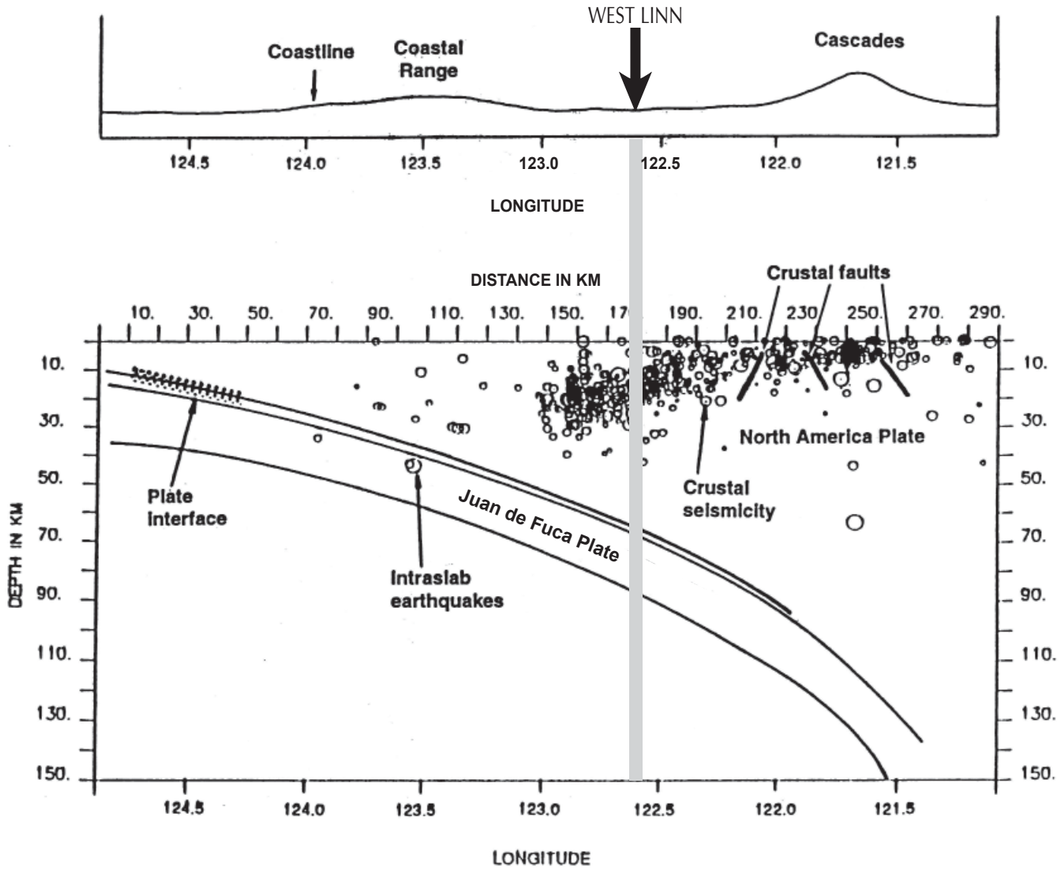
Yeats, R.S., Graven, E.P., Werner, K.S., Goldfinger, C., and Popowski, T., 1991, Tectonics of the Willamette Valley, Oregon: U. S. Geological Survey Open-File Report 91-441-P, 47 p.

Youngs, R.R., Chiou, S.J., Silva, W.J. and Humphrey, J.R., 1997, Strong ground motion attenuation relationships for subduction zone earthquakes: Seismological Research Letters, vol. 68, no. 1, pp. 58-73.

Zhao J.X., Zhang, J., Asano, A., Ohno, Y., Oouchi, T., Takahashi, T., Ogawa, H., Irikura, K., Thio, H., Somerville, P., Fukushima, Y., and Fukushima, Y., 2006, Attenuation relations of strong ground motion in Japan using site classification based on predominant period: Bulletin of the Seismological Society of America, v. 96, p. 898–913.



A) TECTONIC MAP OF PACIFIC NORTHWEST, SHOWING ORIENTATION AND EXTENT OF CASCADIA SUBDUCTION ZONE (MODIFIED FROM DRAGERT AND OTHERS, 1994)

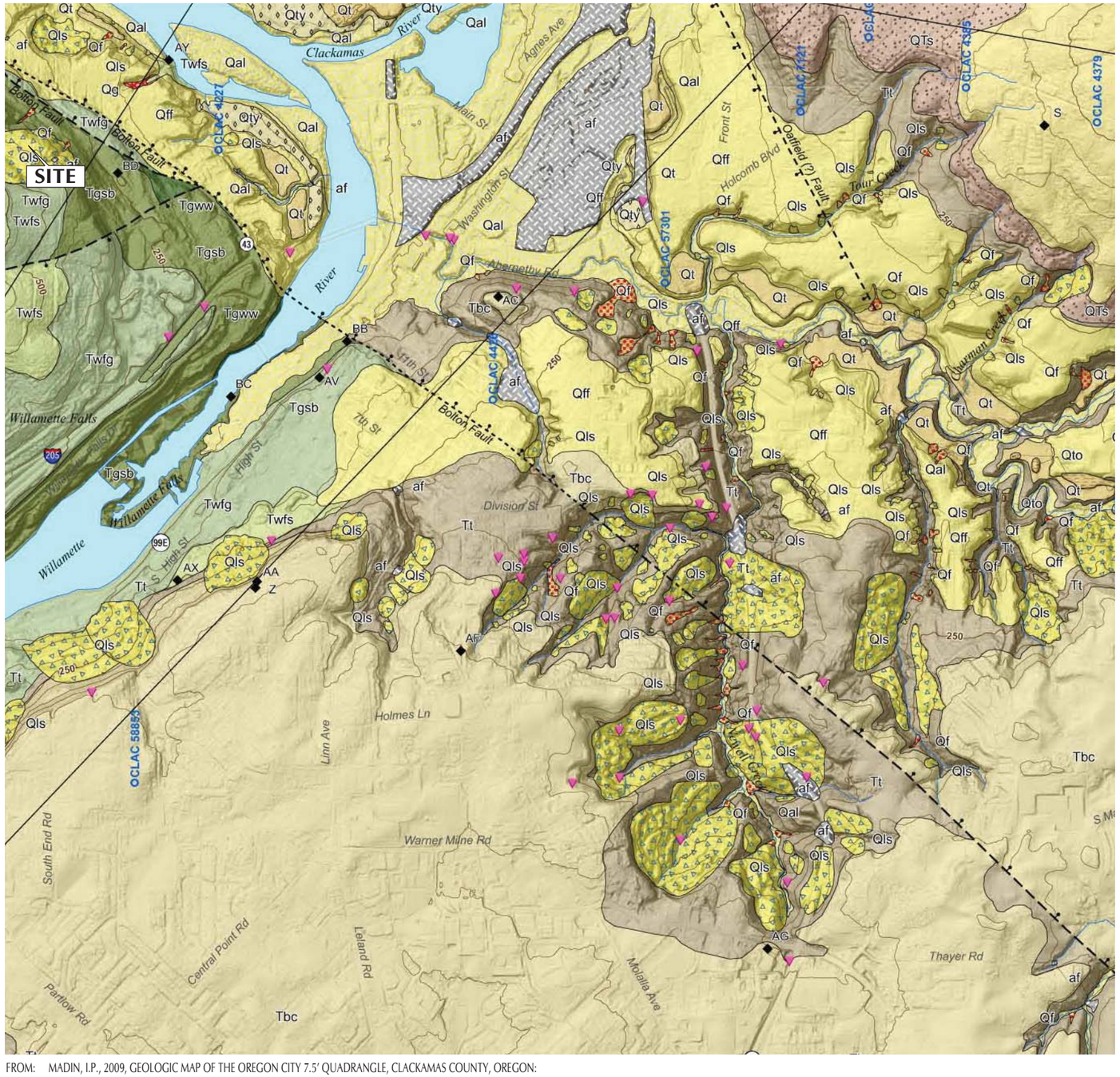


B) EAST-WEST CROSS-SECTION THROUGH WESTERN OREGON AT THE LATITUDE OF PORTLAND, SHOWING THE SEISMIC SOURCES CONSIDERED IN THE SITE-SPECIFIC SEISMIC HAZARD STUDY (MODIFIED FROM GEOMATRIX, 1995)

GRI

TECTONIC SETTING SUMMARY

AUG. 2015 JOB NO. 5338-A FIG. 1C



FROM: MADIN, I.P., 2009, GEOLOGIC MAP OF THE OREGON CITY 7.5' QUADRANGLE, CLACKAMAS COUNTY, OREGON: OREGON DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES GEOLOGIC MAP SERIES 119.

MAP SYMBOLS

- Contact, approximately located
- Normal fault, approximate location
- Normal fault, concealed location
- Normal fault, inferred location
- A—A′ Cross section line
- Water body
- Geochemical sample site, labeled with map code
- Location of water well used to construct cross section, labeled with Oregon Water Resources Department log identification number
- Location of minor debris flow from 1996-1997 storms (Hofmeister, 2000)
- Volcanic vent

EXPLANATION OF MAP UNITS

af — **Artificial fill** — Man-made deposits of mixed clay, silt, sand, gravel, and debris and rubble.

Quaternary Surficial Deposits

Qal — **Alluvial deposits (Quaternary)** — Gravel, sand, silt, and clay deposited in the active channels and floodplains of rivers and streams.

Qty — **Terrace deposits, younger (Quaternary)** — Lowest silt and sand (?) deposits capping strath terraces inset into Missoula Flood deposits along Abernethy Creek and the Willamette River near its confluence with the Clackamas River.

Qt — **Terrace deposits (Quaternary)** — Intermediate-elevation silt and sand (?) deposits capping strath terraces inset into Missoula Flood deposits along Abernethy Creek and the Willamette River near its confluence with the Clackamas River.

Qff — **Missoula (Bretz) flood deposits (Quaternary)** — Silt, sand, and minor gravel deposited by floods caused by the repeated failure of the glacial ice dam that impounded glacial Lake Missoula.

Qls — **Landslides (Quaternary)** — Chaotically mixed and deformed masses of rock, colluvium, and soil that have moved downslope in one or more events.

Miocene-Pleistocene fluvial sedimentary rocks

Qg — **Conglomerate (Pleistocene?)** — Pebble to cobble conglomerate exposed in the walls of a small stream canyon in the extreme northwest corner of the map area.

Tt — **Troutdale Formation sandstone, siltstone and mudstone (Miocene-Pliocene)** — Mudstone, claystone, sandstone, and minor conglomerate and tuff.

Miocene Columbia River Basalt Group lavas

Twfs — **Wanapum Basalt, Frenchman Springs Member, basalt of Sand Hollow (Miocene)** — Black medium-grained basalt flows with sparse plagioclase phenocrysts, well developed columnar jointing.

Twfg — **Wanapum Basalt, Frenchman Springs Member, basalt of Gingko (Miocene)** — Black medium-grained basalt flows with abundant plagioclase phenocrysts, well developed columnar jointing.

Tgww — **Grande Ronde Formation, basalt of Winter Water (Miocene)** — Flow or flows of fine-grained basalt.

