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) */**
- [X] 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.

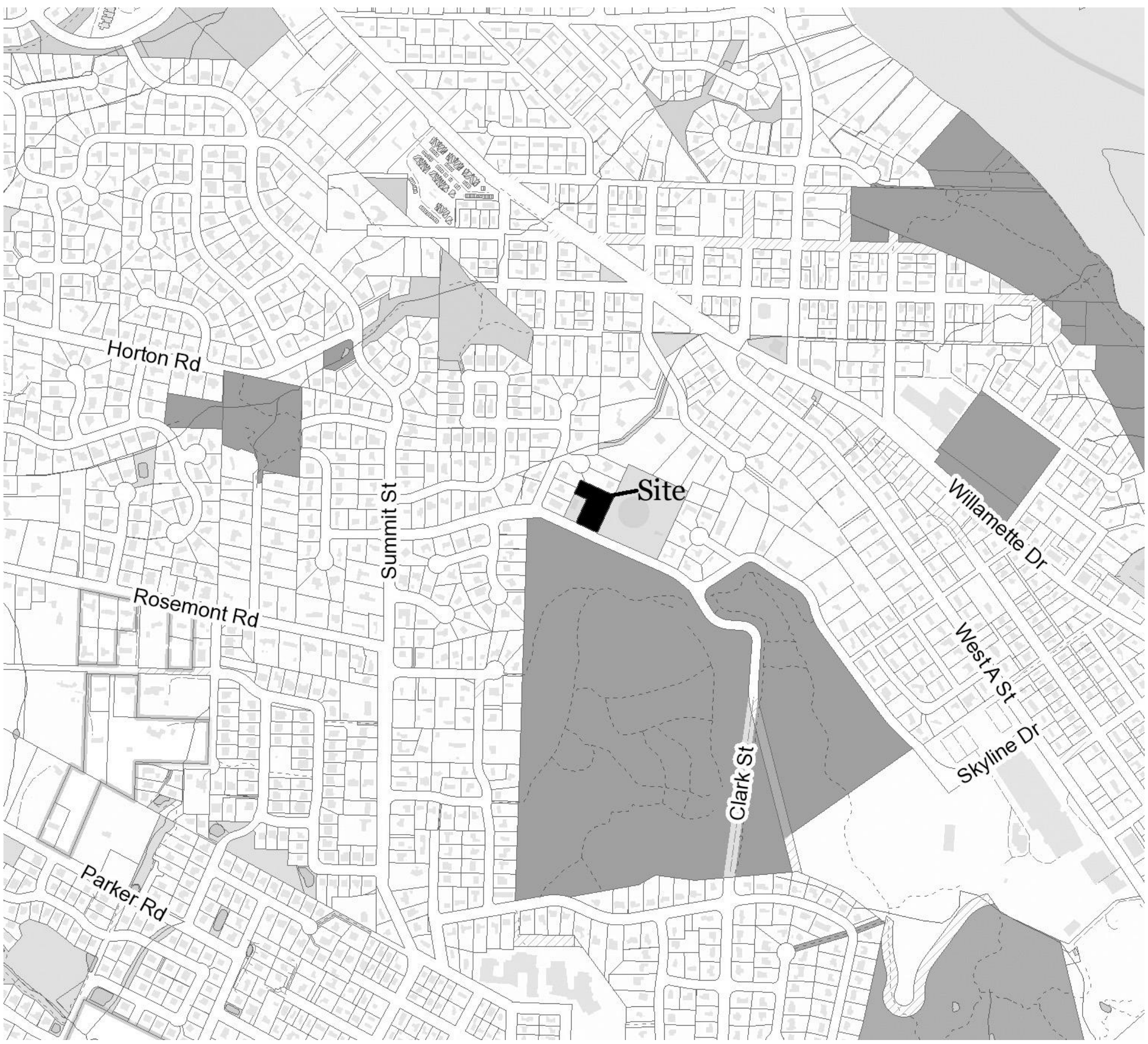
Applicant's signature — Date 9/19/18 — Owner's signature *(required)* — Date 10/2/18

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. <u>Streets</u>.

Comment: No new streets are proposed. The lots front on Skyline Dr. Because of the existing development pattern, which is fully developed to R-10 density to the north and west, and with the water reservoir to the east, there is no opportunity to provide for additional local street connections. Required street frontage improvements and dedication will be provided as needed.

B. <u>Blocks and lots</u>.

1. <u>General</u>. 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. <u>Sizes</u>. 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. <u>Lot size and shape</u>. 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. <u>Large lots or parcels</u>. 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.

<u>C. Pedestrian and bicycle trails.</u>

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. (Deleted)

I. Utility easements.

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. <u>Access options</u>. 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) <u>Option 1</u>. 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) <u>Option 2</u>. 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) <u>Option 3</u>. 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. <u>Subdivisions fronting onto an arterial street</u>. 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. <u>Double-frontage lots</u>. 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. <u>Access spacing</u>.

> *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. <u>Number of access points</u>. 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. <u>Shared driveways</u>. 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. <u>Exception</u>. 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. <u>Street connectivity and formation of blocks required</u>. 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. <u>Block length and perimeter</u>. The maximum block length shall not exceed 800 feet or 1,800 feet along an arterial.

2. <u>Street standards</u>. 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. <u>Exception</u>. 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.

Drainage Report

Address: 6175 Skyline, West Linn, Oregon

Date: August 8, 2018

NARRATIVE:

The existing manufactured house is to be removed and replaced with a new house. The property is approximately 10,000 square foot tract that slopes easterly away from the street. It is not possible to direct storm water to Skyline. The existing roof drains are a combination of splash pads and connections of an unknown underground system. 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. There is no indication of a high water table. At the infiltration test pit I found 0-4" inches of organic top soil followed by brown silt loam with no rock. There were no seeps or perched ground water. The total depth of the test pit was 43-inches. The test pit was hand dug the day before the infiltration test and charged with water to pre-soak the ground. Three successive one hour test for infiltration were conducted using the City of Portland procedure on August 8, 2018. The results of the third test were used for the design solution.
The observed rate of 2.5"/hr was reduced by a safety factor of 2 for this design, resulting in 1.25 in/hr or 48minutes per inch.

The new impervious area as reported by the designer/architect and used in these calculations:

3000 SF Roof + 1049 SF Drive = 0.09 acres

REFERENCE:

The King County Department of Public Works, Hydrographic Program, ver 4.21B and using a 10year event of 3.2 inches/hour.

ASSUMPTION:

5 SC- 740 chambers with 24 inches of additional drain rock below the chambers for a total depth of 5.5 feet.
The overall size of the facility is 10.45 feet X 25.75 feet = 269.1SF

depth	rock	SC unit	total	area
2.0	215.3	0.00	215.3	269.1
2.5	231.5	33.8	265.3	269.1
3.0	247.6	106.6	354.2	269.1
3.5	263.7	176.2	439.9	269.1
4.0	279.8	241.0	520.8	269.1
4.5	296.0	298.3	594.3	269.1
5.0	312.1	340.7	652.8	269.1
5.5	328.2	374.5	702.7	269.1

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)
10,24.3.2
Xxxxxxxxxxxxxxxxxxxxxxxx S.C.S.TYPE-1A DISTRIBUTION xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx
XXXXXXXXXXXX 25-YEAR 24-HOUR STORM xxxx 3.90" TOTAL PRECIP. Xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx

---

ENTER: A(PERV),CN(PERV),A(IMPERV),CN(IMPERV),TC FOR BASIN NO. 1
0.0,86,0.09,98,5
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)
.09	7.67	1197

ENTER [d:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:

C:rd

SPECIFY: C - CONTINUE, N - NEWSTORM, P - PRINT, S – STOP

ENTER OPTION

RESERVOIR ROUTING INFLOW/OUTFLOW ROUTINE

SPECIFY [d:][path]filename[.ext] OF ROUTINE DATA)

C:ydata

DISPLAY ROUTING DATA (Y or N)

Y

ROUTING DATA:

STAGE (FT)	DISCHARGE (CFS)	STORAGE (CU-FT)	PERN-AREA(SQ-FT)
.00	.00	.0	.0
2.00	.00	215.3	269.1
2.50	.00	265.3	269.1
3.00	.00	354.2	269.1
3.50	.00	439.9	269.1
4.00	.00	520.8	269.1
4.50	.00	594.3	269.1
5.00	.00	652.8	269.1
5.50	.00	702.7	

AVERAGE PERM-RATE: 48.0 MINUTES/INCH

SATURATED PERM-RATE: 48.0 MINUTES/INCH

GROUND STORAGE BEFORE SATURATION: .00 CU-FT/SQ-FT

ENTER [d:][path]filename[.ext] OF COMPUTED HYDROGRAPH:

C:rd

INFLOW/OUTFLOW ANALYSIS:

PEAK-INFLOW(CFS)	PEAK-OUTFLOW(CFS)	OUTFLOW-VOL (CU-FT)
.09	.00	0
INITIAL-STAGE (FT)	TIME-OF-PEAK(HRS)	PEAK-STAGE-ELEV(FT)
.00	23.83	4.84

PEAK STORAGE: 630 CU-FT

INFILTRATED VOLUME: 1235 CU-FT

ENTER [d] [path] filename [.ext] FOR STORAGE OF COMPUTED HYDROGRAPH

CONCLUSION:

Calculations show that the 4049 square feet of impervious area can infiltrate into the proposed facility exceeding the 25-year event, using a safety factor of 2. The infiltration system needs to be at least 10-feet from foundations and 5-feet from property lines. With the safety factor it is not anticipated that there will be any overflow condition, but if they did occur there is a significant distance from the facility to any structures.

MAINTENANCE:

A sediment trap is to be installed ahead of the facility with an 18-inch sump to keep leaves and other materials out of the storage areas. The inspection port will allow inspection of the infiltrators

Prepared by:

Bruce D. Goldson, PE

Theta,LLC

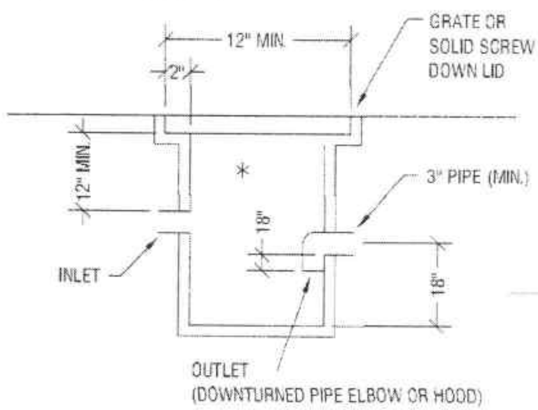
PO Box 1345

Lake Oswego, Oregon 97035

REGISTERED PROFESSIONAL ENGINEER
7120
OREGON
July 16, 1971
BRUCE D. GOLDSON

EXPIRES: 06/30/2019
SIGNATURE DATE: 8/8/18

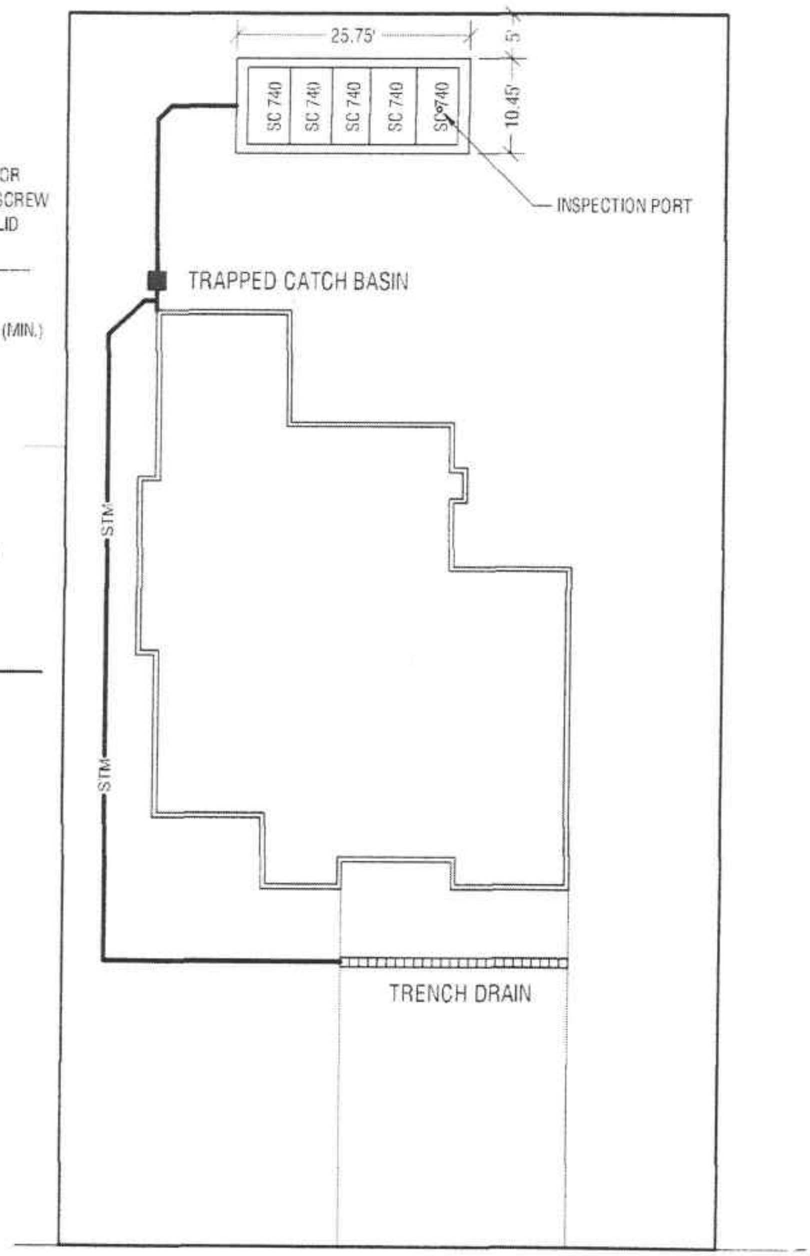
TRAPPED CATCH BASIN TO BE
USED FOR ROOF DRAINS



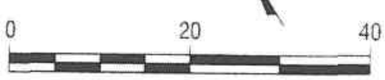
* THE BOTTOM OF THE INLET PIPE MUST NOT BE LOWER THAN THE TOP OF THE OUTLET PIPE.

TRAPPED SILT BASIN DETAIL

SCALE: NTS



SKYLINE DRIVE



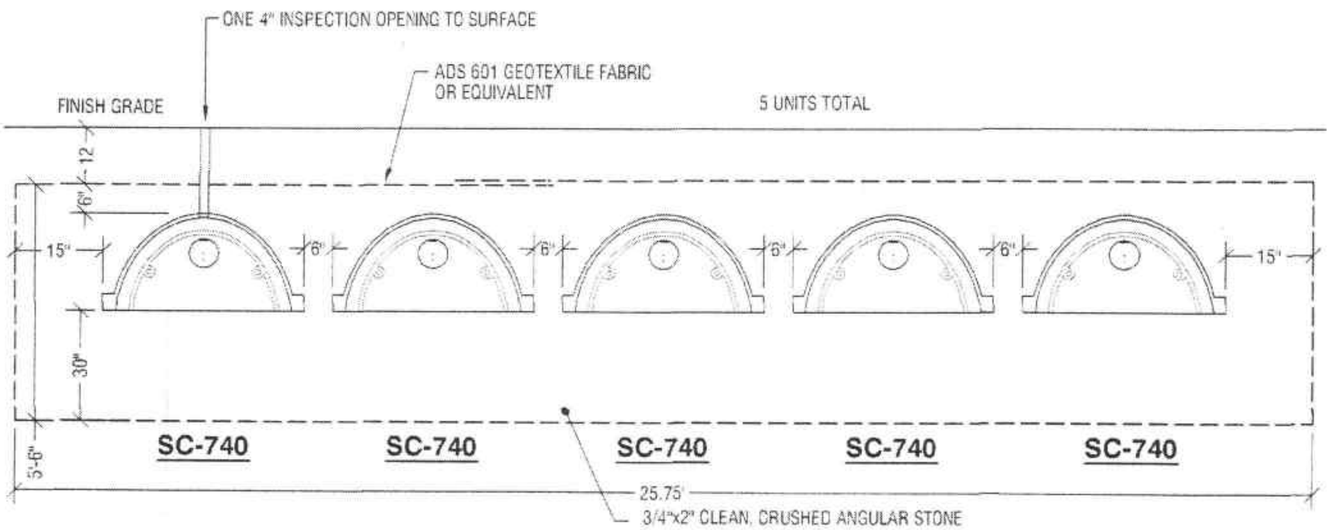
SCALE: 1" = 20'

2014-129U August 7, 2018

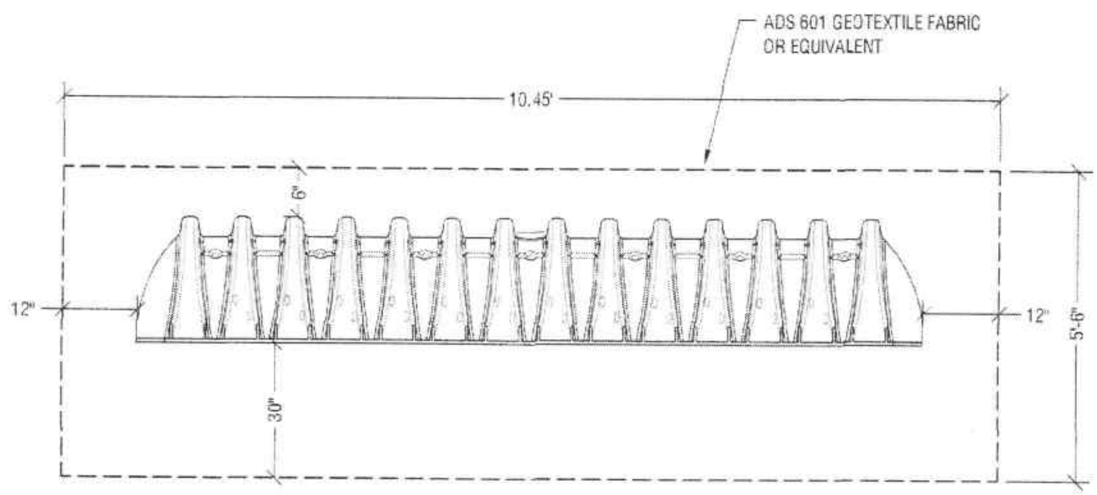
STORM DRAINAGE PLAN

Theta, llc
ENGINEERING - SURVEYING - PLANNING
PO Box 1345
Lake Oswego, Oregon 97035
503-481-8822
email: thetaeng@comcast.net

6175 Skyline Drive
West Linn, Oregon 97068



STORM FACILITY SECTION A



SC-740

STORM FACILITY SECTION B

2014-129U August 7, 2018

STORM DRAINAGE PLAN

Theta, llc
ENGINEERING - SURVEYING - PLANNING
PO Box 1345
Lake Oswego, Oregon 97035
503-481-8822
email: thetaeng@comcast.net

6175 Skyline Drive
West Linn, Oregon 97068

2/2

Clackamas County Area, Oregon (OR610)

Clackamas County Area, Oregon (OR610)

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
13B	Cascade silt loam, 3 to 8 percent slopes	0.1	17.4%
92F	Xerochrepts and Haploxerolls, very steep	0.3	82.6%
Totals for Area of Interest		**0.4**	**100.0%**

Detention-Retention-Recharge

The StormTech SC-740 chamber optimizes storage volumes in relatively small footprints by providing 2.2 ft^3/ft^2 (0.67 m^3/m^2) (minimum) of storage. This can decrease excavation, backfill and associated costs. The StormTech SC-310 chamber is ideal for systems requiring low-rise and wide-span solutions. The chamber allows the storage of large volumes, 1.3 ft^3/ft^2 (0.4 m^3/m^2) (minimum), at minimum depths.

StormTech SC-740 Chamber
(not to scale)

Nominal Chamber Specifications

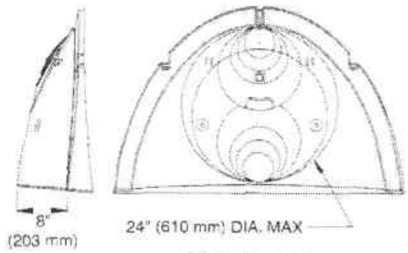
Size (L x W x H)
85.4" x 51.0" x 30.0"
(2170 x 1295 x 762 mm)

Chamber Storage
45.9 ft^3 (1.30 m^3)

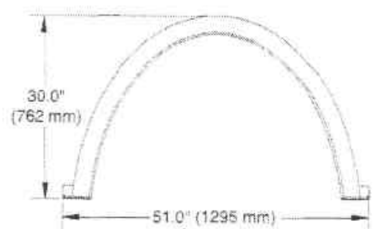
Minimum Installed Storage*
74.9 ft^3 (2.12 m^3)

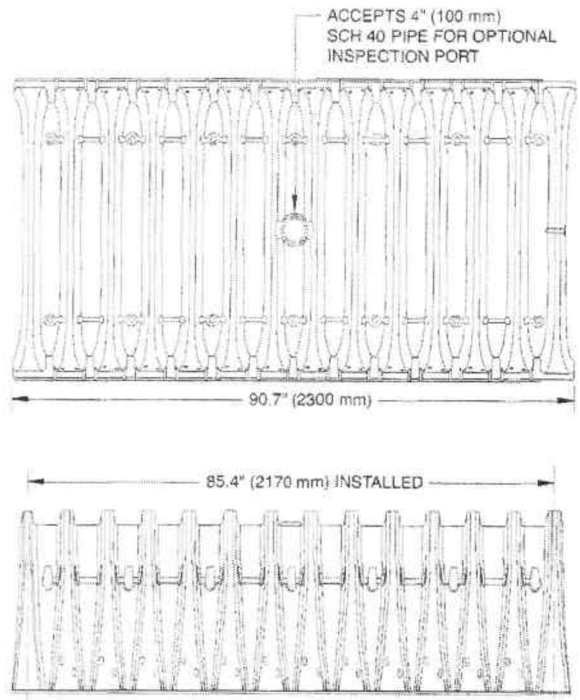
Weight
74.0 lbs (33.6 kg)

Shipping
30 chambers/pallet
60 end caps/pallet
12 pallets/truck



SC-740 end cap





SC-740 chamber

StormTech SC-310 Chamber
(not to scale)

Nominal Chamber Specifications

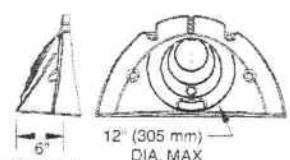
Size (L x W x H)
85.4" x 34.0" x 16.0"
(2170 x 864 x 406 mm)

Chamber Storage
14.7 ft^3 (0.42 m^3)

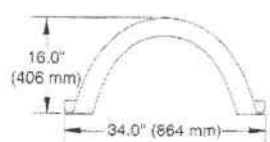
Minimum Installed Storage*
31.0 ft^3 (0.88 m^3)

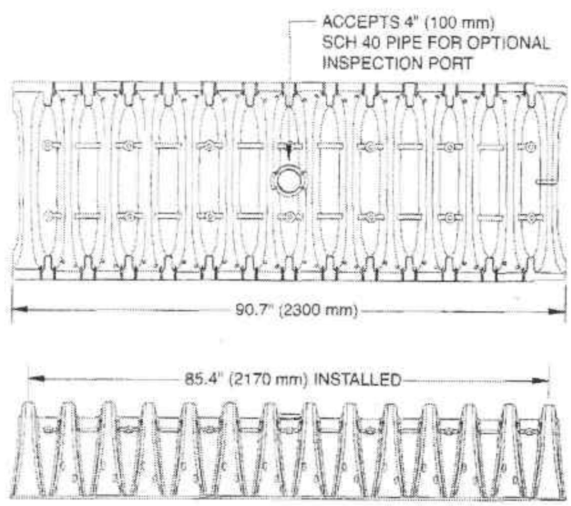
Weight
37.0 lbs (16.8 kg)

Shipping
41 chambers/pallet
108 end caps/pallet
18 pallets/truck



SC-310 end cap





SC-310 chamber

*This assumes a minimum of 6 inches (150 mm) of stone below, above and between chamber rows.

