THE GALLI GROUP
Engineering Consulting

ASHLAND CANAL PIPING PROJECT
PROJECT PHASE 1A
ASHLAND, OREGON

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1.0 INTRODUCTION

The City of Ashland is evaluating the probability of enclosing approximately 9,000 lineal feet of open (partially enclosed) canal that traverses across the ridges and gullies southeast and upslope of downtown Ashland. See Figure 1, Vicinity Map for the route location. At this time the canal has numerous apparent leaks and unauthorized users. Preliminary discussions indicate the canal will likely be replaced with a 24-inch diameter HDPE, smooth interior culvert. It is also likely that the invert will be lowered somewhat for better flow along the route.

This enclosing in a pipe should result in a significant savings in water loss and help improve soggy conditions below portions of the canal. The purpose of this phase of the work is to identify areas of probable instability and other areas that could create potential problems to the canal or adjacent parcels. It is also to help collect/provide design information for planning and preliminary design of the culvert installation.

2.0 SITE AND PROJECT DESCRIPTION

The subject site is a 9,000-foot length of canal that cuts across moderate to steep slopes in SW Ashland, Oregon. See Photos 1 and 2. The canal goes beneath driveways and streets (See Photos 3 and 4) over ridgeline formations and through or over ravines which carry significant runoff during the winter. See Photos 5 and 6. The canal also runs very close to structures and there have been newer structures built over or on the edge of the canal. See Photos 7 through 9. Trees encroach on the canal sides and numerous bridges cross the canal. See Photos 10 through 15. Several areas have pipes discharging into the canal and others have large boulders encroaching at the edge of the canal. See Photos 16 to 19. Figures 2 & 3 show the canal route through tax lot maps and with area topography.

In several locations, the canal has been encapsulated in culverts. See Photos 20 to 24. In a few areas, the culverts extend for a long distance beneath the ground with only hatches for access. See Photos 25 to 28.

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We also observed several areas where water was leaving the canal. Some were legal diversions to paying customers. See Photo 29. Some were due to fractures in the canal's concrete/gunite lining. See Photos 30 and 31.

The canal weaves its way through terrain that can be compromised by debris flows, slope instability and larger scale failures unless the area has been stabilized by engineered structures. See Photos 32 to 35. All of these items listed above present challenges and engineering opportunities for the construction of a new fully enclosed canal culvert along this 9,000-foot length of existing canal. Much of this will be addressed in general terms in this report. Specific designs for problematic sections or locations will be addressed in the next phase of the work.

3.0 SITE RECONNAISANCE

On several occasions in the spring and summer of 2018, personnel from The Galli Group walked the length of the canal. Mr. Busby, P.G., C.E.G., our Senior Engineering Geologist, and William F. Galli, P.E., G.E., our Senior Principal, walked the full length of the canal and took notes and photographed areas of interest regarding design and construction challenges and areas of potential instability or which are susceptible to damage due to landslides or mud and debris flows.

Mr. Galli and Mr. Busby also walked and observed numerous areas upslope of the canal. Observations were made of steep slopes which could potentially fail, ravines which can channel mud or debris flows into the canal and other areas of interest.

Overall, concerns were focused on the vertical and lateral support of the new canal as well as all potential ways the canal pipe could be damaged by earth movement. Also of interest are issues along the route which could compromise or make more difficult the culvert design and construction.

4.0 SITE GEOLOGY

4.1 REGIONAL GEOLOGIC SETTING

The project site is in southwestern Oregon, near the southern boundary of the city of Ashland. The site is immediately southwest of the broad Bear Creek Valley, formed by Bear Creek, the Rogue River, and their smaller tributaries. Bear Creek Valley is bounded on the west by mountains within Oregon's Klamath Mountain Physiographic Province; immediately east of the valley begins the foothills of the Cascade Volcanic Physiographic Province. The project site is situated atop granitic bedrock of the Mount Ashland Pluton, which is part of the Klamath Mountain Physiographic Province.

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

The Hayfork subterrane of the Klamath Province (Applegate Terrane) occurs in the western portion of the Bear Creek Valley area, and consists of volcaniclastic arc rocks (cherts, argillites, limestone, and meta-andesite). The Hayfork subterrane was intruded by the Mount Ashland pluton producing varying grades of metamorphism. Regional uplift "unroofed" and exposed the pluton, and a relatively broad, flat surface was created twenty-five million years after intrusion of the pluton (Wiley, et al, 2011). The Hornbrook Formation, Cretaceous marine sedimentary rocks ranging from sandstone, siltstone, mudstone, and conglomerate, was deposited unconformably upon this eroded surface. The Hornbrook Formation was deposited as an extensive seaway which made repeated transgressions into what is now southwestern Oregon in early to middle Cretaceous time (approximately 100 to 75 Ma). The Payne Cliff Formation, upper Eocene fluviatile braided-stream deposits of sandstone, conglomerate, mudstone and minor coal deposits, overlies the Hornbrook Formation with a slight angular unconformity, and is present along the eastern portion of Bear Creek Valley.

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

Oregon's Klamath Mountain Province experienced episodic regional uplift and faulting into the Tertiary Period. Numerous high-angle, northeast-trending faults are present in the Bear Creek Valley area. In the central part of Bear Creek Valley, between Medford and Ashland, these nearly vertical faults are common. Offset along these faults appears to diminish eastward, where they have been mapped into the younger Western Cascade units (Wiley, 1993). In the southern part of the valley near Emigrant Lake, these northeast trending faults are offset by a younger, northwest-trending set of faults; Cenozoic intrusions have aligned along the younger northwest trending faults (Wiley et al, 2011). Immediately south of the valley the larger Siskiyou Summit, Ager, and Stateline faults offset the Hornbrook Formation by significant amounts (Nilsen, 1983).

Bear Creek Valley has formed by differential erosion of the softer sedimentary units (Hornbrook, Payne Cliffs), compared to the more resistant metamorphic units along the west side and volcanic units along the east edge of the valley. The valley may also have been aligned along the structural fabric of the younger northwest trending faults (Wiley et al, 2011).

In the Bear Creek Valley and project area, faults are observed to offset formations as young as late Miocene. No Quaternary fault activity, however, has been established for Bear Creek Valley or the immediate project area (Walker and MacLeod, 1991; Wiley and Smith, 1993; Madin and Mabey, 1996; Wiley et al, 2011; USGS; 2018a).

Four stages of Quaternary alluvial fans and valley fill have been mapped in the Bear Creek Valley area (Wiley and Smith, 1993; Parsons and Herriman; 1976; Wiley et al, 2011).

A geologic map of the specific project area is provided as Figure 4 of this report, as well as a Hillshade and 2-foot contour Map (Figure 5) which shows the canal course and general landforms along the route.

4.2 SITE GEOLOGY

The project area is within the Ashland 7.5-minute USGS topographic quadrangle (see Vicinity Map, Figure 1). Mapped geologic units at the project area consist of granitic bedrock of the Jurassic Mount Ashland pluton (OGDC-6, 2015; Wiley et al, 2011). The Mount Ashland pluton covers an area of approximately 150 square miles in southern Oregon and northern California and has a radiometric age of 161 Ma (Wiley et al, 2011). Emplacement depth of the pluton is estimated by geochemistry to have been approximately six miles below the ground surface, and subsequent uplift and erosion has "unroofed" the pluton.

In the project area the Mount Ashland pluton consists of two distinct units. Jagd bedrock is mapped over the eastern portion of the canal route, while Jaq bedrock comprises the western section of the route (See Geology Map, Figure 4). Jagd is described as biotite-hornblende granodiorite, and Jaq is described as quartz monzonite. The Jagd and Jaq units represent different phases of magma emplacement during formation of the pluton. Engineering properties of the Jagd and Jaq rock units appeared similar, with no significant distinction between the two mapped units noted during the reconnaissance field traverse.

4.3 GEOLOGIC RECONNAISSANCE FIELD INVESTIGATION

A reconnaissance field traverse along the Ashland Canal was done in May, 2018 to observe the general geology and slope conditions of the project route. No subsurface exploration (borings, test pits) was done as part of this study, but a database of subsurface exploration completed by TGG in the Ashland pluton was available for reference.

Discontinuities (jointing) within the granitic units typically are the most important factors affecting engineering performance. Few outcrops of bedrock were observed along the canal course, and no strongly jointed granite outcrops were noted. Generally, slopes had developed a soil horizon with variable understory vegetation (grasses, brush), and scattered overstory trees including Pine, Madrone, and Oak.

Based on the field traverse and the Slope Map-% (Figure A-3), developed from 1-meter resolution 2015 Lidar dataset (Dogami, 2018), the granitic bedrock forms native hillslopes that range from less than 25% up to approximately 75% in steepness. Most of the native slopes adjacent the canal, immediately upslope and downslope of the canal, fall within the 25% to 50% range. Fills on the downslope side of the canal were generally 2 to 6 feet in height, up to 10 feet, and were in the 25% to 50% range with occasional zones up to 75%. Cut and fill sections, either for the canal itself or adjacent house pads, can be steeper and occasionally have slopes that range upwards of 100%. Several cuts in granite bedrock, up to approximately 10 feet in height were observed, with the steepest being approximately 63°, ½:1 (H:V). Large cuts on the Ashland Loop Road above the project range in height from 5 feet up to nearly 25 feet and inclined at 55° to nearly 75° from horizontal. These cut slopes along the Loop Road have remained essentially stable, with some minor sloughing, over the years including significant storm events.

Slope conditions will be further discussed in the Slope Stability section (6.3) of this report.

Based on other TGG projects in the general vicinity, excavation by conventional equipment is generally possible in the granitic bedrock to depths of nearly 20-25 feet. The upper weathered granite profile typically has a R-0 to R-1 rock hardness rating (ODOT, 1987) to a depth of 20-25 feet, transitioning to harder, less weathered rock (R2-R3) below those depths.

Surface seepage was not observed on native slopes or cut faces during the reconnaissance traverse of the canal. Regional domestic wells in the granitic bedrock typically produce from depths of 100 feet, and as great as 600 feet (ODWR, 2018). However, perched zones of groundwater could occur in shallow fractures of the Jagd or Jaq bedrock, and such perched groundwater elevations can be expected to vary seasonally.

4.4 TECTONIC SETTING

The project site is in proximity to several zones of active seismicity. The region is affected by the Cascadia Subduction Zone (CSZ), an active subduction zone off the Oregon coast considered capable of producing Magnitude 8.5 or greater earthquakes. The offshore surface expression of the CSZ, near the base of the continental slope, is approximately 200 kilometers from the project site. Various tectonic models estimate the eastern, down-dip seismogenic zone of the CSZ to be as close as 90 to 110 kilometers from the project area.

Average recurrence intervals for such great earthquakes, as determined by recent investigations, range approximately between 300-600 years. A 10,000-year history of turbidite deposits offshore of the Pacific northwest coast records 41 CSZ events (Goldfinger et al. 2012). Of those 41 events 19-20 are interpreted to have been great earthquakes resulting from rupture along all or most of the CSZ, 3-4 have involved rupture of the southern 50 to 70 percent of the CSZ, and 18-20 have involved rupture in the southern part, offshore of the project site. Recurrence of great earthquakes resulting from full length rupture is 500-530 years and the recurrence of all CSZ events during the 10,000-year period of record is 240 years.

At depths of 40-60 kilometers, relatively deep focus intraplate earthquakes of Magnitude 7.0 are considered possible within the subducted Juan de Fuca plate beneath western Oregon and Washington. The recurrence interval is not established, but the devastating earthquakes in Puget Sound (M7.1, 1949; M6.5, 1965; and M6.8, 2001) occurred in the intraplate zone. Based on the very limited historic seismic record, intraplate earthquakes are considered rare in southern Oregon.

Relatively shallow crustal earthquakes up to Magnitude 6.5 can occur in the upper North American plate at depths of 5-25 km. This is the zone which generally produces most of the earthquakes in Western Oregon, and in the project region. Such earthquakes occur once every decade to two decades, and historically have not exceeded M 4.5 within an 80-kilometer radius of the project area.

The project area has an additional tectonic source from earthquakes on active Basin and Range faults as close as 45 kilometers to the northeast. The Klamath Falls earthquake of 1993, with M5.9 and M 6.0 main shocks, occurred in this seismic setting. This region has produced numerous earthquakes, including the significant events near Klamath Falls, as well as Warner Valley.

A list of seismic events greater than M 4.5 having epicenters within an 80-kilometer radius of the project site is provided in Table 1.

5.0 SITE SEISMICITY AND SEISMIC DESIGN

5.1 HISTORIC SEISMICITY OF AREA

Within a radius of approximately 80 kilometers (50 miles) of the project area, eight earthquake epicenters with a magnitude equal to or greater than M 4.5 have been reported since 1833 (Johnson & Others, 1993; ANSS, 2018; ComCat, 2018). The Klamath Falls earthquake of 1993 contributed three events to this listing, including the M5.9 and M6.0 main shocks, and one smaller aftershock occurring several months later.

Some of these earthquakes were not recorded by instruments but were estimated by Mercalli Intensities. Magnitudes were then estimated from calculations made on the intensity data. Table 1 provides a list of recorded earthquakes in the project region.

Table 1- Historic earthquakes ≥ M4.5 within 80 km of project site. (Source: Johnson and others, 1993; ANSS, 2018; ComCat, 2018).

Date	Latitude	Longitude	Magnitude	Depth (Km)	Reference
9/2/1931	41.8000	-123.000	4.5	-	Johnson & others, 1993/ANSS
6/12/1978	41.450	-121.850	4.6	2.0	Johnson & others, 1993
8/1/1978	41.4298	-121.8505	4.67	2.0	Johnson & others, 1993
8/19/1978	41.450	-121.850	4.7	2.0	Johnson & others, 1993
1/10/1981	41.550	-121.867	4.5	5.0	Johnson & others, 1993
9/21/1993	42.3575	-122.0583	6.0	10.30	Johnson & others, 1993/ANSS
9/21/1993	42.3877	-122.0508	4.3	0.02	Johnson & others, 1993/ANSS
12/4/1993	42.2915	-122.0087	5.1	6.53	ANSS

Three of the four largest seismic events in Oregon's recorded history have occurred near southwestern Oregon.

The 1873 Port Orford, considered Oregon's largest earthquake, is estimated to be M 7.0 (Johnson, 1993) to M 7.3 (USGS, 2011). Some researchers place this event east-southeast of Brookings near the Oregon/California border, and refer to it as the Crescent City earthquake. Chimneys were toppled in Grants Pass and Jacksonville during this event, indicating Modified Mercalli Intensities of VI and VII in the Rogue Valley area. The quake was felt as far north as Portland, and in San Francisco to the south. This event had an epicenter distance of approximately 95 kilometers from the project area.

Most recently, the September 20, 1993 Klamath Falls quakes (M5.9 and M6.0) are the third and fourth largest events reported in Oregon. Mercalli Intensities of VII were experienced in the Klamath Falls area; effects of this earthquake were felt in Medford as Mercalli Intensity V. In the Grants Pass and Roseburg areas Mercalli Intensities of IV and V were reported. The focus of the M6 event is immediately east of Lake of the Woods, on a normal fault system extending from the Basin and Range Province. The quake had a focal depth of 12 km, and epicenter distance from the project site of approximately 50 kilometers.

A summary of regional Quaternary faults in the project area is provided in Table 2.

Table 2- Regional Faults for Ashland, Oregon (approximately 42.18N°; 122.70W°)

USGS Fault #	USGS Fault Class	Fault Name	Distance to site (km)	Movement	Mapped length (km)	Most Recent Deformation/Recurrence Interval	Slip Rate mm/yr
781	A	Cascadia Subduction Zone (CSZ)	**±90**	Thrust 9°-11° dip; strike N4W	754	300 yr/500-600 yrs avg. for past 2-7 ka	>5.00
843C	A	Klamath Graben Fault system-South Klamath Lake Section	**±70** to west side; **±82** to east side	Normal N-17° to 31°W strike; 51°-58° W or E dip.	59	Late Quaternary (<15ka)/ 3 events in 7ka	0.2-1.0 mm/yr
844	A	Sky Lakes Fault Zone	**±50**	Normal; N18W;70° dip (E or W)	85	Latest Quaternary <15 ka/ >10-30 ka	<0.2 mm/yr

From USGS, 2018a; see explanation of terms below

Explanation of terms in Table 2
USGS Fault class- A*: Geologic evidence demonstrates the existence of a Quaternary fault of tectonic origin, whether the fault is exposed by mapping or inferred from liquefaction or other deformational features.*
Fault type notation*; N (Normal); RL (right lateral strike slip); R (Reverse); T (Thrust)*
ma*= mega-annum (period of one million years before present)*
ka*= kilo-annum- kilo-annum (period of one thousand years before present)*

5.2 DEAGGREGATION OF USGS PROBABILISTIC SEISMIC HAZARD ANALYSIS (PSHA)

To determine the contribution of individual faults to the total PSHA, deaggregation of the data is required. This is made possible by a web-page service provided by the USGS (USGS, 2016b). For the 2008 USGS dataset (used in IBC 2012), deaggregation of the PSAH for PGA at the project site, with a 2% in 50-year exceedance rate earthquake (2475-year return period), is provided in Table 3. A summary of the contribution of individual attenuation models used in the USGS 2008 PSHA for PGA is provided in Table 4.