North

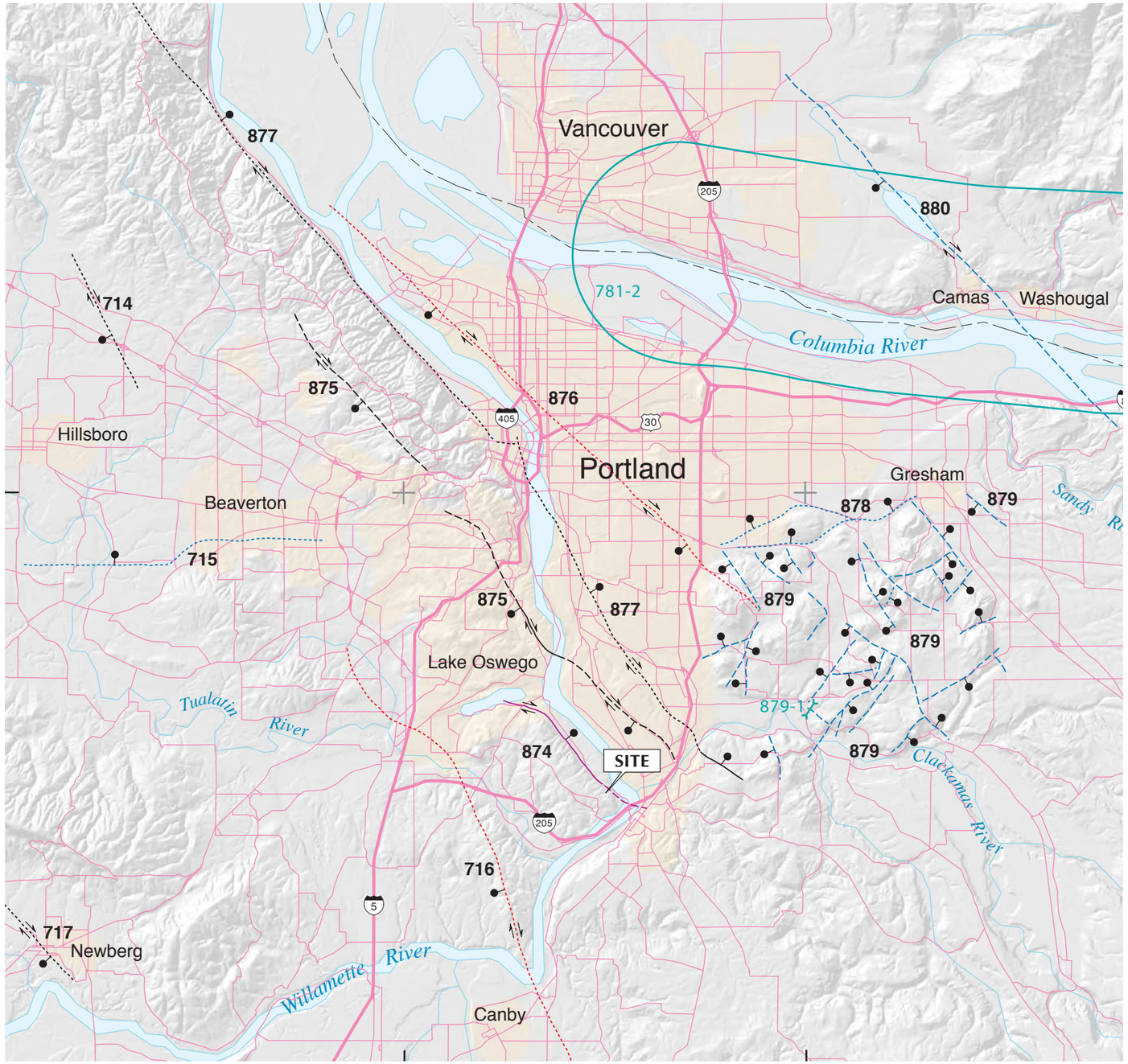
0 1/2 1 MILE

0 2,000 4,000 FT

GRI

GEOLOGIC MAP

AUG. 2015 JOB NO. 5338-A FIG. 2C



MAP EXPLANATION

TIME OF MOST RECENT SURFACE RUPTURE

- Holocene (<10,000 years) or post last glaciation (<15,000 years; 15 ka); no historic ruptures in Oregon to date
- Late Quaternary (<130,000; post penultimate glaciation)
- Late and middle Quaternary (<750,000 years; 750 ka)
- Quaternary, undifferentiated (<1,600,000 years; <1.6 Ma)
- Class B structure (age or origin uncertain)

SLIP RATE

- >5 mm/year
- 1.0-5.0 mm/year
- 0.2-1.0 mm/year
- <0.2 mm/year

TRACE

- Mostly continuous at map scale
- Mostly discontinuous at map scale
- Inferred or concealed

STRUCTURE TYPE AND RELATED FEATURES

- Normal or high-angle reverse fault
- Strike-slip fault
- Thrust fault
- Anticlinal fold
- Synclinal fold
- Monoclinal fold
- Plunge direction of fold
- Fault section marker

DETAILED STUDY SITES

- 731-2 Trench site
- 781-2 Subduction zone study site

CULTURAL AND GEOGRAPHIC FEATURES

- Divided highway
- Primary or secondary road
- Permanent river or stream
- Intermittent river or stream
- Permanent or intermittent lake

FAULT NUMBER	NAME OF STRUCTURE
716	CANBY-MOLALLA FAULT
874	BOLTON FAULT
875	OATFIELD FAULT
877	PORTLAND HILLS FAULT
879	DAMASCUS-TICKLE CREEK FAULT ZONE

FROM: PERSONIUS, S.F., AND OTHERS, 2003, MAP OF QUATERNARY FAULTS AND FOLDS IN OREGON, USGS OPEN FILE REPORT OFR-03-095.

North

0 5 10 MILES

0 5 10 KILOMETERS

GRI

LOCAL FAULT MAP

AUG. 2015 JOB NO. 5338-A FIG. 3C

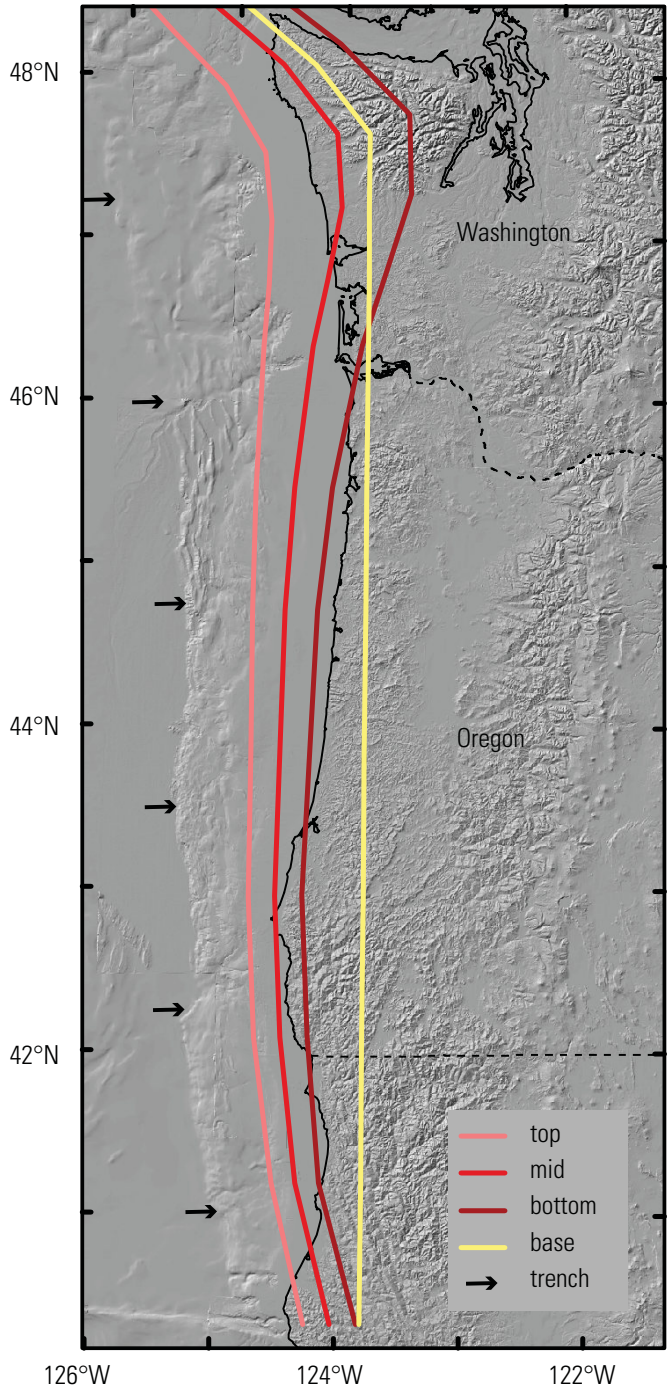


FIGURE 21. LOCATION OF THE EASTERN EDGE OF EARTHQUAKE RUPTURE ZONES ON THE CASCADIA SUBDUCTION ZONE FOR THE VARIOUS MODELS USED IN THIS STUDY RELATIVE TO THE SURFICIAL EXPRESSION OF THE TRENCH: TOP, BASE OF THE ELASTIC ZONE; MID, MIDPOINT OF THE TRANSITION ZONE; BOTTOM, BASE OF THE TRANSITION ZONES; BASE, BASE OF THE MODEL THAT ASSUMES RUPTURES EXTEND TO ABOUT 30-KILOMETERS DEPTH. FIGURE PROVIDED BY RAY WELDON.

FROM: PETERSEN, MD, FRANKEL, AD, HARMSEN, SC, AND OTHERS, 2008, DOCUMENTATION FOR THE 2008 UPDATE OF THE UNITED STATES NATIONAL SEISMIC HAZARD MAPS: US GEOLOGICAL SURVEY, OPEN FILE REPORT 2008-1128

GRI

ASSUMED RUPTURE LOCATIONS
(CASCADIA SUBDUCTION ZONE)

AUG. 2015 JOB NO. 5338-A FIG. 4C

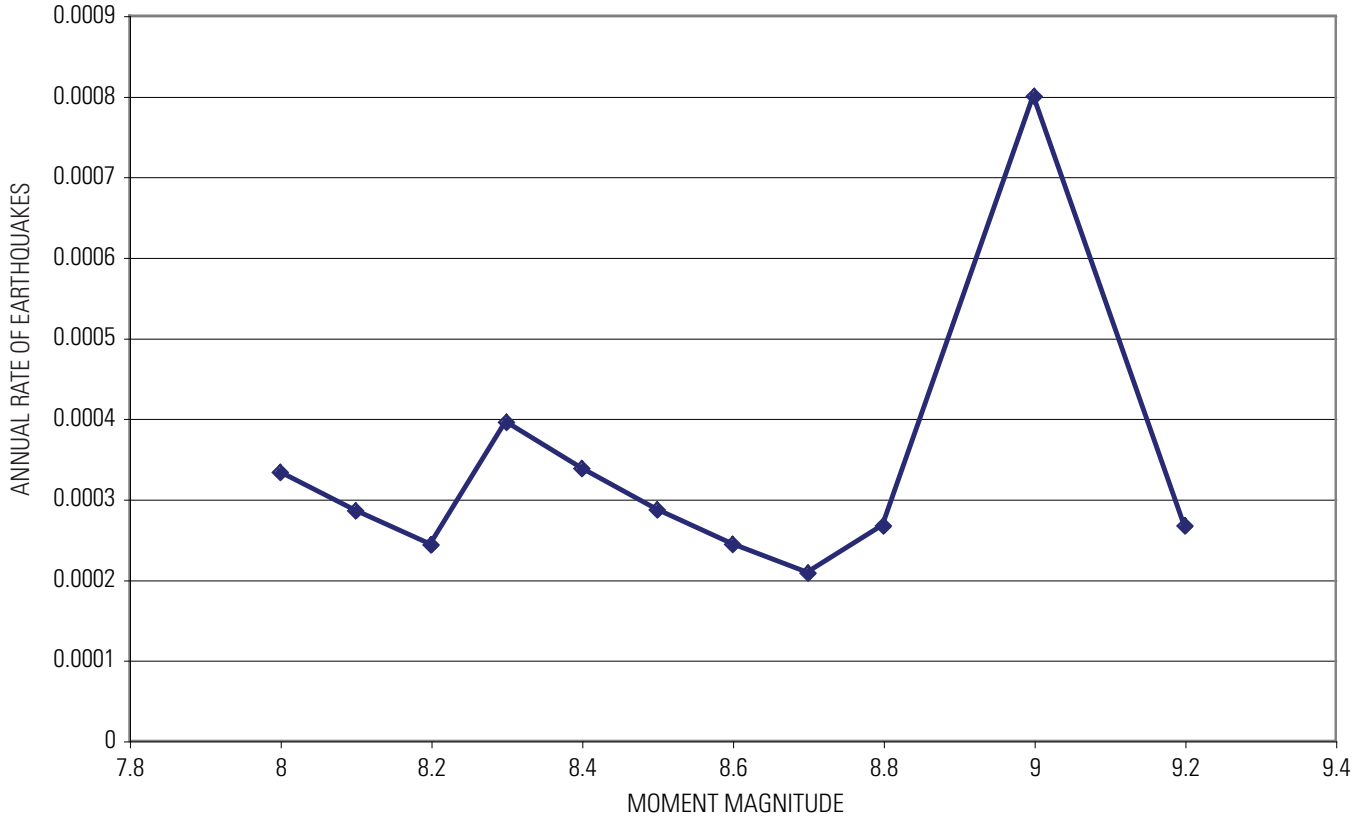


Figure 22. Magnitude-frequency distribution of the Cascadia subduction zone.