8.0 Incremental Storage Volumes

Table 5 and **Table 6** provide incremental storage volumes for SC-310 and SC-740 chamber systems. This information may be used to calculate a detention/retention system's stage storage volume. A spreadsheet is available at www.stormtech.com in which the number of chambers can be input for quick cumulative storage calculations

Product Specifications: 1.1, 2.2, 2.3, 2.4 and 2.6

TABLE 5 – SC-310 Incremental Storage Volumes Per Chamber
Assumes 40% Stone Porosity. Calculations are Based Upon a 6" (152 mm) Stone Base Under the Chambers.

Depth of Water in System Inches (mm)	Cumulative Chamber Storage ft³ (m³)	Total System Cumulative Storage ft³ (m³)
28 (711)	14.70 (0.416)	31.00 (0.878)
27 (686)	14.70 (0.416)	30.21 (0.855)
26 (680)	Stone 14.70 (0.416)	29.42 (0.833)
25 (610)	Cover 14.70 (0.416)	28.63 (0.811)
24 (609)	14.70 (0.416)	27.84 (0.788)
23 (584)	14.70 (0.416)	27.05 (0.766)
22 (559)	14.70 (0.416)	26.26 (0.748)
21 (533)	14.64 (0.415)	25.43 (0.720)
20 (508)	14.49 (0.410)	24.54 (0.695)
19 (483)	14.22 (0.403)	23.58 (0.668)
18 (457)	13.68 (0.387)	22.47 (0.636)
17 (432)	12.99 (0.368)	21.25 (0.602)
16 (406)	12.17 (0.345)	19.97 (0.566)
15 (381)	11.25 (0.319)	18.62 (0.528)
14 (356)	10.23 (0.290)	17.22 (0.488)
13 (330)	9.15 (0.260)	15.78 (0.447)
12 (305)	7.99 (0.227)	14.29 (0.425)
11 (279)	6.78 (0.192)	12.77 (0.362)
10 (254)	5.51 (0.156)	11.22 (0.318)
9 (229)	4.19 (0.119)	9.64 (0.278)
8 (203)	2.83 (0.081)	8.03 (0.227)
7 (178)	1.43 (0.041)	6.40 (0.181)
6 (152)	0	4.74 (0.134)
5 (127)	0	3.95 (0.112)
4 (102)	Stone 0	3.16 (0.090)
3 (76)	Foundation 0	2.37 (0.067)
2 (51)	0	1.58 (0.046)
1 (25)	0	0.79 (0.022)

Note: Add 0.79 cu. ft. (0.022 m³) of storage for each additional inch (25 mm) of stone foundation.

TABLE 6 – SC-740 Incremental Storage Volumes Per Chamber
Assumes 40% Stone Porosity. Calculations are Based Upon a 6" (152 mm) Stone Base Under the Chambers.

Depth of Water in System Inches (mm)	Cumulative Chamber Storage ft³ (m³)	Total System Cumulative Storage ft³ (m³)
42 (1067)	45.90 (1.300)	74.90 (2.121)
41 (1041)	45.90 (1.300)	73.77 (2.089)
40 (1016)	Stone 45.90 (1.300)	72.64 (2.057)
39 (991)	Cover 45.90 (1.300)	71.52 (2.025)
38 (965)	45.90 (1.300)	70.39 (1.993)
37 (948)	45.90 (1.300)	69.26 (1.961)
36 (914)	45.90 (1.300)	68.14 (1.929)
35 (889)	45.85 (1.298)	66.98 (1.897)
34 (864)	45.69 (1.294)	65.75 (1.862)
33 (838)	45.41 (1.286)	64.46 (1.825)
32 (813)	44.81 (1.269)	62.97 (1.783)
31 (787)	44.01 (1.246)	61.36 (1.737)
30 (762)	43.06 (1.219)	59.66 (1.689)
29 (737)	41.98 (1.189)	57.89 (1.639)
28 (711)	40.80 (1.155)	56.05 (1.587)
27 (686)	39.54 (1.120)	54.17 (1.534)
26 (660)	38.18 (1.081)	52.23 (1.479)
25 (635)	36.74 (1.040)	50.23 (1.422)
24 (610)	35.22 (0.977)	48.19 (1.365)
23 (584)	33.64 (0.953)	46.11 (1.306)
22 (559)	31.99 (0.906)	44.00 (1.246)
21 (533)	30.29 (0.858)	41.85 (1.185)
20 (508)	28.54 (0.808)	39.67 (1.123)
19 (483)	26.74 (0.757)	37.47 (1.061)
18 (457)	24.89 (0.705)	35.23 (0.997)
17 (432)	23.00 (0.651)	32.96 (0.939)
16 (406)	21.06 (0.596)	30.68 (0.869)
15 (381)	19.09 (0.541)	28.36 (0.803)
14 (356)	17.08 (0.484)	26.03 (0.737)
13 (330)	15.04 (0.426)	23.68 (0.670)
12 (305)	12.97 (0.367)	21.31 (0.608)
11 (279)	10.87 (0.309)	18.92 (0.535)
10 (254)	8.74 (0.247)	16.51 (0.468)
9 (229)	6.58 (0.186)	14.09 (0.399)
8 (203)	4.41 (0.125)	11.66 (0.330)
7 (178)	2.21 (0.063)	9.21 (0.264)
6 (152)	0	6.76 (0.191)
5 (127)	0	5.63 (0.160)
4 (102)	Stone 0	4.51 (0.125)
3 (76)	Foundation 0	3.38 (0.095)
2 (51)	0	2.25 (0.064)
1 (25)	0	1.13 (0.032)

Note: Add 1.13 cu. ft. (0.032 m³) of storage for each additional inch (25 mm) of stone foundation.

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

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 $1^1/_2$ 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 $1\frac{1}{2}$-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 $1\frac{1}{2}$ in., with not more than about 2% passing the No. 200 sieve (washed analysis). Crushed rock of $\frac{3}{4}$- to $1\frac{1}{2}$-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 $1\frac{1}{2}$ 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 $\frac{3}{4}$ to $1\frac{1}{4}$ 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 $\frac{1}{4}$ to $\frac{1}{2}$ 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 $^{1}/_{3}$H 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.

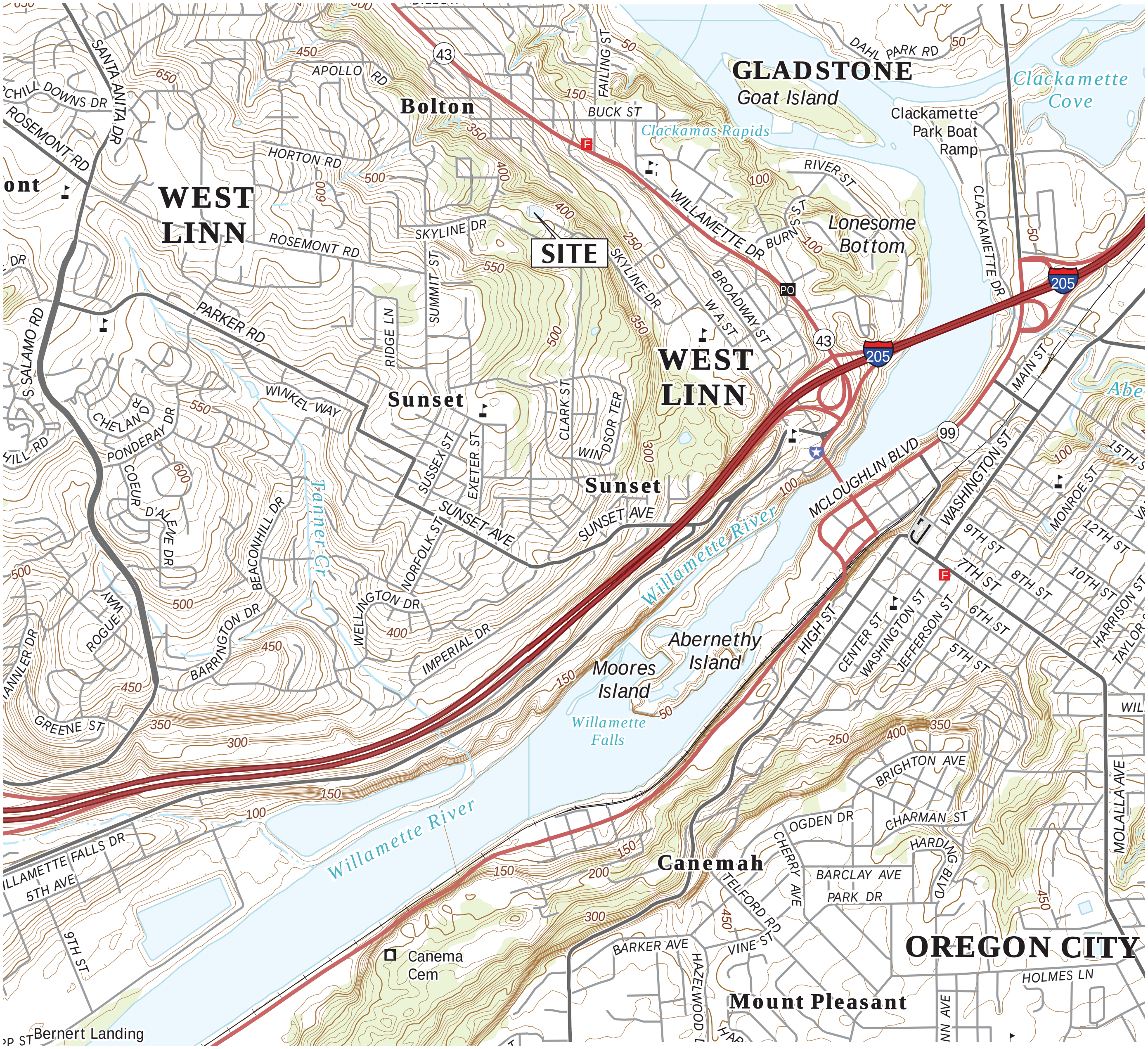
Oregon Department of Geology and Mineral Industries (DOGAMI), September 29, 2009, News release regarding GMS 119.

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

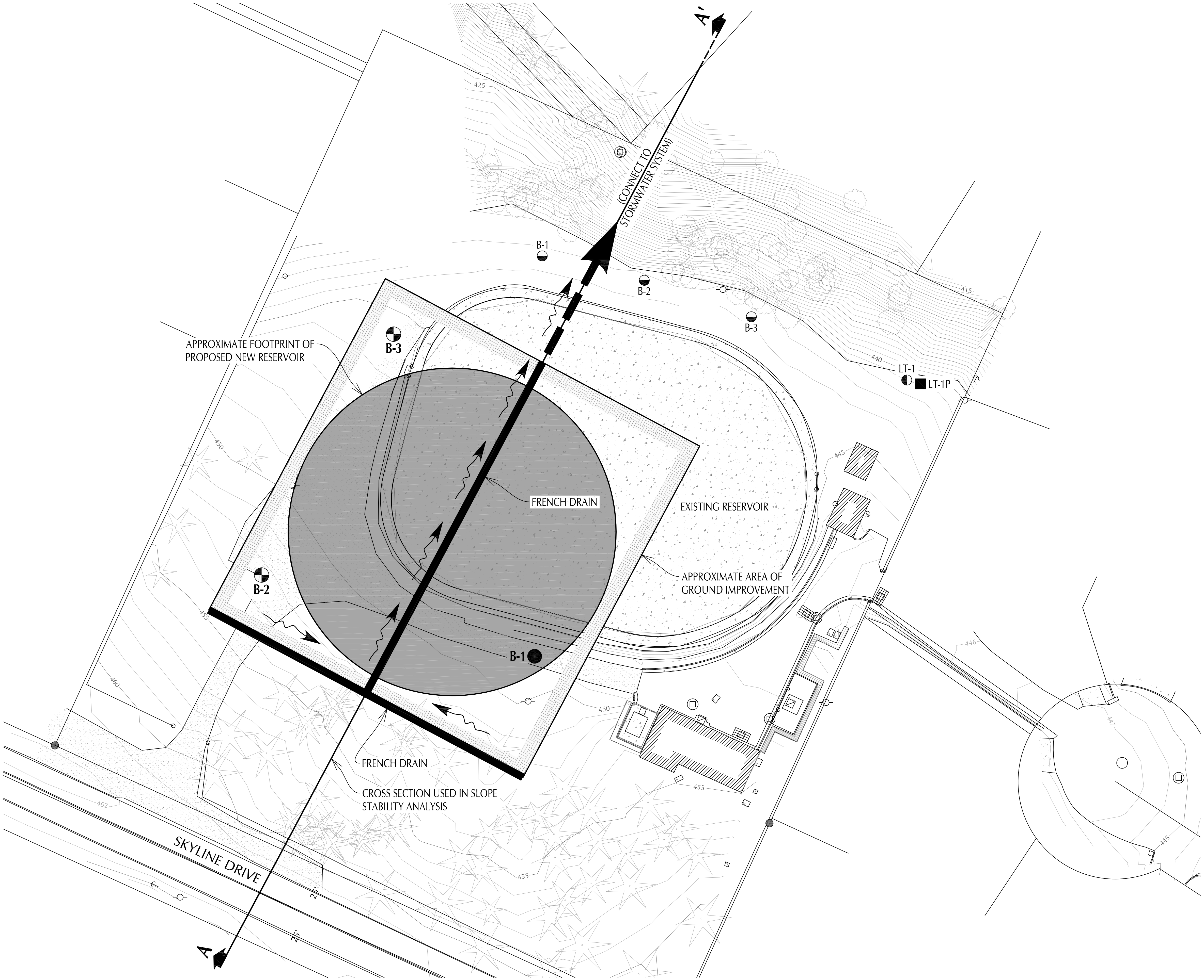
0 1/2 1 MILE

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

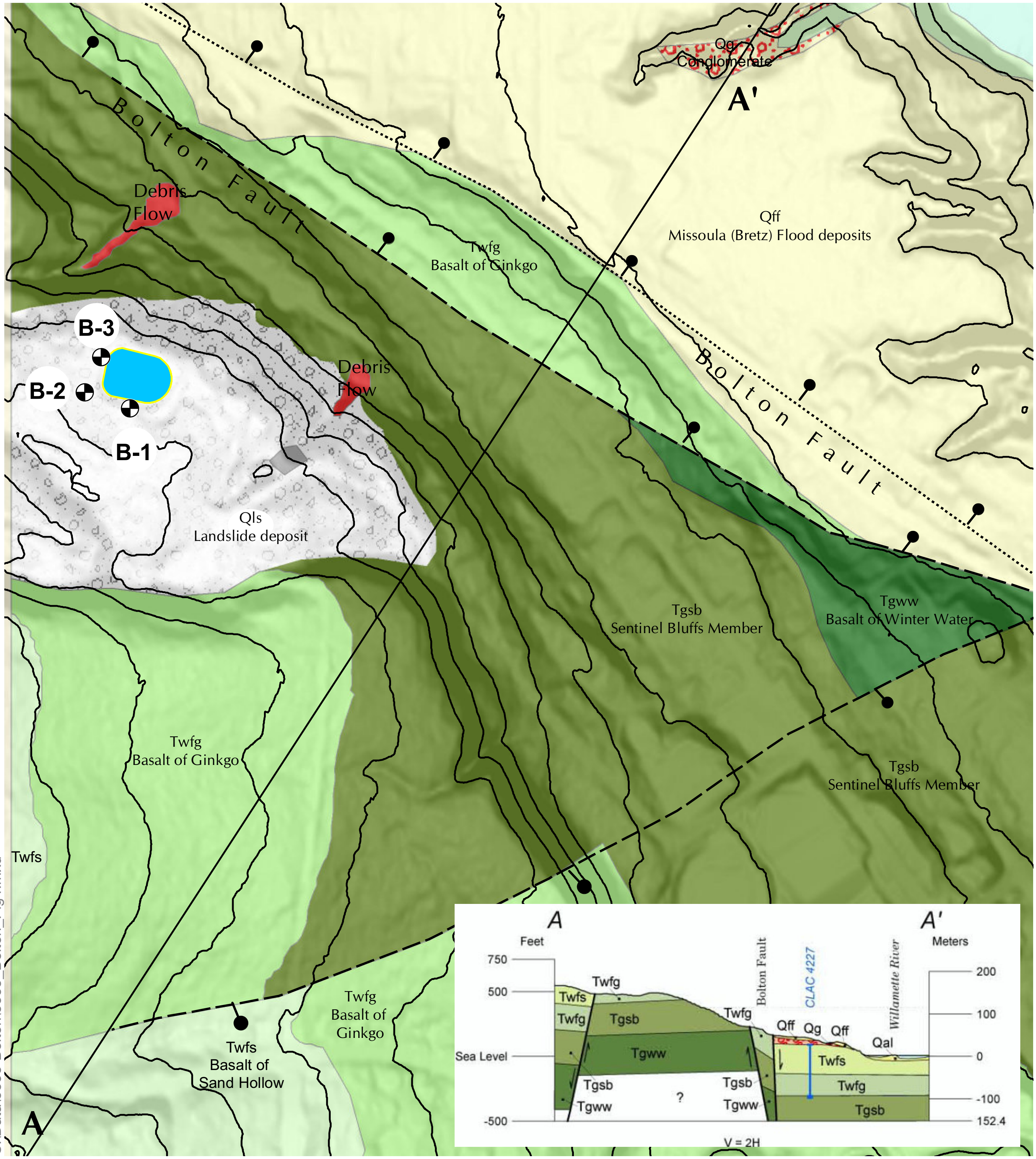
0 250 500 FT

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

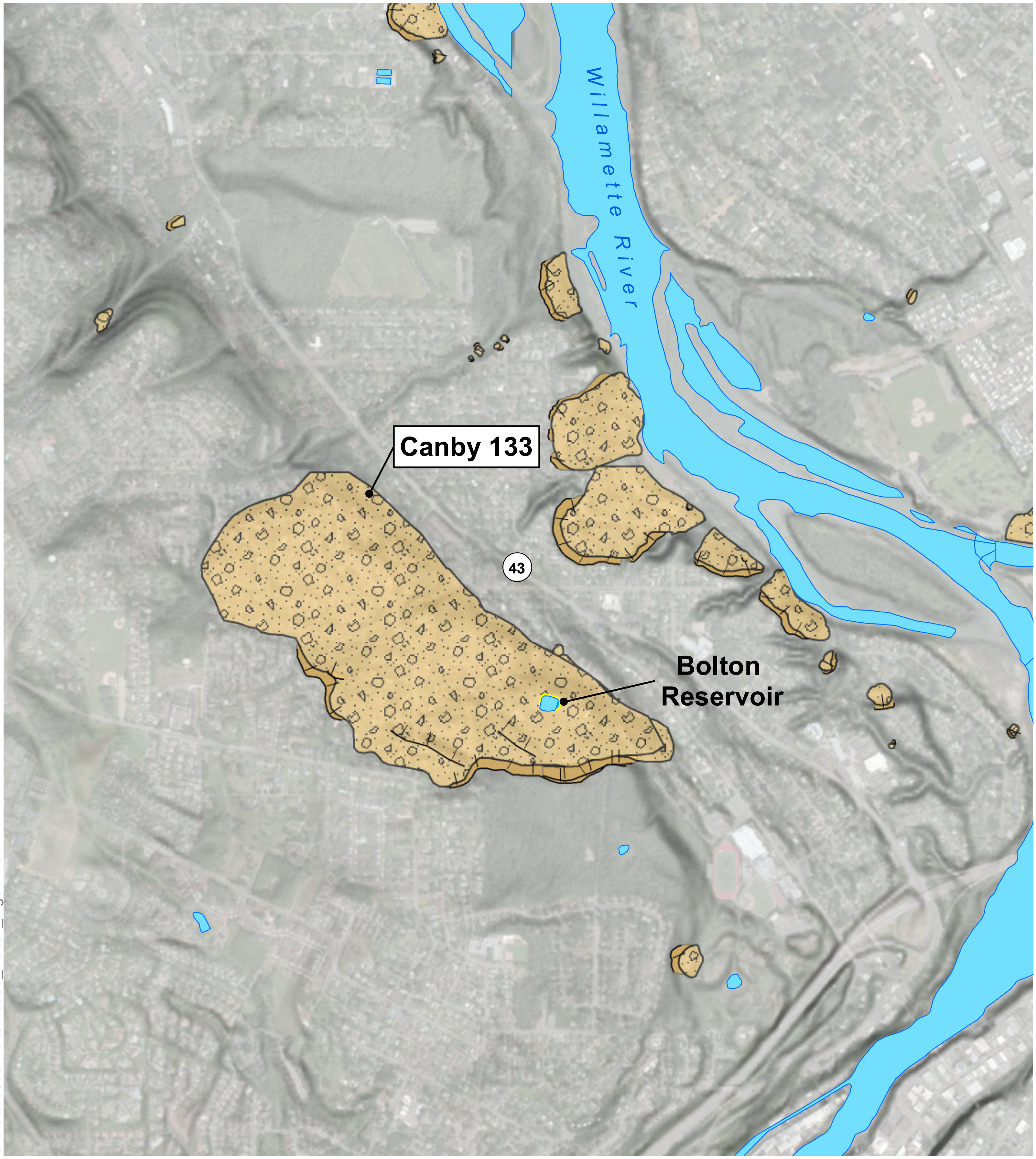
0 300 600 Ft

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

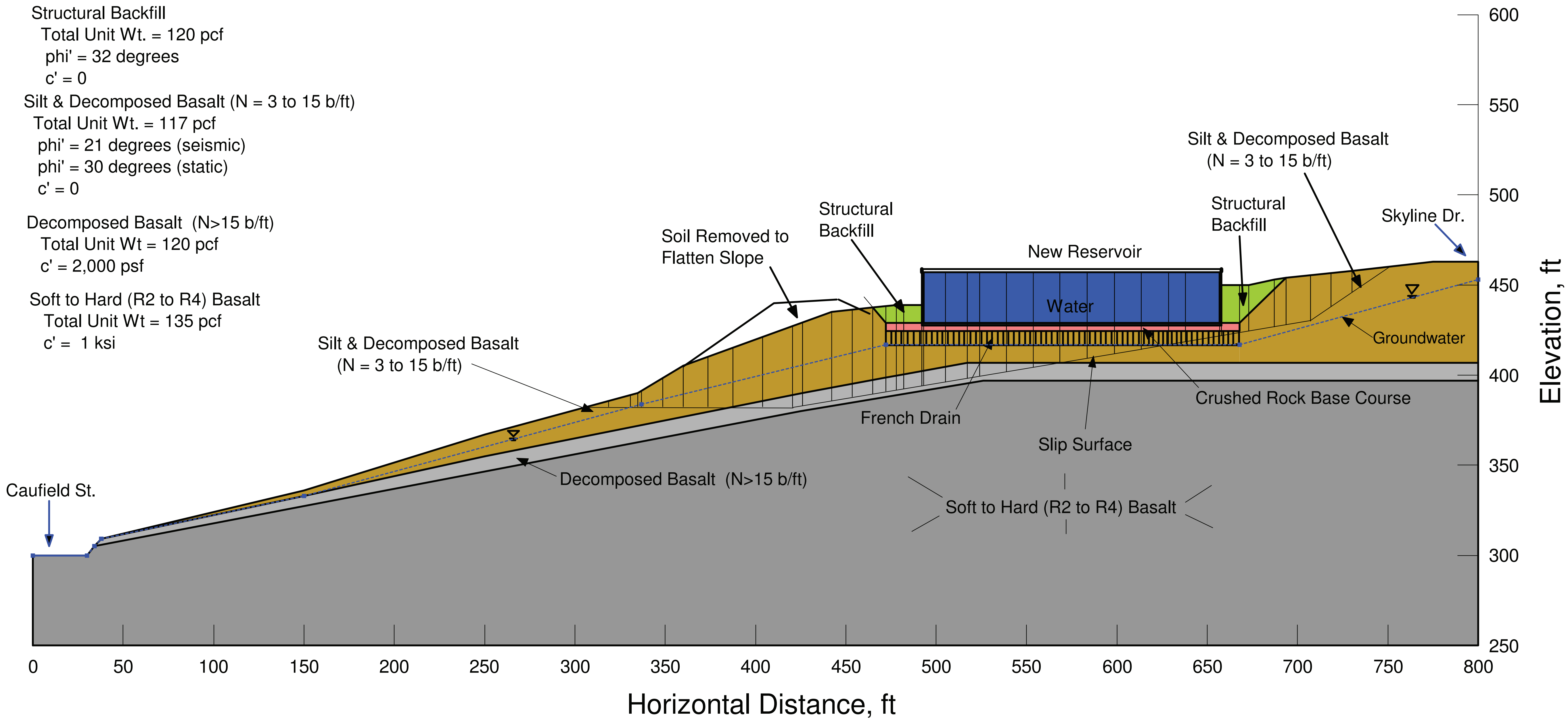
Silt & Decomposed Basalt (N = 3 to 15 b/ft)
- Total Unit Wt. = 117 pcf
- phi' = 21 degrees (seismic)
- phi' = 30 degrees (static)
- c' = 0

