GEOTECHNICAL REPORT – AMENDMENT 1
DEADHORSE CANYON LANDSLIDE STABILIZATION ALTERNATIVES
SEATTLE, WASHINGTON

Work Authorization No.: E314059 MG2
July 2017

Seattle Public Utilities
Geotechnical Engineering

707 South Plummer Street
Seattle, Washington 98134

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SEATTLE, WASHINGTON

1.0 PURPOSE AND SCOPE

Seattle Public Utilities (SPU) Geotechnical Engineering was retained to complete a geotechnical engineering study for the Deadhorse Canyon Sewer in the Lakeridge neighborhood of Seattle, Washington. The project location is shown on Figure 1.

A 10-inch diameter sanitary sewer line is located in the east slope of Deadhorse Canyon in Lakeridge Park. An approximately 100 foot long section of the slope has subsided up to 1 foot. There is concern that additional slope movement could cause damage to the underlying sewer line. In 2014, we completed a subsurface exploration program and a preliminary slope stability analysis, and provided recommendations for several conceptual slope stabilization techniques.

Based on the results of the slope stability analysis, the project team requested that we further investigate two of the slope stabilization options for the site and assess the slope conditions within the canyon along the length of sewer. Our work was requested and authorized by Betsy Lyons of the SPU Capital Portfolio Management Division. Our scope of work included:

- Reviewing GIS and LIDAR maps of Deadhorse Canyon;
- Completing a field reconnaissance to verify conditions observed on the maps;
- Developing detailed scopes for two slope stabilization alternatives; and,
- Preparing this report summarizing our investigations and conclusions.

2.0 BACKGROUND INFORMATION

We understand that the sewer line was installed in Deadhorse Canyon and a recreational trail was constructed above the sewer in 1997. In 2009, the Seattle Parks and Recreation Department (Parks) first noticed settlement along a section of the trail located approximately 1800 feet south of the 68th Avenue S trailhead. A member of the volunteer group that assists with maintenance of the trail indicated that an additional 6 inches of movement occurred after a period of heavy rain during the winter of 2013 and 2014. In 2014, SPU Geotechnical Engineering completed subsurface explorations and analyzed the stability of the slope above and below the settling trail. The results of the analyses indicated that the slope is marginally stable and stabilization was recommended. Based on these results, the project team requested that we assess the slope conditions within the canyon along the length of sewer and further investigate two of the slope stabilization options for the site.

3.0 SLOPE CONDITION ASSESSMENT

The slope condition assessment for Deadhorse Canyon included reviewing published LIDAR and topographic maps, and completing a limited slope stability reconnaissance. The results of the site characterization were used to assess the condition of the slopes along Deadhorse Canyon. Our study is confined to an approximately 2,200 foot long portion of Deadhorse Canyon that is located between the confluence of the east and west forks of Taylor Creek and 68th Avenue South (Figure 1).

3.1 SURFACE CONDITIONS

Taylor Creek flows along the base of the north-south oriented canyon dropping from elevation 185 feet at the confluence of the east and west forks to elevation 75 feet at 68th Avenue South. After passing under 68th Avenue South, Taylor Creek continues north for approximately 0.2 miles to Lake Washington. The ground surface along the top of the canyon varies from elevation 270 feet at the south to elevation 215 feet at the north. These approximate ground surface elevations were determined based on LIDAR topography of the area, and are referenced to the NAVD88 datum.

The Seattle Department of Planning and Development (DPD) have classified the entire slope along the east side of the canyon and portions of the slope along the west side of the canyon as steep slopes (greater than 40 percent slope inclination with at least 10 feet of elevation difference). In general, the east slopes have a constant inclination from top to bottom, while the west slope tends to be steep near the top of the slope, flatter near mid slope and becomes steeper again near the toe of the slope. The aggregate surfaced recreational trail and sewer line are located on the west slope between 20 and 40 feet above Taylor Creek. The active failure is located on a 27 percent slope, which is flatter than the average slope inclination within the study area.

3.2 HISTORICAL LANDSLIDES

In 2005, the United States Geological Survey (USGS) (Schulz, 2005) used LIDAR imagery to create a landslide inventory map for Seattle. Based on landforms visible in the LIDAR imagery, possible headscarps, landslide deposits, and denuded slopes were identified. The historical landslides identified using LIDAR were compared to data from a landslide inventory based on historical records (Shannon and Wilson, Inc., 2003) to develop a relationship between landslide potential and the three identified landform types plus an additional landform representing the areas within Seattle where no landslide-related landforms were observed.