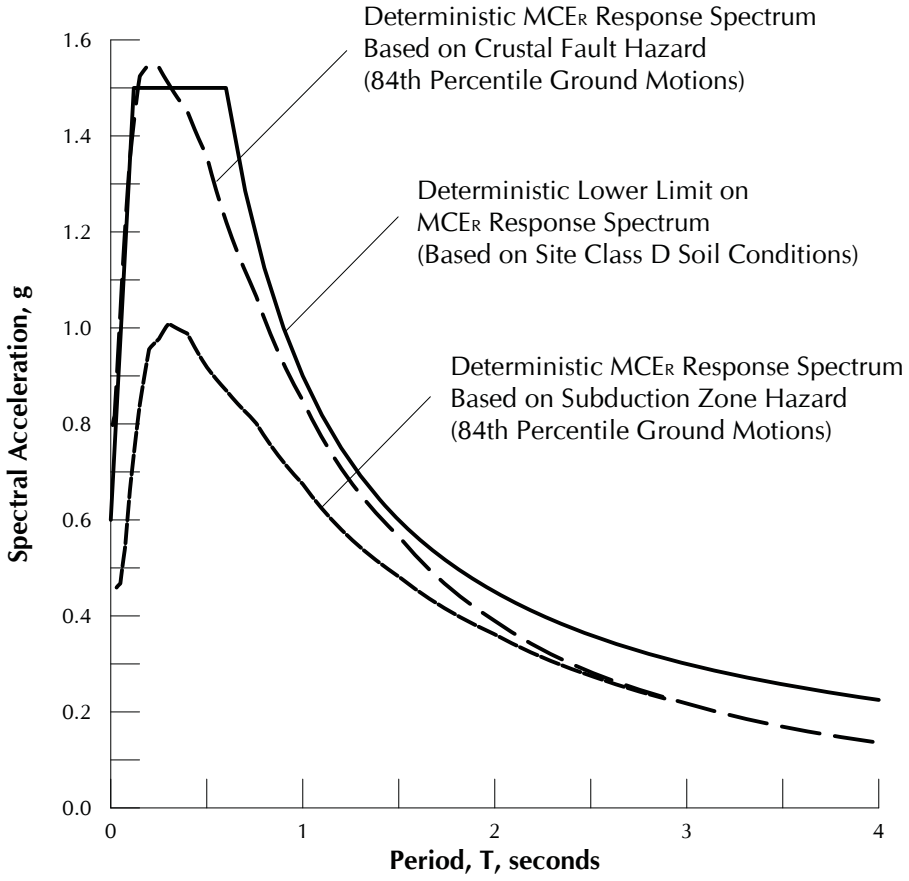
FROM: PETERSEN, M, FRANKEL, A, HARMSEN, S, AND OTHERS, 2008, DOCUMENTATION FOR THE 2008 UPDATE OF THE UNITED STATES NATIONAL SEISMIC HAZARD MAPS: US GEOLOGICAL SURVEY, OPEN FILE REPORT 2008-1128

GRI

ASSUMED MAGNITUDE-FREQUENCY DISTRIBUTION

(CASCADIA SUBDUCTION ZONE)

AUG. 2015 JOB NO. 5338-A FIG. 5C

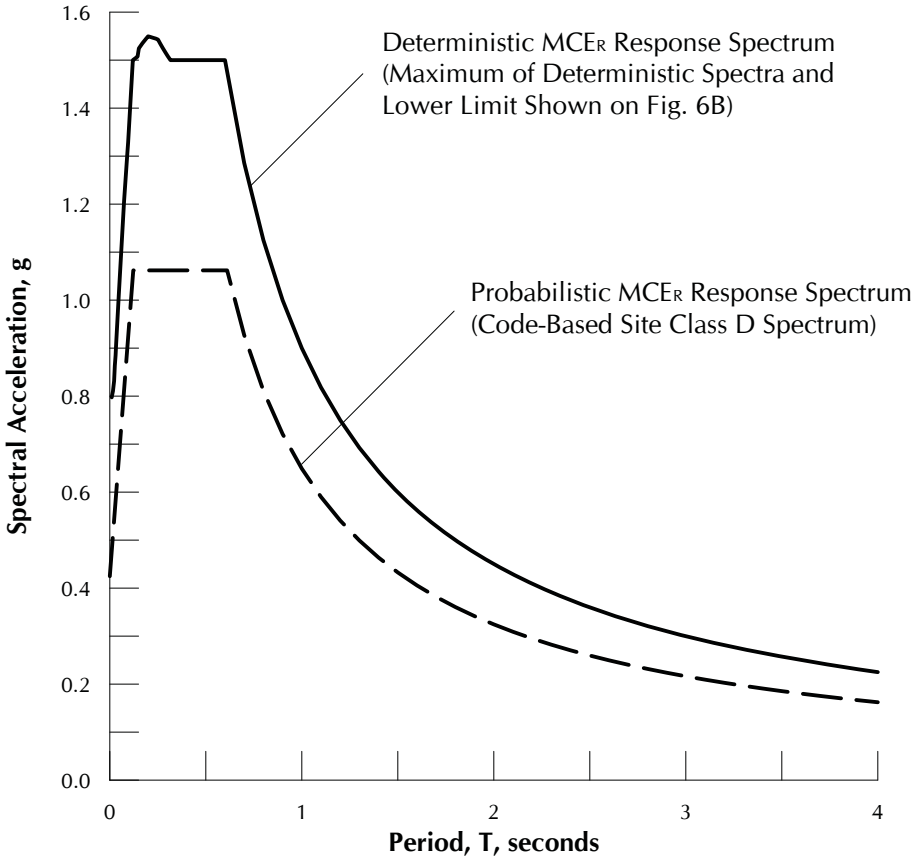


GRI

DETERMINISTIC MCER RESPONSE SPECTRA

(5% DAMPING)

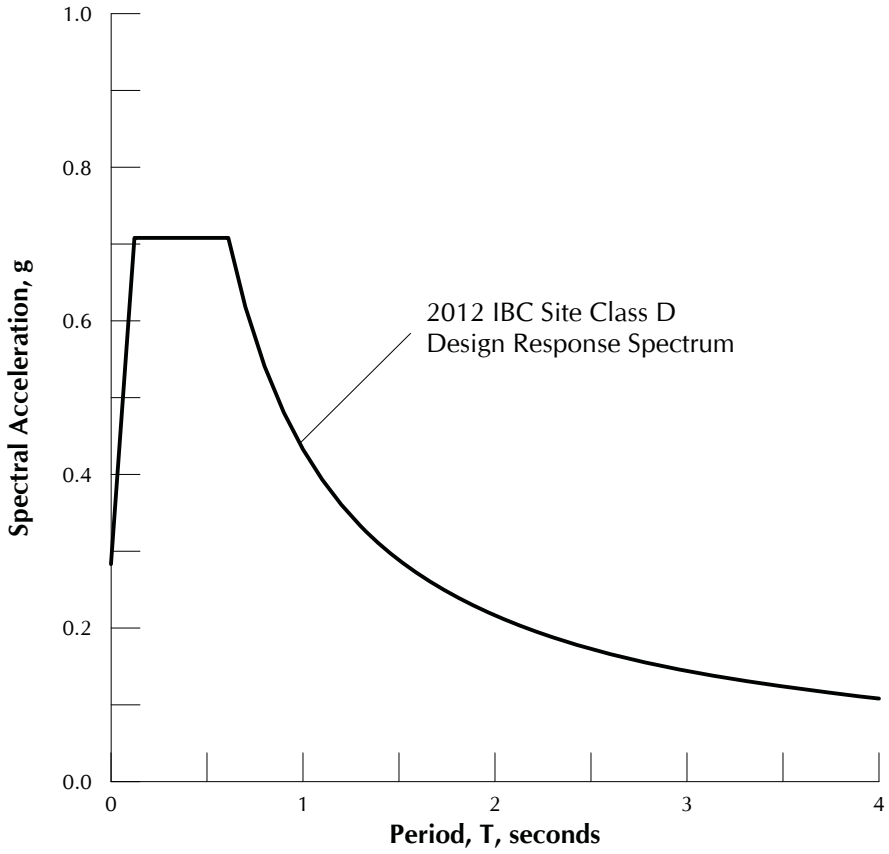
AUG. 2015 JOB NO. 5338-A FIG. 6C



GRI

PROBABILISTIC AND DETERMINISTIC MCER
RESPONSE SPECTRA COMPARISON
(5% DAMPING)

AUG. 2015 JOB NO. 5338-A FIG. 7C



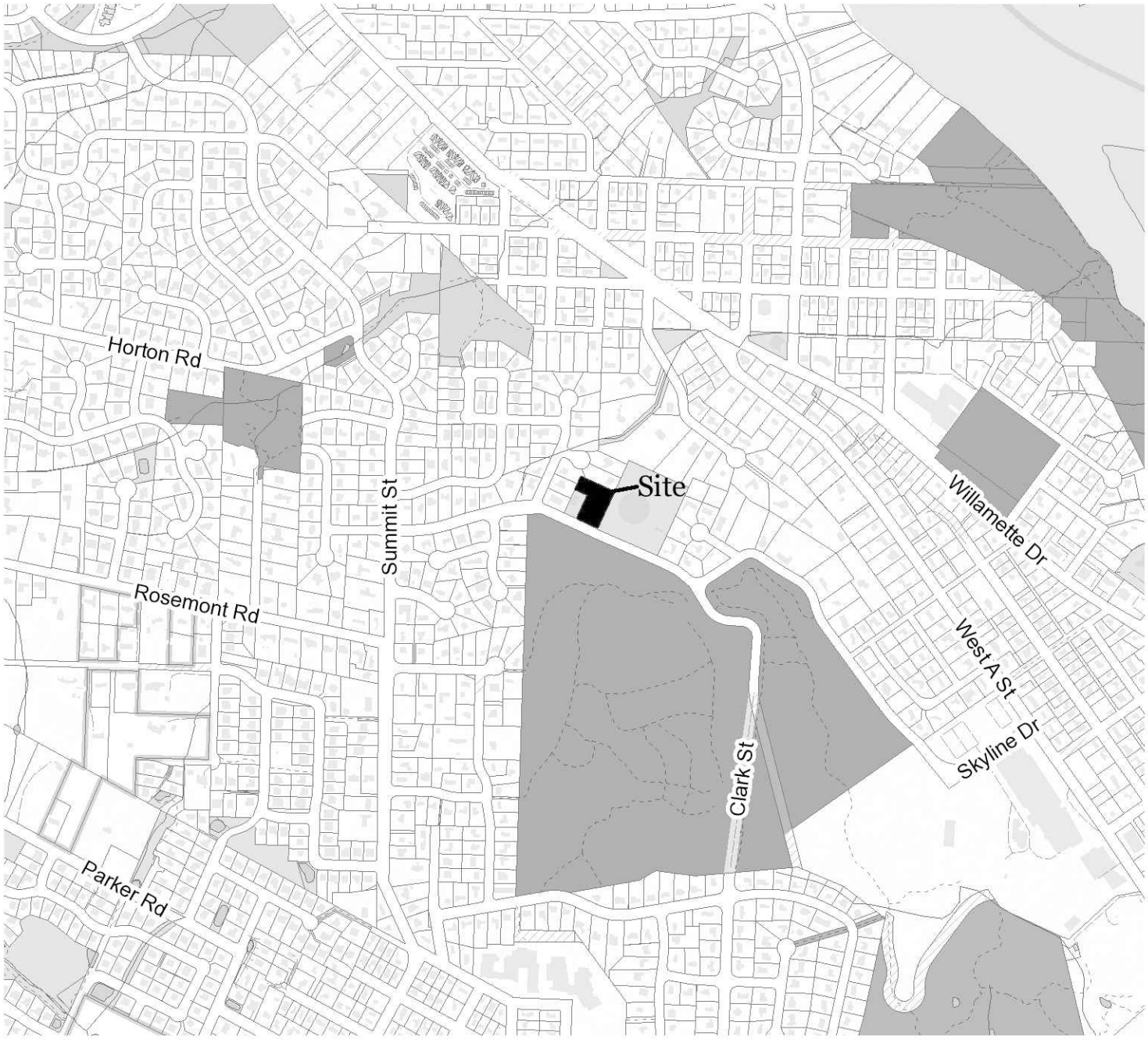
GRI

DESIGN RESPONSE SPECTRUM

(5% DAMPING)

AUG. 2015 JOB NO. 5338-A FIG. 8C

Vicinity Map
6123 Skyline Drive



EXISTING CONDITIONS MAP

TAX LOT 9900, MAP 2S, 1E, 25AD
LOCATED IN THE N.E. 1/4 SECTION 25, T.2S., R.1E., W.M.,
CITY OF WEST LINN, CLACKAMAS COUNTY, OREGON
AUGUST 29, 2018 SCALE 1"=20'

SURVEY NOTES:

THE DATUM FOR THIS SURVEY IS BASED UPON A STATIC GPS OBSERVATION OF LOCAL CONTROL POINTS, PROCESSED THROUGH OPUS. DATUM IS NAVD 88.

A TRIMBLE S6-SERIES ROBOTIC INSTRUMENT WAS USED TO COMPLETE A CLOSED LOOP FIELD TRAVERSE.

THE BASIS OF BEARINGS FOR THIS SURVEY IS PER MONUMENTS FOUND AND HELD PER PARTITION PLAT NO. 1996-115, RECORDS OF CLACKAMAS COUNTY.

THE PURPOSE OF THIS SURVEY IS TO RESOLVE AND DETERMINE THE PERIMETER BOUNDARY OF THE SUBJECT PROPERTY, TO SHOW ALL PERTINENT BOUNDARY ISSUES AND ENCROACHMENTS. NO PROPERTY CORNERS WERE SET IN THIS SURVEY.