Table 3- Seismic Deaggregation of PGA
2008 USGS Probabilistic Seismic Hazard Analysis

Project Province	USGS PSHA (2008) PGA Deaggregation on NEHRP C (Vs=2033 ft/sec) for 2% in 50 years exceedance
Ashland- Rogue Valley, Southern Oregon	• **4.08%** of PSHA -Cascadia M8.11 at 104.8 km • **17.51**% of PSHA -Cascadia M8.5 at 110.3 km • **63.7%** of PSHA- Cascadia Megathrust M_w9.01 at 106.1 km • **11.3%** of PSHA M6.02 at 14 km (WUS crustal gridded) • **Modal R,M, εo –** 92.0 km; M9.0; 0.55

Table 4- Summary of Attenuation Models Used in USGS PSHA for the Project Area

Site	Crustal NGA	Subduction Attenuation model	USGS 2008 PSHA (2% in 50 yrs) **Crustal** Models used (% of total contribution to PSHA)	USGS 2008 PSHA (2% in 50 yrs) **Subduction** Attenuation models used (% of total contribution to PSHA)
Ashland	B&A; C&B; C&Y	Young ; Zaho; A&B, '03	B&A – 3.0%; C & B- 4.6%; C&Y- 5.3%	Young - 18.0%; Zhao- 52.3%; A & B -16.8%

The abbreviations shown in Table 4 are as follows: B&A-Boore and Atkinson (2008); C&B-Cambell and Borzornia (2008); C&Y-Chiou and Young (2008); Young et al (1997); Zhao et al (2006); A&B-Atkinson and Boore (2003).

5.3 IBC SITE CLASS

TGG has completed numerous geotechnical projects in Ashland granitic terrain. Field investigations for these projects included subsurface SPT borings, test pit excavations, and seismic refraction studies. It was concluded from these data, as well as several published shear wave values in these granitic units (Madin and Wang, 1999), that a **Site Class C** designation best represents the overall conditions of the granite terrain along the canal project.

5.4 IBC (2012) AND OSSC (2014) DESIGN EARTHQUAKE

The design earthquake for the project area is based upon established values and methodologies in the International Building Code (IBC; 2012), ASCE 07-10, and Oregon Structural Specialty Code (OSSC; 2014). Seismic design information referenced in this report is from the 2012 IBC and ASCE 07-10.

The Maximum Considered Earthquake (MCE_R) and spectral response accelerations were established as set forth in Section 1613 (IBC, 2012) and Section 11.4 (ASCE 7-10), and were obtained from the online USGS Seismic Design Maps (USGS, 2018b). Table 5 of this report outlines the recommended seismic design values based on site conditions and the above referenced codes.

Table 5- DESIGN EARTHQUAKE (IBC, 2012; ASCE 7-10; OSSC, 2014)

Parameter	Value
Project Latitude/ Longitude (WGS-84); Ashland Canal 02-5407	Lat. 42.18549N Long. 122.71301W
Occupancy/Risk Category (Table 1.5-1 ASCE/SEI 7-10)	I, II, III
Mapped Spectral Response Acceleration (MCE_R) - Short Period (S_S)	0.620g
Mapped Spectral Response Acceleration (MCE_R) - 1-Second Period (S_1)	0.318g
Site Class - *(Table 20-3-1 ASCE/SEI 7-10*)	**C**
Short Period Site Coefficient based on Site Class - (F_a)	1.152
1-Second Site Coefficient based on Site Class - (Fv)	1.482
MCE_R Spectral Response Acceleration - (S_{MS})	$S_{MS}= F_{a*}S_S= 0.715g$
MCE_R Spectral Response Acceleration for 1-Second - (S_{M1})	$S_{M1}= Fv*S_1= 0.472g$
Design Spectral Response Acceleration for Short Periods - (S_{DS})	**S_{DS} = 2/3 S_{MS}= 0.476g**
Design Spectral Response Acceleration for 1-Second - (S_{D1})	**S_{D1} = 2/3 S_{M1} = 0.315g**
PGA= MCE_G (Section 11.8.3.2; and Figures 20-7; ASCE/SEI 7-10)	PGA= 0.279g
F_{PGA} (Table 11.8-1 ASCE/SEI 7-10)	1.121
$PGA_M= F_{pga}$*PGA	**0.313g**
Design PGA (PGAm*2/3)	**0.209g**
Seismic Design Category (Section 11.6 and Table 11.6-1 and Table 11.6-2; ASCE/SEI 7-10)	**D**

6.0 GEOLOGIC HAZARDS EVALUATION

6.1 EXPANSIVE SOIL

The project has a relatively thin native soil profile approximately 4 to 5 feet in thickness (see NRCS Soil Map, Figure 6). The Shefflein loam (166E) which has developed on the granitic bedrock at the project, is classified in USCS as a CL to SC soil material and has a Plasticity Index of 15-20 to its total depth of 56 inches (NRCS, 2018).

Much of any remaining native soil profile along the canal route will be stripped during excavation and construction of the project's pipelined canal. TGG's experience in numerous projects in Ashland's granitic soils concludes they have low expansion potential.

6.2 LIQUEFACTION

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

A seismic source potential for liquefaction certainly exists for the project site as discussed in the earlier portions of this report. The project's subsurface consists of a relatively thin (approximately four to five-foot thickness) CL to SC soil horizon, which is immediately underlain by weathered granite bedrock (R0 to R1). Therefore, the site is not subject to liquefaction hazard.

6.3 LANDSLIDES / SLOPE INSTABILITY

General slope conditions along the canal were discussed earlier in this report in Section 3- Recconnaissance Field Investigation. In that section it was stated that native slopes adjacent the canal were generally within the 25% to 50% range, with some cut and fill features having greater inclinations. Several zones of steeper side slope adjacent or near the canal route were either observed in the field reconnisance or taken from the slope map (Figure 7). These zones, while appearing to be generally stable, may need additional attention during the construction phase, particularly if new excavations, slope cuts, or fills are needed within these zones. A reommendation to address some relatively minor slope erosion is provided for the site near West Fork Beach Creek (see below).

- Adjacent sideslope immediately east of Roca Creek- 700-foot length along canal of 75%-100% slope above and below the canal route.
- Adjacent sideslope immediately east of East Fork Beach Creek- 200 foot length along canal of 75%-100% slope above canal. Ecoblock retaining wall has been placed at east edge of drainage way.
- At East Fork Beach Creek- 75%, 100%, +100 % sideslope extends up the drainage 175 feet from canal, on east slope of drainageway.
- Adjacent sideslope imediately east of West Fork Beach Creek- 200 foot length

along canal of 75% -+100% slope above canal. *Some slope erosion observed and need to flatten or stabalize present slopes.*

- Approximatley 500 feet along canal of 75%-100% native side slopes above and some below canal at location directly east of Weller cul de sac and its intersection with Forest Road. Two short, incised drainages are present in this zone of canal's route, extending up past Waterline Road to Ashland Loop Road.
- 75%-100% sideslope extends 200 feet along canal on upslope side of canal near Long Way.
- 75%-+100% slope extends 160 feet along canal on upslope side of canal near a point beginning 350 feet east of Terrace Ave.

Exposures of granite bedrock along the canal were limited. A granite outcrop was noted near the East Fork of Beach Creek at the point of canal crossing with the drainage way. No adverse joint sets were recorded for this outcrop. However, based on several other nearby projects by TGG in the same granitic terrain, strongly developed joint-set patterns can be present. One strongly developed orthogonal joint pattern observed in the general area is: N-S, vertical and E-W, vertical with approximate 3-foot spacing between vertical joints and 3 to 6-foot spacing height on sub-horizontal joints. It is possible such prominent joint sets might be exposed during trench excavation or slope cuts into the hillside during construction phase of project and could influence general slope stability. They would need to be addressed with in-progress grading inspections at that time.

Generally, slopes adjacent to the canal route were stable with no significant instability noted.

No large deep-seated active or ancestral slope failures, which could directly impact the project, were observed at the time of our field investigation, or during review of aerial photos (Google Earth, 2018; USGS, 2018c). Similarly, no large translational or rotational landslides that might directly impact the project were noted on the Oregon Landslide Inventory Database (SLIDO; 2016). Figure 5 in this report shows 2-foot contours generated from available LIDAR imagery (Dogami, 2018) overlain onto a hillshade map of the project. Review of these data in ArcGIS Pro did not reveal obvious large-scale slope instability that would affect the project. No slope instability is indicated on published geologic mapping at project area (Wiley et al, 2011 in OGDC, 2015).

Recommendations for site grading and proper methods of cut-and-fill construction will be provided in the geotechnical recommendations section of the report. It is essential these recommendations be followed closely to minimize slope instability both during and after construction. Similarly, recommendations addressing surface drainage in the project area are provided and must be followed during, and in some cases, after construction to maintain slope stability in the project area. In-progress grading inspections must be made during construction to note any adverse conditions which could negatively affect cut slopes or general site grading.

6.3.1 Debris Flow/Slide or Rapidly Moving Landslides

It should be noted that considerable space is given to this type of occurrence, not because there is a high likelihood of it happening by being caused by the canal construction. Rather because when such "flows" do take place they can be devastating in nature.

Channelized debris flows often originate on steep side slopes or headwall areas adjacent drainages (Zone of Initiation) as relatively small volumes of material- several cubic yards. These steep slopes, headwalls, or upper channel ways are commonly in the 60%-75% range, with channel adjacent side slopes approaching 80%. Once detached, they then enter a flowing channel, becoming more fluid as they accelerate within this steep channel (Zone of Transport). Moving down the channel, the original mass of soil and debris can erode the channel banks and entrain considerably more soil, water, and debris (bulking). This often results in "gutted channels" which have been stripped down to bedrock. The slurry of material is then discharged, often as much as several miles from its initiation point, and deposited in canyon bottoms, stream channel sections with a lower gradient, and alluvial fans at the outlets of canyons (Zone of Deposition). Velocities of debris flows can exceed 50 miles per hour (Hofmeister et al, 2002). An example of a channelized debris flow is shown in Plate 1.

After enough antecedent rainfall in Fall or early Winter, debris flows can be triggered by a single heavy rainstorm or series of storms.

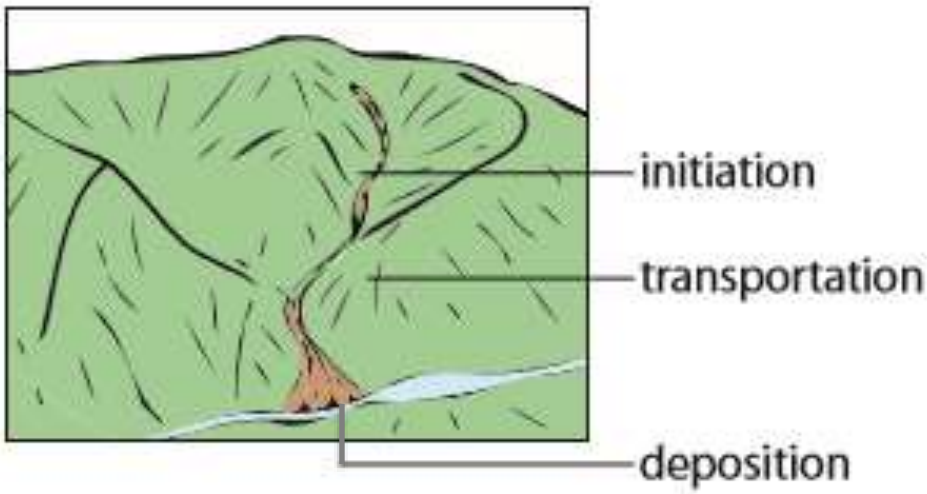

Plate 1 - Channelized Debris Flow (from USGS, 2004)

Numerous landslides occurred in southern Oregon during the New Year's Day storm of 1997. In Ashland and its watershed, several significant landsides (debris flows) occurred and have been mapped into the State landslide inventory (SLIDO, 2018). Heavy rainfall, which fell on snowpack present at lower elevations, resulted in saturated soil conditions as well as extreme surface run-off and flooding. A total of 2.86 inches of rain in a 24-hour period was recorded at the Ashland North Station (NOAA; 1997).

The 2.86 inches of rainfall at Ashland North station was recorded at the north edge of Ashland in the open valley area. The canal project is in the foothills at the south edge of Ashland, and likely receives approximately 10%-15% greater precipitation, based on personal precipitation data recorded mid-point between these two locations. Approximately 2.86 inches/24hours * 1.15 = 3.29 inches/ 24-hours is estimated to have fallen at the project location in the 1996/97 storm. Based on NOAA data in Table 6, the 1996/97 storm was not quite a 100-year precipitation event.

Table 6 lists precipitation frequency/intensity values typically used in civil design for the Ashland area, as provided in the NOAA Atlas 2 dataset (NOAA, 2018).

Table 6- NOAA Atlas 2 Frequency/Intensity Precipitation Dataset for Project Lat/Long.

From NOAA Atlas 2	Precipitation (inches)	Precipitation Intensity (in/hr)
2 year, 6-hour	1.00	0.17
2 year, 24-hour	2.30	0.10
100-year, 6-hour	1.99	0.33
100-year, 24-hour	3.95	0.167

Because of the severe storm events in western Oregon during 1996-1997, including the project area on New Year's Day 1997, Oregon Department of Geology and Mineral Industries (Dogami) conducted a study which produced a GIS map of potential rapidly moving landslides (debris flows) for western Oregon counties (Hofmeister, et al, 2002). The GIS modeling used a framework developed by Earth Systems Institute (ESI), based on the use of DEMs and a suite of rules to model initiation, transport, and deposition zones. The study used higher resolution 10-meter DEMs which had become available from the USGS compared to earlier studies which used 30-meter DEMs. Polygons generated from the Hofmeister study are shown on the "Potential Debris Flow Map" - Figure 8 in this report.

It should be emphasized that the Hoefmeister report was intended to be a "regional screening tool" when published in 2002, with site specific evaluations needed at a project. The model is known to be over-conservative in some site-specific cases, or under-conservative in others. The model is strongly driven by slope steepness and used a 50% slope as the cutoff for defining initiation susceptibility; this cutoff provided a capture range of 60%-85% when the model was checked against known landside locations in the State database at that time (Hoefmeister, 2002). The ODF 's extensive study of debris flows in western Oregon from the 1996-97 storms (Robison et al, 1999) observed no debris flows to occur in their study areas on slopes below 40% and most of the landslides were initiated where slopes exceeded 70%. These ODF values are for slopes measured directly in the field, compared to slope maps of the same terrain derived from DEMs. Thus, slope maps derived from DEMs often show somewhat less slope steepness than actual field measurements of the same terrain. Regardless, the 50% slope defining debris flow initiation susceptibility in the Hoefmeister model is a very conservative (cautious) value.

More recent DEM data, including the 1-meter resolution Lidar datasets for Ashland, would likely provide better estimates of debris flow initiation, transport, and deposition zones. At this time, Dogami has not conducted such studies or published information using the higher-resolution DEMs, and only the Hoefmeister report is available.

Five prominent drainageways interesect the canal route, and can be seen on the Geology Map, Figure 4, Hillshade Map, Figure 5, and Potential Debris Flow Hazard Map, Figure 8. As seen in Figure 8 (Potential Debris Flow Hazard), potential runout from debris flows generated in steeper headwall areas above the project area were mapped into several of the main drainageways that intersect the canal project and are listed below.

- Near the eastern end of the project, Roca Creek interescts the canal route near Pinecrest Terrace, located immediately above the canal.
- Moving westward along the canal route, an east and west fork of Beach Creek intersect the canal.
- Two steep channelways cross the canal route, originating above Waterline Road (near Ashland Loop Road) and extending down to a point eastward of the cul-de-sac on Weller Lane near Forest Lane.
- A fourth crossing of the canal route occurs above Long Way, with a seep headwall area present above Ashland Loop Road, continuing downward across Waterline Road and the canal route. The Ecoblock stabilization from 1997 storm is present in this headwall.
- A fifth crossing near the west end of the project extends down from near Crowson reservoir tank, paralleling Terrace Street.

No historic debris flows are shown on slope stability mapping for the immediate canal route (SLIDO, 2016). However, damage was done to portions of Waterline Road, immediately above the canal route, and listed in FEMA Damage Survey Reports (DSRs) for the 1997 storm event (SLIDO- event ID-FID 932). The FEMA report references: “approximately 0.2-mile Waterline Road damaged by slides; repair 3 major slide areas and stabilize ground to prevent future problems to the underground utilities, water, gas, telephone and electric lines”. Several large Ecoblocks were placed in the steep headwall area above Ashland Loop Road at a point approximately 970 feet east of its intersection with Terrace Street. This headwall area and the drainageway below it would intersect the canal project at a point just above the Long Way cul-de-sac. Large stacked-rock walls are present along Waterline Road, apparently placed for slope stabilization.