Schulz proposes that because future landslide activity in Seattle is expected to be similar in type and location to recent activity, projections can be made about the relative likelihood of landslides within a given area. The results of the study indicate that compared to areas where no landslide-related landforms have been observed, the likelihood of future landslides is 244, 86, and 47 times greater within areas mapped as headscarps, landslide deposits, and denuded slopes, respectively. Figure 2 shows the landslide-related landforms identified by the USGS draped over LIDAR based digital elevation model of Deadhorse Canyon. The majority of the west slope is within three of the mapped historical landslides, indicating that landslides are 86 to 244 times more likely to occur within Deadhorse Canyon than areas where no landslide-related landforms have been observed.

3. 3 FIELD OBSERVATIONS

On June 18, 2015 an SPU field geologist and geotechnical engineer completed a field reconnaissance that consisted of walking the slopes looking for signs of active movement (e.g., tension cracks, scarps, and leaning trees). Particular attention was paid to areas indicated on Figure 2 as being within previous scarp and landslide deposit areas. The route that was followed and items observed during the reconnaissance are indicated on Figure 3.

In general, the slopes of the canyon are heavily vegetated with mature conifers and deciduous trees and low growing brush. The trees are generally straight with only a few with pistol-butted or leaning trunks.

Several seeps and some ponded water were observed in the northern half of the study area west of the trail and between approximately elevation 130 and 140 feet.

Two areas that exhibit potential signs of movement were observed, however, one of the areas is located outside of an area likely to affect the sewer. The other area is located approximately 100 feet north of the active slide. At this location, the surface of the trail is uneven and there are several discontinuous cracks located along the uphill side of the trail. It appears that surface water flows down the trail to this location, then discharges over the downhill slope, as a result, it is difficult to determine if the uneven ground is caused by surface erosion or by larger scale movement. The two areas that exhibit potential signs of movement are shown on Figure 3. No other signs of slope instability were observed.

3.4 SUMMARY

The presence of historical landslides along the eastern slopes of Deadhorse Canyon indicates that there is a higher probability of future landslides within the canyon, however, no obvious signs of current slope movement were observed during our site visit. One area with possible signs of slope movement was observed near the active slide.

4.0 SLOPE STABILIZATION ALTERNATIVES

In our December 2014 Geotechnical report, we provided four potential slope stabilization alternatives for the site: groundwater control, slope grading, a rock buttress, and structural reinforcement. However, we determined that groundwater control alone could not increase the factor of safety (FS) against slope instability to acceptable levels, and the location of the sewer line limits the ability to regrade the slope. As a result, only a rock buttress and structural reinforcement appear to be feasible given the site constraints. In general, both of these slope stabilization measures are long-term solutions with a design service life of at least 50 years and would not require maintenance during the design service life.

In this section we provide more detailed recommendations and construction details for these two slope stabilization alternatives to aid in the selection of the preferred alternative for the site. We understand that it is standard practice to design sewer lines for static loads alone; however, we provide recommendations for both static and seismic design. The recommendations in this section should be considered preliminary and additional analysis should be completed if either of these two alternatives is selected.

4.1 ROCK BUTTRESS OPTION

Rock buttresses typically consist of rip rap placed along the toe of a landslide. The increased weight of material at the toe acts as a counterforce that resists movement. We estimate that a minimum of 1,200 and 1,470 cubic yards (CY) of rip rap would be needed to provide sufficient counterforce to resist static and seismic loading, respectively. The rock buttress would be approximately 10 to 15 feet tall, an average of 12 to15 feet thick, and extend 185 feet along the toe of the slope. The static design would be at the lower end of this range and the seismic design would be at the upper end. Conceptual cross-section, elevation and plan views of a rock buttress are shown on Figure 4.

4.1.1 Constructability and Access

A temporary access road would need to be constructed to allow dump trucks (10 CY capacity) to deliver rip rap and remove excavated material. The access road could follow the general alignment of the existing trail from NE 68th Avenue S to the site (approximately three tenths of a mile). However, the trail will need to be widened from the existing approximately 4 foot width to a minimum width of 10 feet. Most of this additional width could be re-vegetated after construction to return the trail to approximately the original width and condition. At some locations significant cutting into the hillslope will be required to gain the required width.

Culverts would need to be installed at the bottom of approximately two of the larger drainages that the access road would cross. At these locations, the existing trail bridges would be removed and fill would be placed to create a level roadbed. After construction of the buttress is complete the fill and culverts could be removed and the trail bridges reset.

Construction would require vegetation removal and excavation within the buttress footprint shown on Figure 4. The area between the trail and the buttress will also need to be cleared to allow track mounted excavators to access the buttress area. All of the clearing and excavation would occur within the Taylor Creek riparian area which extends from the creek up to the trail. We estimate that approximately 700 to 800 CY of excavation will be required within the buttress footprint. This includes approximately 3 feet of excavation below the creek bed elevation. The excavation below the creek bed would likely require diversion of the creek and localized dewatering. All excavated material would need to be removed from the site using dump trucks and the temporary access road.