NO WARRANTIES ARE MADE AS TO MATTERS OF UNWRITTEN TITLE, SUCH AS ADVERSE POSSESSION, ESTOPPEL, ACQUIESCENCE, ETC.

NO TITLE REPORT WAS SUPPLIED OR USED IN THE PREPARATION OF THIS MAP.

THE UNDERGROUND UTILITIES AS SHOWN ON THIS MAP HAVE BEEN LOCATED FROM FIELD SURVEY OF ABOVE GROUND STRUCTURES AND AS MARKED BY OTHERS. THE SURVEYOR MAKES NO GUARANTEE THAT THE UNDERGROUND UTILITIES SHOWN COMPRISE ALL SUCH UTILITIES IN THE AREA, EITHER IN SERVICE OR ABANDONED. THE SURVEYOR FURTHER DOES NOT WARRANT THAT THE UNDERGROUND UTILITIES ARE IN THE EXACT LOCATION INDICATED, ALTHOUGH HE DOES CERTIFY THAT THEY ARE LOCATED AS ACCURATELY AS POSSIBLE FROM INFORMATION AVAILABLE. THE SURVEYOR HAS NOT PHYSICALLY LOCATED THE UNDERGROUND UTILITIES. SUBSURFACE AND ENVIRONMENTAL CONDITIONS WERE NOT EXAMINED OR CONSIDERED AS A PART OF THIS SURVEY. NO STATEMENT IS MADE CONCERNING THE EXISTENCE OF UNDERGROUND OR OVERHEAD CONTAINERS OR FACILITIES THAT MAY AFFECT THE USE OR DEVELOPMENT OF THIS TRACT. THIS SURVEY DOES NOT CONSTITUTE A TITLE SEARCH BY SURVEYOR.

LEGEND:

Some Symbols shown may not be used on map

- DECIDUOUS TREE
- EVERGREEN TREE
- STORM SEWER MANHOLE
- CATCH BASIN
- SANITARY SEWER CLEANOUT
- SANITARY SEWER MANHOLE
- WATER VALVE
- WATER METER
- FIRE HYDRANT
- GAS VALVE
- GAS METER
- BOLLARD
- SIGN
- MAILBOX
- COMMUNICATIONS PEDESTAL
- COMMUNICATIONS MANHOLE
- COMMUNICATIONS BOX
- STORM OUTFALL
- FOUND MONUMENT
- DOWN SPOUT TO STORM SYSTEM
- UTILITY AND LIGHT POLE
- UTILITY POLE
- LIGHT POLE
- GUY WIRE
- ELECTRIC BOX
- ELECTRIC METER
- ELECTRICAL POWER PEDESTAL
- ELECTRIC RISER
- HEAT PUMP
- OVERHEAD LINE
- GAS LINE
- ELECTRICAL LINE
- COMMUNICATIONS LINE
- SANITARY SEWER LINE
- STORM DRAIN LINE
- WATER LINE
- FENCELINE
- UTILITY RISER
- DOWN SPOUT TO SPLASH GUARD/GROUND

FD = FOUND
FI = FIR TREE
PI = PINE TREE
CE = CEDAR TREE
IR = IRON ROD
YPC = YELLOW PLASTIC CAP
DE = DECIDUOUS TREE

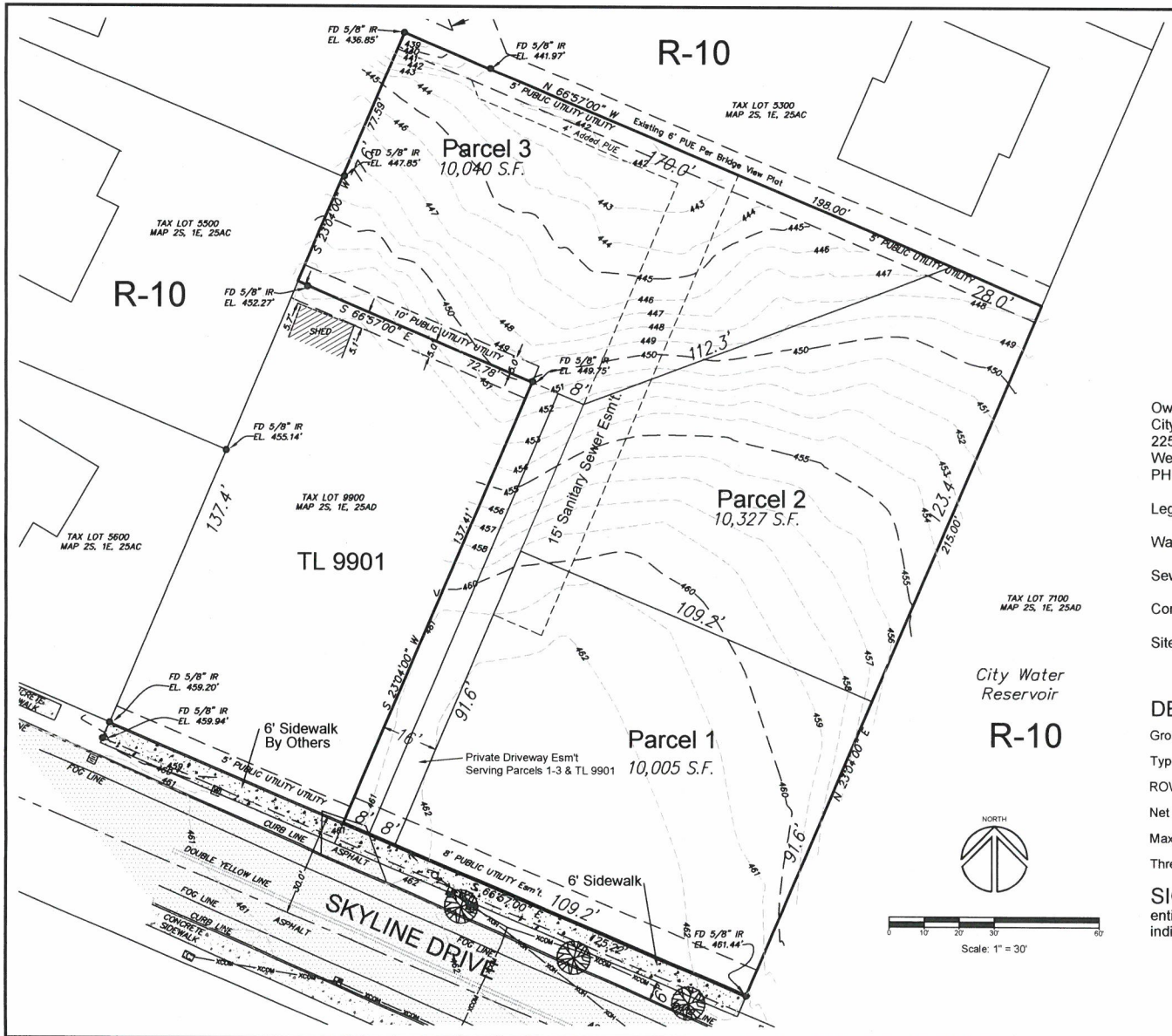
GRAPHIC SCALE
20 0 10 20 40
(IN FEET)
1 INCH = 20 FT.

SIGNED ON: 30/AUG/18

REGISTERED PROFESSIONAL LAND SURVEYOR
OREGON
JULY 13, 2004
TOBY G. BOLDEN
60377LS
RENEWS: DECEMBER 31, 2019

CENTERLINE CONCEPTS
LAND SURVEYING, INC.
19376 MOLALLA AVE., SUITE 120
OREGON CITY, OREGON 97045
PHONE 503.650.0188 FAX 503.650.0189

Plotted: M: \PROJECTS\ICON-SKYLINE DR-6123\dwg\ECM.dwg



RECEIVED
DEC 13 2018
PLANNING & BUILDING
CITY OF WEST LINN
INT. TIME

Owner:
City of West Linn
22500 Salamo Road
West Linn, OR 97068
PH: (503) 657-0331

Engineer:
Theta Engineering
PO Box 1345
Lake Oswego, OR 97035
PH: (503) 481-8822

Legal: 2S 1E 25AD 9900

Zoning: R-10

Water: City of West Linn

Sewer: City of West Linn

Contours: Centerline Concepts, Inc.

Site Area: 32,569 S.F.

Street Tree.
Red Sunset Maple

DENSITY CALCULATIONS:

Gross Site Area: 32,569 square feet.

Type I & II Lands: 312 square feet.

ROW Dedication & Flag Strip 2,199 sq. ft.

Net Site Type III & IV Area: 30,058 sq. ft.

Maximum Density @ 1 Unit Per 10,000 sq. ft. = 3 lots.

Three lots proposed. Both the minimum and maximum density standards are met.

SIGNIFICANT TREES: The City Arborist has determined that the entire grove of trees is significant, but expressed concern about trying to keep individual trees because of threat of windthrow once the grove is disturbed.

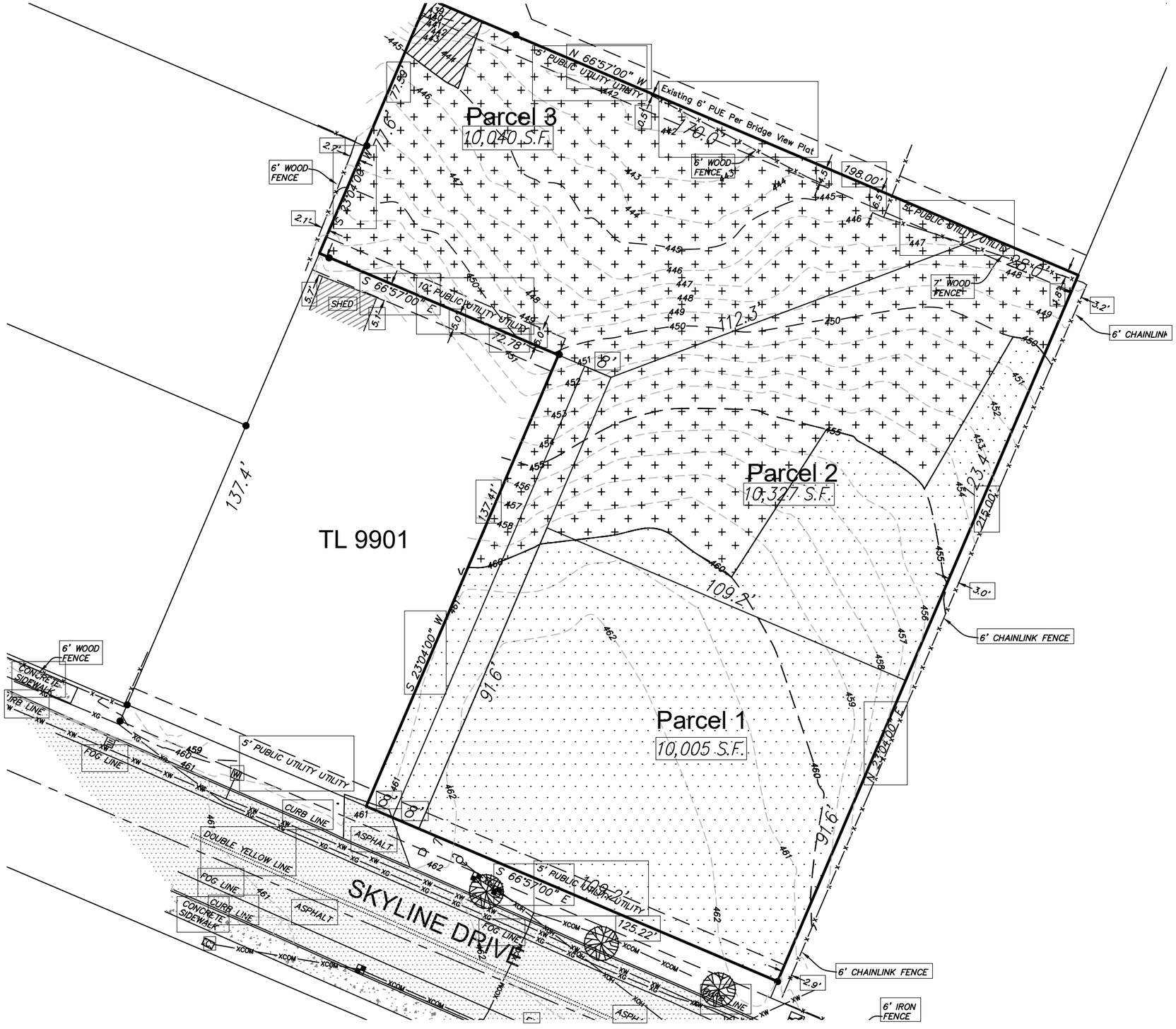
DESIGNED:	R.E.G.	12-13-18	1	Added off-site sidewalk by others.
DRAWN:	R.E.G.			
SCALE:	1" = 30'			
DATE:	September 2018			
FILE:	18-ICN-102	DATE	NO.	REVISION