Review of historical imagery (Google Earth, 8/2000; BLM, 2001) indicates no apparent debris flows/slides were initiated in the immediate canal project area, including the five drainages discussed in the above paragraphs, during the extreme storm events of 1996-97. No readily available documentation of damage to the present canal course caused by debris flows was found.

Based on the reconnaissance field traverse and 2-foot contour data, three of the crossings discussed above have some potential for debris flows impacting the canal. They are:

- East Fork of Beach Creek, and the steep headwall areas directly above where the drainage crosses the canal. Ecoblocks were stacked at the east side of this drainage/canal crossing as a means of additional slope support and erosion protection. It is not clear if this was a result of earlier damage or done to prevent onset of instability.
- Two steep channelways originating above Waterline Road (near Ashland Loop Road) and extending down to a point eastward of the cul-de-sac on Weller Lane near Forest Lane; could generate localized, relatively small volume flows.
- Steep headwall area present above Ashland Loop Road, continuing downward across Waterline Road and the canal route to a point near Long Way. The headwall area above Loop Road had some instability in 1997 storm and Ecoblock stabilization is present.

The canal carries irrigation water during the dry months of May through September. It remains essentially dry during the winter months, except for routing some storm runoff water. Hazard from debris flows is typically greatest during December-March and thus would most likely impact a canal with minor water flow.

Based on the above discussion and historical precipitation records to date, hazard from debris flows significantly impacting the canal route is reasonably low based on recurrence interval. If they do take place they can certainly run over and past the canal alignment with very damaging impacts. While we do not believe, at this time, that the proposed project will increase the likelihood or severity of such flows, it is imperative to review any design recommendations regarding placement of the pipeline, slope drainage, and any other engineering measures, including barriers/deflection structures, provided in the geotechnical report that could influence such events.

6.3.2 Earthquake-Induced Landslides

In a comprehensive study of seismically-induced landslides, 40 historical earthquakes occurring around the world were reviewed to determine characteristics, geologic environments, and associated hazards of various types of landslides caused by seismic events (Keefer, 1984).

The most abundant landslides resulting from earthquakes were rock falls, disrupted soil slides, and rock slides. The greatest loss of life was associated with rock avalanches, rapid soil flows, and rock falls (Keefer, 1984). From this investigation, the lower limit of ground shaking that would likely produce landslides was established as $M_L \approx 4.0$ for rock falls, rock slides, soil falls, and disrupted soil slides. $M_L \approx 5.0$ was estimated to be the minimum magnitude for soil liquefaction or lateral spreads. The maximum area affected by landslides in the Keefer study occurred in the 1964 Alaska earthquake (M9.2), with an estimated area of 500,000 sq km. The relationship between Magnitude and Distance-to-landslide is presented in Plate 2 (from Keefer, 1984). The Y-axis in Plate 2 lists "disrupted slides and falls" which are defined for rock to include rock falls, rock slides, and rock avalanches; disrupted slides and falls in soil included soil falls, soil slides, and soil avalanches.

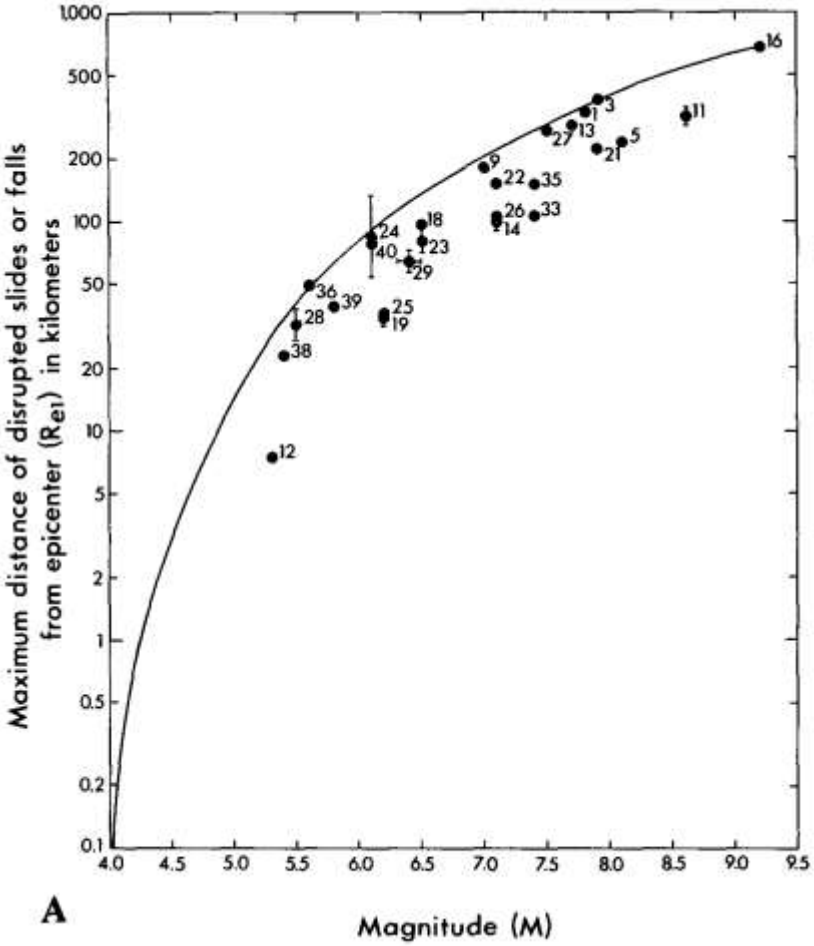

Plate 2- Maximum Distance of Disrupted Slides or Falls from Epicenter in Kilometers (From Keefer, 1984)

Based on the data in Plate 2, an M9.0 event on the CSZ could produce disrupted slides or falls in soil or rock up to 500 kilometers.

The 1906 San Francisco earthquake (M 8.3) triggered slope failures within a strip of coastal California, including the Coast Range Mountains, from southern Monterey County in the south to Eureka at the north end. This zone of slope failures was nearly 370 miles long by approximately 50 miles wide, and, thus, extended over an area of nearly 18,500 square miles (Youd and Hoose, 1978). Most slope failures were adjacent the San Andreas Fault Zone, but it is significant that some failures occurred as far away as 50-60 miles from the fault. *Antecedent rainfall was likely a significant factor contributing to the number of slope failures in the 1906 earthquake*.

The 2011 Mw9.0 Tohoku earthquake off the east coast of Japan resulted in co-seismic landslides which produced, mobilized, and transported large volumes of debris resulting in damage to civil structures, disrupted transportation networks, altered fluvial networks, denuded slopes, and directly caused 14 human casualties (Workman et al, 2012). Much

of this damage was along the eastern coastal plain nearest the epicenter, but landslides also occurred as far away as 200 km to 300 km from the epicenter (Marui et al, 2011).

In an extensive inventory of landslides caused by this subduction zone earthquake, it was observed that 80% of the landslides occurred in Neogene rock units and Quaternary sediments (Wartman, et al, 2012). The most common types of landslides were: 1) disrupted landslides in Neogene sedimentary rocks, and 2) lateral spreading in Quaternary sediments. Secondary modes of landslide were disrupted landslides in Quaternary sediments, Pre-Neogene igneous rocks, and Neogene igneous rocks. The project has bedrock similar to the Pre-Neogene and Neogene igneous rock units (granitics) found in the project area.

The modes of potential seismic-induced slope failure at the project site are most likely shallow soil slips in the native CL and SC soil units, or at the upper 4 to 5 feet near the contact with the highly weathered granite bedrock. Additionally, if any large cuts are needed for the project, the possibility of rock fall or slides could occur along exposed joint sets.

It is essential that the slope and grading design recommendations provided in the geotechnical section of this report be followed to minimize the potential for possible seismically induced slope stability hazard.

6.4 GROUND SHAKING

Project structures, retaining walls, and fills should be designed according to IBC/ASCE and OSSC methodology (IBC, 2012; OSSC, 2014). A Site Class C was determined for the project.

As reference information, several ground motion parameters are provided from a seismic record obtained during the Mw 9.0 2011 Tohoku earthquake in Japan (NIED, 2018). The record is for Station MYGH-6 which has a similar source-to-site distance as the project site has to the CSZ, as well as similar PGA as expected at the project based on the PSHA. The record for MYGH-6 at Tojiri, Japan is summarized in Table 7.

Table 7- Ground Motion Parameters for Seismic Station MYGH-6

Station	Distance from focus	Site Class at station	PGA	Arias Intensity	Bracketed duration	Trifunac-Brady (5-95) duration
MHGH-6 Tojiri, Japan	121 km	C (soil profile for Vs30 at site is 2158 ft/sec)	0.270g	9.91 ft/sec	84.5 sec	88.3 sec

6.5 SEISMIC GROUND AMPLIFICATION OR RESONANCE

An increase in ground motion intensity due to dynamic response of site soil conditions has been incorporated into seismic building codes since the early 1990's. Lessons learned in the Loma Prieta (1989) and Northridge (1994) earthquakes provided much of the data for soil amplification coefficients in today's codes.

Based on the filed evaluation and a TGG database of subsurface conditions in the granitic terrain, a Site Class C was established for the project. Potential amplification or resonance effects from seismic waves associated with the site conditions in the project area are accounted for in the ASCE 7-10 and IBC 2012 seismic design methods, as prescribed in OSSC, 2014.

Topographic amplification sometimes causes an increase in ground motion intensity due to focusing of waves within a hillside (Ashford et al, 1997). This is particularly true for positions along narrowly defined ridge tops with steep slopes. Unfortunately, the problem becomes complex when trying to capture the characteristics of natural slopes in relatively simple models to determine possible amplification. Presently, no standard procedure is in place for estimation of potential topographic amplification. The project is located mid-slope, and not positioned along a sharp ridge line. Therefore, it is not at a typical location of significant topographic amplification.

6.6 FLOODING

The project is not within the 100-year flood zone as mapped on the Jackson County FEMA Special flood Hazard Area (SFHA), effective May 3, 2011 (JC; 2018).

6.7 TSUNAMI/SEICHE HAZARD

The project is located nearly 90 miles inland and is not subject to tsunami hazard**.** The project site is not located adjacent to any large lake or body of water, and, therefore, no seismically induced seiche hazard exists for the project. No large dam reservoirs are in a drainage area upslope from the project site; the project site is not subject to hazard from seismically induced reservoir failure. During a seismic event some potential for "sloshing" of water from the canal exists if any sections of the canal remain as open channel.

6.8 SURFACE FAULT RUPTURE

No active fault traces or Quaternary local faults are mapped within the project site (Walker and MacLeod, 1991; Madin and Mabey, 1996; Wiley et al, 2011; USGS; 2018a). Hazard from surface rupture is considered very low at the project. Several of the northeast trending faults reviewed in the Regional Geology section of this report are present near or within the project and appear on the Project Geology map- Figure 4. As discussed, these are not considered to be active faults and have not demonstrated Quaternary movement. Therefore, they present a low hazard regarding surface fault rupture at the project.

7.0 GEOSCIENCES CONSIDERATIONS

This project winds its way through an area of relatively steep slopes, with ravines subject to flooding and debris flows, with houses, driveways and streets nearby, with large boulders and trees in the path and with numerous marginally stable slopes above and below the alignment. Consideration of all these items and more must be included in the planning, design and construction phases of the work. Without inclusion of these items, the construction process and the short and long-term performance of the newly enclosed canal will likely be disappointing and problematic. Many of these items are discussed below with conceptual comments on how each can be mitigated.

7.1 IMPEDIMENTS TO CANAL ROUTE

Large boulders, trees of all sizes, structures and streets and driveways all will likely be present within the proposed canal/culvert alignment and construction work corridor. These must be avoided, removed or shaped to fit the work.

7.1.1 Large Boulders

There are locations along the current canal route where boulders, some very large, encroach upon the side of the current canal. If the proposed new 24-inch diameter pipe is located within the current canal channel, these boulders will be in the way of the alignment.

7.1.2 Trees

Numerous trees along the alignment encroach into the current canal and/or are located within a minimum width construction corridor that will be required to complete the work. If necessary, these could be removed.

There are a few areas where removal of very large trees (36” to 48”) and their enormous root ball could compromise a portion of the slope above or the foundations for a nearby (very close) structure. Some of these structures may also be within the easement for the current canal.

Slope stabilization in some of these areas may require stacked rip rap and/or MSE walls to minimize long-term erosion and slope failure problems. Underpinning of and/or replacement of structure foundations will be addressed later in this report.

Stump removal can be time consuming and can create a “mess” and compromise stability of that area. Therefore, in areas where slight degradation of a root wad over decades is not a problem, in our opinion, they could be left in place. Of course, in many instances it is really the root wad that is in the way, therefore, it must be removed.

If such tree and root wad removal appears necessary when the final alignment is known, they should be removed in the least disruptive way. This could include using a stump grinder to take out the bulk of the stump and root wad. Consultation with an arborist could help clarify if leaving a portion of the root wad and/or large roots in place could cause any long-term problems for the pipeline or its embankment. These would have to be evaluated on a case-by-case basis.

There is also the possibility that pipe placement and compaction of backfill over the top of large roots for current trees could adversely impact the roots and the tree. This is another area where the advice/consultation of an arborist would be recommended.

7.1.3 Nearby Structures

As can be seen in the attached photos, there are numerous structures (carports, garages, decks, fences, etc.) that encroach upon the edges of the canal route. Some could be an impediment to the construction process due to their location in the desired construction corridor.

The obvious solutions are to adjust the alignment or grade of the culvert or underpin the structure if the canal excavation gets too close.

Adjust the Alignment or Grade. Depending on the cost and effectiveness of the other options this may or may not be feasible. With the size of pipe in most locations the alignment could be altered (like swing out wide on a corner to miss a structure) and still function correctly. These will have to be dealt with on a case-by-case basis if they come up during construction.

Underpin the Structure. There are foundations for decks that have been constructed right <u>on</u> the top edge of the canal. Since the new culvert will be somewhat narrower than the existing canal the concerns would be for stability of the structure during the construction phase and then for how the footing loads would impact the new culvert. We understand the invert may be lowered as much as 18 inches at some locations. This can cause the excavation to undermine structural supports, depending on the alignment of the pipe.

Underpinning these footings allows the area to be excavated right up close to the footings. The underpinning would also support the footing loads after the pipe is in place. Such underpinning can consist of driven small diameter steel pipe piles, helical anchors, drilled piers, push piers and other related items. Any of these could be installed with equipment that could “fit” within a normal construction corridor. Based on past experience, probably the most versatile and least costly of these are the driven small diameter steel pipe piles. These can be used to support footings that could be undermined during construction excavation.

If care is taken in these areas when the final pipe alignment is designated, such extra work may be avoided.

7.2 ROADWAYS, DRIVEWAYS AND PROPERTY

There are several locations where the current canal goes beneath roadways, driveways and areas of property through existing culverts. In several instances this entails dual culverts of smaller size (24-inch) to provide adequate capacity at the proper grade without the top of the culverts being too high. It appears that with the single 24-inch culvert proposed that most or all of this could be accomplished with open cuts.

7.2.1 Shallow Culverts

Many times the culverts are only 2 to 3 or 4 feet below grade. While this will be an inconvenience, open cutting these areas is easily done without the use of shoring or tall steep cuts. These open cuts also would not cause a substantial amount of damage to adjoining property, trees or structures.

Therefore, conventional open-cut excavations should be the least expensive and easiest to deal with by typical construction methods. In most areas where the existing canal culvert goes beneath roads and driveways, the surface grades are not steep (4% to 8%). Therefore, side slopes of the excavations will not become excessively tall (likely 8 feet or less). Such excavations will be able to be cut at relatively steep inclinations into the dense, underlying native granites and somewhat flatter in any areas of fill. These cut slope recommendations will be addressed later in the report. As discussed earlier, care must be taken to insure instability along fracture planes is not initiated by these excavations. This could mean flattening cut slopes such that the fracture plane slope does not exit the face of the cut. All such excavations must be inspected by our Senior Engineering Geologist during their early stages of excavation.

7.2.2 Deep Culverts

There are a few locations where the existing culvert goes beneath driveways and the street at a relatively deep depth (8 to 15 feet). Replacing these by open cut excavation could result in excavation top widths of up to 30 or 40 feet. Narrowing the top width could require steep cut slopes that could develop fracture plane failures. They could avoid such a large excavation by using shoring. However, that can be cumbersome and expensive.