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

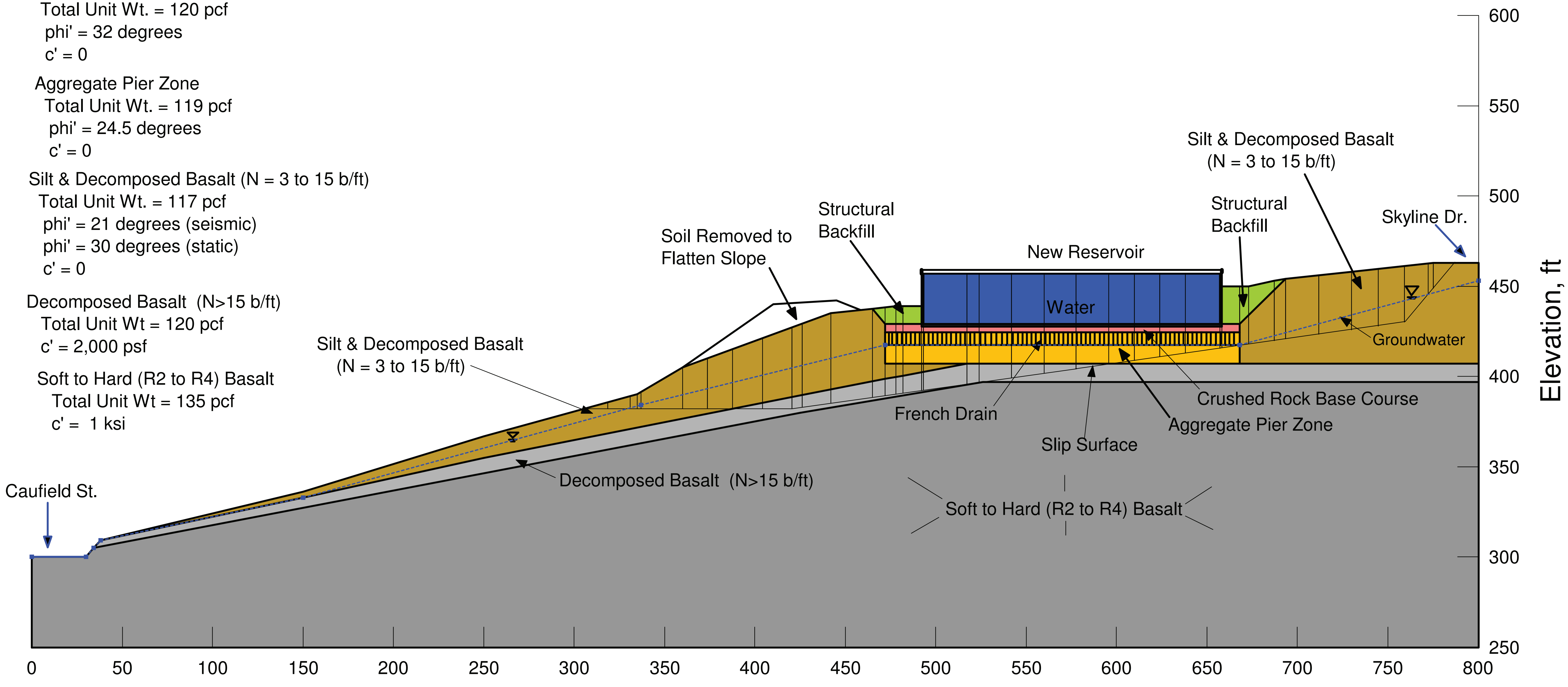
Silt & Decomposed Basalt (N = 3 to 15 b/ft)
- Total Unit Wt. = 117 pcf
- phi' = 21 degrees (seismic)
- phi' = 30 degrees (static)
- c' = 0

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)



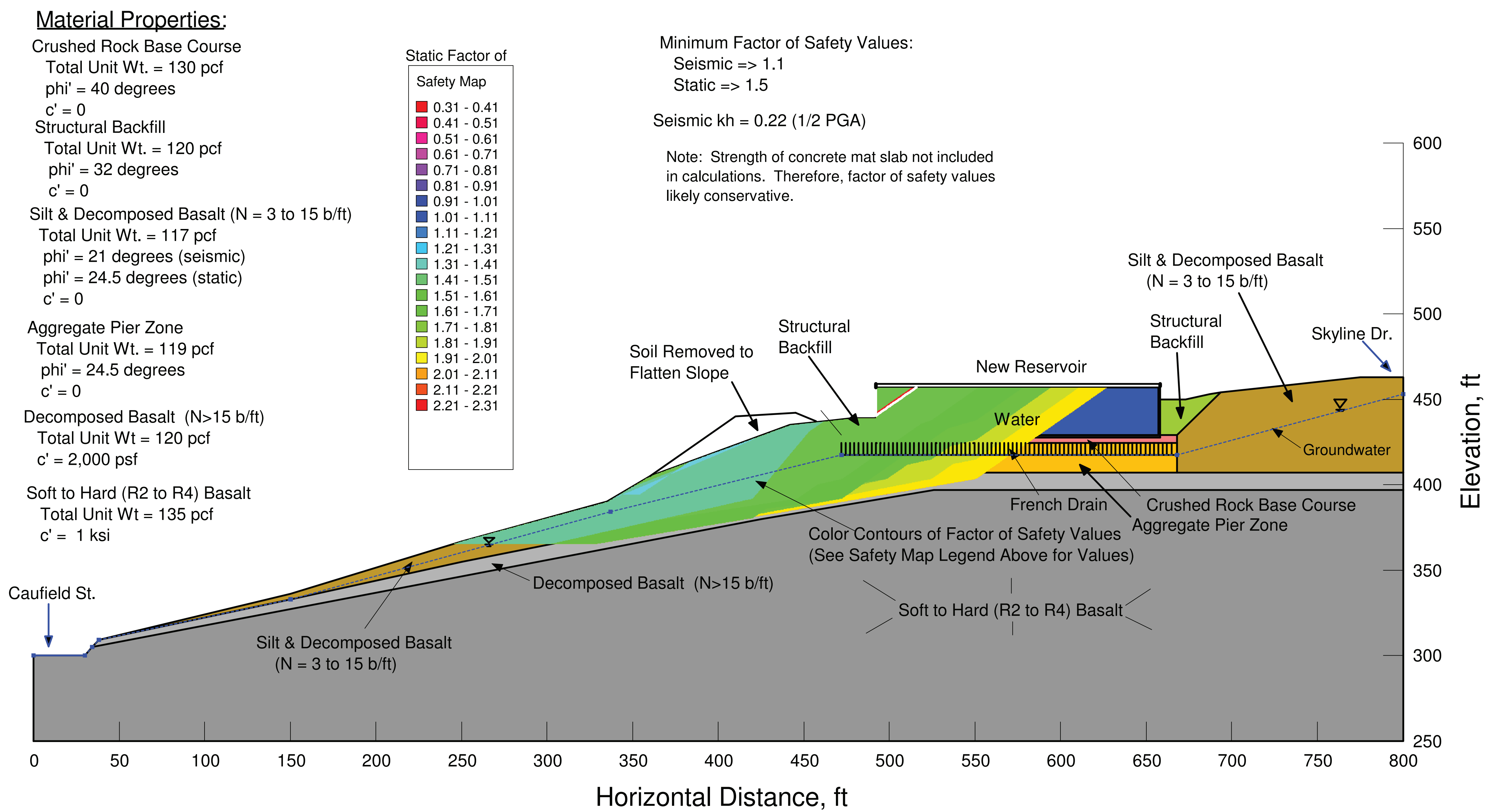
GRI

MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

SLOPE STABILITY MODEL

(NEW RESERVOIR WITH GROUND IMPROVEMENT)

SEP. 2015 | JOB NO. 5338-A | FIG. 7



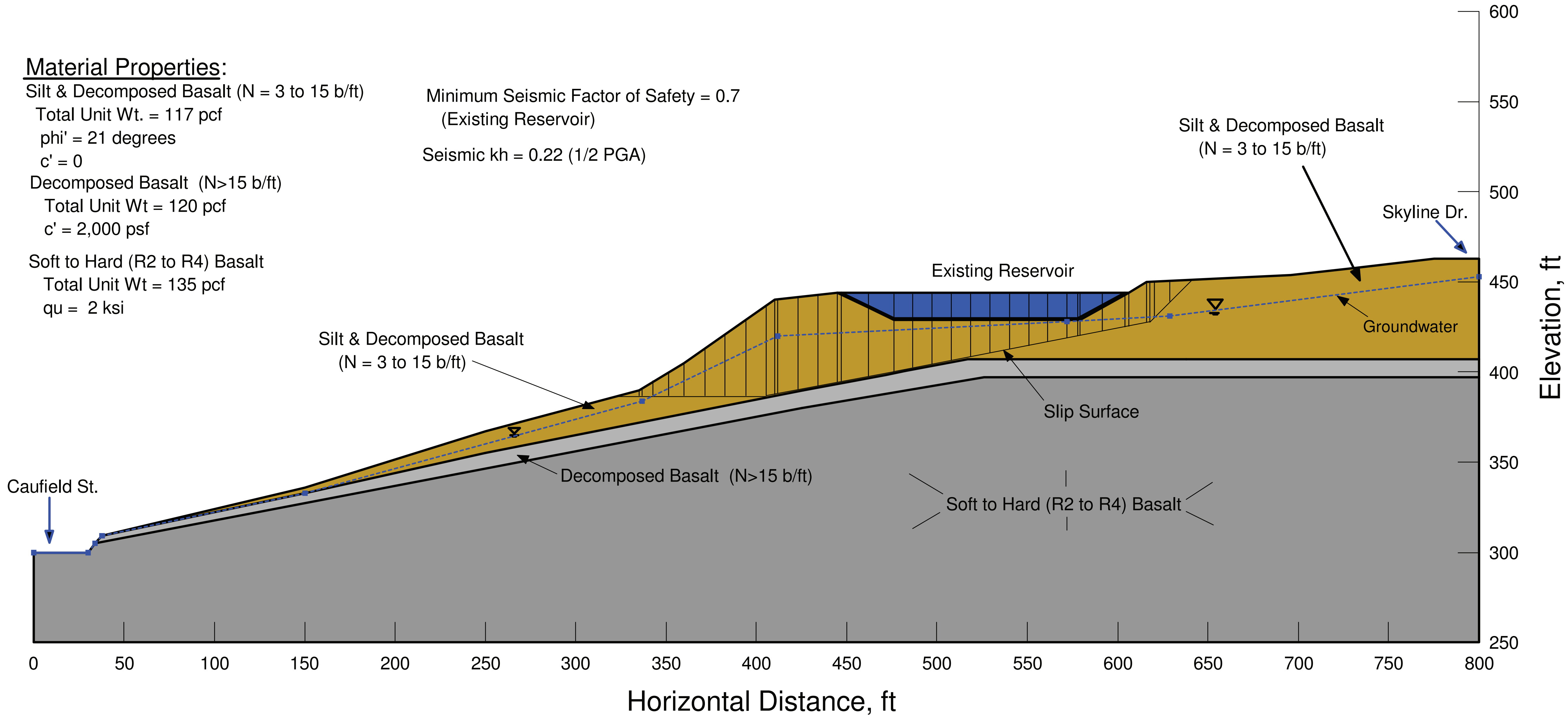
GRI

MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

SLOPE STABILITY MODEL

(NEW RESERVOIR WITH GROUND IMPROVEMENT, NORTH SLOPE)

SEP. 2015 JOB NO. 5338-A FIG. 8

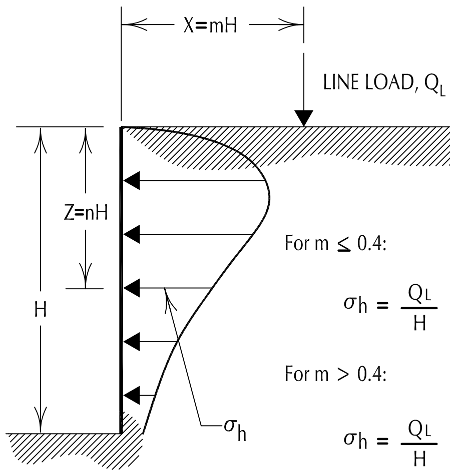


GRI

MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

SLOPE STABILITY MODEL
(EXISTING RESERVOIR)

SEP. 2015 | JOB NO. 5338-A | FIG. 9



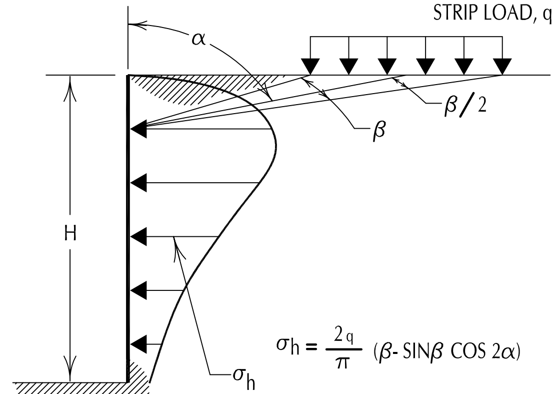
For m ≤ 0.4:

$$\sigma_h = \frac{Q_L}{H}\,\frac{0.2n}{(0.16+n^2)^2}$$

For m > 0.4:

$$\sigma_h = \frac{Q_L}{H}\,\frac{1.28m^2n}{(m^2+n^2)^2}$$

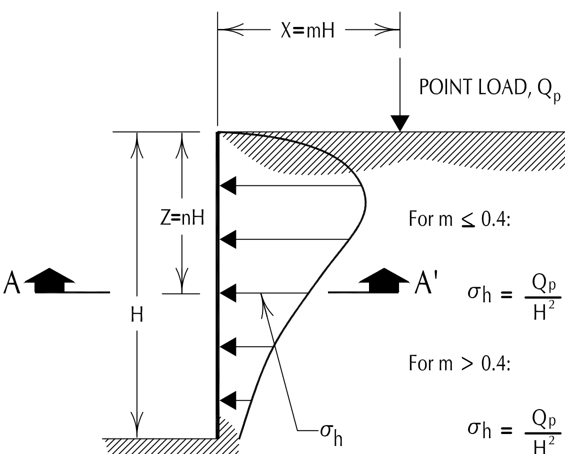
LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{\pi}\,(\beta - \mathrm{SIN}\beta\ \mathrm{COS}\ 2\alpha)$$

(β in radians)

STRIP LOAD PARALLEL TO WALL

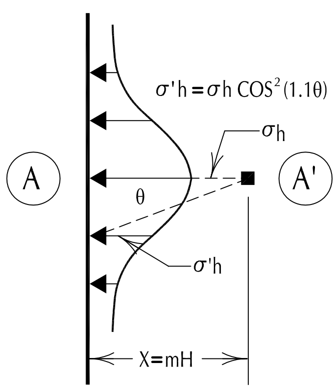


For m ≤ 0.4:

$$\sigma_h = \frac{Q_p}{H^2}\,\frac{0.28n^2}{(0.16+n^2)^3}$$

For m > 0.4:

$$\sigma_h = \frac{Q_p}{H^2}\,\frac{1.77m^2n^2}{(m^2+n^2)^3}$$



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

Boulders:
- >12 in.

Cobbles:
- 3 - 12 in.

Gravel:
- 1/4 - 3/4 in. (fine)
- 3/4 - 3 in. (coarse)

Sand:
- No. 200 - No. 40 sieve (fine)
- No. 40 - No. 10 sieve (medium)
- No. 10 - No. 4 sieve (coarse)

Silt/Clay:
- pass No. 200 sieve

Modifier for Subclassification

Adjective	Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
	Percentage of Other Material (by weight)	
trace:	5 - 15 (sand, gravel)	5 - 15 (sand, gravel)
some:	15 - 30 (sand, gravel)	15 - 30 (sand, gravel)
sandy, gravelly:	30 - 50 (sand, gravel)	30 - 50 (sand, gravel)
trace:	<5 (silt, clay)	*Relationship of clay and silt determined by plasticity index test*
some:	5 - 12 (silt, clay)	
silty, clayey:	12 - 50 (silt, clay)	

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

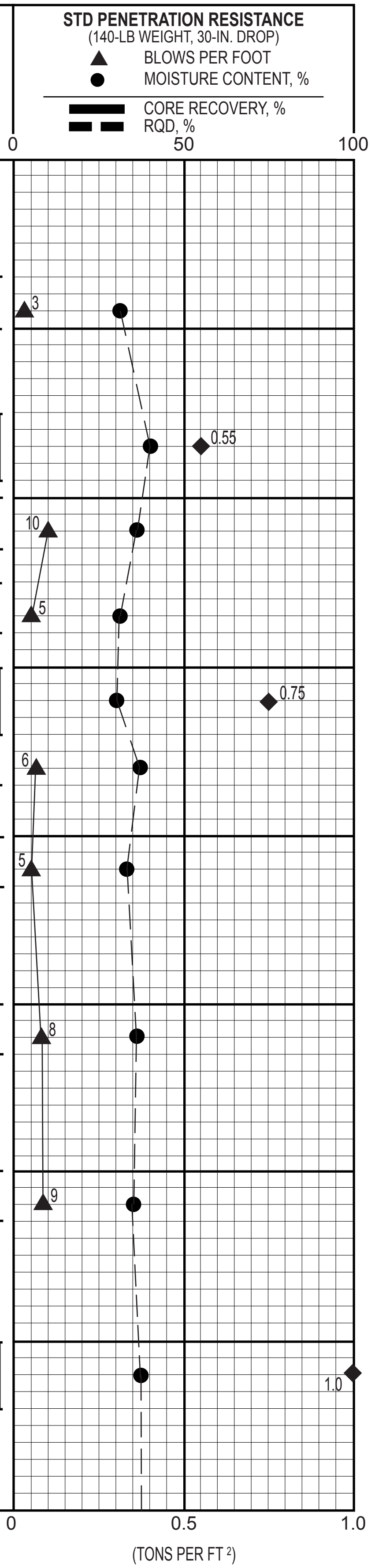
INSTALLATION SYMBOLS

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
P	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)

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	SAMPLES
	SURFACE ELEVATION 449 ft [±] (NAVD 88)		
	FILL: Crushed Rock	0.9	
	FILL: Soft, brown SILT; trace fine-grained sand, medium plasticity	8.0	S-1
	Stiff, brown to dark brown, clayey SILT; some fine- to medium-grained sand, medium plasticity, slight reddish-brown mottling (Landslide Debris)		S-2
	---------medium stiff below 11.5 ft		S-3, S-4
	---------stiff at 15 ft	30.0	S-5, S-6, S-7, S-8
	Stiff, brown and orange-brown, clayey SILT; medium to low plasticity, trace sand- to gravel-size fragments of extremely soft (R0), predominantly decomposed basalt (Landslide Debris)		S-9
	---------stiff to very stiff below 35 ft		S-10



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

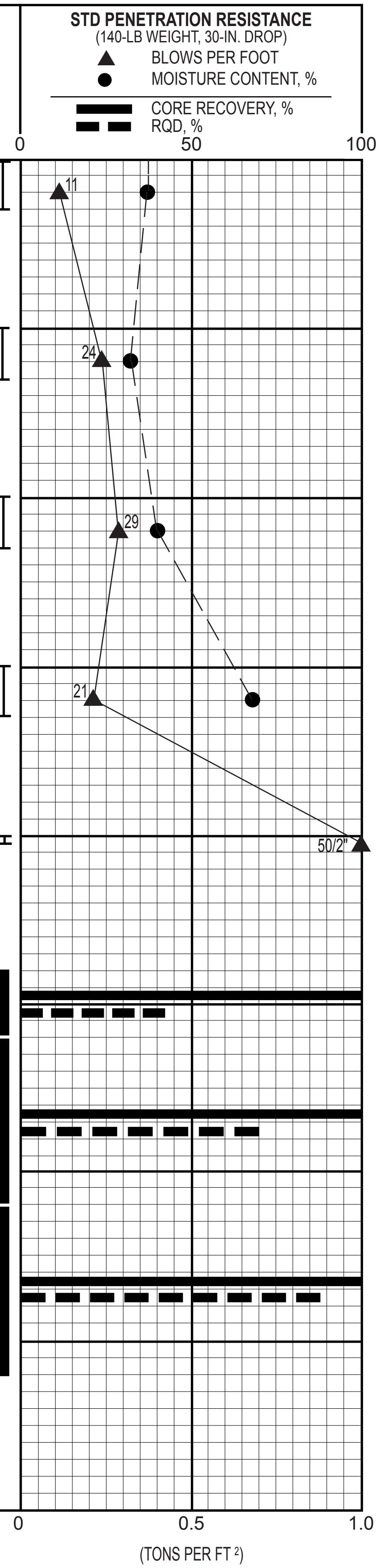
GRI

BORING B-1

AUG. 2015 JOB. NO. 5338-A FIG. 1A

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	SAMPLES
	SURFACE ELEVATION 449 ft [±] (NAVD 88)		
40	Stiff, brown and orange-brown, clayey SILT; medium to low plasticity, trace sand- to gravel-size fragments of extremely soft (R0), predominantly decomposed basalt (Landslide Debris)		S-11
45	Very stiff, red, clayey SILT; trace yellowish-white and black, sand-size fragments of predominantly decomposed basalt, relic rock structure (Landslide Debris)	45.0	S-12
50	Very stiff, gray, clayey SILT; medium to high plasticity, some coarse-grained sand- to fi ne gravel-size fragments of extremely soft (R0), predominantly decomposed basalt (Landslide Debris)	51.0	S-13, S-14
60	Soft to medium hard (R2 to R3), gray and reddish-brown BASALT; coarse-grained sand- to gravel-size fragments of slightly weathered rock (Wanapum Basalt)	60.0	S-15
65	Hard, gray BASALT; vesicular, close to moderately close fractures, fresh to slightly weathered (Wanapum Basalt)	64.0	RUN 1, RUN 2, RUN 3
75	(6/15/2012)	76.0	
80			

GRAPHIC LOG | GROUNDWATER



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

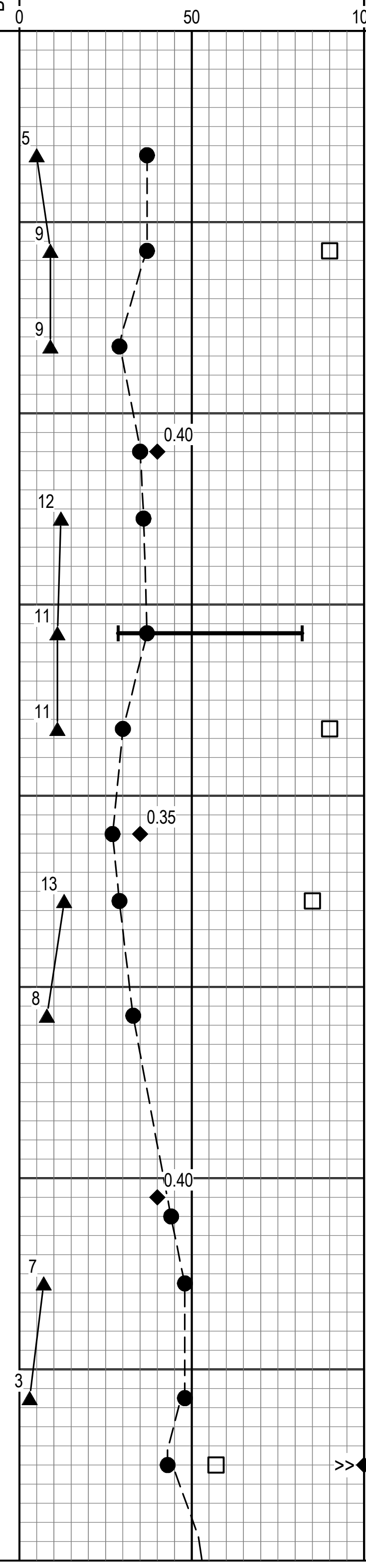
BORING B-1 (cont.)