Richard E. Givens, Planning Consultant
18680 Sunblaze Dr.
Oregon City, OR 97045
PH: (503) 479-0097

APPLICANT: Icon Construction & Development, LLC
1980 Willamette Falls Drive, Suite 200
West Linn, OR 97068
PH: (503) 657-0406

Partition Site Plan

6123 Skyline Drive, West Linn, OR



Slopes 0-10% (Type IV Land)
15,103 sq. ft. (46.3% of site)

Slopes 10-25% (Type III Land)
17,155 sq. ft. (52.7% of site)

Slopes 25-35% (Type II Land)
312 sq. ft. (1.0% of site)

Slopes 35% Plus (Type I Land
0 sq. ft. (0% of site)

NORTH

0 10' 20' 30' 60'

Scale: 1" = 30'

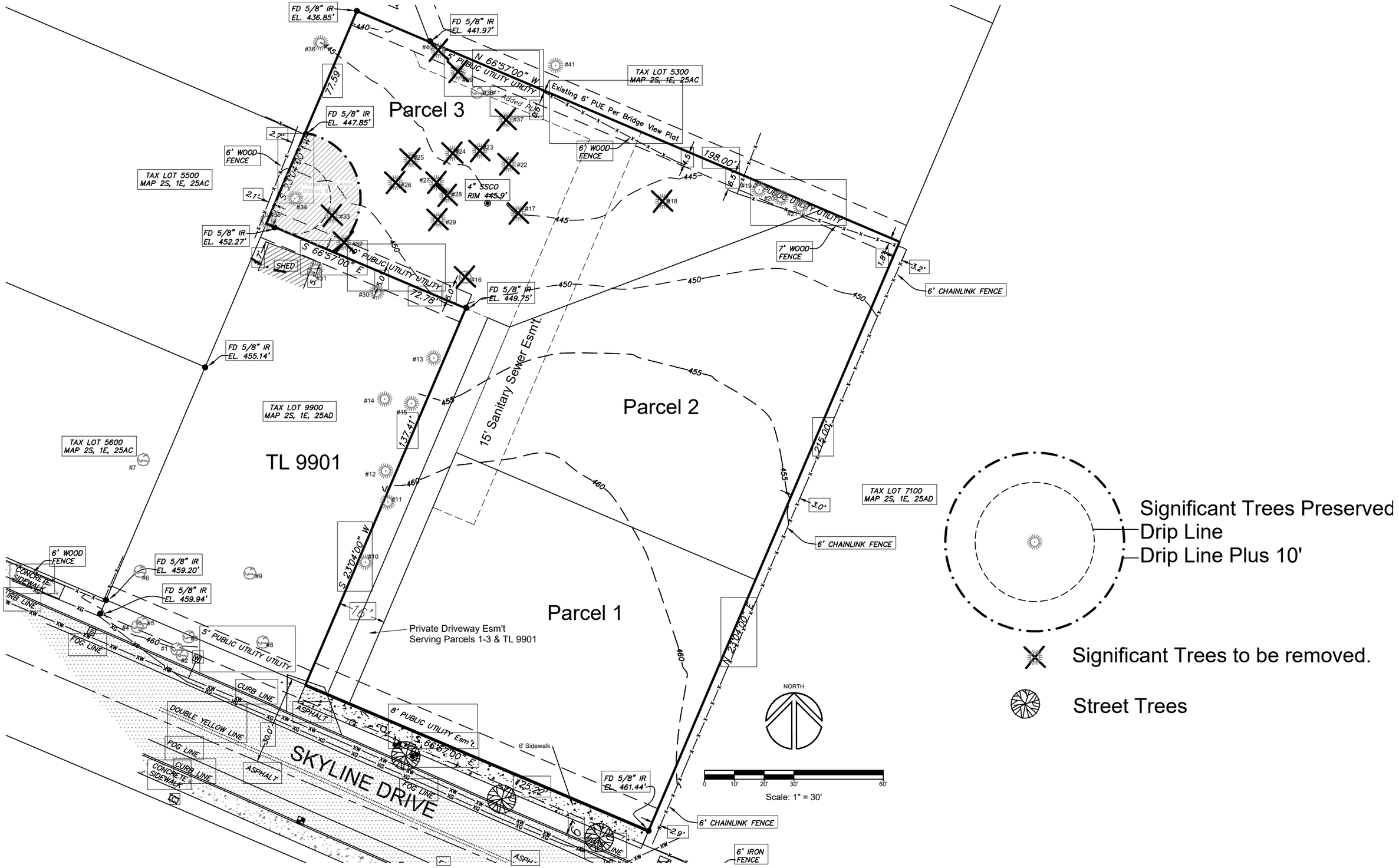
DESIGNED:	R.E.G.				
DRAWN:	R.E.G.				
SCALE:	1" = 30'				
DATE:	September 2018				
FILE:	18-ICN-102	DATE	NO.	REVISION	

Richard E. Givens, Planning Consultant
18680 Sunblaze Dr.
Oregon City, OR 97045
PH: (503) 479-0097

APPLICANT: Icon Construction & Development, LLC
1980 Willamette Falls Drive, Suite 200
West Linn, OR 97068
PH: (503) 657-0406

Partition Slope Analysis

6123 Skyline Drive, West Linn, OR



DESIGNED:	R.E.G.				
DRAWN:	R.E.G.				
SCALE:	1" = 30'				
DATE:	September 2018				
FILE:	18-ICN-102	DATE	NO.	REVISION	

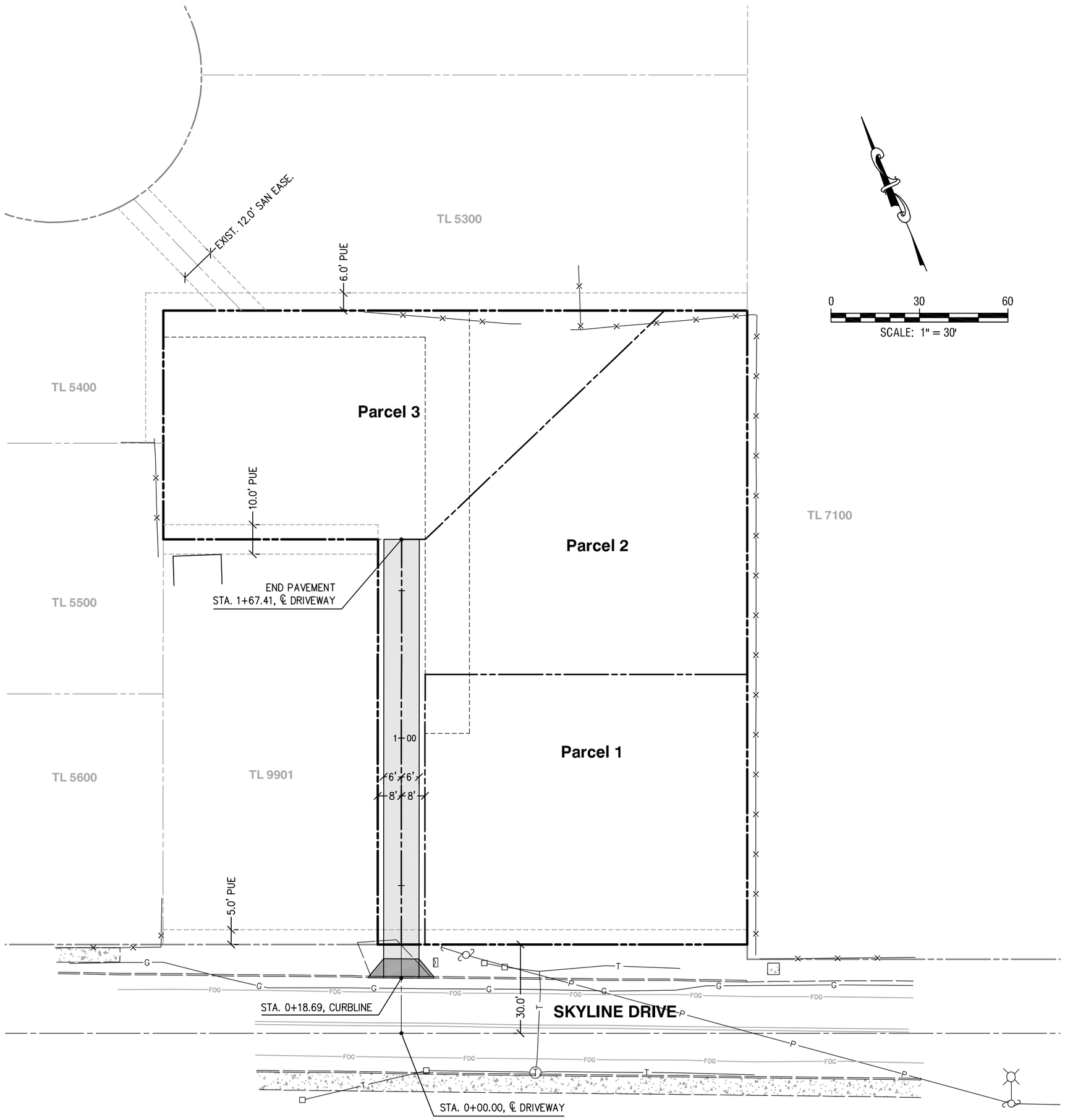
Richard E. Givens, Planning Consultant
18680 Sunblaze Dr.
Oregon City, OR 97045
PH: (503) 479-0097

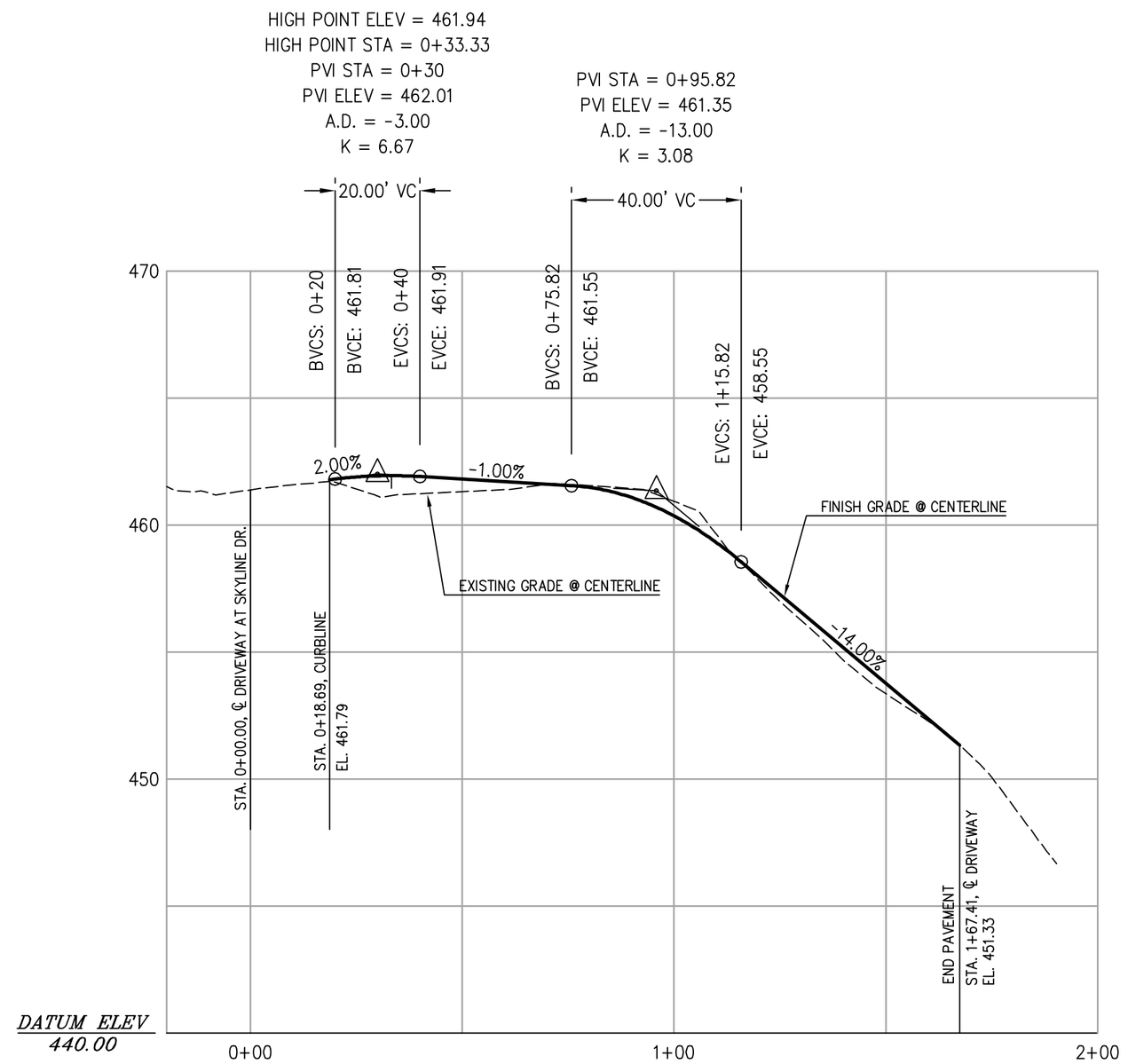
APPLICANT: Icon Construction & Development, LLC
1980 Willamette Falls Drive, Suite 200
West Linn, OR 97068
PH: (503) 657-0406