Leave the Culvert in Place. In some instances, if the culvert size is adequate and it is in good condition, able to function for the long-term needs of the project, this section of culvert could be left in place. It could also be lined, if by so doing the improved n value of the lining would increase the culvert capacity. In these cases, geotechnical issues are minimal. They would include creating adequate exposure at the location where the connection between existing and new culvert will be made. This should be able to be accomplished by open cut excavation methods.

Use a Directional Bore. If costs are too high or the area could be unstable for deep open cuts, directional boring or Horizontal Directional Drilling (HDD) could work very well in the soft weathered rock which exists along much of the canal route. The bore could be controlled to proceed along a specific alignment to miss other underground obstructions. Bore pits at each end of the area to construct this way would be needed. These will likely require excavations 2 or 3 feet deeper than the invert of the normal canal culvert. In most cases the conventional open cut excavation method would be adequate for the "bore pits". Cut slope heights would likely be no more than 6 to 8 or 9 feet.

The biggest drawback for this would be the cost per lineal foot of this type of construction. The Horizontal Directional Drilling consists of two stages. The first is to drill a pilot hole 3 to 8 inches in diameter along the alignment. Then to enlarge it by back reaming to the desired diameter. The selected culvert is attached to the back end of the reamer and is pulled through the enlarged bore during back reaming.

While it may be unlikely that this method of construction will be needed, it is a viable option in areas where deep excavations and damage to surface development would cause a large increase in cost for conventional open cut excavation.

7.3 CROSSING RAVINE AREAS

There are several locations where the current canal and future culvert is, and will be, across ravines. These are areas where, over time, concentrated stormwater and snow melt runoff and small to large debris flows have scoured out deeper channels down the granitic slopes. Such areas create a few interesting challenges. These include runoff (which at times can be large) and debris flows.

7.3.1 Ravine Runoff

Where ravines run down the slope there is usually a concentration of surface runoff. At times the discharge out of the ravines can be great. Such discharge usually will also have suspended and bed load comprised of silt and fine sand, up to gravels, cobbles and boulders, depending upon the discharge rate (cubic feet per second).

This creates the problems of 1) overwhelming the capacity of an open canal, 2) causing the canal to "silt up" with silt, sand and gravel deposits and 3) creating unstable conditions where the canal dike is overwhelmed with such water and rock debris.

The current canal appears to have dealt with this in one of two ways. One is to provide a culvert for the ravine water to pass beneath the canal. This is an effective way to pass the water downslope without using the canal to intercept the water or the sediment. However, if this is the chosen method for the new project, we strongly recommend that the bypass beneath the new culvert have sufficient capacity to carry the 50-year storm. It should also be constructed in a manner that if it becomes plugged it will not pond a large

volume of water upslope of the culvert. In such instances, and where the culvert is buried in a soil dike on the hillside, failure of the dike by "piping" or from being overloaded laterally (when the soils are fully saturated and in a weakened state due to the elevated pore water pressure) can take place. These areas must be constructed to be able to withstand the loads and soil strength loss such flow channel failures can cause.

7.3.2 Mudflows and Debris Flows

Mud flows and debris flows take place in steep water sheds all over western Oregon. These are a result of a buildup of soil, gravels, cobbles and boulders along the length of ravines due to mass wasting of the surrounding slopes. Debris from normal forest duff to twigs, limbs and trees can also build up in these areas. At some point, usually during a very heavy rainfall event, a small to moderate (sometimes large) soil mass breaks loose high in the ravine. As it moves downslope it will gather the saturated deposits in the invert along with the excess water from the storm runoff. This causes a large increase in volume and speed of this rapidly moving fluid mass moving down the ravine. We observed the aftermath of such debris flows in the side canyons of Ashland Creek after the December 1996 storms. We also observed several very large such mudflows which destroyed houses along the Umpqua River during that same storm.

The debris flows in side canyons along Ashland Creek caused significant damage to homes and property. They also carried a large amount of sediment, debris and even trees with root wads into Ashland Creek. It was all this debris and small to large sediment (sand to cobble size) that blocked the Winburn Way culvert and caused significant damage and revenue loss as Ashland Creek overtopped Winburn Way and flooded the entire Plaza area.

As discussed in the Geologic Hazards portion of this report, debris flows are a minor threat in most of the steep ravines in Ashland. Figure 8, Potential Debris Flow Hazard, Hofmeister, 2002, provides an outline of the area's most likely to experience such phenomenon. The Ashland Canal is shown as a blue line which winds along the contours. As can be seen, it crosses about nine (9) ravines noted as subject to debris flows. Therefore, the new culvert crossings must be designed to minimize damage should such debris flows take place.

Based on past experience, having some sort of earthen or rock "structure" across the ravine to decrease the velocity of the flow and to provide some soil and debris storage area can help limit the adverse impacts downslope. Such debris flow abatement structures will be addressed later in the geotechnical recommendations section of the report.

Culvert Protection. It is imperative that the culvert installation itself be constructed in such a way as to minimize damage should a debris flow take place. This usually consists of the culvert pipe being embedded into a stable subgrade or be installed upslope of a stable dike fill. Then the upslope area should be graded such that there are no large high points where the top of culvert cover extends above the surrounding ground. When the

area is relatively planar the debris material can flow over the culvert with very little impact. Having the culvert buried under several feet of scour-resistant material or covering the upper half with a reinforced concrete cap will limit damage caused by scouring out of the surface soils by the debris flow. In very adverse locations the entire culvert may have to be protected in reinforced concrete which is anchored into the dense rock below.

Currently, some areas have a large water catchment just up ravine from the canal. These can be effective but will also likely become fully plugged off when a debris flow comes down the ravine. Such catchments with debris "screens" can also become plugged with debris during winter storms. These must be constructed in a manner that small debris such as grass, leaves and twigs and soil and fine rock will easily pass through the catchment. Larger limbs, trees, cobbles and boulders need to be removed after any large storm event and then periodically throughout the winter each year.

Note: The native granitic soils and pulverized weathered granitic rock is highly susceptible to scour and erosion when subjected to high velocity water or mud flow. Therefore, dike areas on the downslope side of the current canal constructed out of the decomposed granite soils will likely sustain damage when overrun by water or a debris flow. Provisions in the pipeline design shall be included to decrease the chances of the pipe being damaged should this take place.

Alternate to Culvert Protection. This situation could also be looked at from a cost analysis and risk perspective. Debris flows and mud flows almost always happen during the winter heavy rainfall months of December and January. This is a time when the canal is not is service. There are also several months during which repairs could be made before the canal must be filled.

Another consideration is that larger debris flows which cause damage do not happen very often. When a ravine is "gutted" by a debris flow it may take 15 years or more before mass wasting in the basin replenishes the sand, gravel, cobbles and debris to make another debris flow probable. Therefore, it may be most cost effective to install the standard pipe in these areas and then deal with the repair and associated costs if and when such a damaging debris flow takes place.

7.4 SLOPE STABILITY AND LANDSLIDES

These are different than debris flows because they can take place anywhere along the route where conditions are "right" for a slide to take place. The subject underlying granite formation over these slopes tends to be very stable unless severely disturbed or undermined by nature or construction excavations. Slope failures on the native slopes are very rare. However, in areas where manmade cuts have created steep cut slopes, sloughing and failure is possible, and in some cases likely.

7.4.1 Cut Slope Stability

There were a few locations along the route where steep, tall cut slopes combined with apparent overland water flow from above has caused small failures and general degradation of the cut slope. Photos 32 and 35 show two such locations.

Even with a path along the route it is unlikely that the entire width now taken up by the top width of the canal and the path on top of the dike will be needed. Therefore, in some areas where instability appears to be a problem, angular rock fills, MSE walls or other method constructed at the toe of the slope could be used to improve the stability of these areas. Backsloping of the cut slopes also works well to improve its stability. Where large trees or structures are not in the way this could be a viable way to improve slope stability.

Areas which receive runoff or overland flow from above will tend to be more unstable, especially in the upper 3 or 4 feet of the cut slope where the weathered, decomposed granite soil is exposed. Therefore, taking measures to cut off runoff from upslope will help. Many times, this flow is the result of concentrated flow from path or road side ditches that have found an outlet onto the slopes above. We strongly recommend that all such paths and roadways upslope be checked to verify that roadside and pathside ditches are functioning properly. Also, to repair areas that are channeling water onto this site.

Note: For improved long-term stability, all concentrated water flow must be intercepted and conveyed to an erosion protected discharge location.

7.4.2 Fill Slope Stability

There are also areas where fills have been used to construct the canal dike, to construct roadway and driveway fills and to create a level bench that supports a piped section of the current canal. The most obvious area is one towards the east end of the route where sand bags had been used to prevent overtopping. This has been caused by previous overtopping of the dike fill, which has slowly eroded away.

The second obvious area is where the existing culvert has had a level fill bench created by use of an Eco Block-faced MSE wall to create a near vertical 10 to 12-foot tall retention structure on the side of a large ravine. Please see Photo 36. Note that this MSE wall has a 3 to 4-foot embed down into this very steep sided ravine.

There appear to be no records of this wall and its design or construction. It is recommended that the geogrid length and type be verified before this wall is relied upon to support added fills.

If this MSE wall is found to be inadequate for the long-term static and dynamic loads, it can be improved by reducing the load on the wall (use of lightweight fill) increasing its load capacity with some form of tie back anchors, near surface anchor rods to a concrete deadman anchor or by installing a tied back soldier pile wall. <u>At this time it appears that</u>

a major upgrade would not be required. Also that replacing the current pipe with a new 24-inch HDPE pipe will not adversely impact stability of the wall.

We recommend that the load on the wall (consisting mainly of the wall backfill) not be increased by raising the grade behind the wall. Any such upgrades would have to be evaluated after the final alignment and pipe elevation are established. This may require subsurface investigation at these sites.

7.5 SURFACE FLOW INTERCEPTION

Currently, rainfall and snow melt runoff flows generally uncontrolled (except where intercepted by roadways and driveways) down the slope. The long stretches of open canal generally intercept this runoff and convey it to an area where it spills into one of the local unnamed "streams". Numerous pipes were also noted emptying collected runoff from above into the canal. When the canal is completely enclosed in a culvert, such runoff disposal will not be available. That is unless, as part of the design, periodic locations are designed to allow such flow into the pipe. This can only be done at locations where it is known that the new culvert will not be flowing with a pressure head in the pipe. Without some manner of water collection, this excess overland flow will discharge onto the slopes below the current canal alignment. It is possible that this increase in runoff discharge to these locations could cause localized erosion, area inundation and creation of new wetlands. This is especially true if some areas receive discharge of concentrated runoff.

On Site Water Disposal. It has been discussed that Low Impact Development (LID) measures be used for stormwater runoff disposal. These could take the form of bioswales, infiltration ponds or infiltration galleries at various sites along the pipeline route.

Almost all of the route has a shallow soil profile over weathered, then unweathered, fractured granite rock. Such subsurface conditions are not conducive to infiltration of large amounts of runoff. The upper soil layer will absorb some runoff. But once it becomes saturated the underlying rock prevents further downward migration of the water. Therefore, the discharge rate that could be accepted by such LID methods is very small and likely not worth the cost and disturbance of such measures.

7.6 MAINTENANCE AND PUBLIC ACCESS

Access along the 9,000-foot route of the pipeline may be needed. This could take the form of access for light maintenance vehicles (which is not available over much of the route at this time) and/or some form of walking path for local residents or for the public in general. The following paragraphs provide some general concepts of how this could be accomplished.

Light maintenance traffic could consist of ATV type vehicles usually weighing 900 to 1,500 pounds and being only 4-1/2 to 5 feet wide. These would be the type seen used at a

lot of parks and on golf courses. At this time, it seems unlikely that a roadway which is wide enough and able to support larger vehicles (like pickups and bobtail dump trucks) will be considered.

Geotechnical considerations for such access ways or pathways will be 1) loading up the area with added fill to protect the pipe and 2) protecting the pipe from point loads from the maintenance vehicles tires. Some areas of the current canal dike, which would make up the edge of such an access road, are steep granite fill. Therefore, their factor of safety against failure is likely low. If an additional 2 or 3 feet of fill is contemplated, all such areas must be evaluated to assure such new loading will not cause a stability problem, especially in winter when the localized "perched" groundwater table is high, thereby decreasing the soil strength along potential failure planes.

The area along the outboard edge and the material over the top of the pipe could be "reinforced" with geogrids or woven support fabric. The edge areas could have a 2-block high ECO Block MSE wall installed and the top of the pipe could be protected by a reinforced concrete "saddle", depending upon what traffic loads and new fill height will be. Cement treatment of soil fill over the pipe will also increase strength and protection of the pipe. These are easy and quick designs to provide once the final nature and purpose of the access way has been determined.

8.0 GEOTECHNICAL RECOMMENDATIONS

This canal project is in the beginning stages of planning and conceptual design. There has been no subsurface investigation accomplished, no soil samples were obtained and no laboratory tests have been accomplished for this report. The recommendations provided are for conceptual design only. These are based on our previous work on numerous projects in the Ashland hills near and around the subject pipeline route. The soil strength parameters and other engineering properties are based on those found on these earlier projects. Therefore, when specific areas of this project require specific design recommendations, subsurface exploration and laboratory testing may be required at that time to confirm the values in this Phase 1A report. Recommendations will include information to construct the pipeline, reconstruct roadways, keep cut and fill slopes, including construction slopes, stable and maintain the stability of existing structures along the route. Please note that these will have to be confirmed prior to final design for specific sites along the pipeline.

8.1 SITE PREPARATION AND GRADING

This is a canal site winding through wooded hillsides. Therefore, the site has moderate understory and overstory vegetation, concrete canal lining, various canal structures and numerous roadways and associated structures. Therefore, normal methods of debris removal, clearing, grubbing and stripping for organic removal and construction or demolition debris will apply.

8.1.1 Clearing, Grubbing and Stripping

All areas proposed for the canal and access ways or other structures or structural fill beneath these items shall be cleared and grubbed of all organics. *It appears that a stripping depth of from 2 to 10 inches will be required (deeper where disturbed organic soils are present).* Additional stripping (or excavations) will be required to remove large tree stumps, the canal lining and associated old fill or soft soils. The stripped materials and organics removed shall be hauled from the site and wasted at a legal dump site.

All construction demolition debris, old concrete canal lining, old concrete foundations and asphalt from roads and driveways must be removed from the site. Holes or depressions resulting from the removal of underground obstructions that extend below the finish subgrade and will be beneath the canal structures, walkways, parking or roadways shall be cleared of all loose material and dished to provide access for compaction equipment. These areas shall then be backfilled and compacted to grade with compacted structural fill, as described later in this report.

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

8.1.2 Subgrade Preparation and Proofrolling

This is primarily for areas of the route that will require reconstruction of roadways and driveways. Some of the site may have loose and medium dense soils near the subgrade elevation. Stripped subgrades and all areas proposed for fill shall be proofrolled with a loaded truck prior to proceeding with the work when access allows. Soft areas must be removed and replaced with structural fill.

Note: All areas cut into the dense granite will not have to be proofrolled.

The proofrolling may be accomplished with a loaded dump truck, loaded water truck or large heavy roller (no vibration). Proofrolling should be discontinued if it appears the operation is pumping moisture up to the surface or otherwise disturbing the in-place soils. *When proofrolling, the tires of a loaded truck should not deflect the soils more than 3/8 inch.*

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

After completion of site stripping and/or excavation to subgrade, the contractor shall take care to protect the subgrade from disturbance due to construction equipment traffic. ***Note:*** *This will likely not be a problem over the dense, native weathered rock.*

8.2 SITE EXCAVATIONS

During the construction of the project, we anticipate excavations will be required for the pipe installation. These will likely encounter the overlying soils, dense weathered rock and previously placed fill materials.

Excavations. All excavators will be able to remove the overlying less dense soils. Only larger excavators will be able to remove the weathered rock to depths below 5 to 6 feet. Trench excavations during dry weather should stand in shallow vertical trenches in soils (less than 3 feet). *However, these are likely to have some sloughing or rockfall off the walls.* Seepage or wet weather and long-term dry weather, can cause the upper soils to cave and slough into the trench. Excavations deeper than 4 feet may require the use of temporary shoring, trench boxes and/or temporary cut slopes to protect workmen. *Some areas will likely have rockfall off deeper trenches if cut slopes are too steep.*

8.3 CUTS AND FILLS

Cuts and fills of 4 to 20 feet could be required for this project. These must be constructed at proper inclinations and be of the recommended materials to remain stable.

8.3.1 Temporary Cut Slopes

During dry weather, temporary construction slopes may be cut at 3/4H:1V or flatter in the dense granites. During wet weather, the contractor must be prepared to flatten temporary cut slopes in the soils to 1H:1V or flatter. Cut slopes in the weathered rock may be cut at 1H:1V in all weather. As discussed earlier, all steep cut slopes must be examined by our Engineering Geologist to verify adverse fracture planes are not present, which can cause large-scale rock fall.