AUG. 2015 JOB. NO. 5338-A FIG. 1A

DEPTH, FT	CLASSIFICATION OF MATERIAL Surface Elevation: 452.0 ft [±] (NAVD 88)	ELEVATION, FT / DEPTH, FT
0	CRUSHED ROCK (Fill)	451.5 / 0.5
0.5	SILT, trace to some clay, trace to some fine-grained sand, brown, medium stiff, black manganese oxide staining (Landslide Debris)	
5	---stiff below 5 ft	
10	---red-brown, some clay to clayey below 10 ft	
20	---brown to red-brown at 20 ft	
30	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)	422.0 / 30.0

SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
S-1	2 2 3	
S-2	3 4 5	
S-3	3 4 5	
S-4		Dry Density = 87.8 pcf
S-5	3 6 6	
S-6	3 5 6	
S-7	5 5 6	
S-8		
S-9	4 6 7	Dry Density = 101.5 pcf
S-10	3 4 4	
S-11		Dry Density = 80 pcf
S-12	2 3 4	
S-13	0 0 3	
S-14		Dry Density = 76 pcf

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %
□ FINES CONTENT, %
LIQUID LIMIT, %
PLASTIC LIMIT, %



◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

(CONTINUED NEXT PAGE)

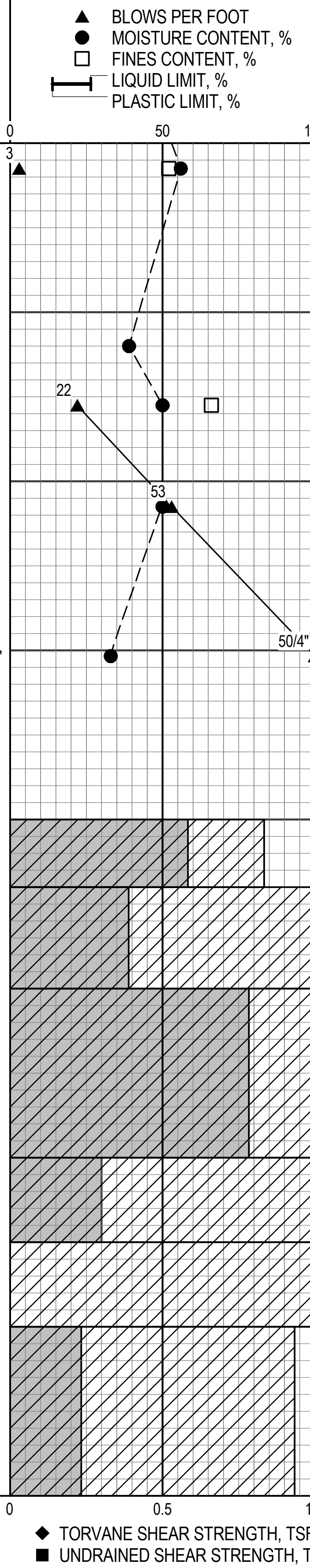
Logged By: M. Marshall	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 10/27/14	**Coordinates:** Not Available
Drilling Method: Mud Rotary **Equipment:** CME 75 HT Truck-Mounted Drill Rig **Hole Diameter:** 3.875 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 71.7 %
Note: See Legend for Explanation of Symbols	

GRI

BORING B-2

AUG. 2015 JOB NO. 5338-A FIG. 2A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** 452.0 ft [±] (NAVD 88)	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		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)			S-15		1 / 1 / 2	41.2 ft (1/7/15) 41.6 ft (11/18/14)
45					S-16			Dry Density = 84 pcf
					S-17		6 / 11 / 11	
50					S-18		23 / 23 / 30	
55		Gravel- to boulder-size fragments of BASALT, gray, decomposed to moderately weathered, soft to medium hard (R2 to R3) (Wanapum Basalt)	397.0 / 55.0		S-19		50/4"	
60		BASALT, some vesicles, gray, fresh, hard (R4), close joints (Wanapum Basalt)	392.0 / 60.0		R-1			
					R-2			
65					R-3			
70		---some vesicles below 70 ft			R-4			
					R-5			
75					R-6			
80			373.0 / 79.0					



(CONTINUED NEXT PAGE)

GRI BORING LOG GRI DATA TEMPLATE.GDT 1/16/15

GRI

BORING B-2

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** 452.0 ft [±] (NAVD 88)	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	▲ BLOWS PER FOOT ● MOISTURE CONTENT, % □ FINES CONTENT, % LIQUID LIMIT, % PLASTIC LIMIT, % (0 – 50 – 100)	COMMENTS AND ADDITIONAL TESTS
		BASALT, scoriaceous, dark red to black, slightly to moderately weathered, soft to medium hard (R2 to R3), 6-in.-thick zone with black amorphous glassy luster (carbonized wood) (Vantage Horizon of the Grande Ronde Basalt)			R-7				
85		BASALT, some vesicles, dark gray, moderately weathered, soft (R2), very close fractures, some healed (Grande Ronde Basalt) ---highly vesicular below 89 ft	367.0 85.0		R-8				
90		(10/28/2014)	362.0 90.0						
95									
100									
105									
110									
115									
120									

0 0.5 1.0

◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

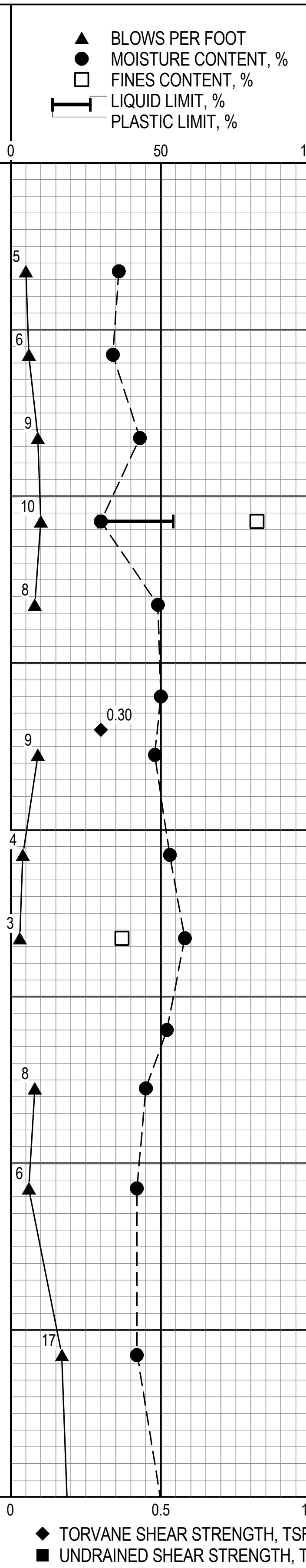
GRI BORING LOG GRI DATA TEMPLATE.GDT 1/16/15

GRI

BORING B-2

AUG. 2015 JOB NO. 5338-A FIG. 2A

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Surface Elevation: 446.0 ft [±] (NAVD 88)				
	CRUSHED ROCK (Fill)	445.5 / 0.5			
	SILT, trace to some clay, trace to some fine-grained sand, brown, medium stiff (Fill)		S-1	1 2 3	
5	SILT, some clay to clayey, trace to some fine-grained sand, brown, medium stiff (Landslide Debris)	441.0 / 5.0	S-2	0 3 3	
			S-3	2 4 5	
10			S-4	3 5 5	
	BASALT, gray-brown to red-brown, decomposed, extremely soft (R0), secondary mineralization, relict rock structure, consistency of soft to hard soil (Wanapum Basalt, Landslide Debris)	433.5 / 12.5	S-5	3 5 3	
15			S-6		Dry Density = 76 pcf
			S-7	3 4 5	
20			S-8	3 3 1	
			S-9	2 1 2	22.7 ft (11/18/14; 1/7/15)
25			S-10		Dry Density = 68 pcf
			S-11	3 4 4	
30			S-12	3 3 3	
	---boulder from 33 to 34.5 ft				
35	---gravel-size fragments of moderately weathered, soft (R2) basalt below 35 ft		S-13	5 8 9	
40					



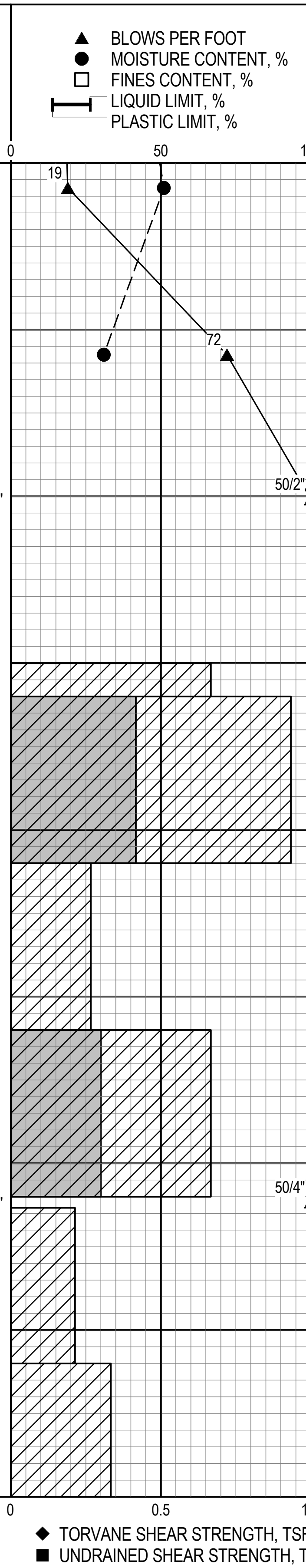
(CONTINUED NEXT PAGE)

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		

GRI

BORING B-3

DEPTH, FT	CLASSIFICATION OF MATERIAL **Surface Elevation:** 446.0 ft [±] (NAVD 88)	ELEVATION, FT / DEPTH, FT	SAMPLE NO.	BLOW COUNT
	BASALT, light brown, decomposed to moderately weathered, extremely soft to soft (R0 to R2), secondary mineralization, relict rock structure, consistency of soft to hard soil (Wanapum Basalt, Landslide Debris)		S-14	4 10 9
45			S-15	20 32 40
50	Gravel- to boulder-size fragments of BASALT, gray, slightly to moderately weathered, medium hard to hard (R3 to R4) (Wanapum Basalt)	396.0 50.0	S-16	50/2"
55	BASALT, gray, slightly weathered, medium hard to hard (R3 to R4), close joints, black carbonized wood within near-vertical (80°) closed fractures, some with chilled margin (Wanapum Basalt)	391.0 55.0	R-1	
			R-2	
60			R-3	
65	---some vesicles, fresh to slightly weathered, hard (R4), several closed near-vertical fractures, iron and manganese oxide staining along joints below 65 ft		R-4	
70	BASALT, highly vesicular, red-brown, moderately weathered, soft to very soft (R2 to R1), secondary mineralization (Vantage Horizon of the Grande Ronde Basalt)	375.0 71.0	S-17	50/4"
			R-5	
75			R-6	
80				



COMMENTS AND ADDITIONAL TESTS

(CONTINUED NEXT PAGE)

GRI

BORING B-3

AUG. 2015 JOB NO. 5338-A FIG. 3A

GRI BORING LOG GRI DATA TEMPLATE.GDT 1/16/15

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	▲ BLOWS PER FOOT ● MOISTURE CONTENT, % □ FINES CONTENT, % LIQUID LIMIT, % PLASTIC LIMIT, % (0 50 100)	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)	365.0 / 81.0						
		(10/29/2014)							
85									
90									
95									
100									
105									
110									
115									
120									