Partition Tree Plan

6123 Skyline Drive, West Linn, OR

SHEET: 3/3





DRIVEWAY CENTERLINE PROFILE 1/1

SCALE: 1" = 30' HORIZONTAL
1" = 5' VERTICAL

2014-129U

PRELIMINARY STREET PLAN

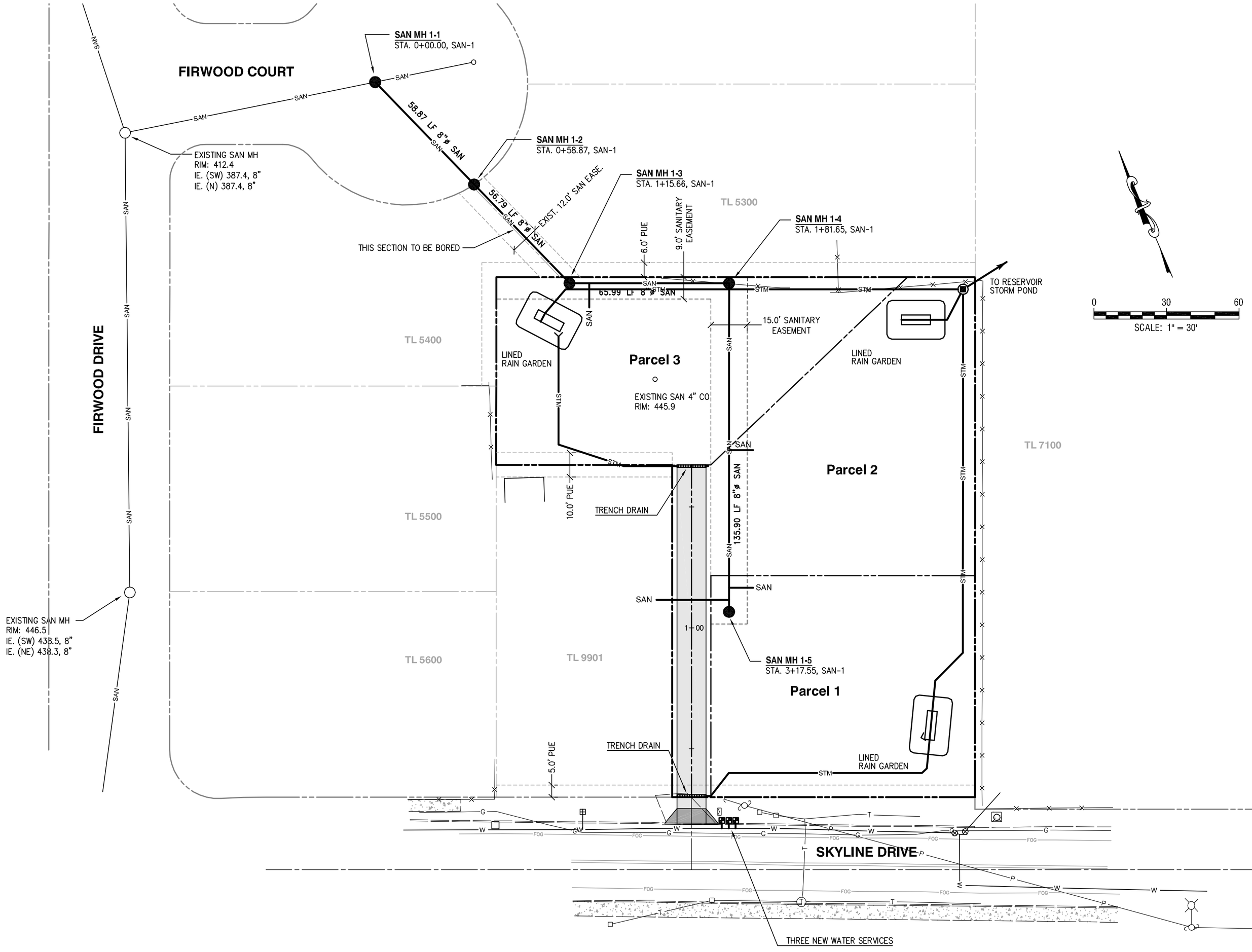
DESIGNED:	BDG			
DRAWN:	BJS			
SCALE:	As Noted			
DATE:	September, 2018			
FILE:	Skyline Partition Prelim1	DATE	NO.	REVISION

Theta, llc
ENGINEERING - SURVEYING - PLANNING
PO Box 1345
Lake Oswego, Oregon 97035
503/481-8822
email: thetaeng@comcast.net

Icon Construction and Developement
1380 Willamette Falls Drive, No. 200
West Linn, Oregon 97068

Partition Site Plan
6123 Skyline Drive
West Linn, Oregon

SHEET: 1/4



PRELIMINARY UTILITY PLAN

2014-129U

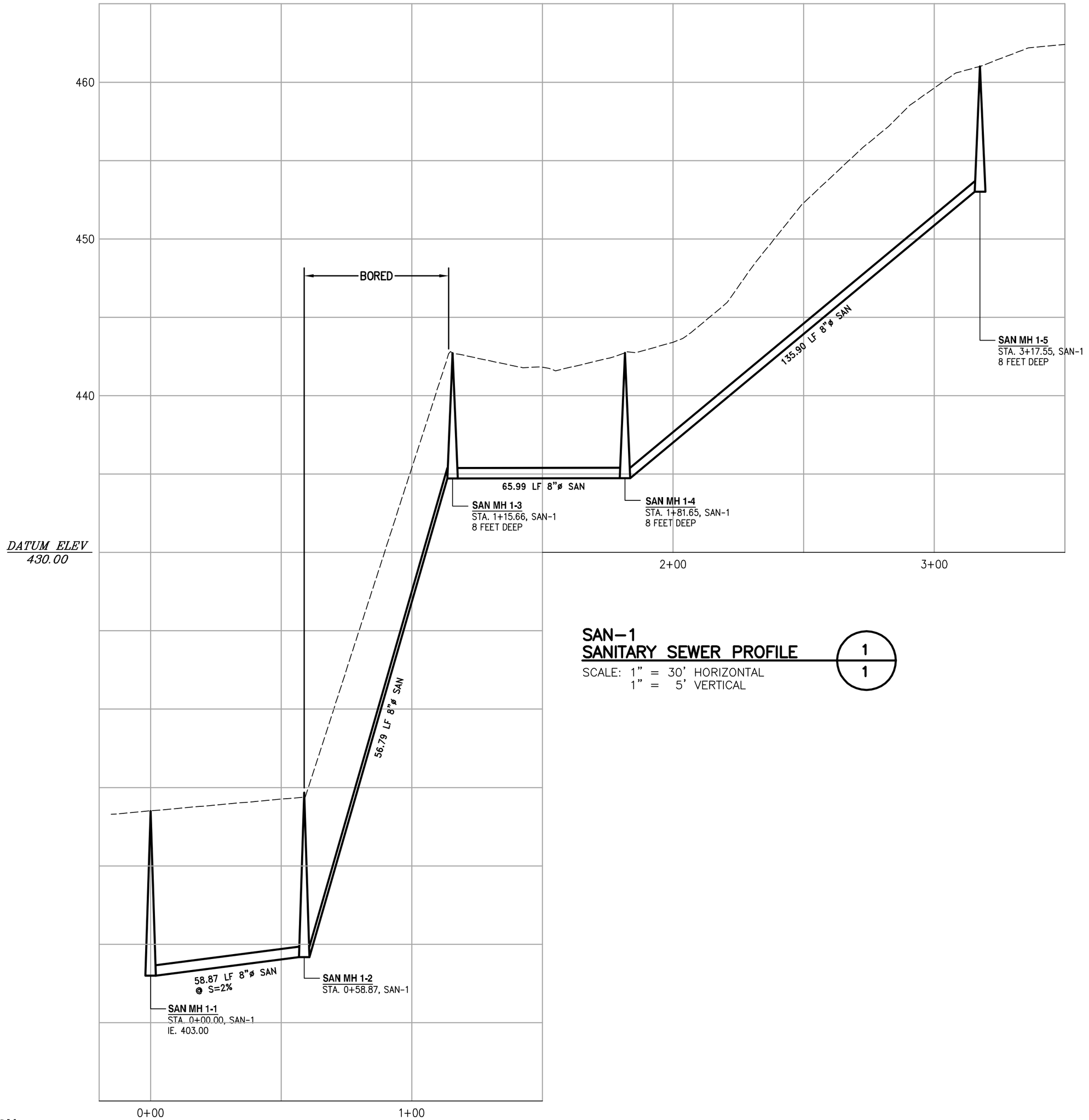
DESIGNED:	BDG			
DRAWN:	BJS			
SCALE:	As Noted			
DATE:	September, 2018			
FILE:	Skyline Partition Prelim1	DATE	NO.	REVISION

Theta, llc
ENGINEERING - SURVEYING - PLANNING
PO Box 1345
Lake Oswego, Oregon 97035
503/481-8822
email: thetaeng@comcast.net

Icon Construction and Developement
1380 Willamette Falls Drive, No. 200
West Linn, Oregon 97068

Partition Site Plan
6123 Skyline Drive
West Linn, Oregon

SHEET: 2/4



SAN-1
SANITARY SEWER PROFILE (1/1)
SCALE: 1" = 30' HORIZONTAL
1" = 5' VERTICAL

2014-129U

PRELIMINARY UTILITY PROFILES

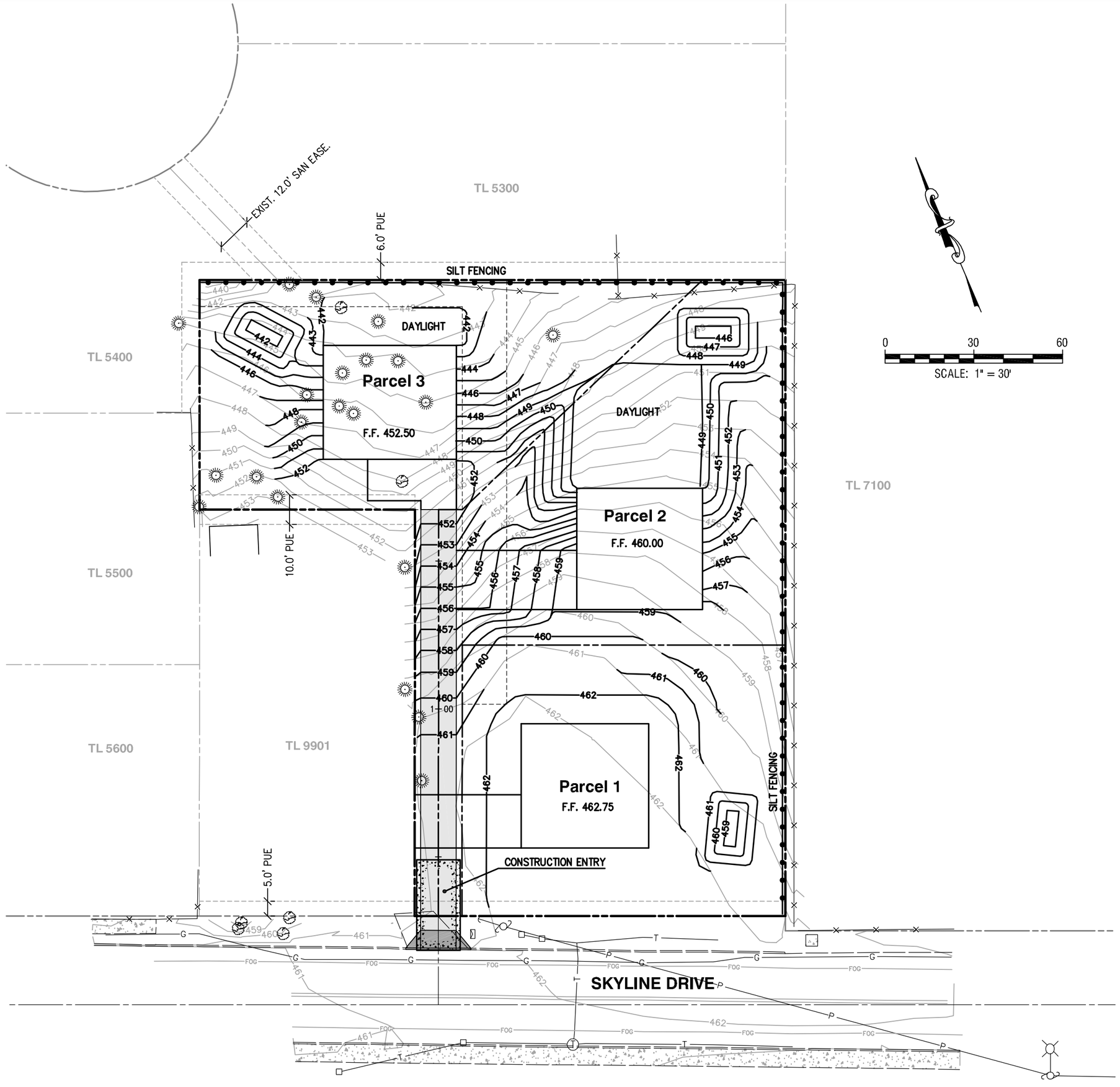
DESIGNED:	BDG			
DRAWN:	BJS			
SCALE:	As Noted			
DATE:	September, 2018			
FILE:	Skyline Partition Prelim1	DATE	NO.	REVISION