8.3.2 Permanent Cut Slopes

All permanent cut slopes into the native materials which will remain shall be excavated at the following inclinations:

Weathered Rock	1H:1V
Upper Soils	2 1/2H:1V

Note: Where soils transition from weathered rock to the soil the cut slopes shall be at 1 1/2H:1V to 2H:1V and grade into the flatter and steeper slopes.

8.3.3 Fill Slopes

Fill slopes may be used to create wide, level areas on the slope for a portion of the project. These fills shall be constructed as described below.

Fill slope inclinations shall be as follows:

Angular Crushed Rock	1.65H:1V
Angular Clean Jaw Raw Shale	2.0H:1V
Pulverized On Site Weathered Rock	2.0H:1V
Decomposed Granite soils	2.0H:1V
Dirty Jaw Run Shale	2.25H:1V

All such fills shall be placed and compacted as Structural Fill as described later in this report. In order to decrease surface sloughing and erosion of all fill except the dirty shale or pulverized weathered rock slopes, these must be overbuilt and then cut back to a compacted fill face.

8.3.4 Fill on Steep Slopes

All fills placed on slopes steeper than 10% shall be placed and configured as shown in Figure 9. This requires a key trench across the toe and level benches be cut back up the slope. Place and compact the fill in level lifts as Structural Fill. As noted, drainage beneath the fill (at least in the key area) may be required by the geotechnical engineer at the time of excavation.

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

8.4 STRUCTURAL FILL PLACEMENT AND COMPACTION

Beneath Structures and Roadways. Structural fill is defined as any fill placed and compacted to specified densities and used in areas that will be under access, structures, driveways, sidewalks and other load-bearing areas or that will create fill slopes or be used as pipe backfill. It appears that the access, pipeline backfill and road subgrade will have structural fill below them. The subgrade needs to be prepared properly as described earlier and the fill must be placed and compacted correctly for proper long-term performance.

Structural Fill Materials. Ideally, and particularly for wet weather construction, structural fill shall consist of a free-draining crushed rock or shale with a maximum particle size of six inches. The material shall be well-graded with less than 5 percent fines (silt and clay size passing the No. 200 mesh sieve) and meet ODOT's requirements for fracture faces on the stones. During dry weather, any organic-free, non-expansive, reasonably well graded crushed rock or clean jaw run material with less than 7% passing

the No. 200 sieve, meeting the maximum size criteria, is typically acceptable for this purpose. Locally available crushed rock and jaw-run crushed "shale" have performed adequately for most applications of structural fill. Pulverized on-site granite and decomposed granite soil also has worked well for most aspects of structural fill. The material must be reasonably well graded and able to be compacted into a dense monolithic unit.

Structural Fill Placement. All structural fill shall be placed in horizontal lifts not exceeding 9 inches loose thickness (less, if necessary to obtain proper compaction), for heavy compaction equipment and four inches or less for light and hand-operated equipment. Each lift shall be compacted to a minimum of 98 percent of the maximum dry density, as determined by ASTM Test Method D-698 (Standard Proctor).

Beneath Footings. Structural fill placed beneath footings or other structural elements must extend beyond all sides of such elements a distance equal to 1/2 the total depth of the structural fill beneath the structural element in question for vertical support (i.e. for 2 feet of structural fill beneath footings, extend the fill at least 1 foot past all edges of the footing) unless altered elsewhere in this report (for vertical support). Use the structural fill materials beneath footings as described in the Foundation Section later in this report.

Note: Lateral support of footings on fill will have to be reviewed on a case by case basis. Typically this requires at least 5 feet of fill on a level slope beyond the downslope edge of the footing.

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

Fill Placement Observation and Testing Methods. The required construction monitoring of the structural fill utilizing standard nuclear density gauge testing and standard laboratory compaction curves (ASTM D-698 specified) is applicable to materials 1 1/2-inch size and smaller. Larger (2” or above) jaw-run “shale”, crushed rock or pulverized weathered rock from the site do not yield consistent results with this type of testing. The high percentage of rock particles greater than 3/4’s of an inch in these materials causes laboratory and field density test results to be erratic and does not provide an adequate representation of the density achieved. Therefore, construction specifications for this type of material typically specify method of placement and compaction coupled with visual observation during the placement and compaction operations and proofrolling of lifts, instead of nuclear density testing.

Observation of Fill Placement. For these larger rock materials, we recommend the 9-inch lift (after being "worked in" with a dozer) be compacted by a minimum of 3 passes with a heavy vibratory roller. One "pass" is defined as the roller moving across an area once in both directions. The placement and compaction should be observed by our representative. After compaction, as specified above, is completed, the entire area should be proofrolled with a loaded dump truck to verify density has been achieved. ***Note:*** *Soft subgrades must not be damaged by proofrolling. All areas which exhibit movement or compression of the rock material more than 1/4 inch, under proofrolling, should be reworked or removed and replaced as specified above.*

Nuclear Density Testing of Fill. Field density testing by nuclear density gage would be adequate for verifying compaction of 1 1/2-inch to 3/4-inch minus crushed base rock and decomposed granite and other materials 1 1/2 inches or smaller in size. Therefore, typical % compaction specifications as described elsewhere in the report would suffice. Testing should be accomplished in a systematic manner on all lifts as they are placed. Testing only the upper lifts is not adequate.

8.5 UTILITY TRENCH BACKFILL

The proposed new culvert will be buried. This needs to be adequately supported and the trench needs to be properly backfilled and compacted to prevent subsidence of the surface or damage to the culvert and/or the overlying hardscapes.

In our experience, utility trench backfill has been the source of the majority of post-construction fill settlement problems. Some utility contractors do not expend the effort necessary to adequately place and compact trench backfill in lifts as specified. As a result, over a relatively short period of time, the trench backfill has a tendency to settle, thereby leaving a hollow or depression in the surface along its alignment. These linear depressions show up particularly well on relatively flat surfaces just after it rains.

Pipe Bedding. The bottom of the trench must be shaped out of acceptable bedding materials (refer to manufacturer's recommendations) to fit the pipe base prior to placement of the pipe. It is critical to the long-term performance of the pipe that the bottom and haunches <u>be fully supported by a dense bedding</u> which decreases pipe distortion from load. The onsite clayey Silt material is not acceptable for pipe bedding due to the difficulty to compact this material in trench areas and the likelihood it will not provide a dense, firm base. The onsite Decomposed Granite may be used as bedding material. The large rock pieces in the pulverized rock will tend to damage the pipe during compaction of the utility line backfill and therefore should not be used. Finer, crushed rock material (¾-inch minus crushed rock) usually performs well as a good bedding material.

Pipe bedding shall be compacted to at least 95% of AASHTO T-99 (Standard Proctor) or to that specified by the pipeline designer. Cement-treated pea-gravel or sand/cement slurry (with at least 200 pounds of cement per cubic yard) will solidify and would typically not require compaction after placement and makes good bedding material. Care

must be taken to make sure the pipe does not "float" up in the fluid mix prior to it "setting".

Pipe Zone Material. The new culvert must be backfilled around and to approximately 12 inches above the top with an acceptable "pipe zone" material. This may consist of finer crushed rock, cement-treated pea-gravel, sand/cement slurry, coarse sand with fine gravel, or other material acceptable to The City of Ashland and pipeline designers. The onsite clayey Silt soils will be very difficult to compact within the confines of the trench and therefore will not provide the desired level of support for the pipe. We also recommend against using the onsite pulverized granite rock unit as a pipe zone backfill material. The large angular rock pieces will tend to damage the conduit during compaction of the backfill. The pipe zone material shall be well compacted on each side of the pipe and to at least 12 inches above the pipe. Mechanical means will be required to densify these materials to the required densities (unless a cement-treated material is used).

Density requirements for "pipe zone" backfill should be per the manufacturer's specifications for the type of pipe being used (we recommend using 95% of ASTM D-698) Care should be taken when compacting close to and immediately above the pipe so as to not damage the pipe.

General Trench Backfill. Above the "pipe zone" the backfill materials may consist of any compactable material that does not have excessive voids (such as gap-graded large gravels and cobbles), organics, expansive clays, debris or other deleterious material. *The clayey Silt soil would not make good trench backfill*. However, the onsite decomposed granite and well-graded pulverized granite rock are adequate for use as general trench backfill. Crushed rock and jaw-run shale also work well for general trench backfill.

We strongly recommend that all general trench backfill be placed and compacted in the same manner as for general structural fill. Trench backfill beneath asphalt pavements shall be compacted to at least 98 percent of the maximum dry density, as determined by ASTM Test Method D-698 (Standard Proctor) for the upper 48 inches. Below 48 inches the trench backfill shall be compacted to at least 95 percent of the maximum dry density. Trench backfill in landscape areas, that are not part of a cut or fill slope, may be compacted to at least 93 percent of the maximum dry density.

8.6 STRUCTURE SUPPORT

Some structures may be undermined and need replacement of footings. Support of all areas of the structure must be founded over materials that will not have adverse impacts on the structure. Support shall be as listed in the sections below.

8.6.1 Foundation Support Recommendations for Spread Footings

Foundations must be placed directly on structural rock fill placed over the weathered rock, directly on the rock or on dense overlying soils. The footings must be constructed and designed as described below.

1. Excavate down to the dense weathered rock or overlying dense soils.
2. Cut the subgrade into level benches for the footings to bear on.
3. Footings placed on the dense native soil/weathered granitic rock covered with at least 4 inches of crushed rock structural fill as listed above may be designed for an allowable bearing pressure of 3,500 pounds per square foot. A 1/3 increase in this allowable bearing pressure may be used when considering short-term transitory wind and seismic loads.
4. Spread footings shall have the base buried a minimum of 18 inches below finish grade in order to provide lateral support and frost protection.
5. We recommend minimum lateral dimensions of 12 inches for continuous load bearing footings and 18 inches for isolated spread footings constructed in this manner.

Floor support should be as follows:

8.6.2 Underpinning of Foundations

Some foundations may become undermined during the excavation process. Therefore, underpinning the footings prior to excavation will help limit damage to the structure. As discussed earlier, one of the better methods to accomplish this is to use driven, small diameter, steel pipe piles. These can typically be designed and constructed as outlined below.

Pipe Pile Design

- Driven 3" diameter galvanized steel pipe piles.
- Standard wall thickness (Sch 40; 0.237" wall thickness).
- Drive closed ended; anticipated depth is 5 to 15 feet.
- Utilize vibratory driver sized for 3" pipe (1100-pound class).
- Final set criteria; drive until less than 1 inch of advancement in 6 seconds or more continuous driving.
- Pile Top; new construction cap of ½"x6"x6" steel plate for each pile (or per Structural Engineer).
- Use sleeved friction couplers; piles are for vertical compression load only (no Uplift Load Capacity).
- Pile capacity is 12 kips with Factor of Safety of 2.0+; a pile load test should be accomplished at the time of production driving; this is strongly recommended.
- Typical Spacing; 4 to 6 feet along strip footings; depending upon the loads above.
- Multiple piles beneath larger spread footings depending upon the load and stability needed.
- Advantage of rapid installation; 30+ per day.
- Low installation costs ($20 to $25/foot of pile).

Embed top of pile with 6" x 6" reaction plate at distance up into footing or grade beam as recommended by project structural engineer (usually 6" to 8" depending upon footing/grade beam thickness).

Note: Number and location of these small diameter piles will be determined by the geotechnical and structural engineers based on loads and design pile capacity.

8.6.3 Hand Dug Caissons

If the dense granite weathered rock is shallow below the footings, hand dug caissons may be used. These would be excavated beneath portions of the footings down into the dense granitics. A size on the order of 2 feet x 2 feet, in plan view, is usually required to allow access for digging. These may be used to support a 15 to 20-kip load. A reinforced concrete caisson constructed in this excavation will be used to provide support of the footing and structure above.

8.7 LATERAL LOAD RESISTANCE

Lateral loads exerted upon structures can be resisted by passive pressure acting on buried portions of the foundations, retaining walls and other buried structures and by friction between the bottom of structural elements of the wall and slabs and the underlying soil. We recommend the use of passive equivalent fluid pressures of the following values for portions of the structure and foundations embedded into the native soils.

- Native silty Sand or sandy Silt Granite 300 pcf
- Dense Compacted Crushed Rock (5' wide minimum) 450 pcf
- Dense Weathered Rock 600 pcf

A coefficient of friction of 0.55 can be used for elements poured neat against crushed rock structural fill. These should be reduced to 0.20 for areas over a vapor barrier or 0.50 over native soils.

8.8 RETAINING WALLS

Retaining walls may be required on portions of the alignment. Lateral earth pressures will be imposed on all grade separation retaining walls. The following recommendations are provided for design and construction of conventional reinforced concrete or CMU block retaining walls:

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

Pulverized Native Rock EFP	45 pcf
Low Grade Angular Rock/Shale EFP	40 pcf
Crushed Rock EFP	35 pcf
Seismic (up to 10 feet tall)	0.11 g

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

Pulverized Native Rock EFP	60 pcf
Low Grade Angular Rock/Shale EFP	50 pcf
Crushed Rock EFP	45 pcf
Seismic (up to 10 feet tall)	0.11 g

- The walls all must have full drainage as described in section 8.9 and as shown on Figure 10.
- These equivalent fluid pressures are to be used for the soil through which the anticipated failure plane will develop (assume envelope beginning 3 feet behind base of wall and rising up and away from wall at 60 degrees off the horizon).
- A wet soil unit weight of 140 pcf should be used for design of retaining walls which are backfilled with crushed rock or jaw-run "shale". Use 130 pcf for native soil backfill.
- These values are for properly compacted, free draining walls. The onsite organic topsoil or very Silty soils shall not be used for wall backfill. Imported crushed rock or clean jaw-run "shale" work well for wall backfill materials.
- These design values assume the wall or structure is fully drained, has a flat backfill and has no surcharge loads from traffic or other structures. The structural designer should include surcharge loading from traffic, building loads and/or sloped backfill.
- We recommend designing retaining walls to resist seismic loading. A horizontal acceleration component of at least 0.11 g should be applied to the mass of an enlarged active wedge of soil behind the walls and utilized in a pseudo-static analysis. The wedge length back from the wall along the ground surface may be taken to be 0.8H, where H is the height of the wall. This relates to an equivalent uniform load over the entire back of the wall of approximately 7 pounds per square foot for each foot of backfill, for walls up to 8 feet tall (i.e. for a 10-foot wall, fully backfilled, uniform seismic load will be on the order of 70 psf over the entire back of the wall).
- The backfill should be placed in lifts at near the optimum moisture content (clayey soils at 2% to 3% above optimum) and compacted to between 93 and 95 percent of the maximum dry density as determined by laboratory procedure ASTM D-698 (Standard Proctor). Loosely placed backfill will exert greater pressures on the wall than the pressures provided above and must be avoided.
- To prevent damage to the wall, backfill and compaction against walls or embedded structures should be accomplished with lighter hand-operated equipment within a distance of 1/2 h to 1/3 h (h being the vertical distance from the level being compacted down to the surface on the opposite side of the wall). Outside this distance, normal compaction equipment may be used.

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

8.9 RETAINING WALL DRAINS

All exterior retaining walls shall have proper drainage.

Wall drains shall have a minimum 12-inch wide drainage zone of drain rock wrapped in non-woven filter fabric immediately behind the wall extending up from the drainage section to within 12 to 18 inches of the surface. A preformed, fabric-wrapped, polymer sheet drain, such as Amerdrain, Linq Drain or Enkamat must be placed against the wall. The wall shall also have a base of wall drainage section (perforated pipe with drain rock, all wrapped in a filter fabric). Exterior wall drains, which will not be sealed on top by asphalt or concrete, should have the upper 12 inches backfilled with compacted onsite silt soils to minimize intrusion of surface waters into the wall drain system. Please see Figure 10.

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

All drains shall be tightlined and positively sloped to an approved stormwater disposal location in the public right-of-way.

8.10 MUDFLOW/DEBRIS FLOW CATCHMENTS

Protection from debris flow damage can be attained by blocking or partially blocking the potential flow path of such flows. It could also include installation methods which create a protective barrier or support for the section of pipe likely to be subjected to such debris flows.

Deflectors consisting of angled barriers constructed out of large angular quarry stone or reinforced concrete can be used. However, at this site there really is very little space to deflect the mud flow mass into.

Flow blockage and storage areas can be created by constructing a reinforced earth soil berm or quarry stone dam across the ravine. This would likely have to be at least 8 to 10 feet tall to be effective. Provision must be made for normal runoff flowing out of this “catchment”.

Alternately, a reinforced concrete wall or geogrid reinforced Ecoblock wall with backfill could also be used to create a catchment.

Such catchments shall be far enough up the ravine that spillover will not immediately overwhelm the large catch basins upslope of the canal pipe. These catchment structures must also have the toe embedded into the native ground and the ends embedded into the side walls of the canyon to help keep it stable through impact and then static loading by a high-density fluid (85pcf to 95pcf).