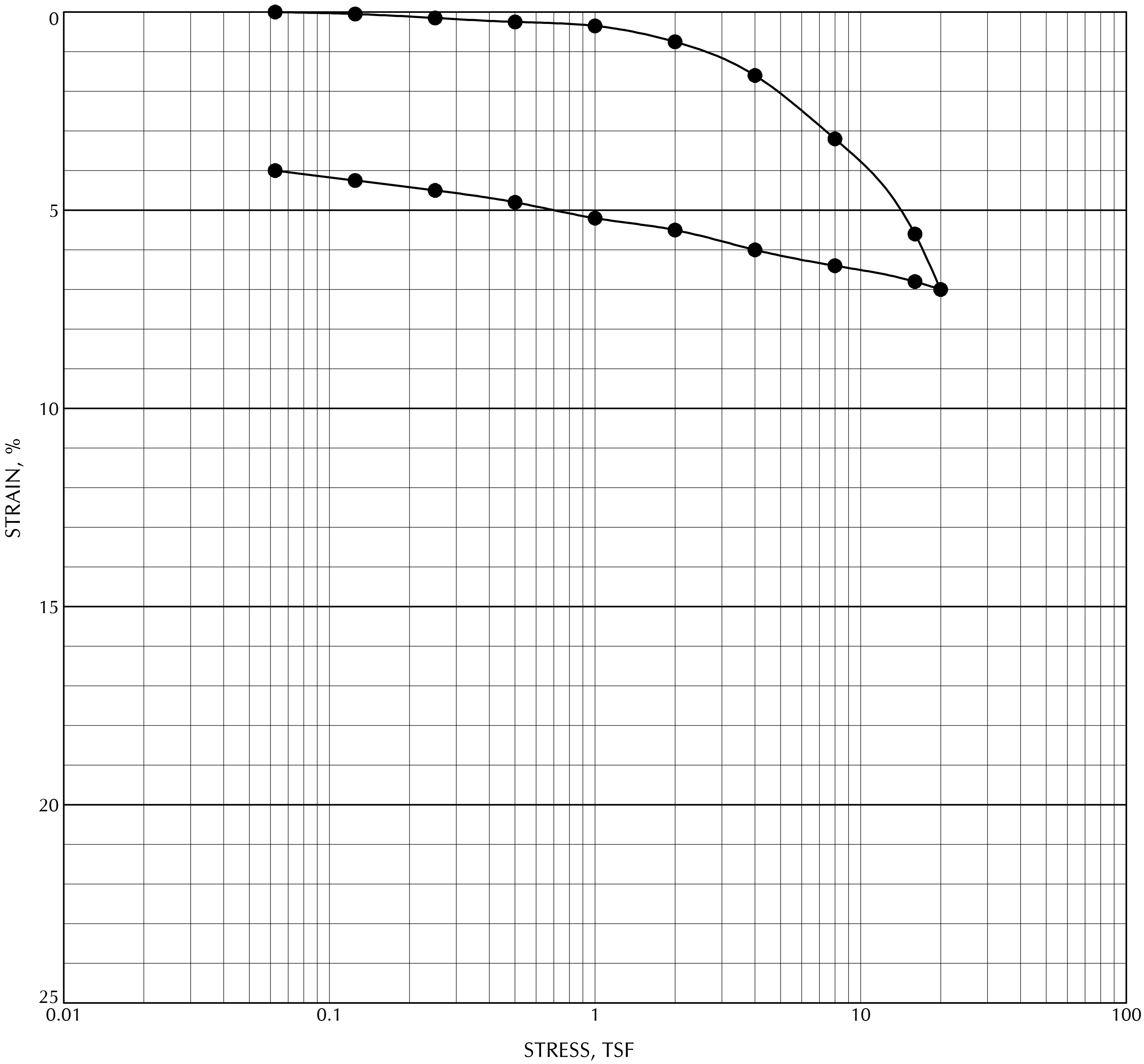
0 0.5 1.0

◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

GRI

BORING B-3

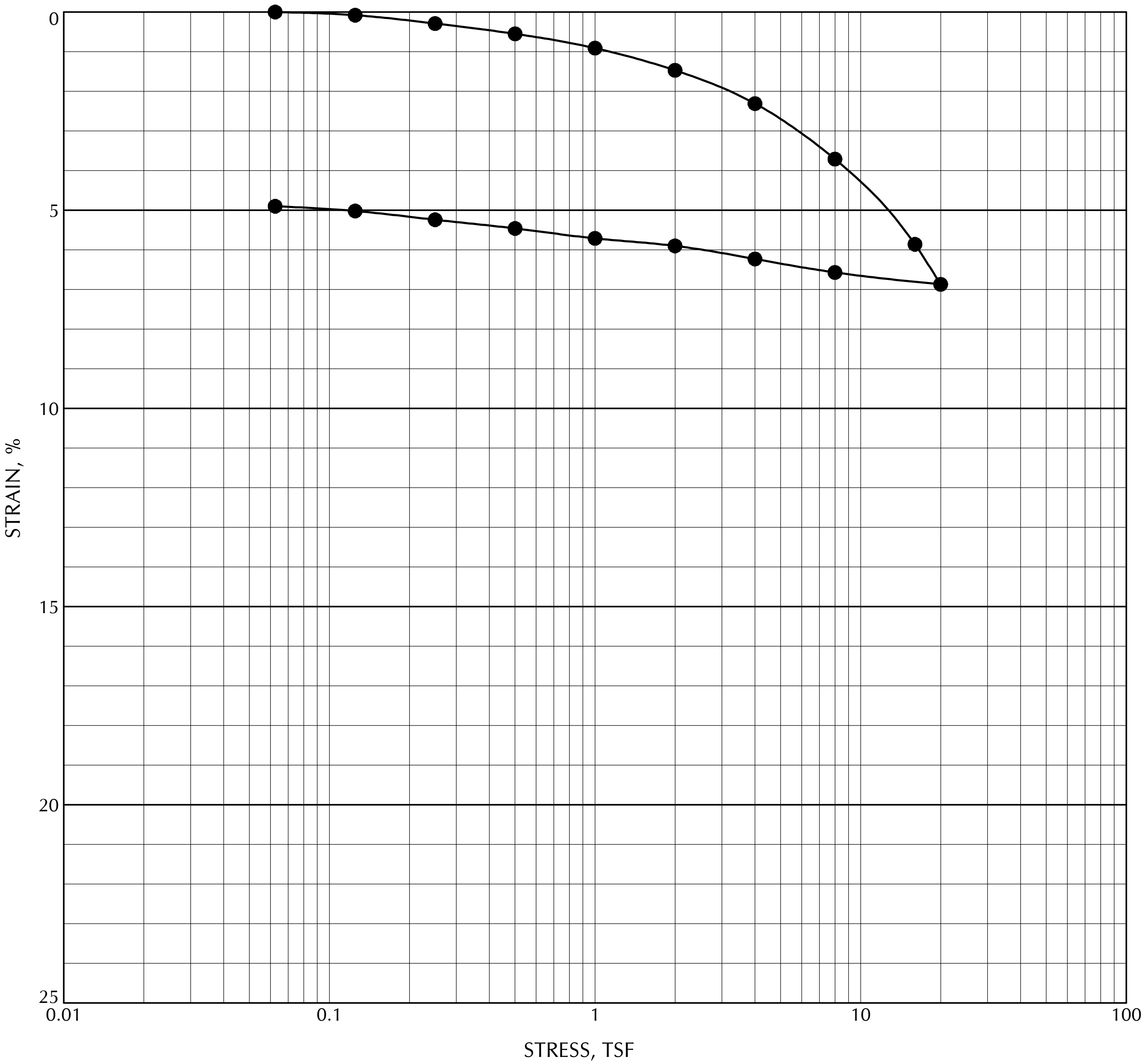
AUG. 2015 JOB NO. 5338-A FIG. 3A



	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



	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

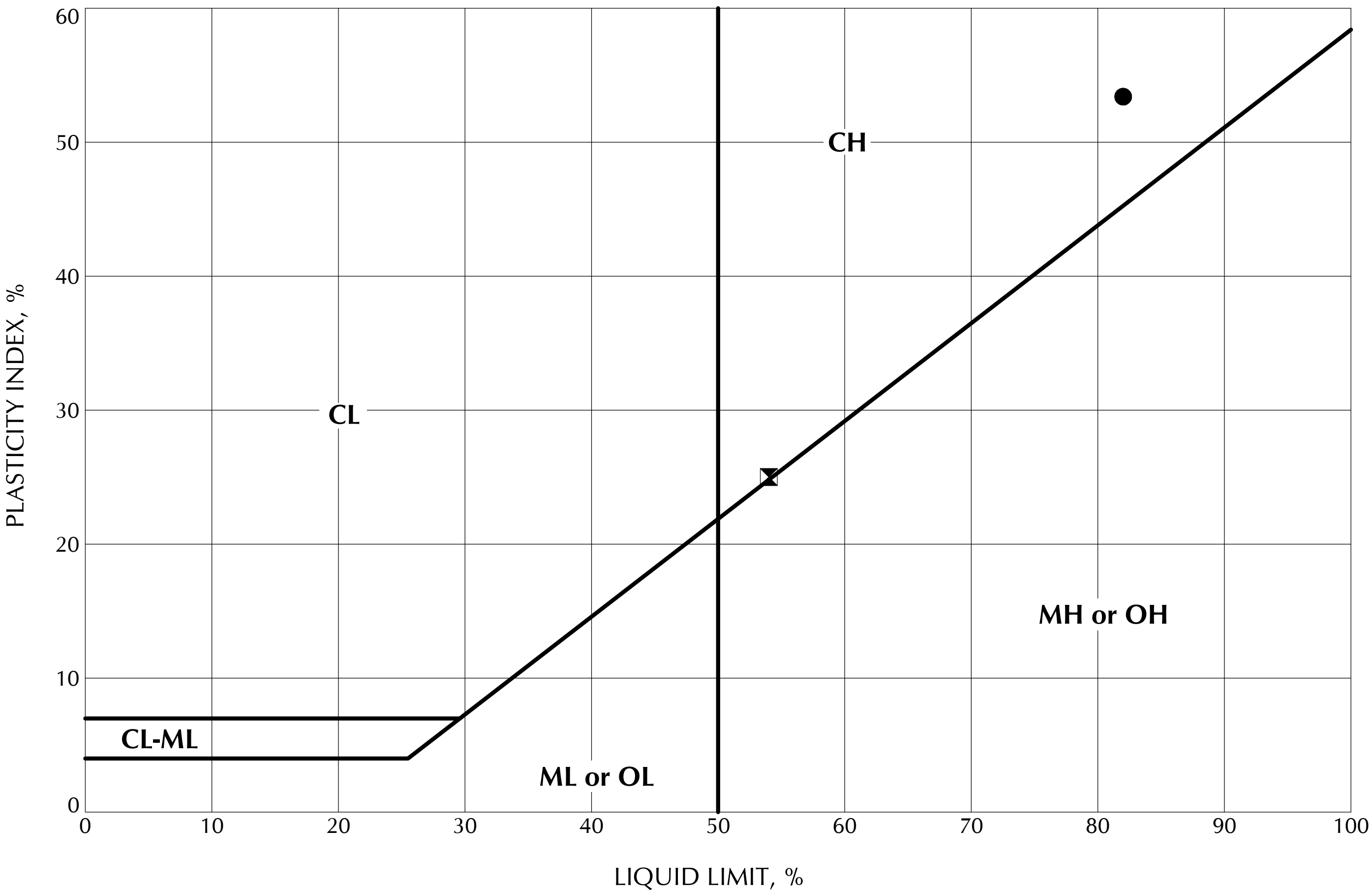
CONSOLIDATION TEST

AUG. 2015 JOB NO. 5338-A FIG. 5A

CONSOL STRAIN GRI-0 TO 25-1 PER PAGE GRI DATA TEMPLATE.GDT 1/16/15

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

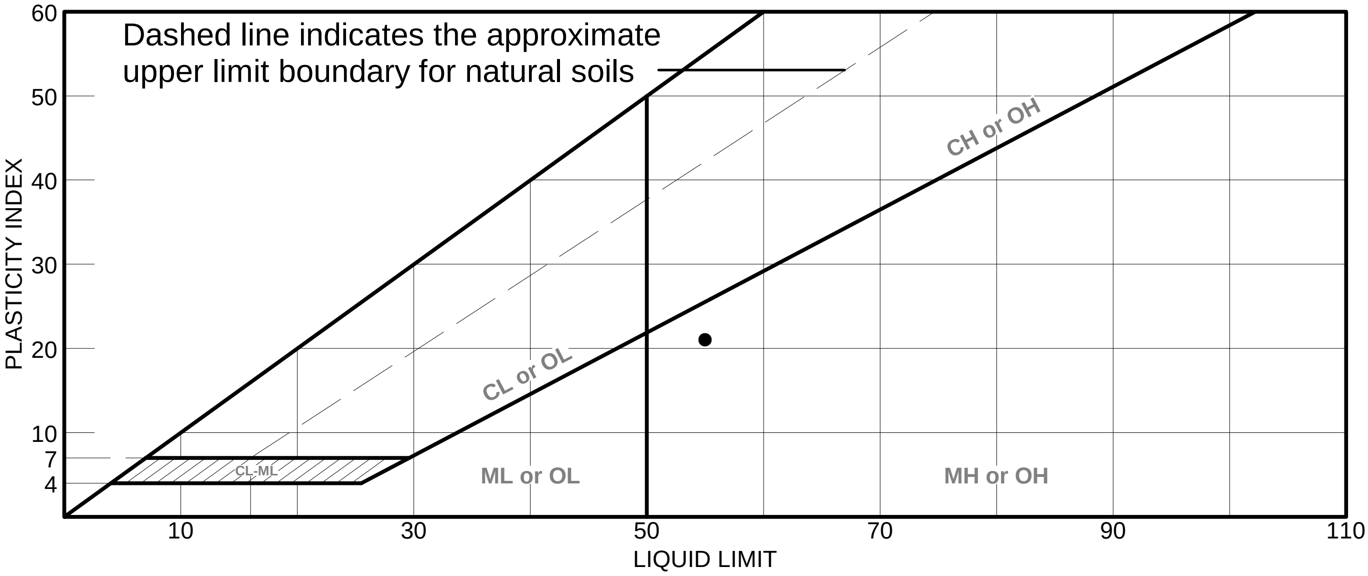


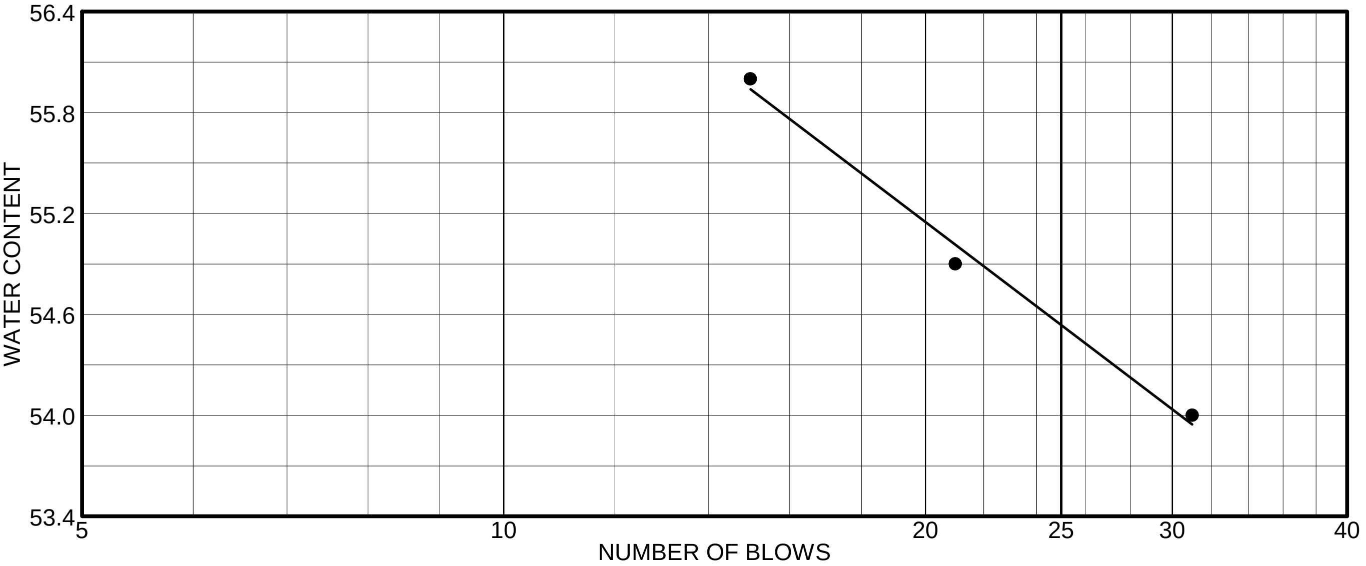
	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-2	S-6	15.0	SILT, some clay to clayey, trace to some fine-grained sand, red-brown (Landslide Debris)	82	29	53	37
⧗	B-3	S-4	10.0	SILT, some clay to clayey, trace to some fine-grained sand, brown (Landslide Debris)	54	29	25	30

GRI

PLASTICITY CHART

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

COOPER TESTING LABORATORY

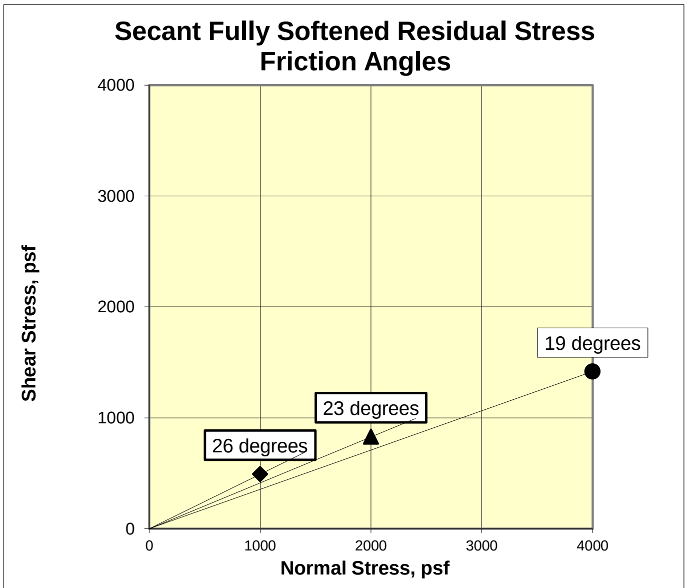
Drained Residual Torsional Shear Strength (ASTM D6467)

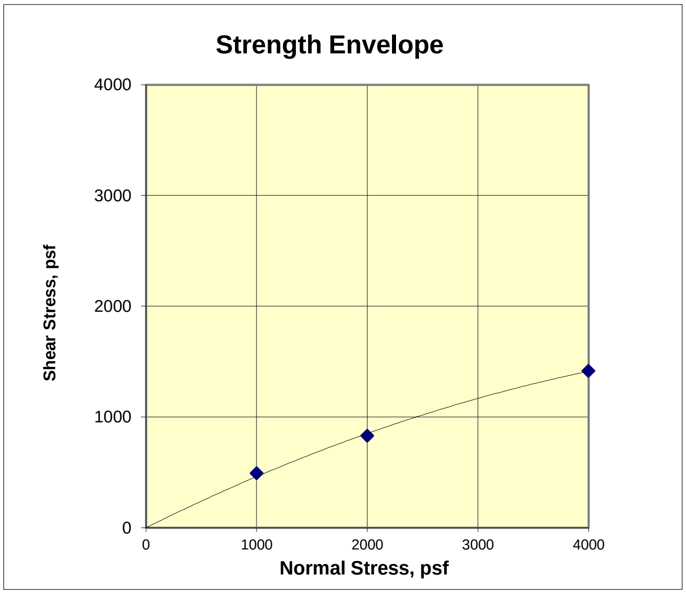
CTL Job No.:	823-005	**Boring:**	B-2	**Date:**	12/9/2014	**Clay, %:**	29.2
Client:	GRI	**Sample:**	S-13	**By:**	PJ	**LL:**	55
Project Name:	Bolton Reservoir	**Depth (ft):**	35	**Checked:**	DC	**PL:**	34
Project Number:	5338-A	**Test Type:**	Fully Softened Residual				

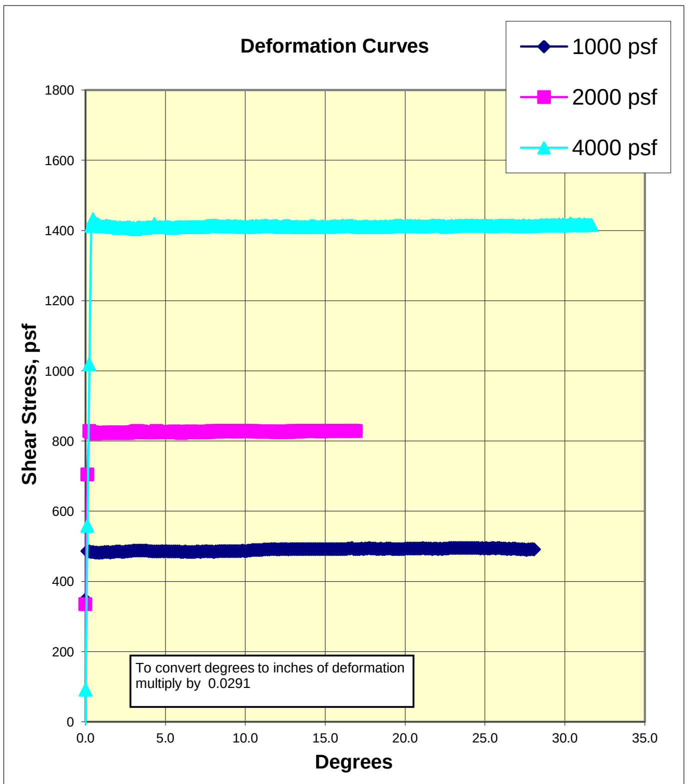
Soil Type: Strong Brown Sandy Elastic SILT

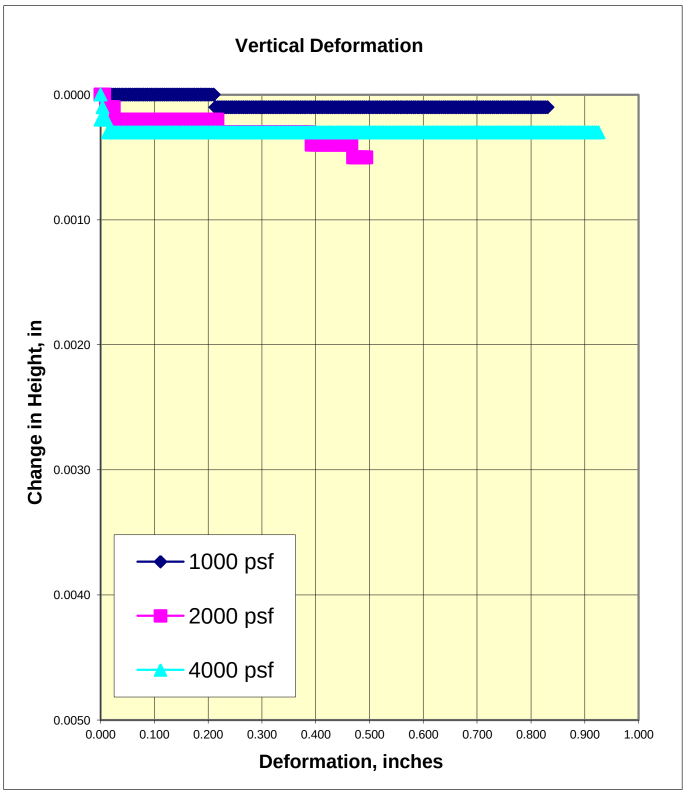
Normal Stress, psf:	1000	2000	4000
Secant Phi, deg.:	26	23	19

Remarks: This sample has an unusually highly curved strength envelope.

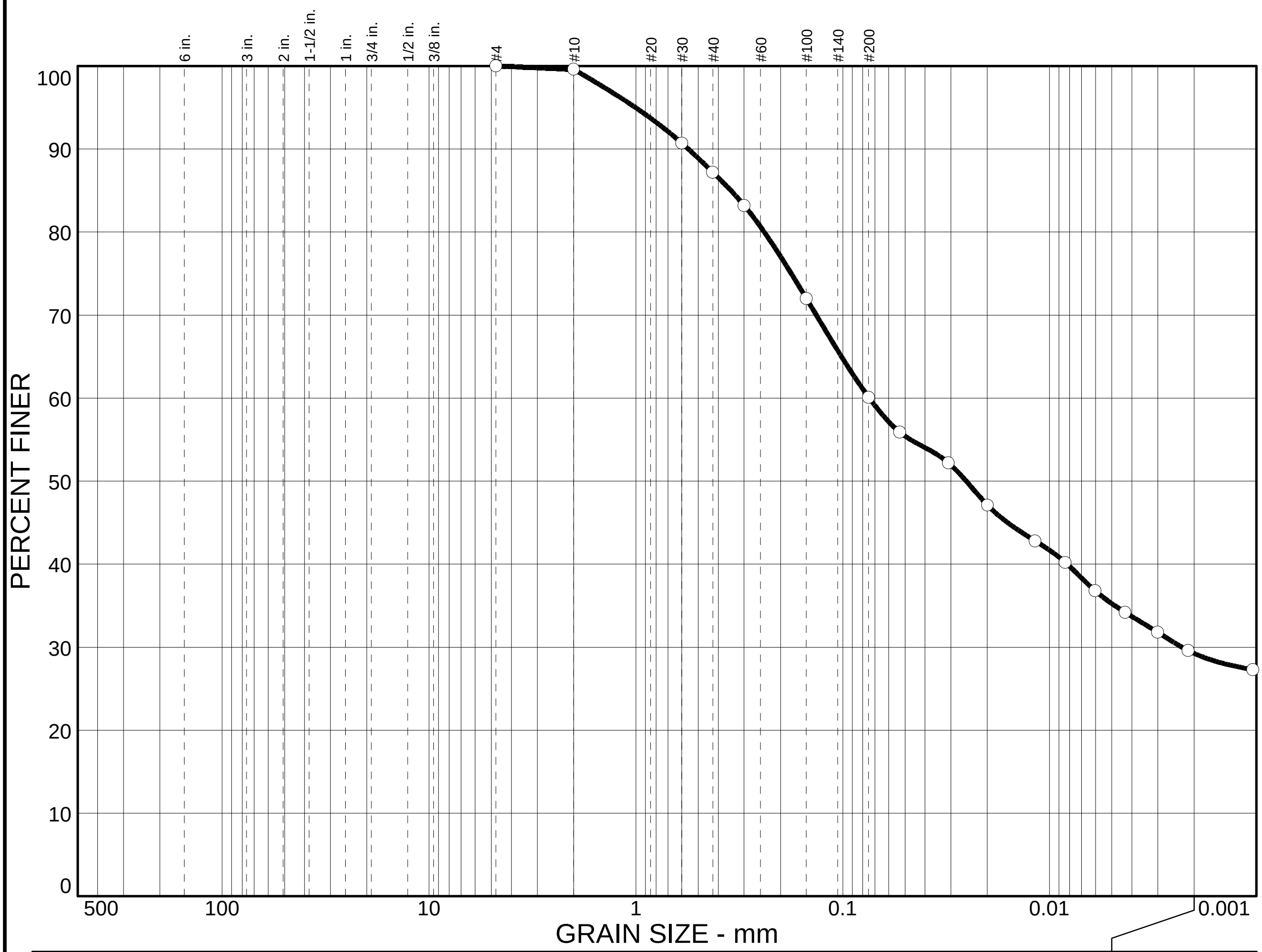








Particle Size Distribution Report



% 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'

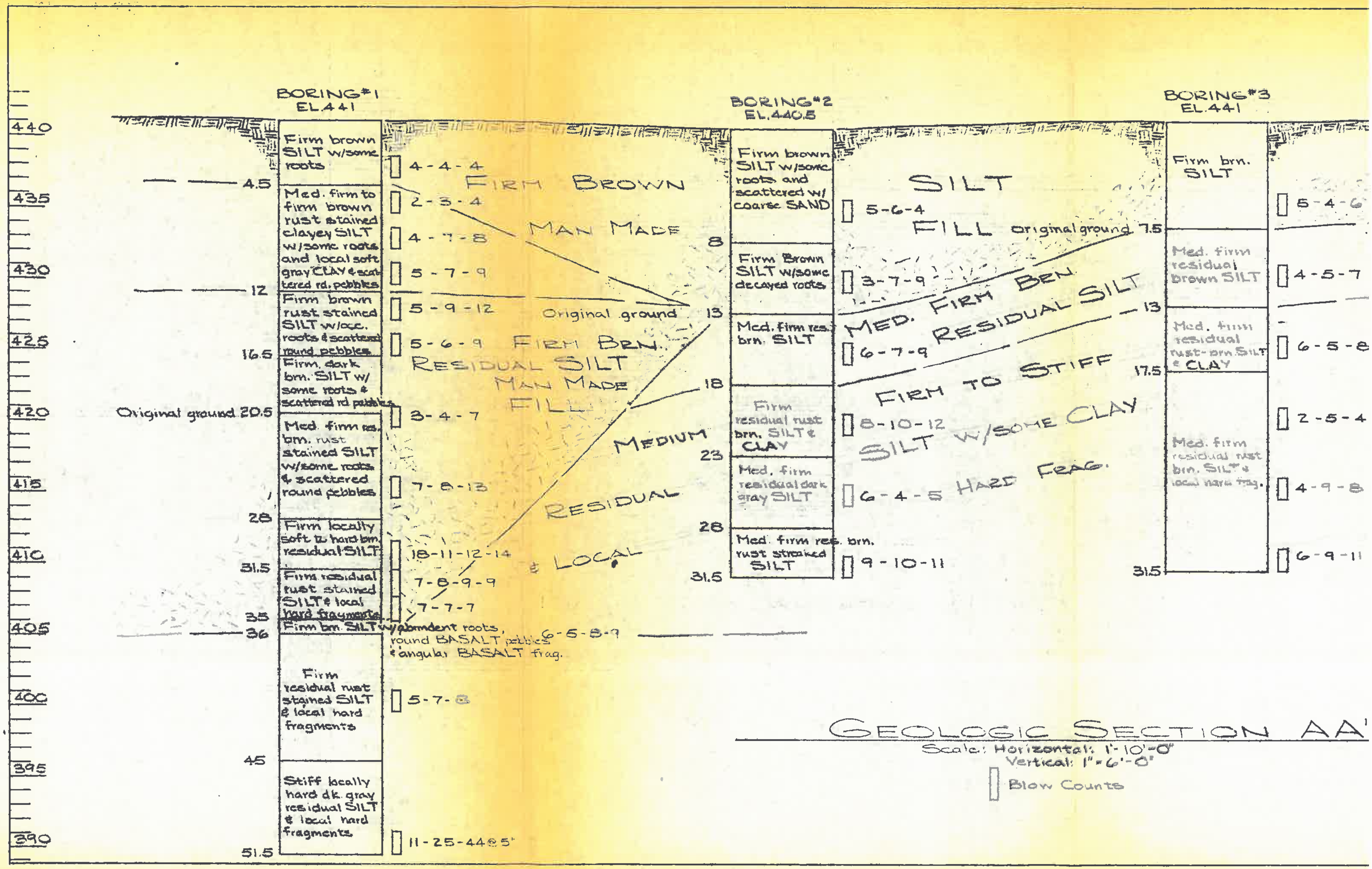
COOPER TESTING LABORATORY

Client: GRI
Project: Bolton Reservoir - 5338-A

Project No: 823-005 **Figure**

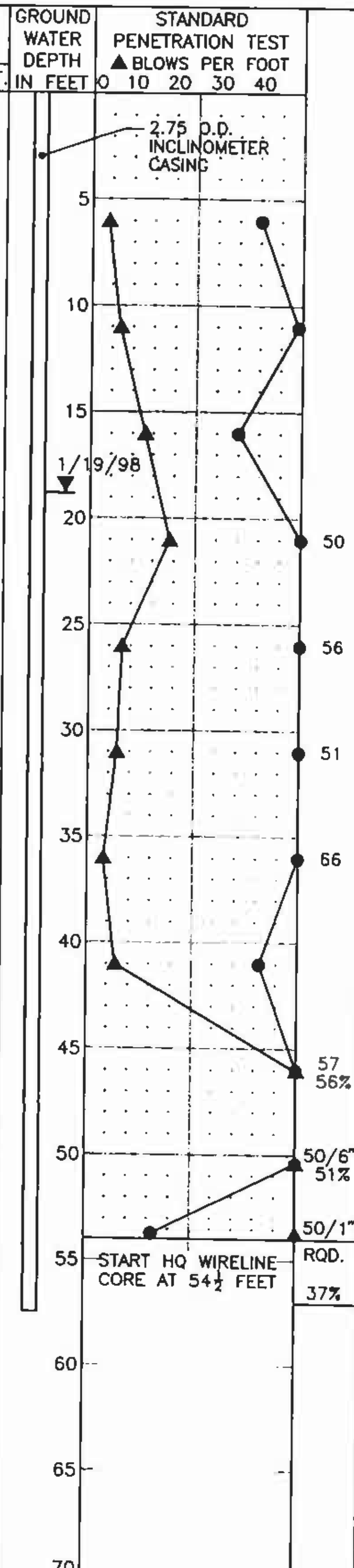
APPENDIX B

Logs of Subsurface Explorations by Others



BORING #1
EL. 441
BORING #2
EL. 440.5
BORING #3
EL. 441
Firm brown SILT w/some roots
Med. firm to firm brown rust stained clayey SILT w/some roots and local soft gray CLAY & scattered rd. pebbles
Firm brown rust stained SILT w/occ. roots & scattered round pebbles
Firm, dark brn. SILT w/ some roots & scattered rd pebbles
Med. firm res. brn. rust stained SILT w/some roots & scattered round pebbles
Firm locally soft to hard brn residual SILT
Firm residual rust stained SILT & local hard fragments
Firm brn. SILT w/abundent roots, round BASALT pebbles & angular BASALT frag.
Firm residual rust stained SILT & local hard fragments
Stiff locally hard dk. gray residual SILT & local hard fragments
4-4-4
2-3-4
4-7-8
5-7-9
5-9-12
5-6-9
3-4-7
7-8-13
18-11-12-14
7-8-9-9
7-7-7
6-5-8-9
5-7-8
11-25-44 @ 5"
Original ground 20.5
FIRM BROWN MAN MADE
Original ground
FIRM BRN. RESIDUAL SILT
MAN MADE FILL
MEDIUM RESIDUAL & LOCAL
Firm brown SILT w/some roots and scattered w/ coarse SAND
Firm Brown SILT w/some decayed roots
Med. firm res. brn. SILT
Firm residual rust brn. SILT & CLAY
Med. firm residual dark gray SILT
Med. firm res. brn. rust streaked SILT
5-6-4
3-7-9
6-7-9
8-10-12
6-4-5
9-10-11
SILT FILL
original ground 7.5
MED. FIRM BRN. RESIDUAL SILT
FIRM TO STIFF SILT W/SOME CLAY
HARD FRAG.
Firm brn. SILT
Med. firm residual brown SILT
Med. firm residual rust-brn. SILT & CLAY
Med. firm residual rust brn. SILT & local hard frag.
5-4-6
4-5-7
6-5-8
2-5-4
4-9-8
6-9-11
440
435
430
425
420
415
410
405
400
395
390
4.5
12
16.5
28
31.5
35
36
45
51.5
8
13
18
23
28
31.5
7.5
13
17.5
31.5
GEOLOGIC SECTION AA'
Scale: Horizontal: 1" - 10'-0"
Vertical: 1" = 6'-0"
Blow Counts

ELEV.	DEPTH IN FEET	MATERIAL DESCRIPTION	SAMPLES NO.	S.P.T.
		SURFACE ELEV. (FT.) 437 (approx.)		
		SOFT, to MEDIUM STIFF, brown, slightly clayey to clayey SILT (FILL).	1	4
			2	7
424	13	SOFT to STIFF, mottled orange-brown and gray, sandy, slightly clayey SILT (DECOMPOSED BASALT).	3	13
			4	19
			5	8
		...becomes gravelly	6	7
		...becomes gray	7	4
			8	7
395	42	VERY DENSE, gray, slightly clayey, sandy SILT; relic rock texture (DECOMPOSED BASALT).	9	57
			10	50/6"
			11	50/1"
382½	54½	SOFT to MEDIUM HARD, grey, moderately weatherd BASALT; highly jointed.	RUN NO. 1	CORE RECV. 90%
379½	57½	Bottom of Boring: 57½ feet		



LEGEND

- 2" S.P.T. SAMPLE
- * SAMPLE NOT RECOVERED
- U 3" THIN WALL SAMPLE
- P PITCHER SAMPLE
- IMPERVIOUS SEAL
- WATER LEVEL
- PIEZOMETER TIP
- LIQUID LIMIT / NATURAL WATER CONTENT / PLASTIC LIMIT
- • WATER CONTENT IN PERCENT

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

JOB
BOLTON RESERVOIR LANDSLIDE
WEST LINN, OREGON

DATE JAN 1998
JOB NO. 1057
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.