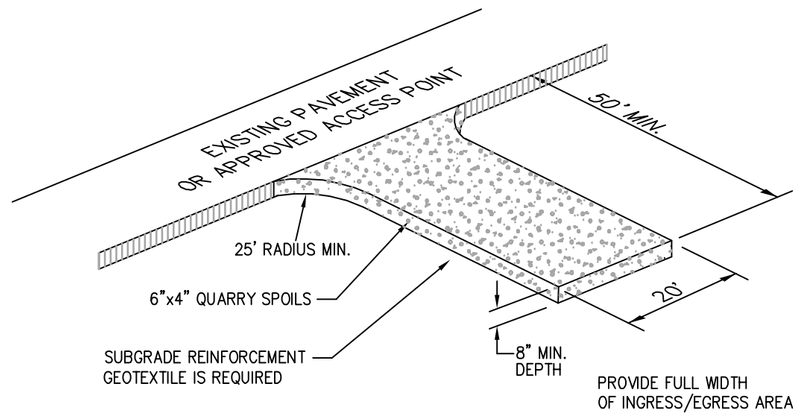
Theta, llc
ENGINEERING - SURVEYING - PLANNING
PO Box 1345
Lake Oswego, Oregon 97035
503/481-8822
email: thetaeng@comcast.net

Icon Construction and Developement
1380 Willamette Falls Drive, No. 200
West Linn, Oregon 97068

Partition Site Plan
6123 Skyline Drive
West Linn, Oregon

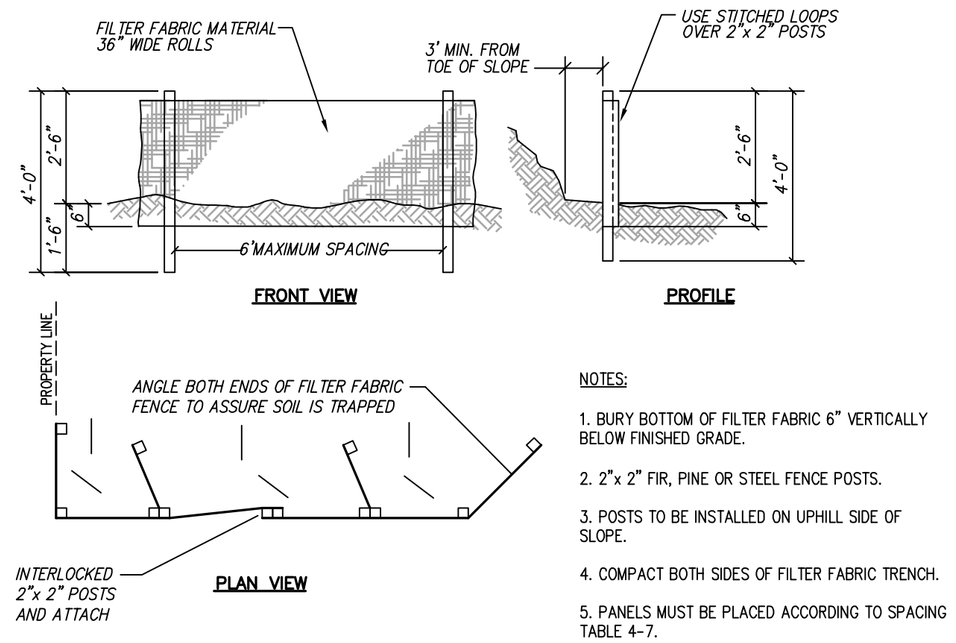
SHEET: 3/4





GRAVEL CONSTRUCTION ENTRANCE

SCALE: NTS



NOTES:

1. BURY BOTTOM OF FILTER FABRIC 6" VERTICALLY BELOW FINISHED GRADE.
2. 2"x 2" FIR, PINE OR STEEL FENCE POSTS.
3. POSTS TO BE INSTALLED ON UPHILL SIDE OF SLOPE.
4. COMPACT BOTH SIDES OF FILTER FABRIC TRENCH.
5. PANELS MUST BE PLACED ACCORDING TO SPACING TABLE 4-7.

SEDIMENT FENCE

SCALE: NTS

NOTES:

PURSUANT TO CDC 85-200 E AND GEOTECHNICAL INVESTIGATION

1. CONCEPTUAL GRADING PLAN ONLY TO DEMONSTRATE FEASIBILITY.
2. ALL CUT AND FILL SLOPES ARE GREATER THAN 2:1.
3. GRADING IS MINIMUM OF 5-FEET FROM PROJECT BOUNDARY.
4. GRI GEOTECHNICAL INVESTIGATION (9-10-15)

"THE SURROUNDING NEIGHBORHOOD WAS ALSO EXAMINED FROM THE PUBLIC RIGHT OF WAY FOR INDICATIONS OF SLOPE MOVEMENT (CRACKED AND SEPARATED SIDEWALKS OR CURBS). THE RECONNAISSANCE DID NOT DISCLOSE OBVIOUS INDICATIONS OF RELATIVELY RECENT MOVEMENT, SUCH AS CRACKED STREETS, SIDEWALKS, OR CURBS. LIMITED INTERVIEWS WITH CITY MAINTENANCE PERSONNEL DID NOT DISCLOSE REPORTS OF BROKEN OR SHEARED UNDERGROUND UTILITIES."

"THE RESERVOIR SITE IS LOCATED ON A VERY LARGE, ANCIENT LANDSLIDE. HOWEVER, RECONNAISSANCE BY GRI AS PART OF THIS STUDY AND DURING OUR 2012 STUDY DID NOT DISCLOSE INDICATIONS OF RECENT LANDSLIDE MOVEMENT. A RECONNAISSANCE RECENTLY COMPLETED BY CORNFORTH CONSULTANTS (DECEMBER 2014) ALSO DID NOT IDENTIFY SIGNS OF ACTIVE MOVEMENT. IT IS OUR OPINION THE RISK OF SIGNIFICANT FUTURE MOVEMENT OF THE LARGE, ANCIENT LANDSLIDE IS LOW."

STORM DRAINAGE NOTES:

1. ABOVE GROUND FACILITIES - RAINGARDENS
2. SIZING OF FACILITIES FOR TOTAL INFILTRATION WIITH NO OUTFLOW.
3. PRIVATE DRIVE IMPERVIOUS AREA TO BE INCLUDED IN A ROOF RAINGARDEN.
4. FINAL SIZING TO BE BASED ON ACTUAL IMPERVIOUS AREA.

PRELIMINARY GRADING AND EROSION CONTROL PLAN

2014-129U

DESIGNED:	BDG			
DRAWN:	BJS			
SCALE:	As Noted			
DATE:	September, 2018	03/12/2019	1	CONCEPTUAL HOUSE GRADING
FILE:	Skyline Partition Prelim1	DATE	NO.	REVISION

Theta, llc
ENGINEERING - SURVEYING - PLANNING
PO Box 1345
Lake Oswego, Oregon 97035
503/481-8822
email: thetaeng@comcast.net

Icon Construction and Developement
1380 Willamette Falls Drive, No. 200
West Linn, Oregon 97068

Partition Site Plan
6123 Skyline Drive
West Linn, Oregon

SHEET: 4/4

PD-2 PUBLIC COMMENT

From:	Phillip Kreiger
To:	Wyss, Darren
Cc:	Krista Kopina
Subject:	Planning Manager Decision File No. MIP-18-05 - Comment/Concerns
Date:	Tuesday, March 5, 2019 4:55:02 PM

Good afternoon Darren,

Following up on the Notice of Upcoming Planning Manager Decision (File No. MIP-18-05) we received in the mail regarding the three-lot partition at 6123 Skyline Drive. We would like to raise a few concerns and request some supplemental information if available. All of the following questions or concerns are being made in reference to the Applicant Submittal and Chapter 85, General Provisions.

1. Please reference 85.160 - E. 5, Submittal Sheet 1/3 titled *Partition Site Plan*, and Sheet 3/3 titled *Partition Tree Plan*. The Partition Tree Plan (3/3) shows complete removal of the grove of trees on Parcel 3 with a note regarding "Significant Trees" on the Partition Site Plan (1/3) indicating the City Arborist "expressed concern about trying to keep individual trees because of threat of windthrow once the grove is disturbed". The grove of significant trees on our lot (Tax Lot 5300) is a continuation of this particular grove proposed to be removed in its entirety. There appears to be no information regarding the assessment of the resulting effect on the remaining grove on our lot. Could you please provide a copy of the arborist report or assurances that the Arborist concerns regarding windthrow does not now transfer to the trees remaining on Tax Lot 5300?
2. Please reference 85.160 – C.3.d. The GRI Geotechnical report dated August 10, 2015 included in the applicant submittal appears to be specifically for the Bolton Reservoir project and does not specifically identify the proposed land use for development of these three parcels. With consideration that our lot is downslope of the proposed development, and our driveway is significantly downslope at a much lower elevation, we are interested in understanding the location of the structure proposed on Parcel 3 and what potential effect that has on the stabilization of the downslope (uphill side of our driveway). Parcel 3 is shown to have a slope of 10-25% per the Partition Slope Analysis Sheet 2/3. Has a geotechnical assessment (for this proposed development) with relation to adjacent properties been performed for this development? If so, please provide a copy of the latest Geotechnical report or assurances that the proposed developments will not have a negative impact on the downslope property during and after the proposed development.
3. Please reference multiple surveys (e.g. Partition Slope Analysis Sheet 2/3) that shows portions of our existing fence located partially into the property line of Parcels 2 & 3. A portion of the fence was existing and built by the previous home owner and appears to conflict with the new Sanitary line. This fence can be removed and reinstalled as necessary. We have a section of new cedar fence that was inadvertently built encroaching into Parcels 2 & 3 at the North-East corner of the proposed development. We would like to understand what the expectations are of us for this issue with the City or developer. Understandably this is an error on my part as I should have surveyed the exact property line prior to constructing the fence in 2015 (Built prior to City surveys) in lieu of following assumed natural property delineation at the time. We are hoping to keep this portion of the fence in place if at all possible due to the recent expense of constructing it and would continue to take all responsibility for maintenance and repairs. Please advise for further discussion or expeced action on this matter.

Thank you for providing the opportunity to comment on this proposed development.

Sincerely,

Phillip & Krista Kreiger
5147 Firwood Ct
West Linn, OR 97068

PD-3 AFFADAVIT AND NOTICE PACKET

AFFIDAVIT OF NOTICE

We, the undersigned do hereby certify that, in the interest of the party (parties) initiating a proposed land use, the following took place on the dates indicated below:

GENERAL
File No. MIP-18-05 Applicant's Name Icon Construction
Development Name _____
Scheduled ~~Meeting~~/Decision Date _____

NOTICE: Notices were sent at least 20 days prior to the scheduled hearing, meeting, or decision date per Section 99.080 of the Community Development Code. **(check below)**

TYPE A ✓

A.	The applicant (date) 2-14-19	(signed)	S. Shroyer
B.	Affected property owners (date) 2-14-19	(signed)	S. Shroyer
C.	School District/Board (date) _____	(signed)	_____
D.	Other affected gov't. agencies (date) _____	(signed)	_____
E.	Affected neighborhood assns. (date) 2-14-19 (ALL)	(signed)	S. Shroyer
F.	All parties to an appeal or review (date) _____	(signed)	_____

At least 10 days prior to the scheduled hearing or meeting, notice was published/posted:

Tidings (published date)	N/A	(signed)	S. Shroyer
City's website (posted date)	2-14-19	(signed)	S. Shroyer

SIGN

At least 10 days prior to the scheduled hearing, meeting or decision date, a sign was posted on the property per Section 99.080 of the Community Development Code.