The site should be configured in such a way that these catchments will be able to be cleaned out after a mudflow event.

9.0 MATERIALS SPECIFICATIONS

The following materials specifications shall apply to the materials used on this project unless otherwise changed by the City of Ashland.

Note: All such materials to be used on the project <u>must</u> be submitted for compliance testing or review, at least two weeks prior to use at the site.

Aggregate Base Rock (Acceptable for Structural Fill)

- Angular Crushed Rock (3/4 or 1" Minus); R=85 or greater; Well Graded (No Gaps and at least 60% retained on the No. 4 sieve).
- Exceeds the fracture, durability and sand equivalent requirements outlined in Section 00641 of the Oregon Standard Specifications for Construction.
- Maximum passing the No. 200 sieve ≤ 5% Total; ≤ 2% Clay Size.
- Compacted to 98% of the maximum dry density as determined by ASTM D698 or AASHTO T-99.

Aggregate Subbase Rock (Acceptable for Structural Fill)

- Angular Clean Crushed (jaw run) hard "Shale" (4" Minus Jaw-Run) or Crushed Rock (2" to 4" Minus); R=60 or greater; Angular and Reasonably Well Graded.
- At Least 60% retained on the No. 4 Sieve.
- Exceeds the fracture, durability and sand equivalent requirements outlined in Section 00641 of the Oregon Standard Specifications for Construction.
- Maximum passing the No. 200 sieve ≤ 10% Total; ≤ 3% Clay Size.
- <u>During wet weather; passing No. 200 sieve ≤ 5%.</u>
- Compacted to 95% of the maximum dry density as determined by ASTM D698 or AASHTO T-99; initial lift may not attain 95% due to soft subgrade; Engineer to decide in the field.
- Care must be taken to avoid very silty subbase that will not support construction loads, especially when wet (will not meet specifications).

On-Site Soil Fill 1

- Pulverized Native Granite Rock, 2" minus to 4" minus.
- Where specifically allowed in the Geotechnical Recommendations.

On-Site Soil Fill 2

- Decomposed Granite, Silty Sand.
- Where specifically allowed in the Geotechnical Recommendations.

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

Sand

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

Drain Rock (For drainage sections)

- Clean, washed, rounded or angular openwork drain rock.
- Gradation to be 1/4" and greater, sized to not move into and through perforations in the pipe.
- 1/4" to 3/4" clean crushed, 3/4" to 1" clean rounded rock, 1" to 2" clean angular rock are all acceptable.
- Clean means washed rock with NO coating of silt, clay or sand.

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

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

Geotextile Filter Fabric

- Non-woven geotextile filter fabric for wrapping drainage sections and separation of openwork rock from sands or soils fines.
- Meet specifications as per Mirafi 140N or equivalent.
- Overlap all edges at least 24 inches (12" for drainage section envelope).
- Secure in place such that overlaps will not move during covering operation.

Geotextile Support Fabric

- Woven geotextile support fabric designed for separation of crushed rock and subgrade soil and for rock section support.
- Meet specifications as per ACF180 woven support fabric.
- Overlap edges at least 2 feet and ends at least 5 feet.
- Align roll lengthwise with direction of traffic in all drive lanes.
- Pull tight full length and keep tight during placement of crushed rock above fabric.
- Do not drive on the fabric until it is covered with rock.\

Perforated Pipe

- 3", 4" or 6" rigid wall, smooth interior, perforated pipe.
- Secure all joints with solvent weld glue. DO NOT use only compression push together fittings.
- Slope to drain per specifications in report or on plan sheets.
- Align perforations in the downward direction.
- Must always be placed within filter fabric wrap unless specifically specified otherwise.
- Protect from construction traffic until buried at least 2 times pipe diameter (minimum 8 inches) of angular rock fill.

Wall Sheet Drain

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

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

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

10.0 SITE DRAINAGE

Control of storm water and snow melt runoff is critical during the construction process.

During construction water should not be allowed to pond within the canal excavation or on other areas that will support roadways or structures. The project construction staging design shall assure that all runoff is collected, conveyed and discharged into the major drainageways already in use in this area of Ashland. We strongly recommend against allowing such water to collect during construction and then to run uncontrolled onto the cut and fill slopes or into yard areas downslope of the pipeline. The contractor must be admonished to have a clear plan of how to control runoff should heavy rain take place during construction.

11.0 LIMITATIONS

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

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

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

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Principal Engineer

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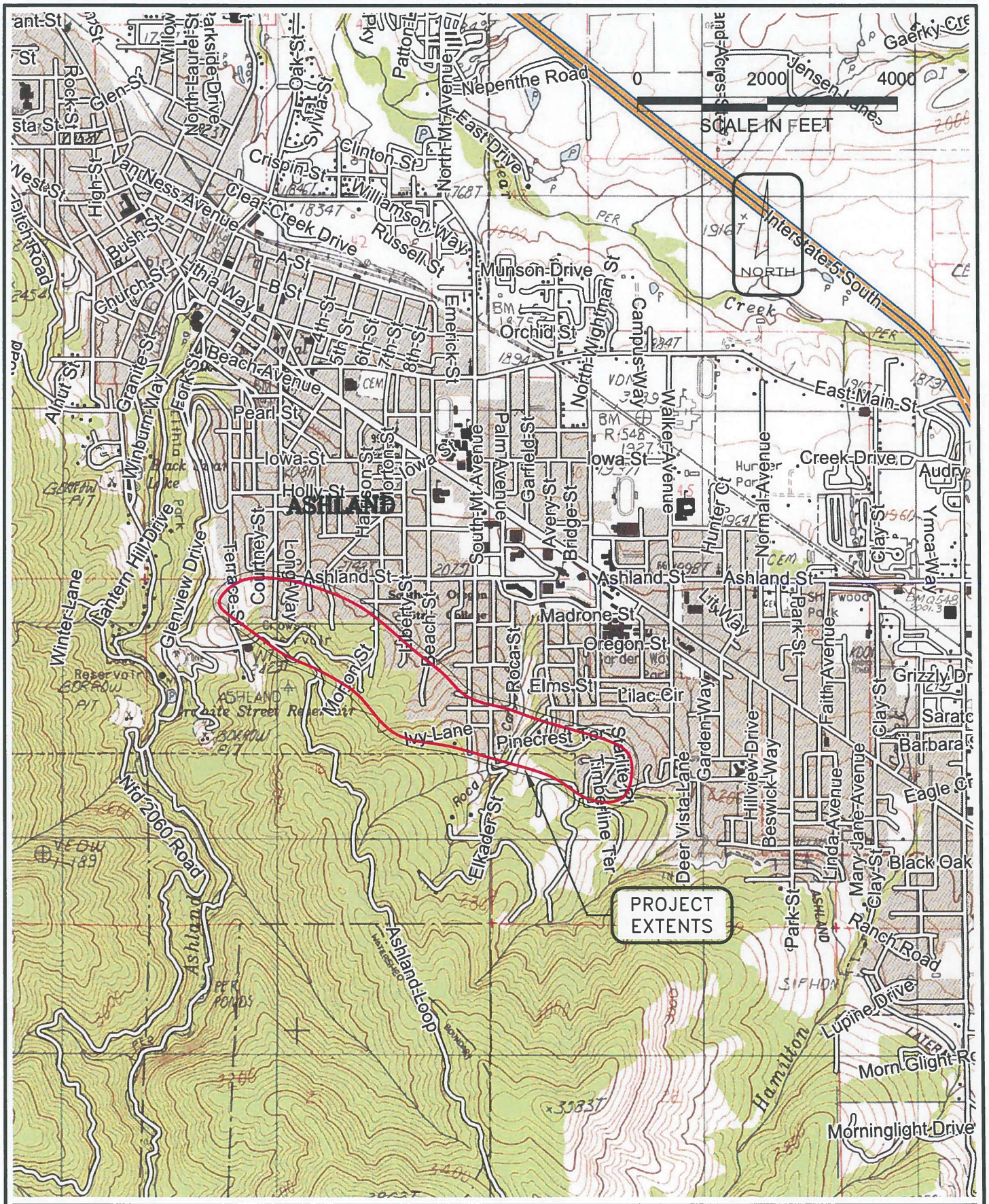
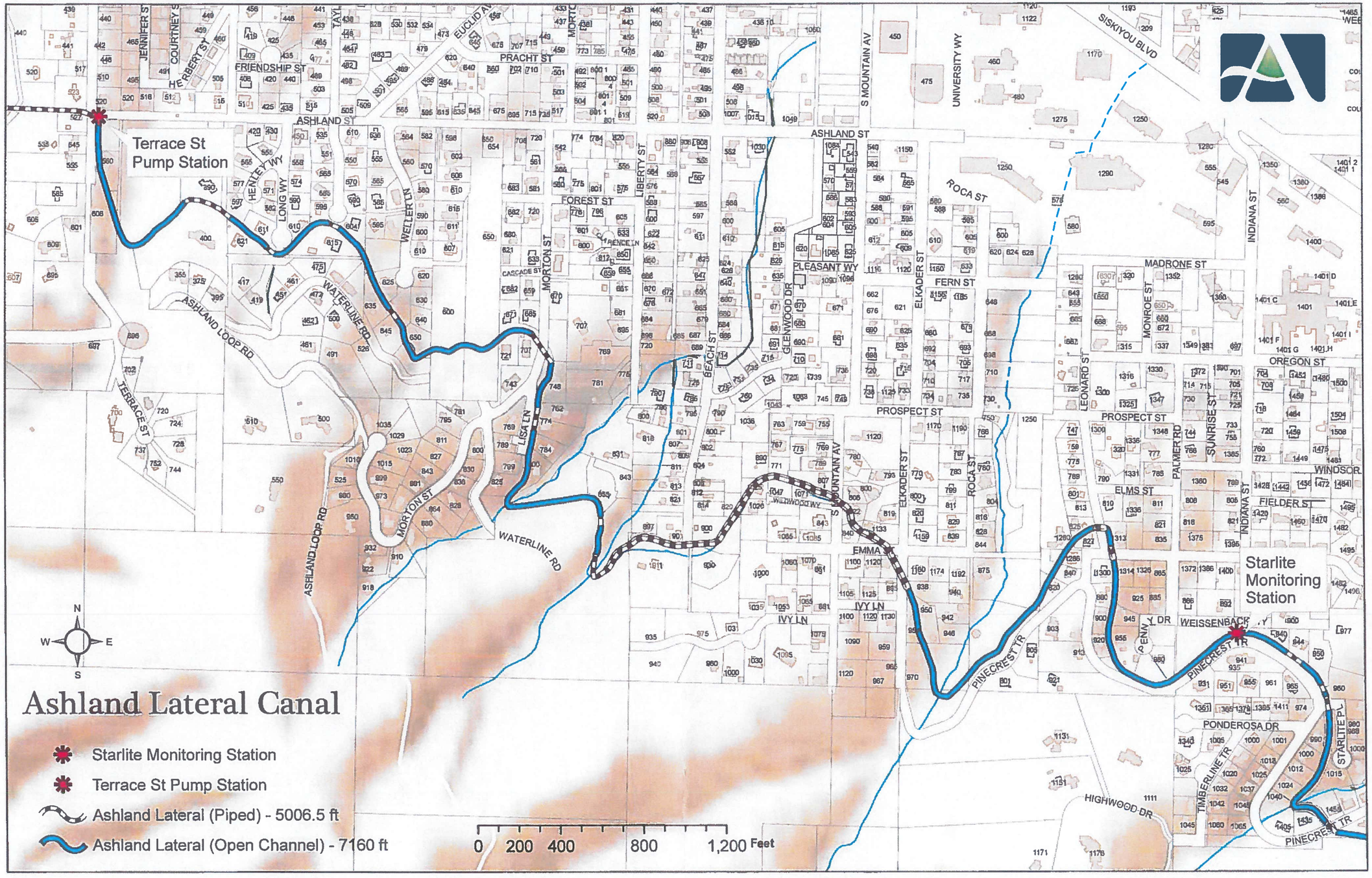
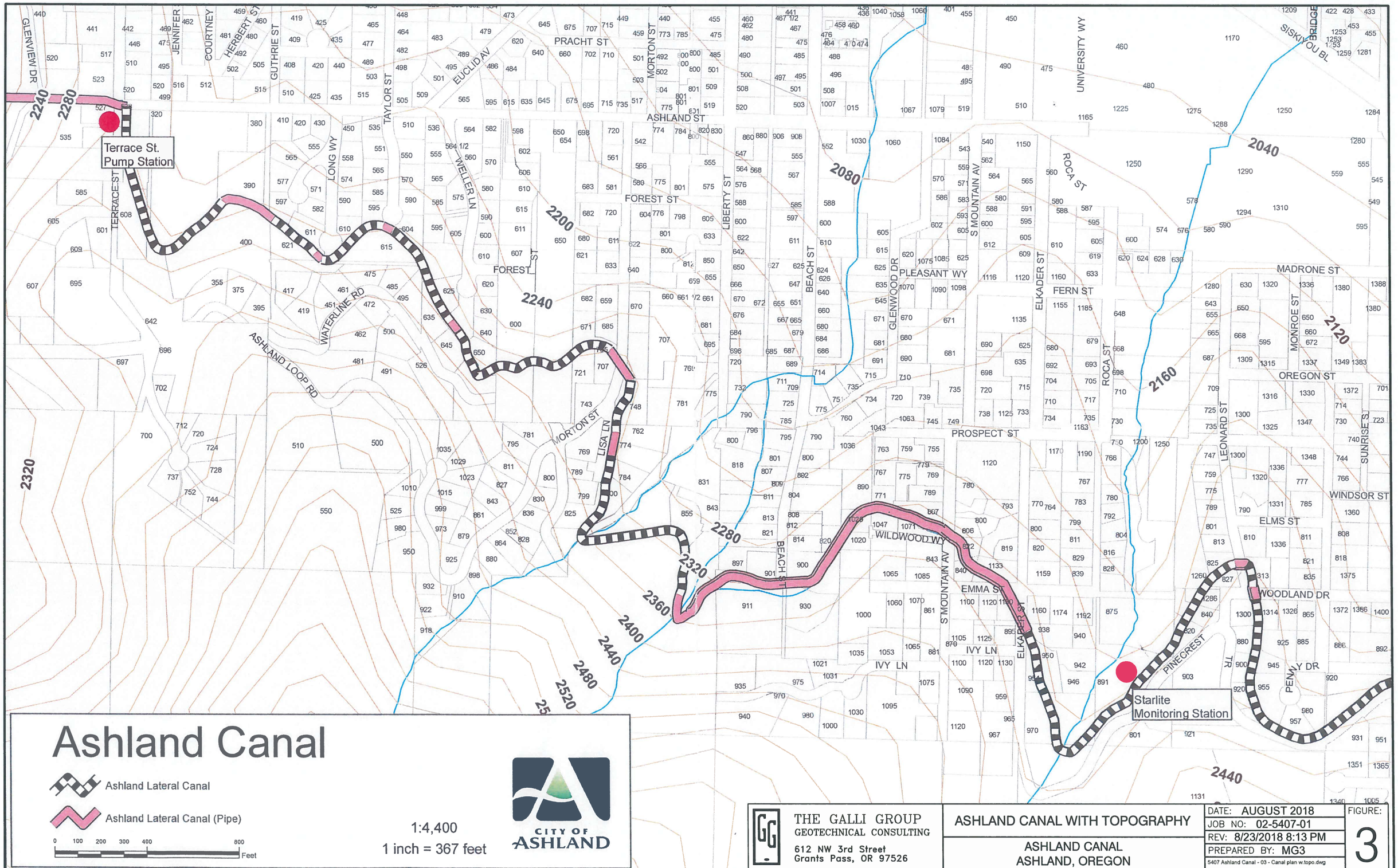


THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	VICINITY MAP ASHLAND CANAL ASHLAND, OREGON	DATE: AUGUST 2018 JOB NO: 02-5407-01 REV: 8/23/2018 6:10 PM PREPARED BY: MG3 5407 Ashland Canal - 01 - Vicinity.dwg	FIGURE: 1



THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	ASHLAND CANAL WITH TAX LOTS ASHLAND CANAL ASHLAND, OREGON	DATE: AUGUST 2018 JOB NO: 02-5407-01 REV: 8/23/2018 8:04 PM PREPARED BY: MG3 5407 Ashland Canal - 02 - Canal plan w taxlots.dwg	FIGURE: 2



Ashland Canal

Ashland Lateral Canal

Ashland Lateral Canal (Pipe)