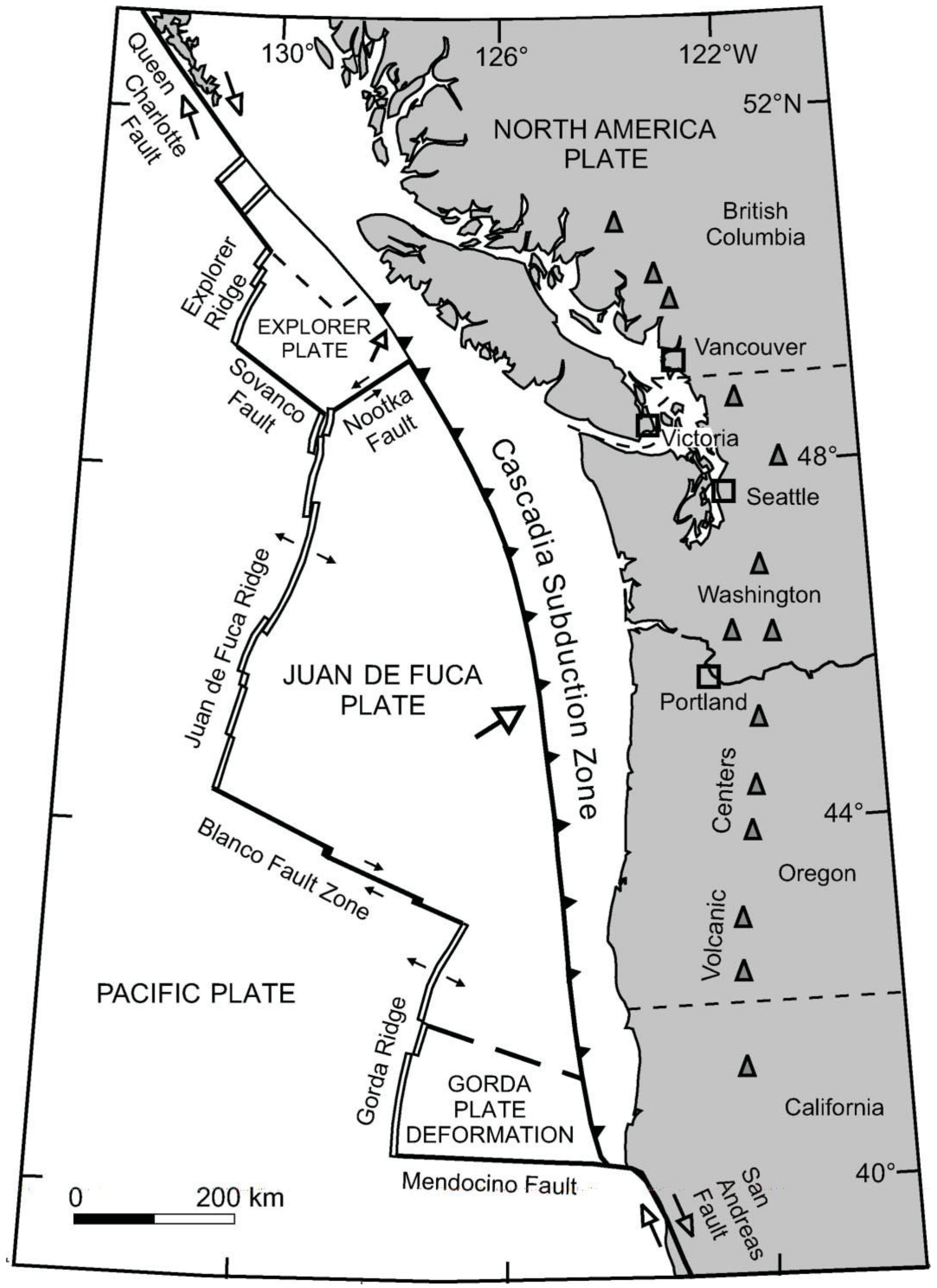
Witter, R.C., Kelsey, H.M., Hemphill-Haley, E., 2003, Great Cascadia earthquakes and tsunamis of the past 6700 years, Coquille River estuary, southern coastal Oregon. Geological Society of America Bulletin 115, 1289–1306.

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.

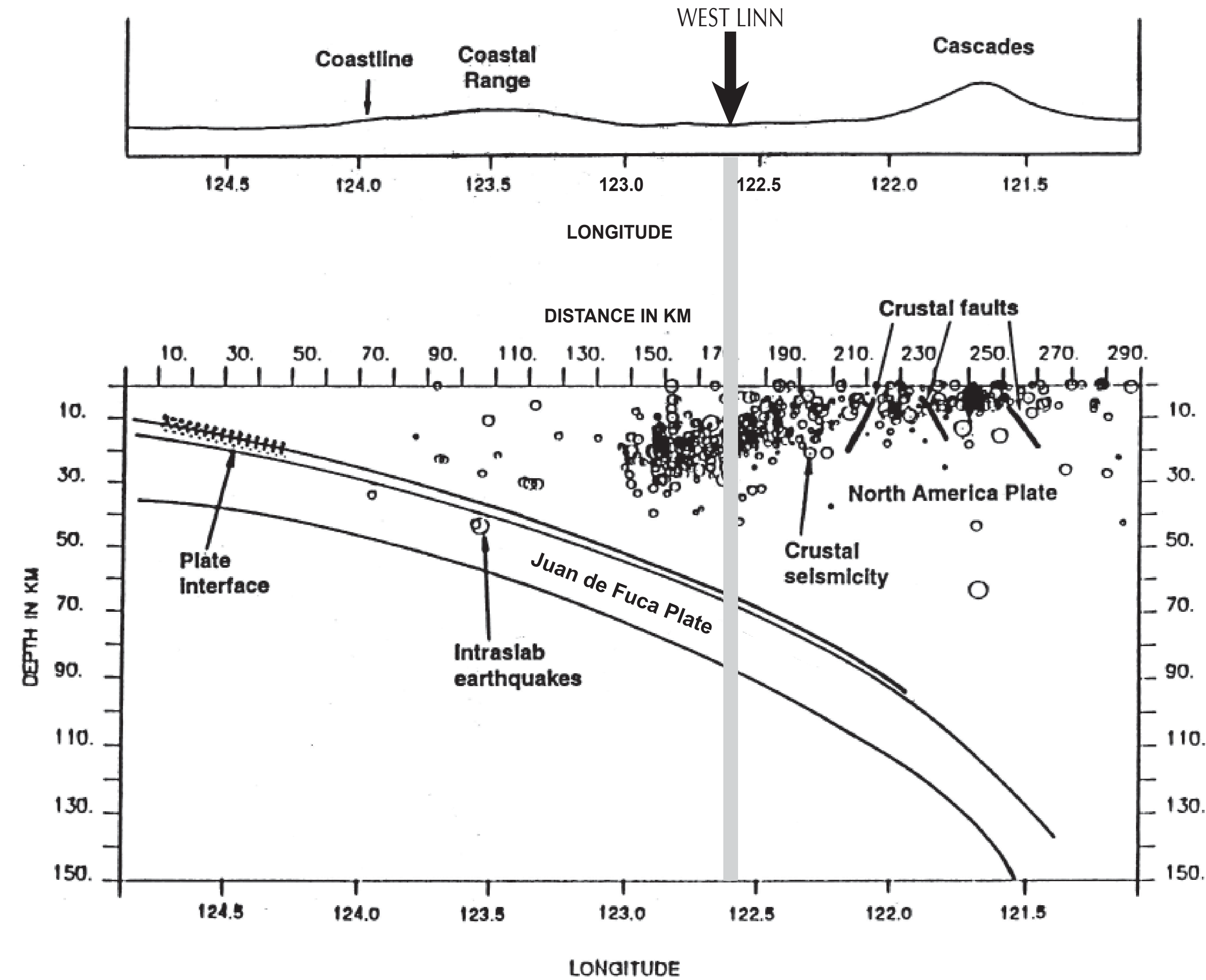
Yeats, R.S., Graven, E.P., Werner, K.S., Goldfinger, C., and Popowski, T., 1991, Tectonics of the Willamette Valley, Oregon: U. S. Geological Survey Open-File Report 91-441-P, 47 p.

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)

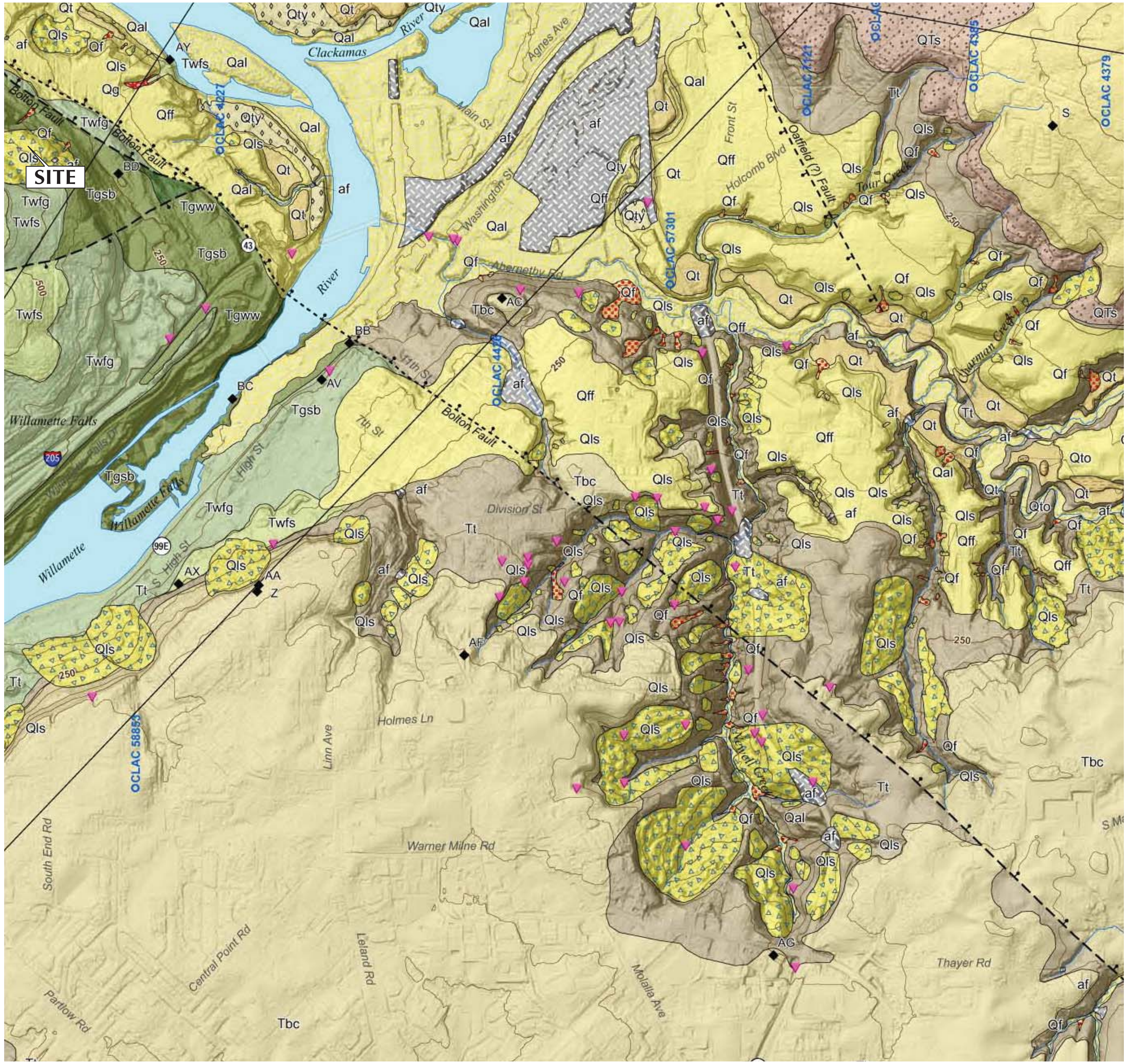


B) EAST-WEST CROSS-SECTION THROUGH WESTERN OREGON AT THE LATITUDE OF PORTLAND, SHOWING THE SEISMIC SOURCES CONSIDERED IN THE SITE-SPECIFIC SEISMIC HAZARD STUDY (MODIFIED FROM GEOMATRIX, 1995)

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

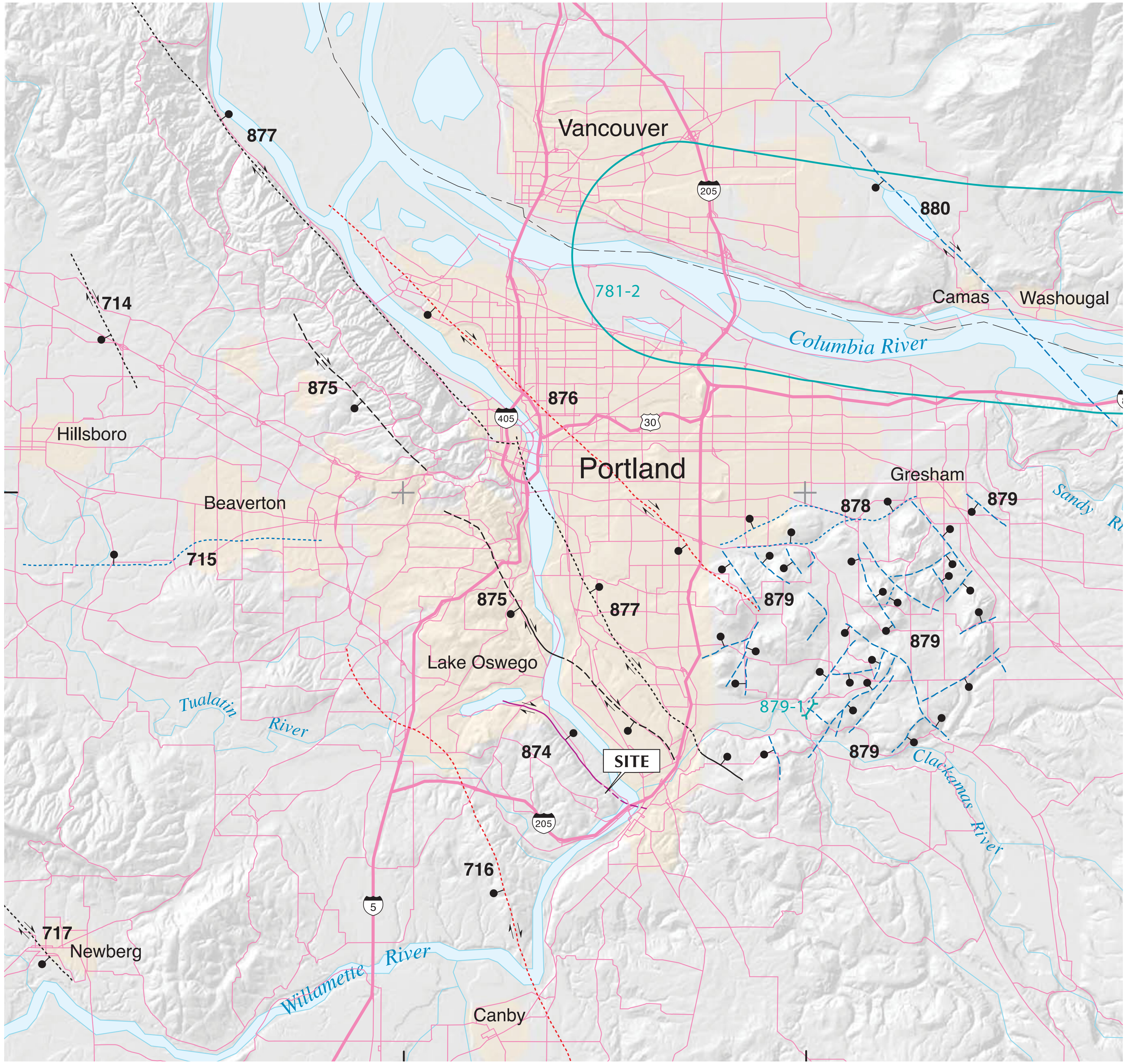
0 1/2 1 MILE

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

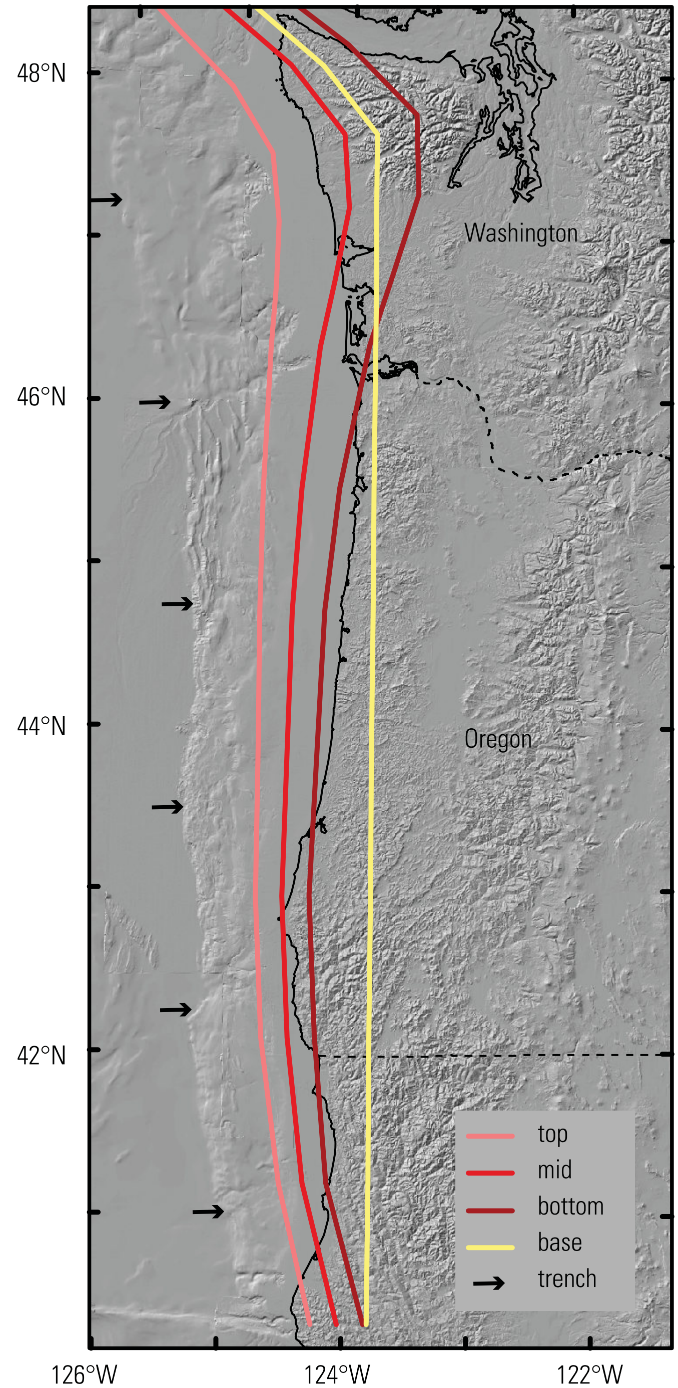


FIGURE 21. LOCATION OF THE EASTERN EDGE OF EARTHQUAKE RUPTURE ZONES ON THE CASCADIA SUBDUCTION ZONE FOR THE VARIOUS MODELS USED IN THIS STUDY RELATIVE TO THE SURFICIAL EXPRESSION OF THE TRENCH: TOP, BASE OF THE ELASTIC ZONE; MID, MIDPOINT OF THE TRANSITION ZONE; BOTTOM, BASE OF THE TRANSITION ZONES; 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)