(date) 2/22/2019 (signed) [signature]

NOTICE: Notices were sent at least 14 days prior to the scheduled hearing, meeting, or decision date per Section 99.080 of the Community Development Code. **(check below)**

TYPE B _____

A.	The applicant (date) _____	(signed)	_____
B.	Affected property owners (date) _____	(signed)	_____
C.	School District/Board (date) _____	(signed)	_____
D.	Other affected gov't. agencies (date) _____	(signed)	_____
E.	Affected neighborhood assns. (date) _____	(signed)	_____

Notice was posted on the City's website at least 10 days prior to the scheduled hearing or meeting.
Date: _____ (signed) _____

STAFF REPORT mailed to applicant, City Council/Planning Commission and any other applicable parties 10 days prior to the scheduled hearing.

(date) _____ (signed) _____

FINAL DECISION notice mailed to applicant, all other parties with standing, and, if zone change, the County surveyor's office.

(date) 5/15/2019 (signed) [signature]

CITY OF WEST LINN
NOTICE OF UPCOMING
PLANNING MANAGER DECISION
FILE NO. MIP-18-05

The West Linn Planning Manager is considering a request for a three-lot minor partition at 6123 Skyline Drive.

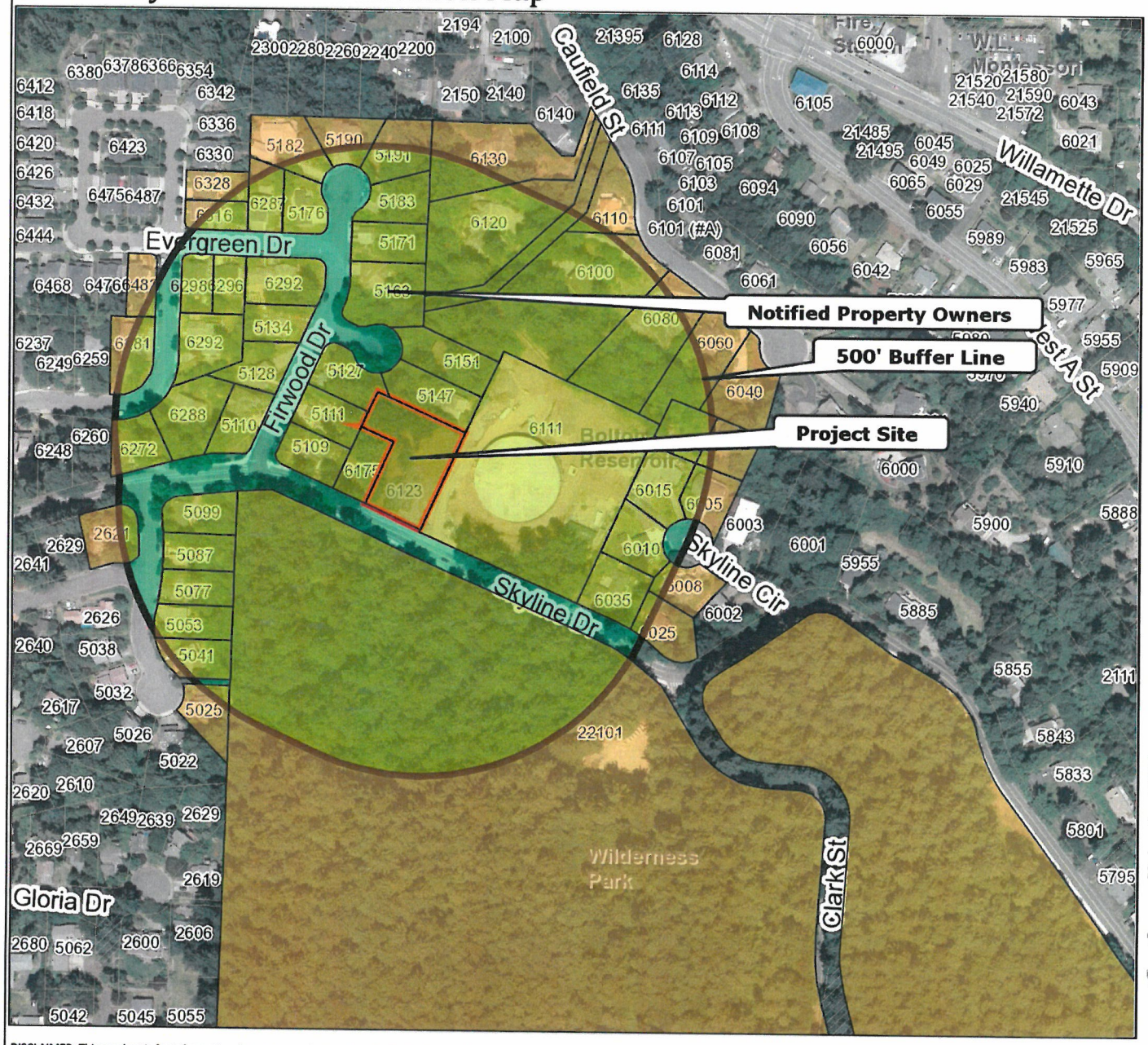
The decision will be based on the approval criteria in Chapters 11, 48, 85, 92, and 99 of the Community Development Code (CDC). The approval criteria from the CDC are available for review at City Hall, at the City Library, and at http://www.westlinnoregon.gov/cdc.

You have received this notice because County records indicate that you own property within 500 feet of this property (Tax Lot 9900 of Clackamas County Assessor's Map 21E 25AD) or as otherwise required by Chapter 99 of the CDC.

All relevant materials in the above noted file are available for inspection at no cost at City Hall, and on the city web site https://westlinnoregon.gov/planning/6123-skyline-drive-minor-partition or copies may be obtained for a minimal charge per page. A public hearing will not be held on this decision. **Anyone wishing to present written testimony for consideration on this matter shall submit all material before 4:00 p.m. on March 6, 2019. Persons interested in party status should submit their letter along with any concerns related to the proposal by the comment deadline.** For further information, please contact Darren Wyss, Associate Planner, City Hall, 22500 Salamo Rd., West Linn, OR 97068, (503)742-6064, dwyss@westlinnoregon.gov.

Any appeals to this decision must be filed within 14 days of the final decision date with the Planning Department. **It is important to submit all testimony in response to this notice.** Failure to raise an issue in person or by letter, or failure to provide sufficient specificity to afford the decision-maker an opportunity to respond to the issue, precludes the raising of the issue at a subsequent time on appeal or before the Land Use Board of Appeals.

6123 Skyline Drive Notification Map



CITY OF West Linn

N
W E
S

Feet
0 150

Scale 1:3,600 - 1 in = 300 ft
Scale is based on 8-1/2 x 11 paper size

Map created by: SSHROYER
Date Created: 17-Dec-18 02:14 PM

WEST LINN GIS

CITY OF West Linn

CITY OF WEST LINN
NOTICE OF UPCOMING
PLANNING MANAGER DECISION

PROJECT # MIP-18-05
MAIL: 2/14/19 TIDINGS: N/A

CITIZEN CONTACT INFORMATION

To lessen the bulk of agenda packets, land use application notice, and to address the worries of some City residents about testimony contact information and online application packets containing their names and addresses as a reflection of the mailing notice area, this sheet substitutes for the photocopy of the testimony forms and/or mailing labels. A copy is available upon request.

PD-4 COMPLETENESS LETTER

CITY OF
West Linn

December 17, 2018

Icon Construction & Development, LLC
1980 Willamette Falls Drive, Suite 200
West Linn, OR. 97068

SUBJECT: MIP-18-05 application for 3-Lot Minor Partition at 6123 Skyline Drive.

Dear Icon:

You submitted this application on October 2, 2018. The application was deemed incomplete on November 1, 2018. You submitted additional information on November 27, 2018 and the application was found to be incomplete again on December 13, 2018. After receiving additional information on December 13, 2018 the Planning and Engineering Departments find that this application is now **complete.** The city has 120 days to exhaust all local review; that period ends April 16, 2019.

Please be aware that determination of a complete application does not guarantee a recommendation of approval from staff for your proposal as submitted – it signals that staff believes you have provided the necessary information for the Planning Director to render a decision on your proposal.

A 20-day public notice will be prepared and mailed. This notice will identify the earliest potential decision date by the Planning Director. Please contact me at 503-742-6057, or by email at jarnold@westlinnoregon.gov if you have any questions or comments.

Sincerely,

Jennifer Arnold
Associate Planner

PD-5 PROPERTY MAPS

6123 Skyline Drive Aerial

CITY OF West Linn

N
W E
S

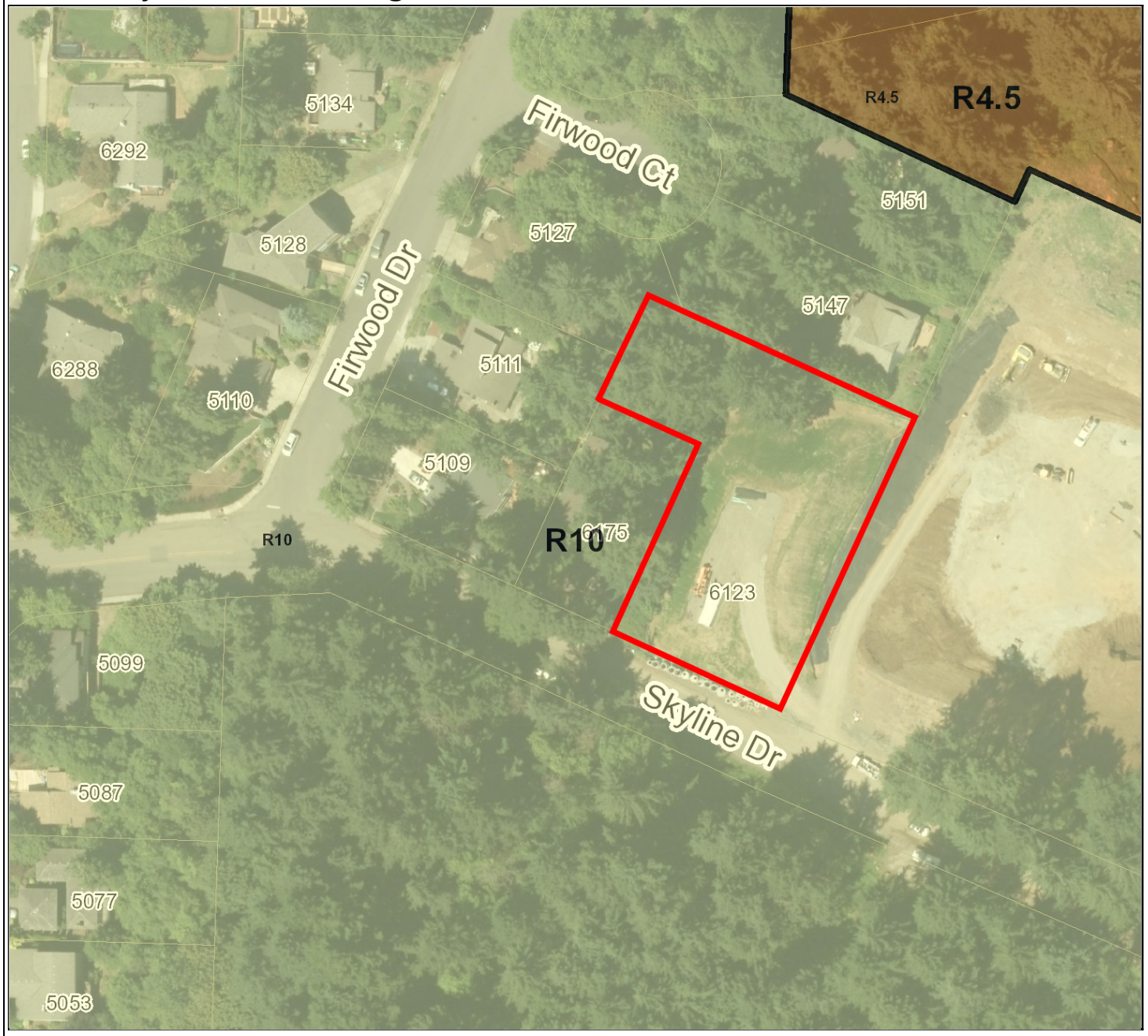
Feet
0 50

Scale 1:1,200 - 1 in = 100 ft
Scale is based on 8-1/2 x 11 paper size

Map created by: DWYSS
Date Created: 13-May-19 10:22 AM

WEST LINN GIS

6123 Skyline Drive Zoning



CITY OF West Linn

N
W E
S

Feet
0 50

Scale 1:1,200 - 1 in = 100 ft
Scale is based on 8-1/2 x 11 paper size

Map created by: DWYSS
Date Created: 13-May-19 10:24 AM

WEST LINN GIS

6123 Skyline Drive Habitat Map

CITY OF West Linn

Legend

Metro Habitat Protection
Published October 2005

Habitat Conservation Area

Allow Development

N
W E
S

Feet
0 50

Scale 1:1,200 - 1 in = 100 ft
Scale is based on 8-1/2 x 11 paper size

Map created by: DWYSS
Date Created: 13-May-19 10:25 AM

WEST LINN GIS