0 100 200 300 400 800 Feet

1:4,400

1 inch = 367 feet

CITY OF ASHLAND

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

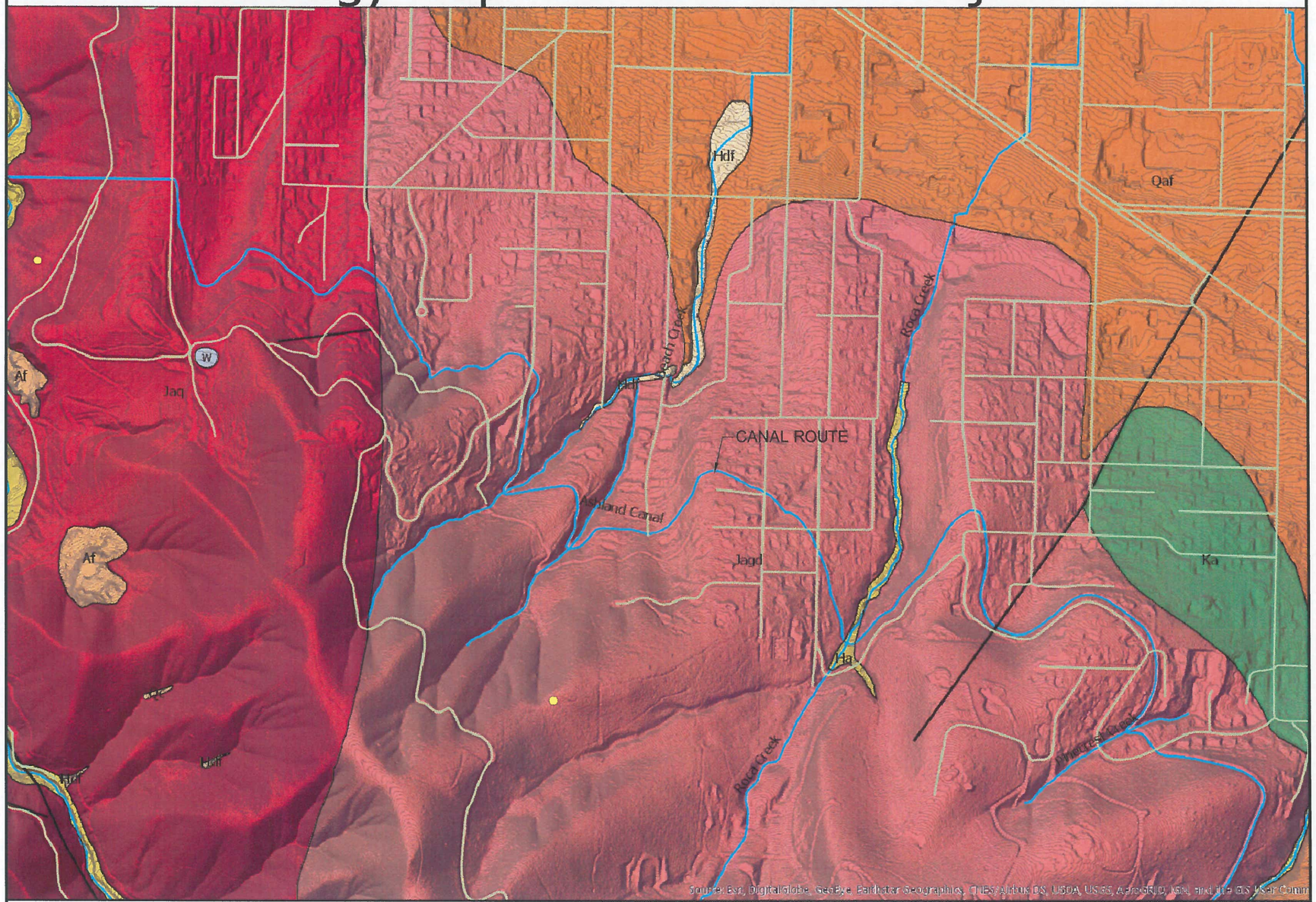
ASHLAND CANAL WITH TOPOGRAPHY

ASHLAND CANAL
ASHLAND, OREGON

DATE: AUGUST 2018
JOB NO: 02-5407-01
REV: 8/23/2018 8:13 PM
PREPARED BY: MG3
5407 Ashland Canal - 03 - Canal plan w.topo.dwg

FIGURE: 3

Geology Map- Ashland Canal Project



LEGEND

Contact approximately located.

Fault--dashed where approx. located; dotted where concealed;

Geology from:
OGDC-6, 2015

12-foot contours and hillshade from DOGAMI Lidar 2015 (1-meter grid resolution)

LIST OF GEOLOGIC UNITS

- Af — Modern fill and construction material
- Ha — Holocene Alluvium
- Hdf — Holocene debris from deposits
- Qaf — Quaternary alluvial fan
- Ka — Cretaceous Hornbrook Fan - Ashland Sandstone member
- Jaq — Jurassic Mt. Ashland Pluton - quartz monzonite
- Jagd — Jurassic Mt. Ashland Pluton - biotite-hornblende granodiorite

NORTH

0 700 1,400
APPROXIMATE SCALE IN FEET

THE GALLI GROUP
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Grants Pass, OR 97526

GEOLOGIC MAP
ASHLAND CANAL
ASHLAND, OREGON

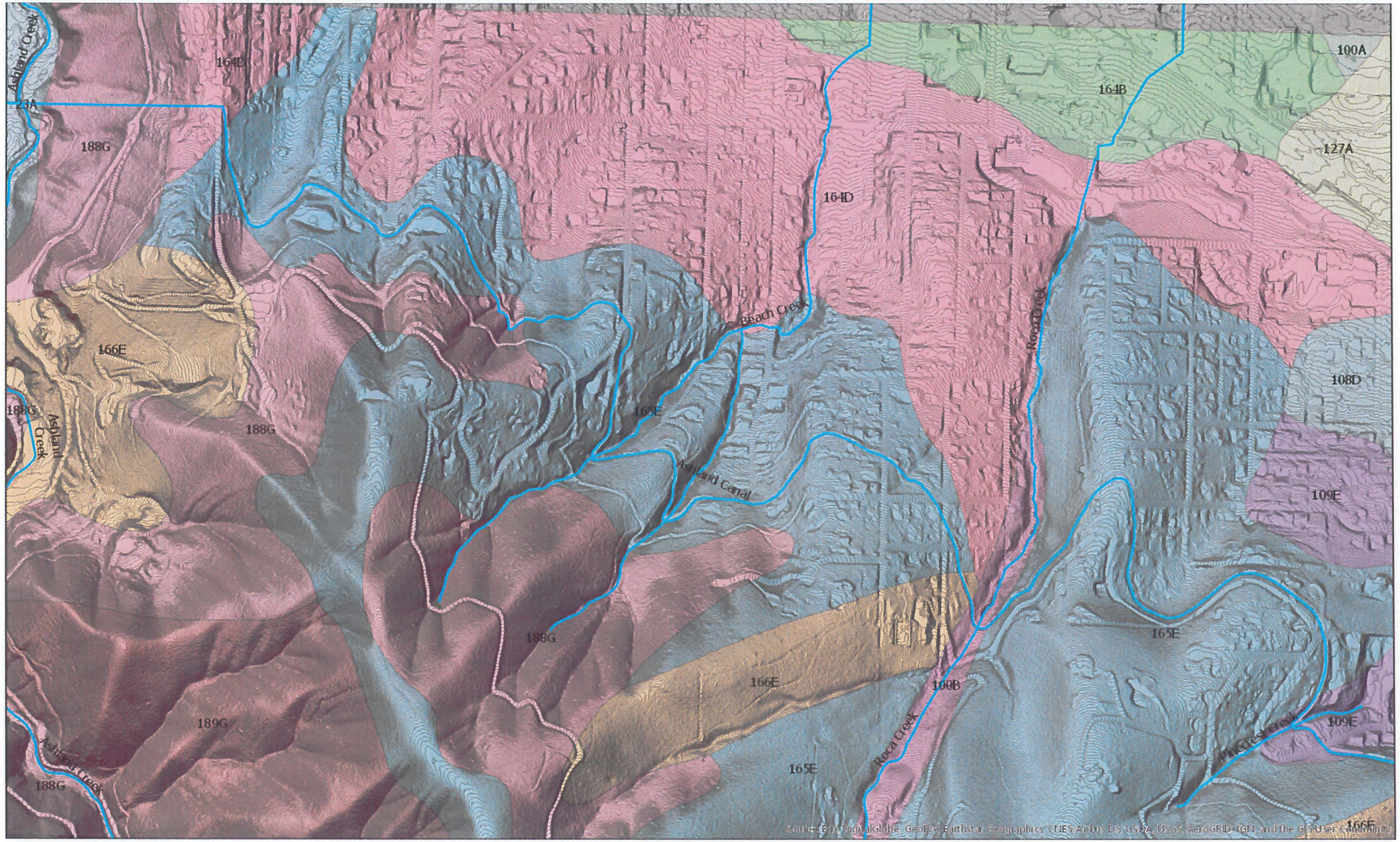
DATE:	AUGUST 2018
JOB NO:	02-5407-01
REV:	8/23/2018 7:32 PM
PREPARED BY:	MG3
5407 Ashland Canal - 04 - Geo Map.dwg	

FIGURE: 4

LS_points
Streets
Streams
contour
Hillshade 2015.tif
Value
255
1

N

THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	HILLSHADE AND 2-FOOT CONTOURS ASHLAND CANAL ASHLAND, OREGON	DATE: AUGUST 2018 JOB NO: 02-5407-01 REV: 8/23/2018 7:35 PM PREPARED BY: MG3 5407 Ashland Canal - 05 - Hillshade topo.dwg	FIGURE: 5



0 0.1 0.2 0.4 Miles

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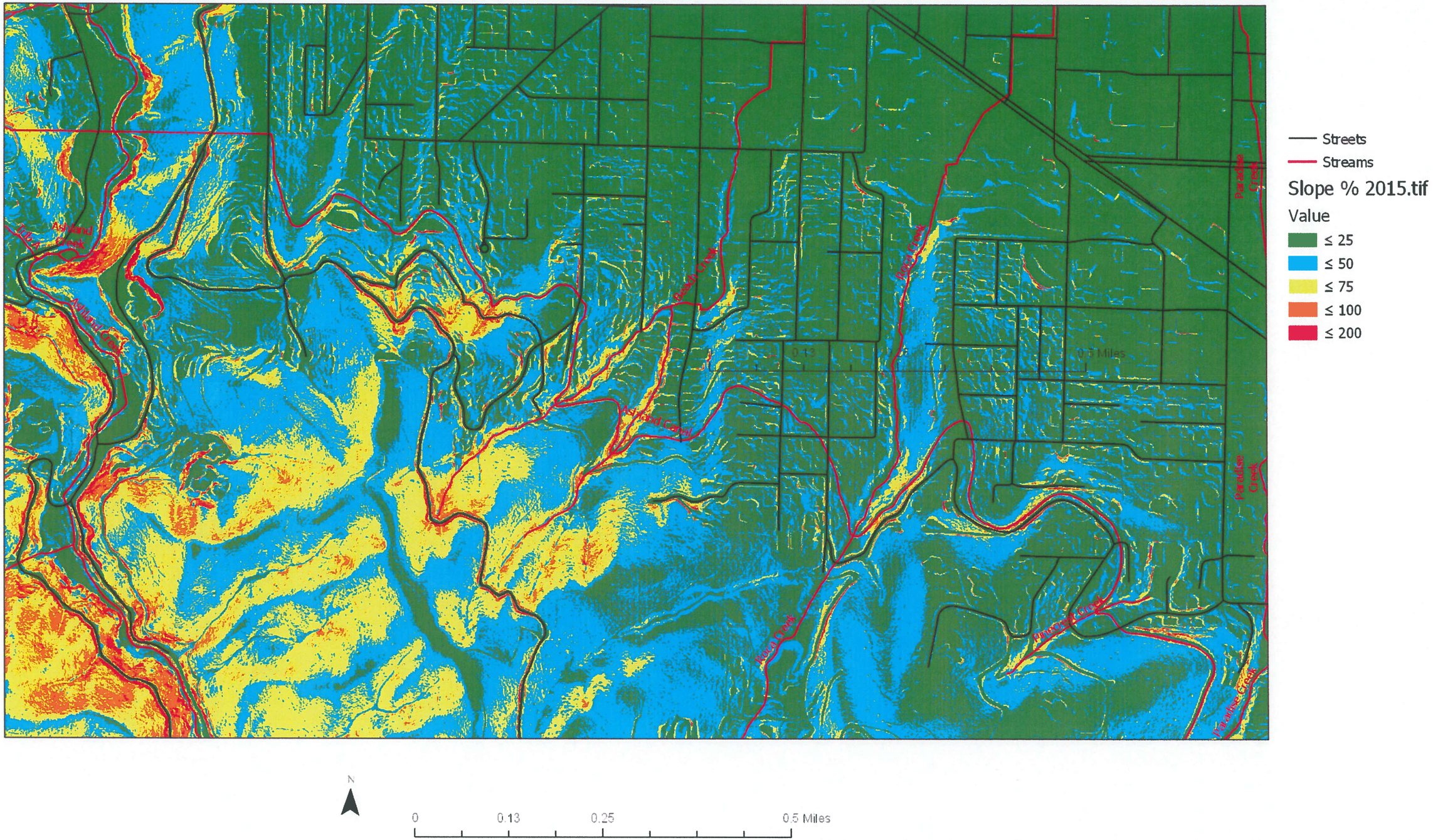
612 NW 3rd Street
Grants Pass, OR 97526

NRCS SOIL MAP

ASHLAND CANAL
ASHLAND, OREGON

DATE:	AUGUST 2018
JOB NO:	02-5407-01
REV:	8/23/2018 7:43 PM
PREPARED BY:	MG3
5407 Ashland Canal - 06 - NRCS soils.dwg	

FIGURE: 6



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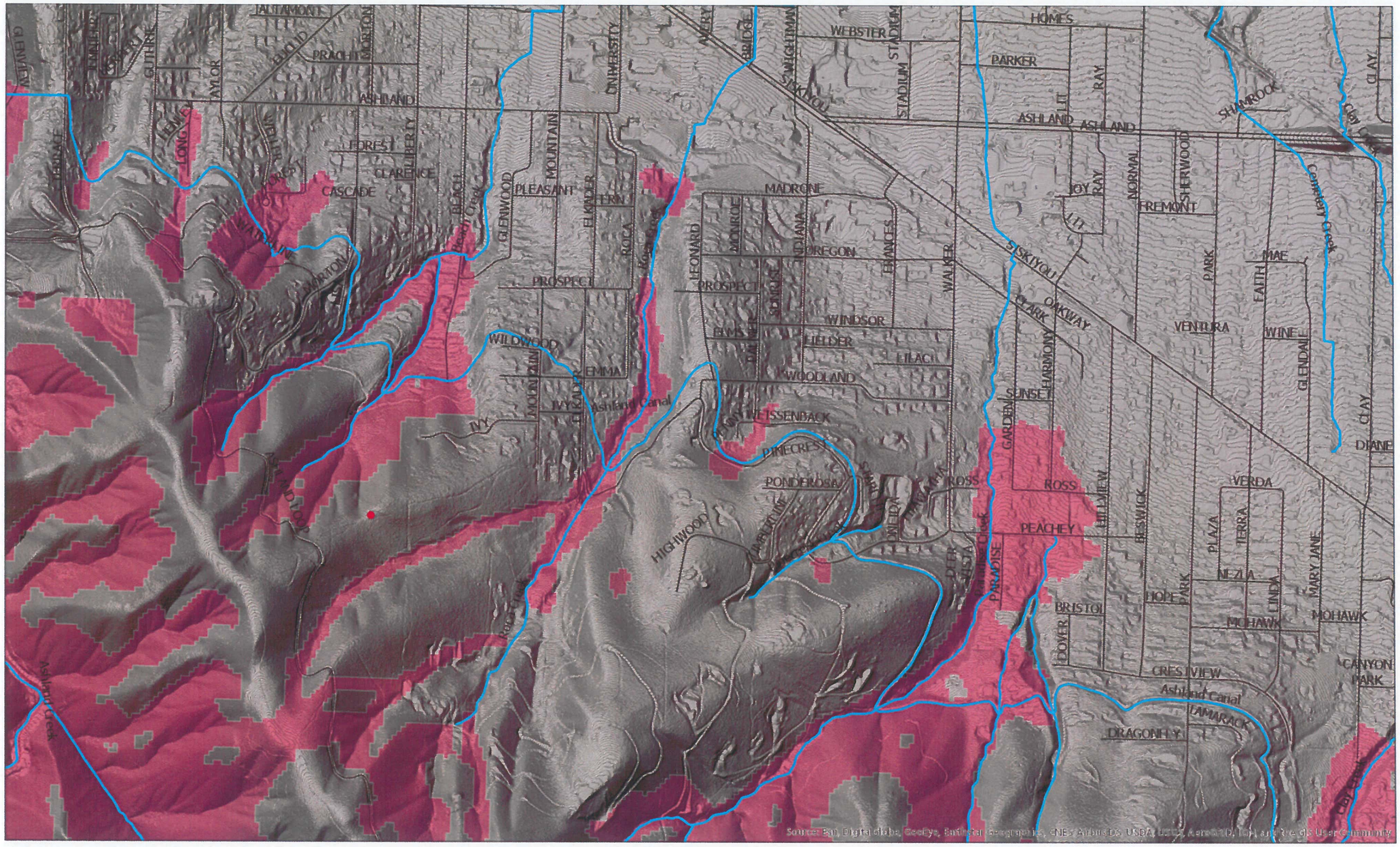
612 NW 3rd Street
Grants Pass, OR 97526