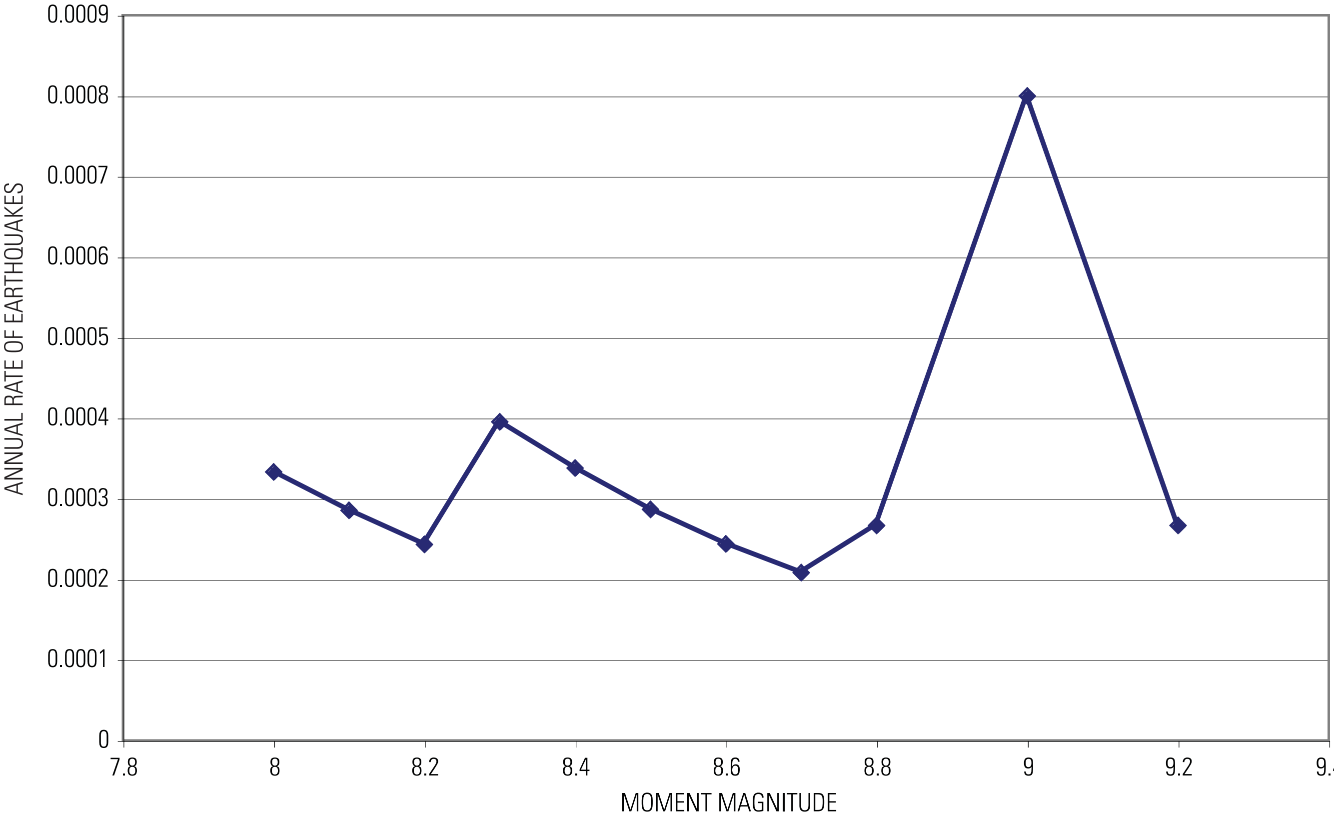


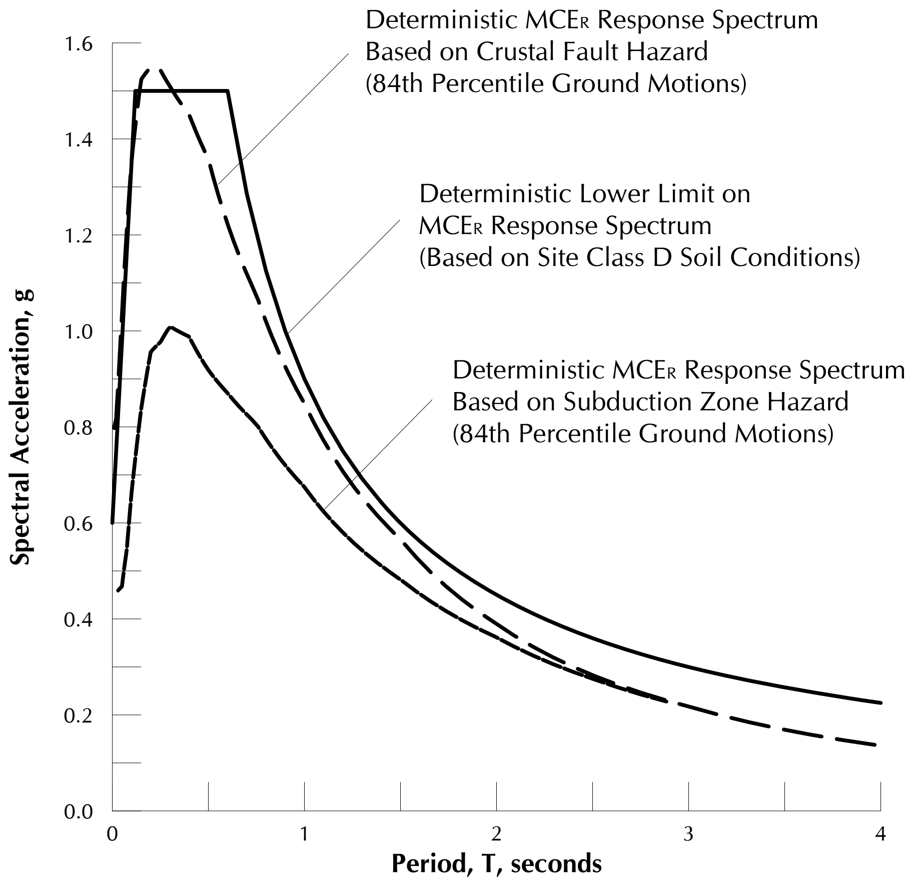
Figure 22. Magnitude-frequency distribution of the Cascadia subduction zone.

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

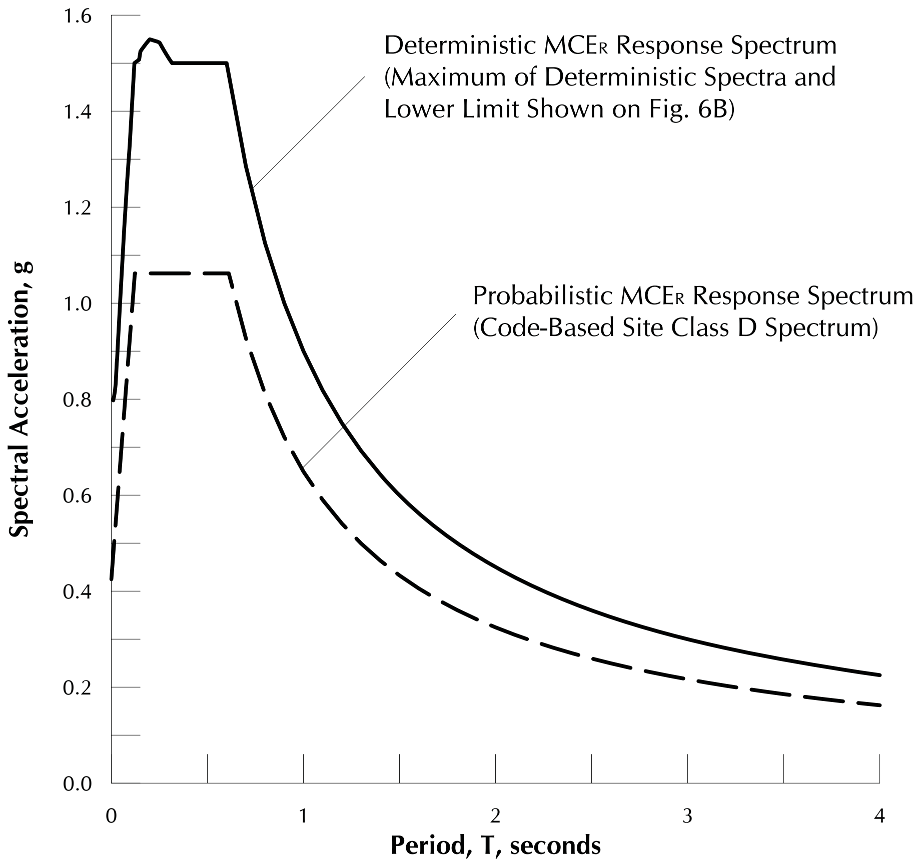


GRI

DETERMINISTIC MCE$_R$ RESPONSE SPECTRA

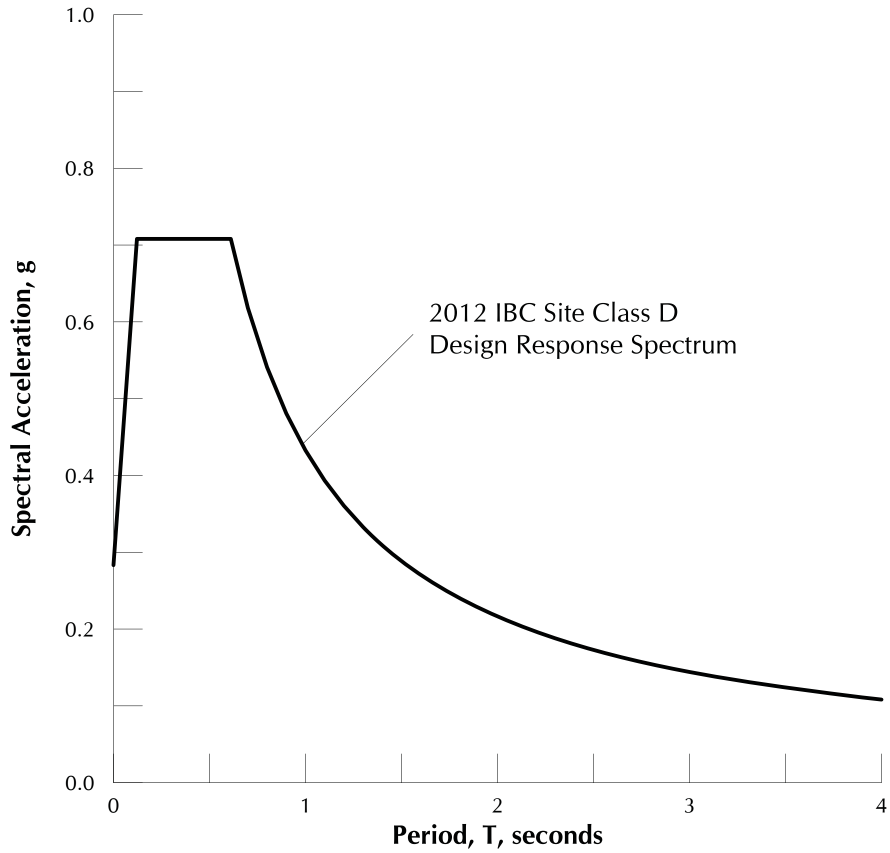
(5% DAMPING)

AUG. 2015 | JOB NO. 5338-A | FIG. 6C



GRI

PROBABILISTIC AND DETERMINISTIC MCE$_R$
RESPONSE SPECTRA COMPARISON
(5% DAMPING)

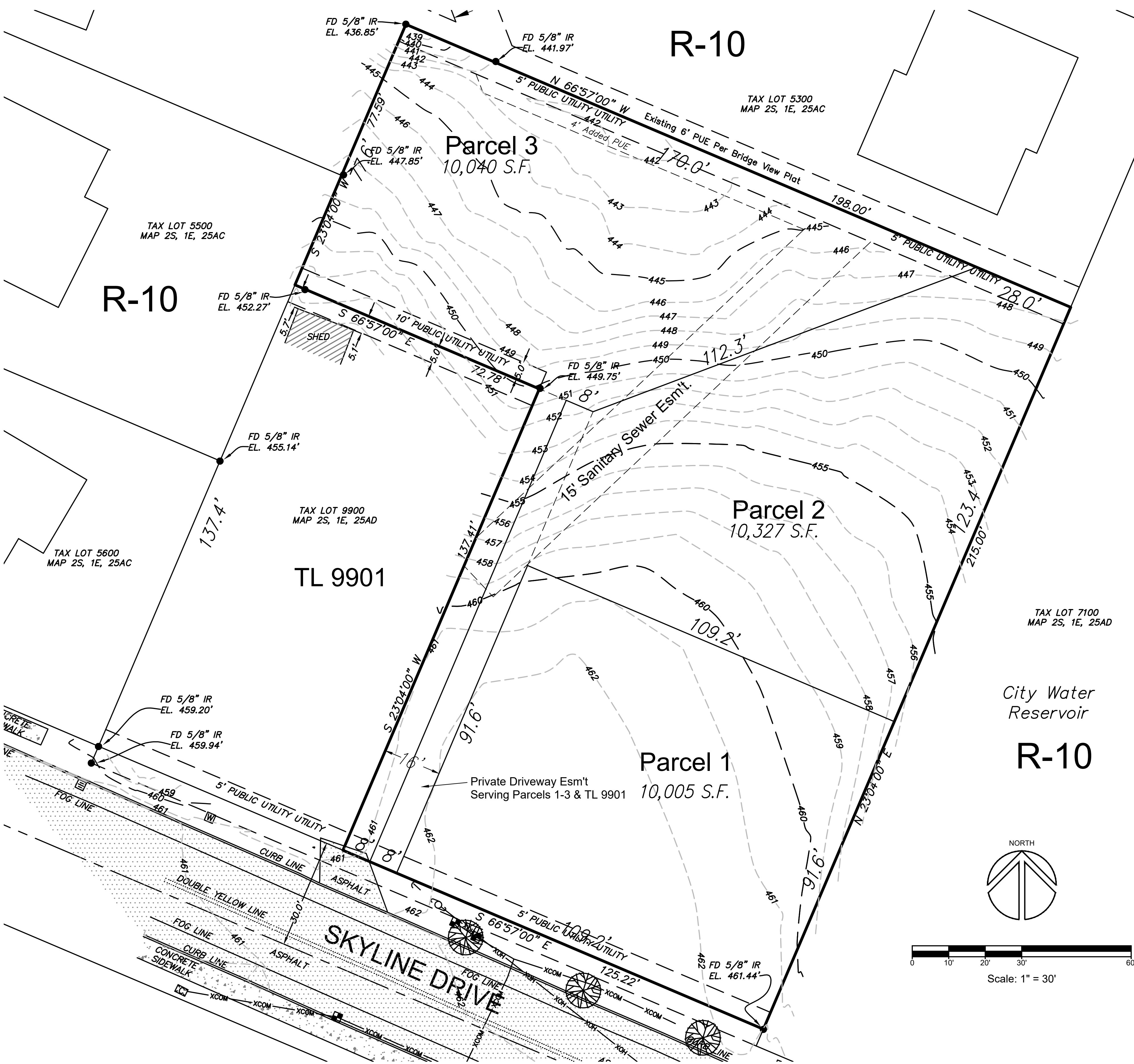


GRI

DESIGN RESPONSE SPECTRUM
(5% DAMPING)

AUG. 2015 JOB NO. 5338-A FIG. 8C

Vicinity Map
6123 Skyline Drive



Owner:
City of West Linn
22500 Salamo Road
West Linn, OR 97068
PH: (503) 657-0331

Legal: 2S 1E 25AD 9900

Water: City of West Linn

Sewer: City of West Linn

Contours: Centerline Concepts, Inc.

Site Area: 32,569 S.F.

Engineer:
Theta Engineering
PO Box 1345
Lake Oswego, OR 97035
PH: (503) 481-8822

Zoning: R-10

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.			
DRAWN:	R.E.G.			
SCALE:	1" = 30'			
DATE:	September 2018			
FILE:	18-ICN-102	DATE	NO.	REVISION

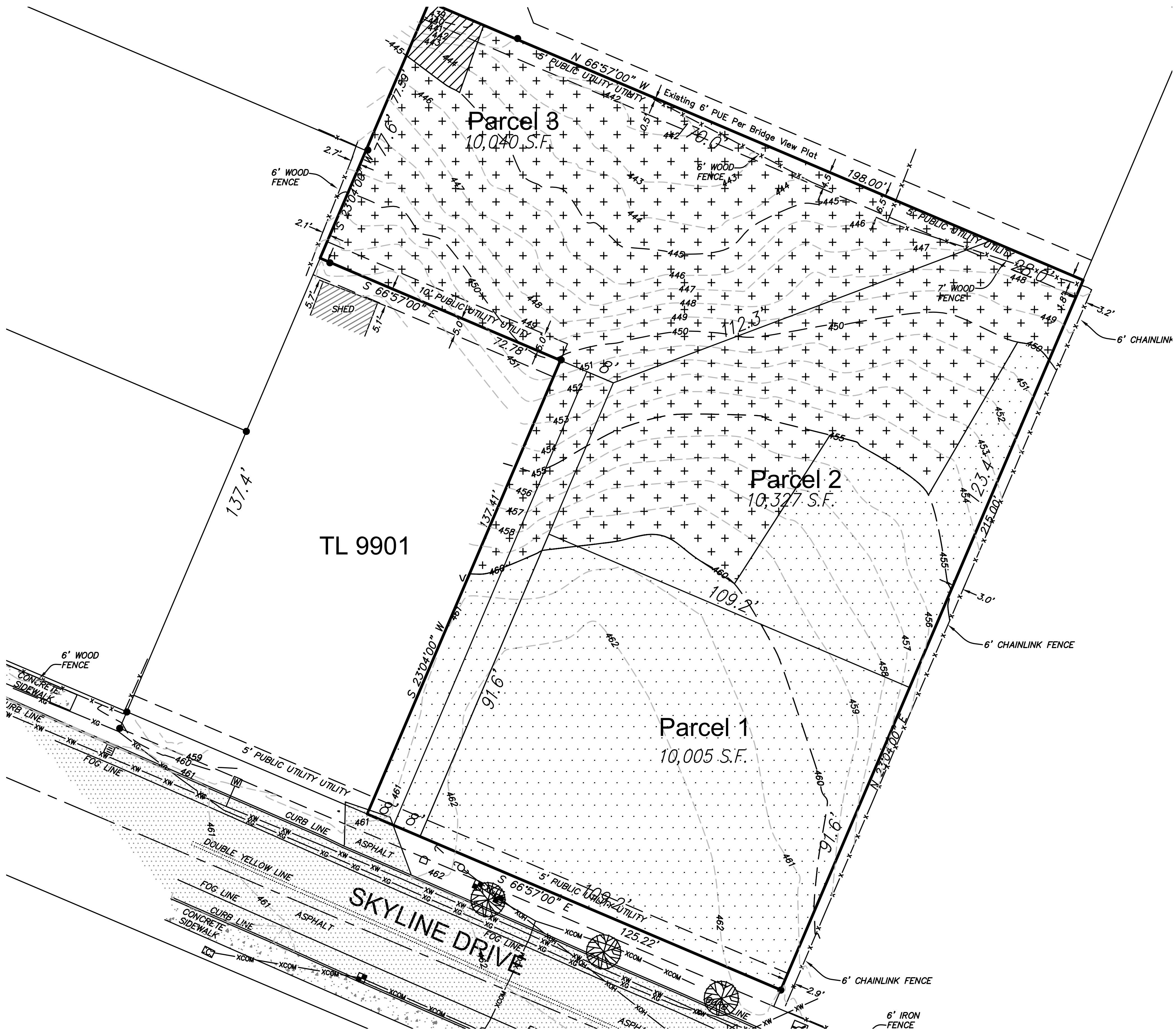
Richard E. Givens, Planning Consultant
18680 Sunblaze Dr.
Oregon City, OR 97045
PH: (503) 479-0097

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

SHEET:
1/3



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

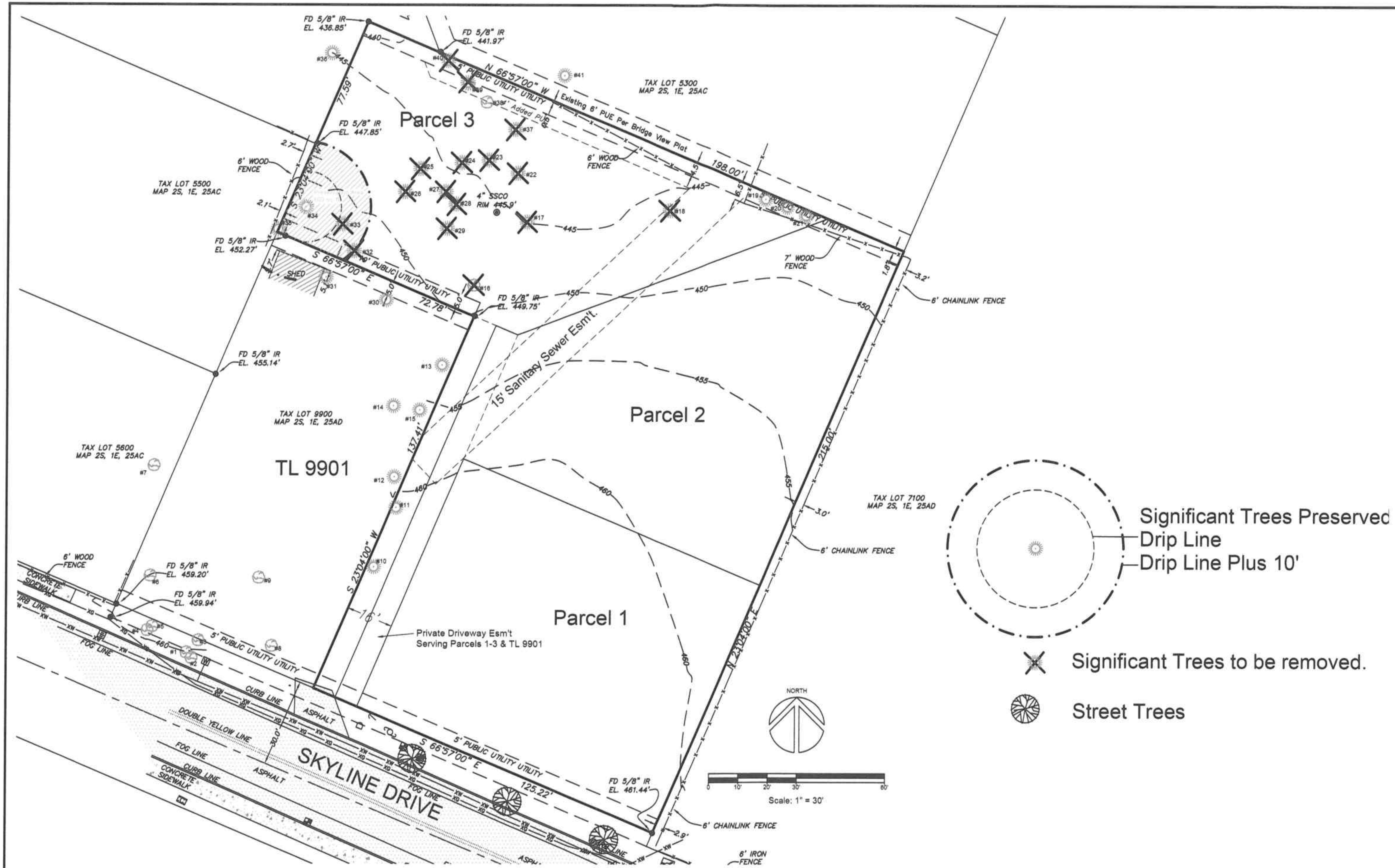
Richard E. Givens, Planning Consultant
18680 Sunblaze Dr.
Oregon City, OR 97045
PH: (503) 479-0097

APPLICANT: Icon Construction & Development, LLC
1980 Willamette Falls Drive, Suite 200
West Linn, OR 97068
PH: (503) 657-0406