SLOPE MAP IN PERCENT

ASHLAND CANAL
ASHLAND, OREGON

DATE:	AUGUST 2018
JOB NO:	02-5407-01
REV:	8/24/2018 9:10 AM
PREPARED BY:	MG3
5407 Ashland Canal - 07 - Slopes Percent Map.dwg	

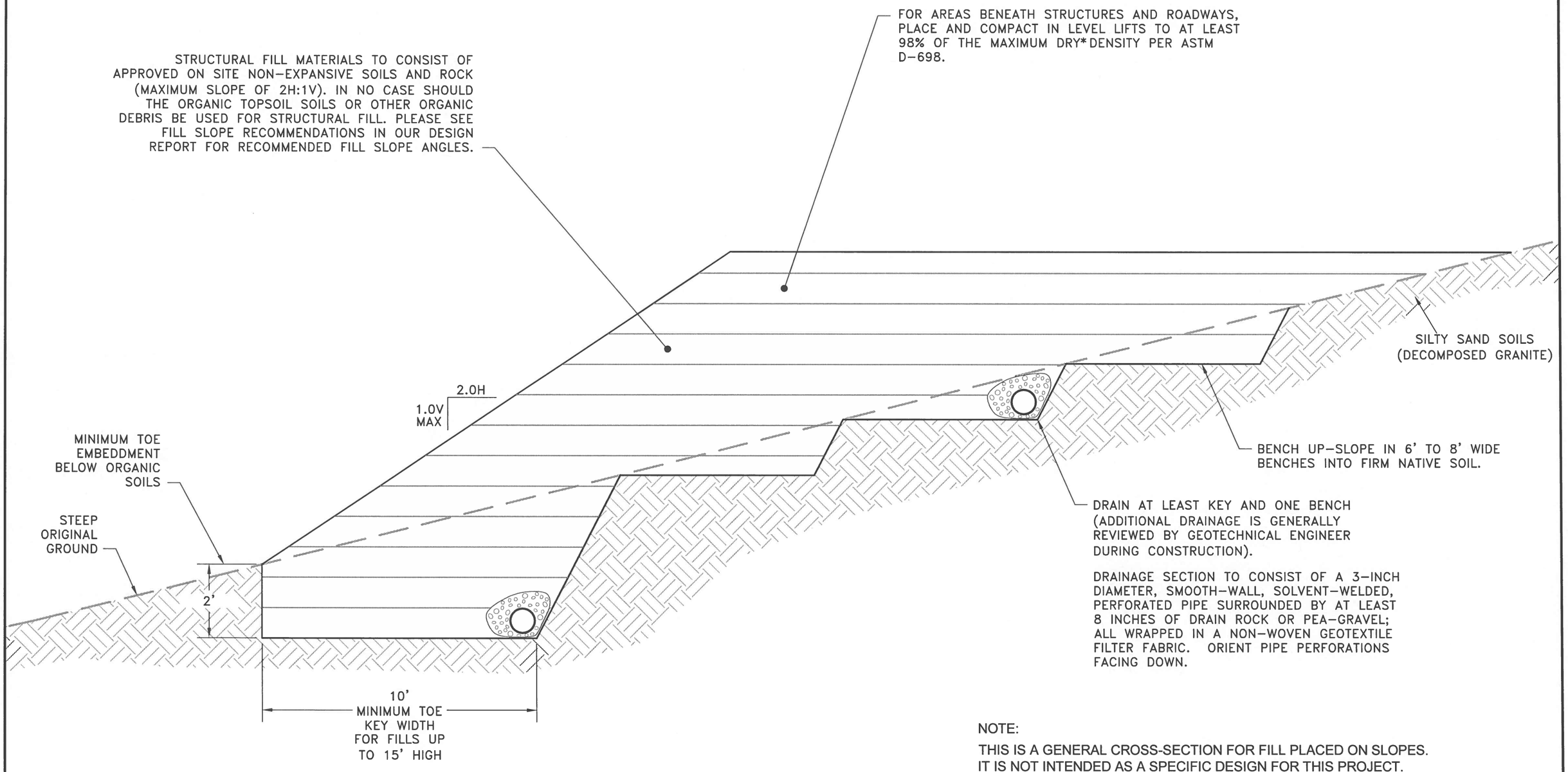
FIGURE: 7



N

0 0.15 0.3 0.6 Miles

THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	POTENTIAL DEBRIS FLOW HAZARD MAP ASHLAND CANAL ASHLAND, OREGON	DATE: AUGUST 2018 JOB NO: 02-5407-01 REV: 8/24/2018 9:10 AM PREPARED BY: MG3 5407 Ashland Canal - 08 - Debris flow.dwg	FIGURE: 8



NOTE:

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

FOR ILLUSTRATION PURPOSES ONLY
NOT TO SCALE

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612 NW 3rd Street
Grants Pass, OR 97526

FILL ON STEEP SLOPE
CROSS-SECTION

ASHLAND CANAL
ASHLAND, OREGON

DATE:	AUGUST 2018
JOB NO:	02-5407-01
REV:	8/24/2018 9:18 AM
PREPARED BY:	MG3
5407 Ashland Canal - 09 - slope fill.dwg	

FIGURE: 9

TYPICAL RETAINING WALL CROSS-SECTION

NOTE: TWO COATS (OR ONE THICK COAT) OF A HIGH QUALITY WALL SEALER. FLEXIBLE BITUMEN-BASED, SPRAYED, ROLLED OR TROWELED-ON MATERIALS SHALL BE USED. WE RECOMMEND MASTERBLEND HLM5000, OR EQUIVALENT. BENTONITE PANELS AND STICKY-BACKED MEMBRANES ALSO WORK WELL. THIS IS CRITICAL FOR WALLS WHICH HAVE DRY LIVING SPACE INSIDE

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

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

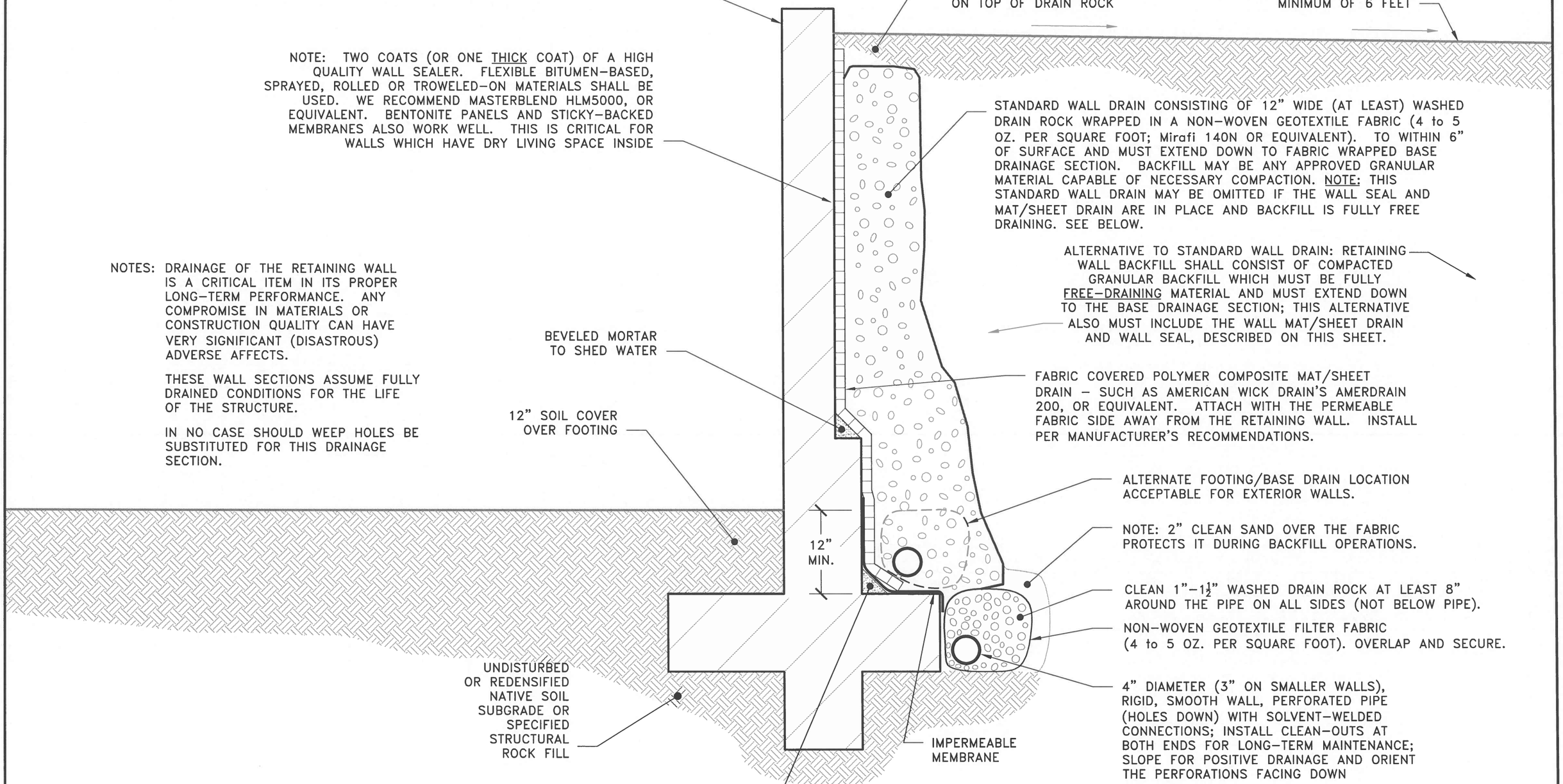
IN NO CASE SHOULD WEEP HOLES BE SUBSTITUTED FOR THIS DRAINAGE SECTION.

BEVELED MORTAR TO SHED WATER

12" SOIL COVER OVER FOOTING

UNDISTURBED OR REDENSIFIED NATIVE SOIL SUBGRADE OR SPECIFIED STRUCTURAL ROCK FILL

BEVELED MORTAR TO SHED WATER



CLAYEY SOIL SEAL OR PLASTIC SHEETING ON TOP OF DRAIN ROCK

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

STANDARD WALL DRAIN CONSISTING OF 12" WIDE (AT LEAST) WASHED DRAIN ROCK WRAPPED IN A NON-WOVEN GEOTEXTILE FABRIC (4 to 5 OZ. PER SQUARE FOOT; Mirafi 140N OR EQUIVALENT). TO WITHIN 6" OF SURFACE AND MUST EXTEND DOWN TO FABRIC WRAPPED BASE DRAINAGE SECTION. BACKFILL MAY BE ANY APPROVED GRANULAR MATERIAL CAPABLE OF NECESSARY COMPACTION. NOTE: THIS STANDARD WALL DRAIN MAY BE OMITTED IF THE WALL SEAL AND MAT/SHEET DRAIN ARE IN PLACE AND BACKFILL IS FULLY FREE DRAINING. SEE BELOW.

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

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

ALTERNATE FOOTING/BASE DRAIN LOCATION ACCEPTABLE FOR EXTERIOR WALLS.

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

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

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

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

IMPERMEABLE MEMBRANE

FOR ILLUSTRATION PURPOSES ONLY

NOT TO SCALE

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

EXTERIOR RETAINING WALL DRAINAGE CROSS-SECTION

ASHLAND CANAL
ASHLAND, OREGON

DATE: AUGUST 2018
JOB NO: 02-5407-01
REV: 8/24/2018 9:19 AM
PREPARED BY: MG3
5407 Ashland Canal - 10 - retwall drain-ext-03.dwg

FIGURE: 10

APPENDIX A

SITE PHOTOS

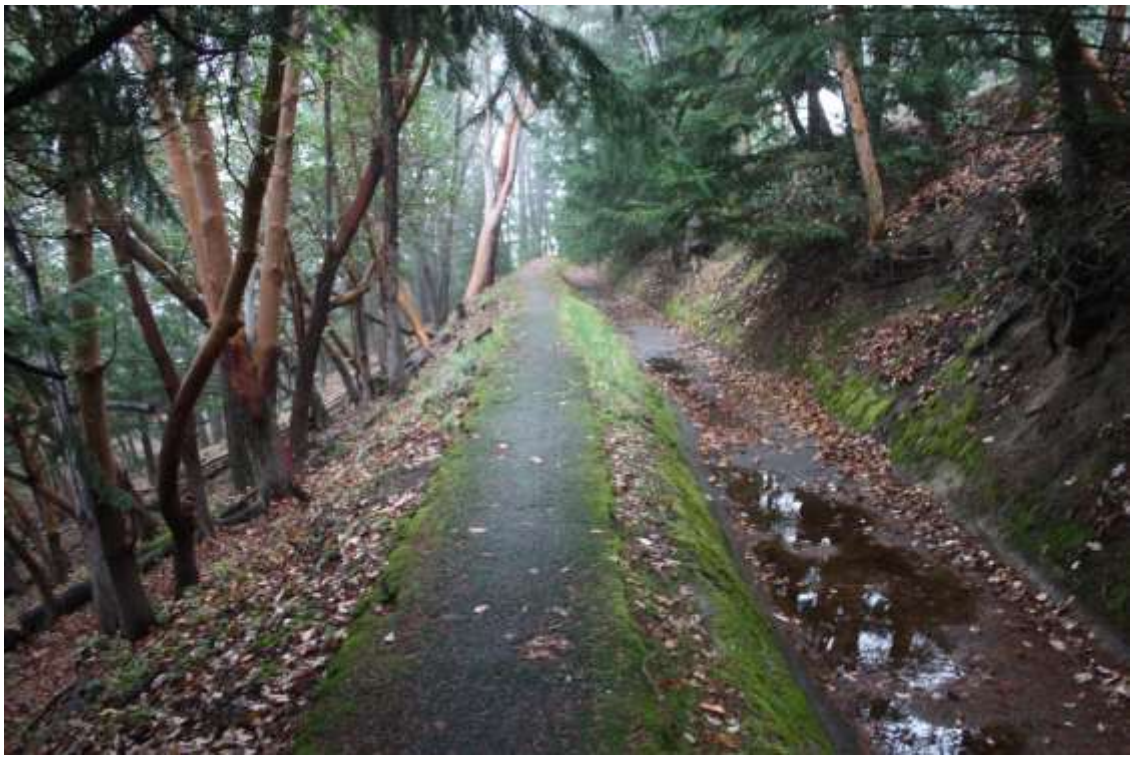

PHOTO 1: East End of Canal on Steep Slopes

PHOTO 2: Steep Slopes Near Ashland Loop Road

PHOTO 3: Dual Culvert Beneath Roadway

PHOTO 4: Culvert Beneath Driveway

PHOTO 5: Canal Goes Over Ravine Culvert

PHOTO 6: Ravine Comes into and Below Canal Near Ashland Loop Road

PHOTO 7: Car Shelter Built Right Next to Buried Portion of Canal

PHOTO 8: House and Fence Next to Canal

PHOTO 9: Deck Supports on Bank of Canal

PHOTO 10: Footbridge Over Canal Near East End

PHOTO 11: Footbridge Over Canal

PHOTO 12: Footbridge Over Canal

PHOTO 13: Driveway Bridge Over Canal Near West End

PHOTO 14: Tree Buckled Side of Canal

PHOTO 15: Tree Buckled Side of Canal

PHOTO 16: Conjugated Pipe Discharging into Canal

PHOTO 17: Steel Pipe Discharging into Canal

PHOTO 18: Conjugated Pipe form House Discharges into Canal

PHOTO 19: Large Boulder Encroaching into Canal Area

PHOTO 20: Canal in Double Culvert Under the Road

PHOTO 21: Double Culvert Under Roadway

PHOTO 22: Double Culvert Under Property

PHOTO 23: Culvert Under Driveway then Roadway and Several Lots

PHOTO 24: Culvert Under Driveway with Lions

PHOTO 25: Culvert Under Access Pathway

PHOTO 26: Culvert Under the Driveway

PHOTO 27: Hatch to Buried Canal Culvert

PHOTO 28: Legal Spill from Canal

PHOTO 29: Legal Spill from Canal

PHOTO 30: Large Cracks that Create Significant Leakage

PHOTO 31: Fractures in Bottom and Sides of Canal

PHOTO 32: Area Subject to Debris/Mud Flows

PHOTO 33: Area Subject to Mud Flows by the Roadway

PHOTO 34: Cut Slope Instability

PHOTO 35: Ecology Block Support of Existing Canal Culvert