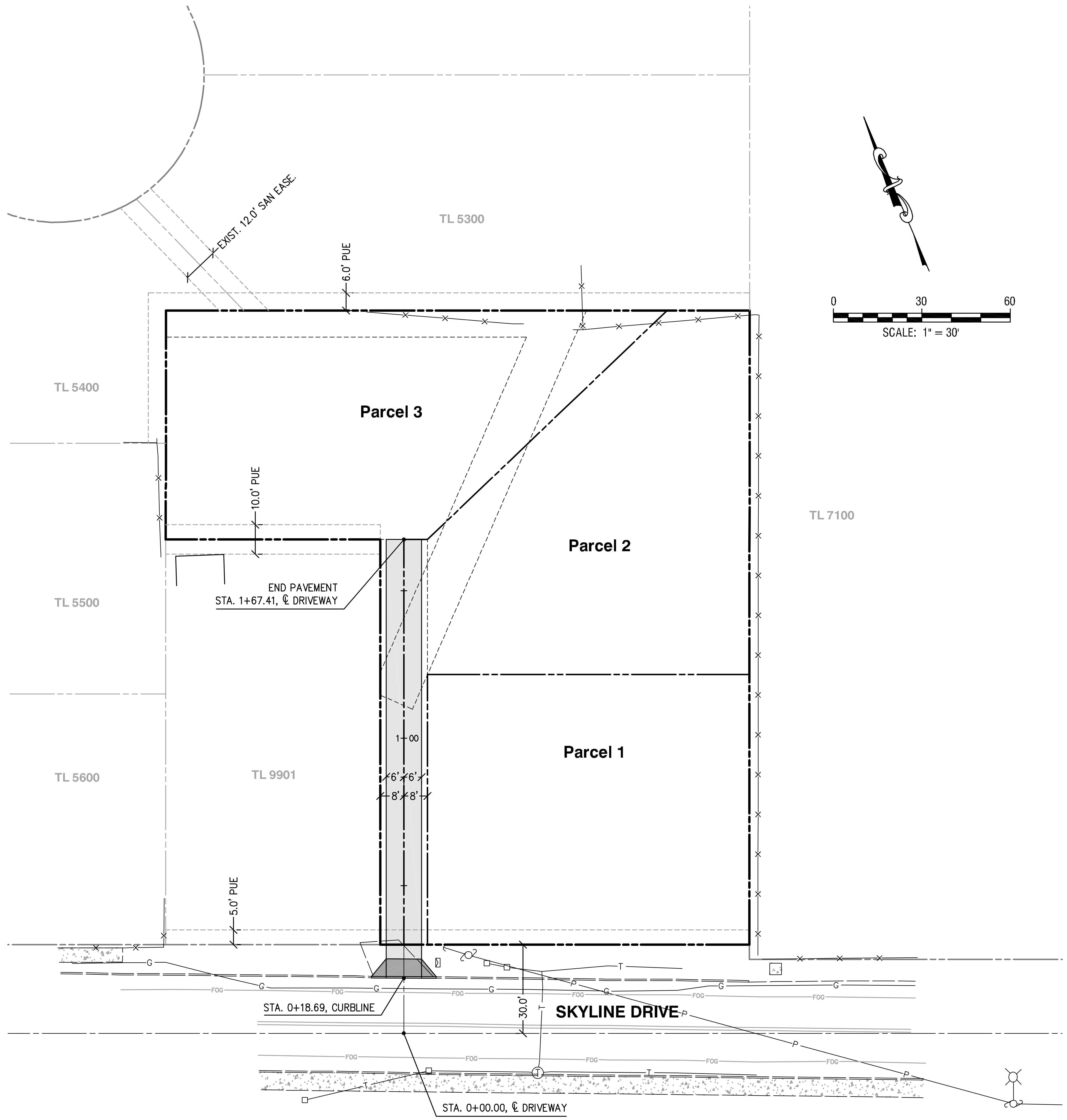
Partition Slope Analysis

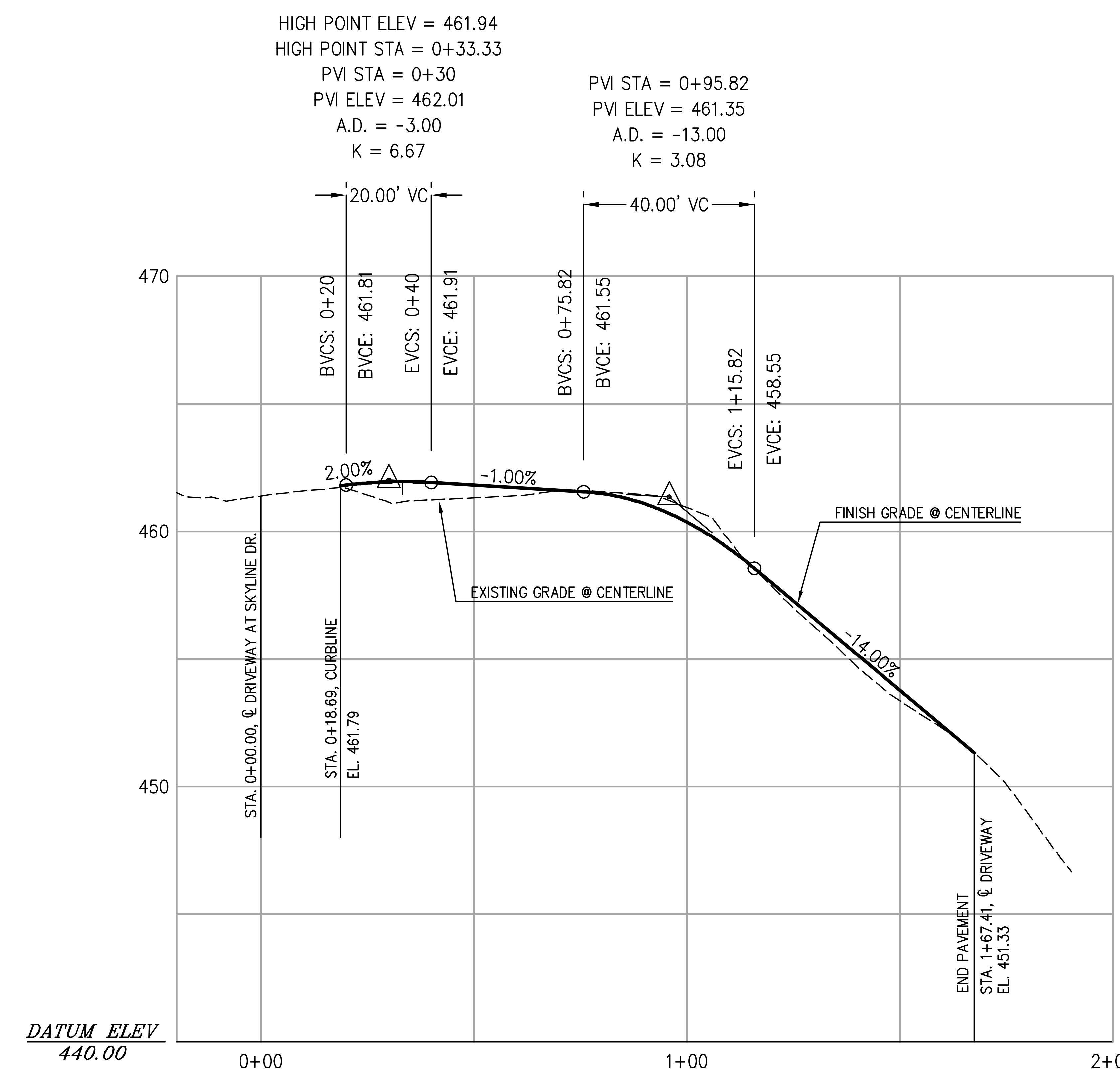
6123 Skyline Drive, West Linn, OR

SHEET: 2/3



DESIGNED:	R.E.G.	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
DRAWN:	R.E.G.				
SCALE:	1" = 30'				
DATE:	September 2018				
FILE:	18-ICN-102				
DATE	NO.	REVISION			





DRIVEWAY CENTERLINE PROFILE 1/1

SCALE: 1" = 30' HORIZONTAL
1" = 5' VERTICAL

2014-129U

PRELIMINARY STREET PLAN

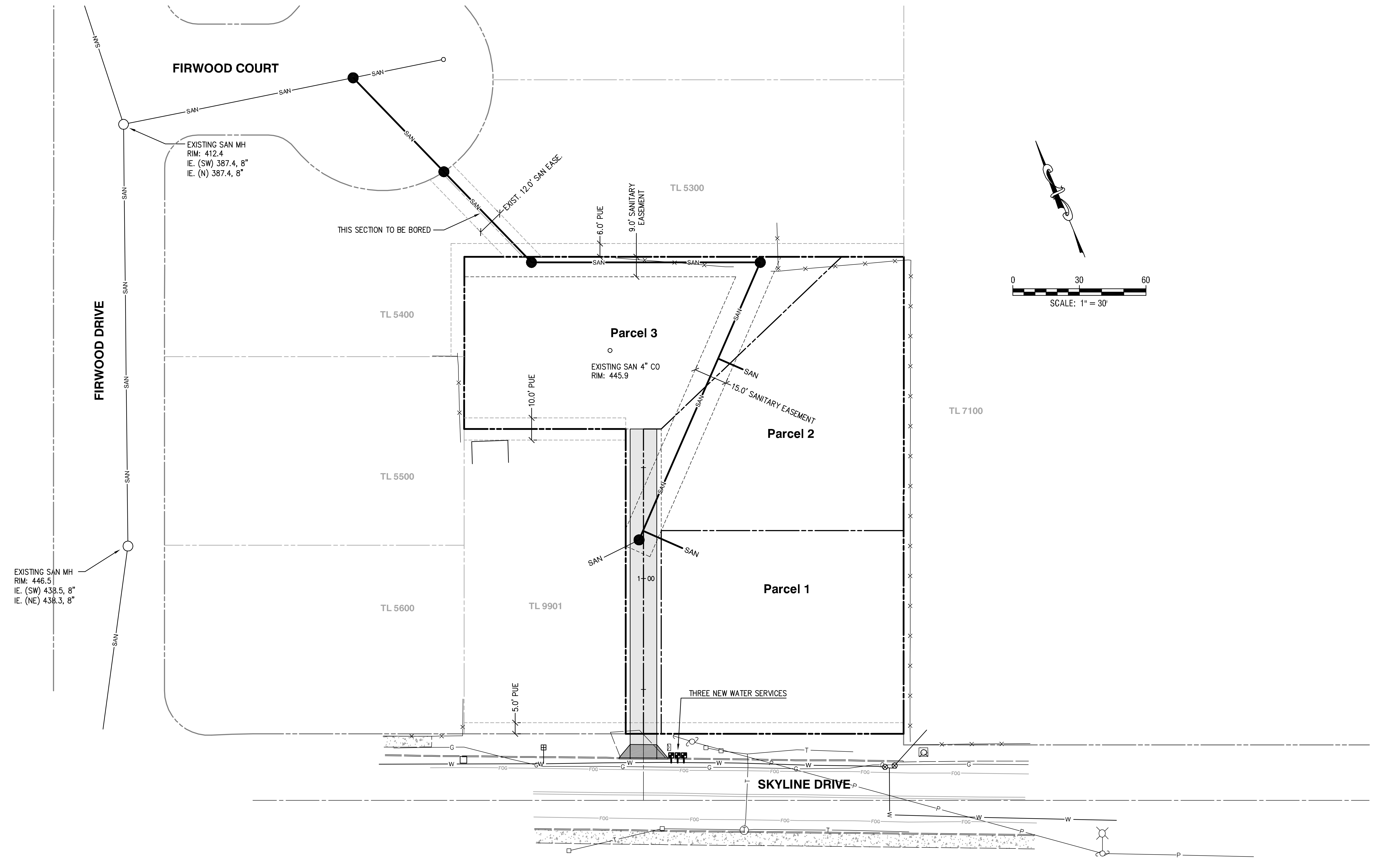
DESIGNED:	BDG			
DRAWN:	BJS			
SCALE:	As Noted			
DATE:	September, 2018			
FILE:	Skyline Partition Prelim1	DATE	NO.	REVISION

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:
1/3



PRELIMINARY UTILITY PLAN

2014-129U

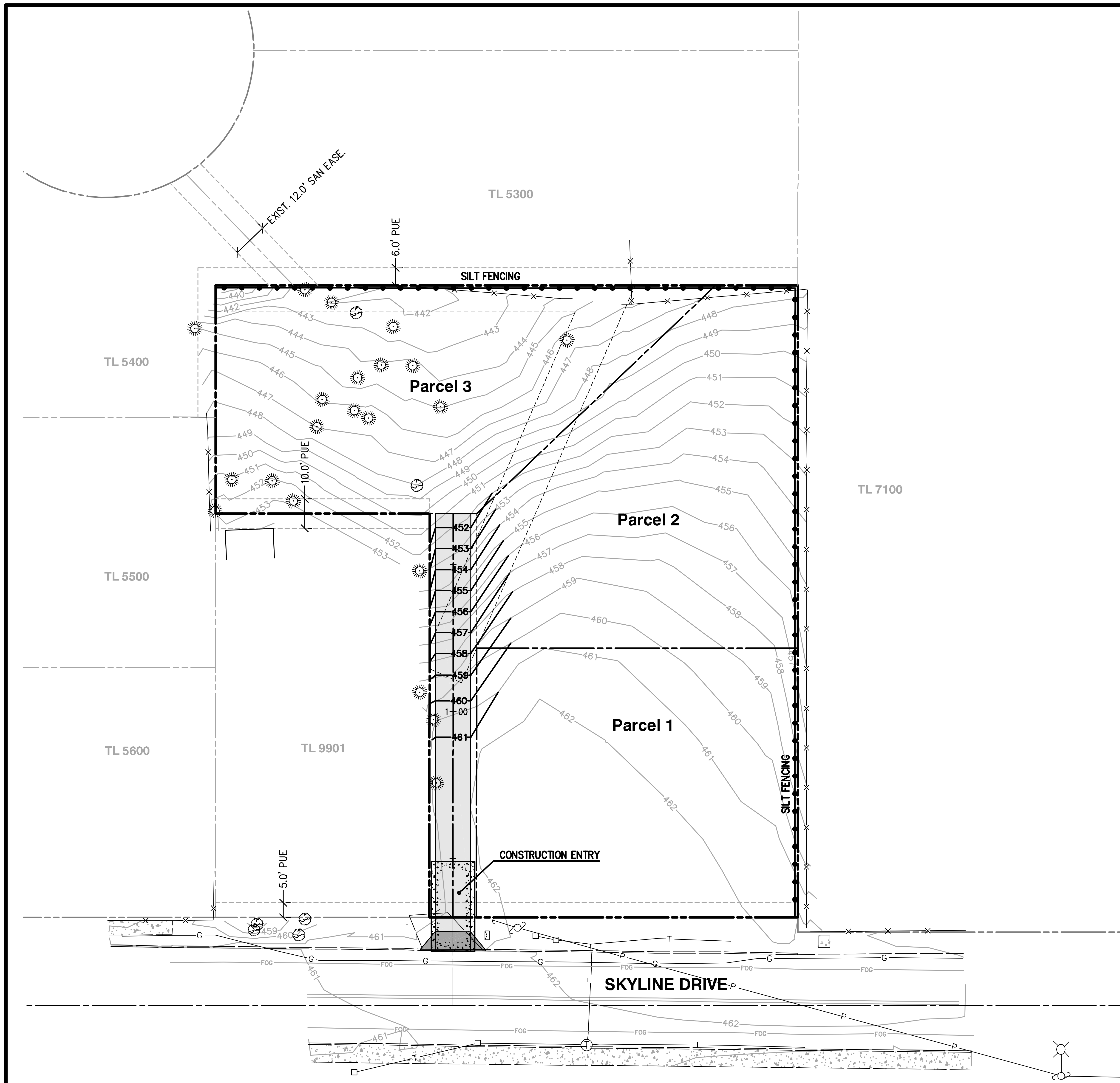
DESIGNED:	BDG			
DRAWN:	BJS			
SCALE:	As Noted			
DATE:	September, 2018			
FILE:	Skyline Partition Prelim1	DATE	NO.	REVISION

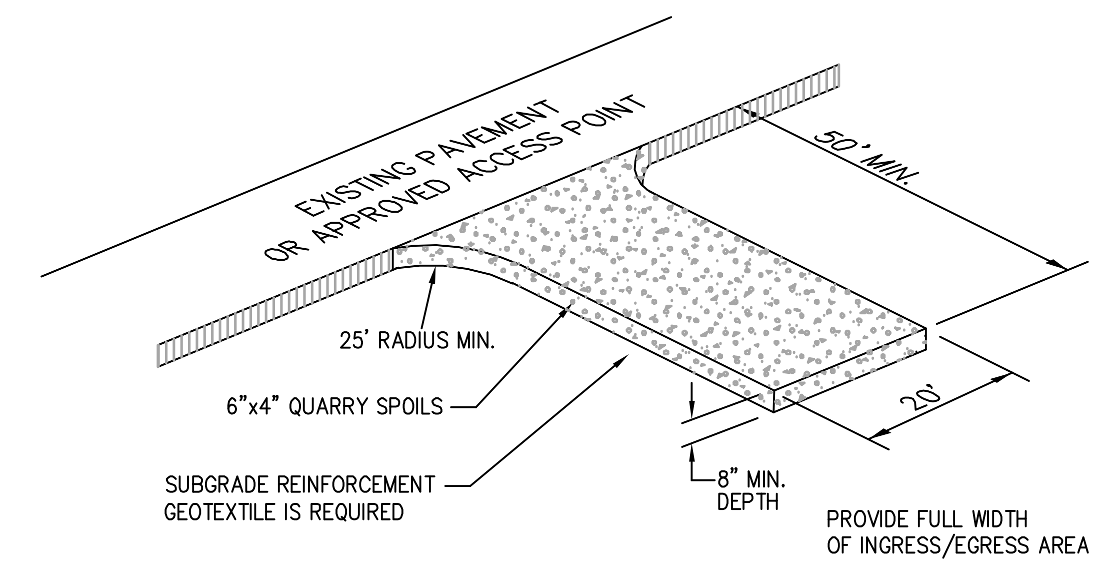
Theta, llc
ENGINEERING - SURVEYING - PLANNING
PO Box 1345
Lake Oswego, Oregon 97035
503/481-8822
email: thetaeng@comcast.net

Icon Construction and Developement
1380 Willamette Falls Drive, No. 200
West Linn, Oregon 97068

Partition Site Plan
6123 Skyline Drive
West Linn, Oregon

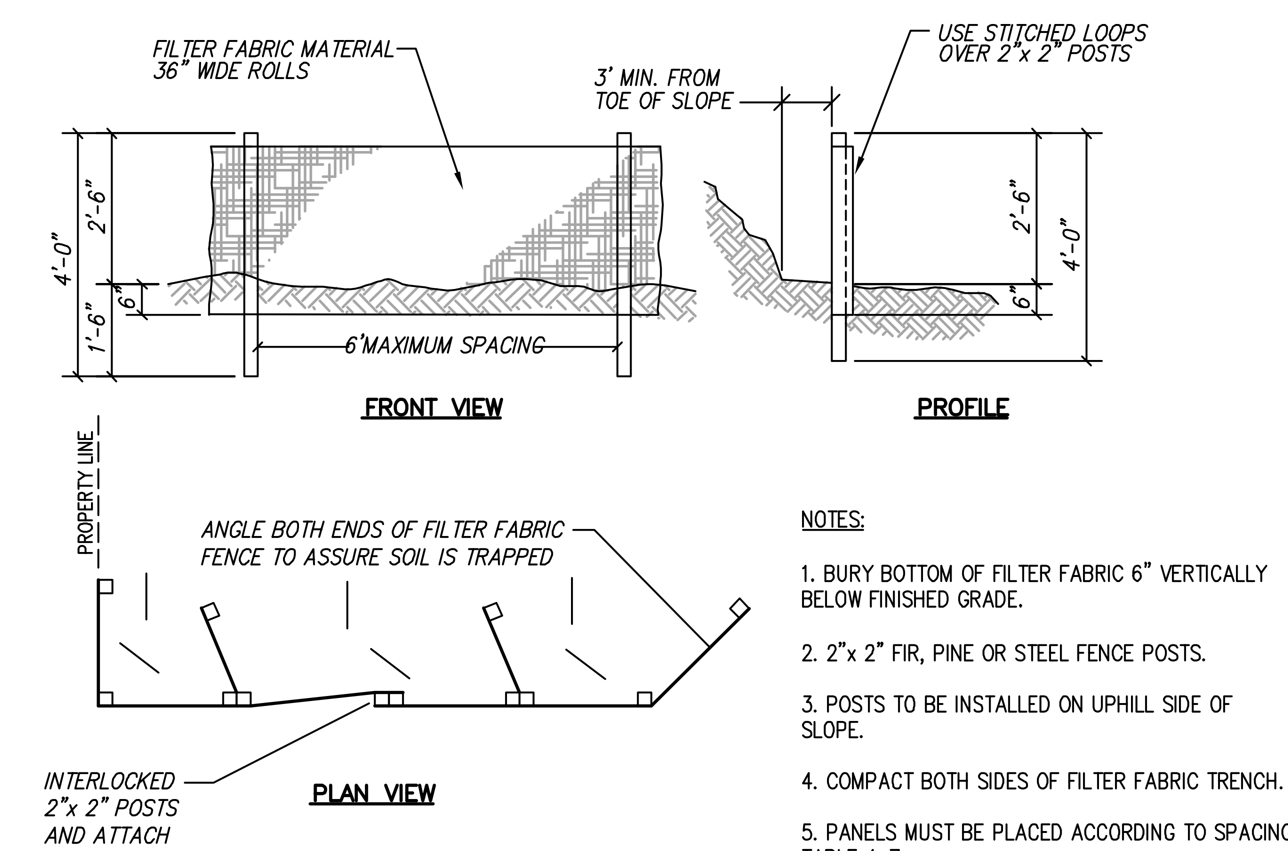
SHEET:
2/3





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

2014-129U

PRELIMINARY GRADING AND EROSION CONTROL 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:
3/3

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.

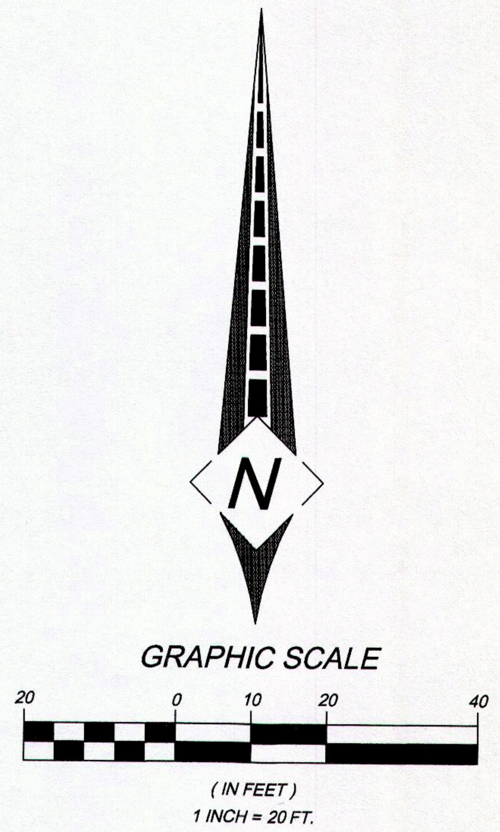
NO TITLE REPORT WAS SUPPLIED OR USED IN THE PREPARATION OF THIS MAP.

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
- FD = FOUND
- FI = FIR TREE
- PI = PINE TREE
- CE = CEDAR TREE
- IR = IRON ROD
- YPC = YELLOW PLASTIC CAP
- DE = DECIDUOUS TREE
- UTILITY AND LIGHT POLE
- UTILITY POLE
- LIGHT POLE
- GUY WIRE
- ELECTRIC BOX
- ELECTRIC METER
- ELECTRICAL POWER PEDESTAL
- ELECTRIC RISER
- HEAT PUMP
- XOH — OVERHEAD LINE
- XG — GAS LINE
- XE — ELECTRICAL LINE
- XCOM — COMMUNICATIONS LINE
- XSS — SANITARY SEWER LINE
- XSD — STORM DRAIN LINE
- XW — WATER LINE
- x — x FENCELINE
- UTILITY RISER
- DOWN SPOUT TO SPLASH GUARD/GROUND



SIGNED ON: 30AUG18

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\CON-SKYLINE DR-6123\dwg\ECM.dwg