4.1.2 Schedule

We anticipate that construction of the rock buttress could be completed within a four to five week period which includes approximately three weeks of in-water work. Deadhorse Canyon Park would need to be closed to the public during construction.

Because some of the work will take place within the ordinary high water mark of Lower Taylor Creek, multiple local, state and federal permits will be required. Typically the applications for these permits are processed within 60 days; however, the time period can vary from project to project.

We recommend that construction be completed during the dry season to minimize the potential for slope instability during construction and to minimize the amount of dewatering that is required. In addition the work must be completed within state and federally mandated in-water work windows.

4.1.3 Cost Estimate

Ten percent conceptual design level itemized cost estimates for construction of rock buttresses meeting static and seismic design criteria are provided in Appendix A. We anticipate that regulating agencies would require mitigation in another area to offset the loss of natural habitat along the creek. The cost estimate does not include the mitigation costs.

4.1.4 Risk Assessment

The rock buttress option has several risks that are present during the design, construction, and post-construction phases of the project.

The design risks are fundamentally the same for the rock buttress and the slope reinforcement options. Both designs are based on three discrete borings completed along one cross-section of the slide. The actual conditions may differ from those observed in the borings and assumed based on available information. In addition, the geometry of the slide is based on visual cues observed at the ground surface, soil properties determined from our subsurface exploration, and our experience with other landslides. As a result, there is some degree of uncertainty in the assumed location of the slip surface and the mass of the slide. If the actual slide is deeper than we have assumed, the designed fix may not be able to retain the sliding mass of soil. The location of the slip surface could be more accurately located with a long term monitoring program that would include slope inclinometers and groundwater monitoring wells.

During construction, excavation along the toe of the slide increases the possibility of destabilizing the slope. This can be minimized by completing the buttress in prescribed stages to reduce the volume that is excavated, and completing the work during the dry season. Additional slope stability analyses should be completed for slope configurations that are representative of different phases of construction to determine if the work needs to be completed in stages.

The rock buttress will create a hard surface along the creek bank. During high flows, the creek will reflect off of this hard surface and could cause increased scour and erosion along the opposite bank. This is of particular concern because there is an existing problem with excessive stream bed loading downstream of the site that would be worsened by an increase in streambed erosion.

After construction there is a risk that surface erosion in the large denuded area could result in increased volumes of sediment in the creek during the first winter. Erosion could be minimized by including an aggressive re-vegetation plan in the contract. However, because the cleared area is relatively large, there is a moderate risk of excessive sediment loads even with proper re-vegetation.

4.2 STRUCTURAL REINFORCEMENT OPTION

Structural reinforcement of a slope typically includes the installation of a row of structural reinforcing elements along the length of the slope failure plus 10 to 15 feet beyond the edges of the failure. As a result, the reinforcing elements would extend approximately 160 feet along the slope. The reinforcing elements are installed approximately mid-way between the head and toe of the slide. At the Deadhorse site the structural elements would be advanced across the failure plane into the stable glacial till material to provide passive shear capacity, disrupt the slide plane, and reinforce the soil mass. We determined that an increase in shear strength of 5.5 and 18 kips per lineal foot was required to provide the target FS for the static and seismic cases, respectively.

Reinforcing elements can consist of drilled shafts, micropiles, soil nails or other structural elements. The access limitations at the site preclude the use of larger diameter reinforcing elements (including drilled shafts) that would require large construction equipment. Smaller diameter reinforcing elements including micropiles and soil nails are better suited to the site. While vertically installed micropiles and soil nails have been used to stabilize landslides, soil nails are not typically used for this application in the United States (Lazarte et al., 2003). As a result, we recommend micropiles for stabilization of the Deadhorse Canyon site.

Micropiles are small diameter drilled and grouted elements that are typically reinforced using either a drill casing or high strength reinforcing bar. Although micropiles can also be driven, we recommend drilled micropiles because the hammer required to drive the micropiles sufficiently in the till would likely be too large to access the site via the existing trail. Once installed, the micropiles will not require maintenance and can be designed to achieve a service life of between 50 and 75 years.

A preliminary design chart (Armour, 1997) indicates that battered 6-inch-diameter micropiles spaced approximately 2-foot on-center should provide the required shear resistance for the seismic case. We anticipate that the micropiles would extend approximately 25 feet below ground surface. An approximately 2-foot-wide reinforced concrete cap beam would be installed along the length of the micropiles to structurally tie the piles together. We assume that SPU and Parks would prefer to minimize visual impacts within the park and as a result the top of the piles and cap beam would be buried below the existing ground surface. Conceptual cross-section, elevation and plan views of the micropile slope stabilization are shown on Figure 5.

4.2.1 Constructability and Access

Construction of a buried micropile slope stabilization system would begin with the excavation of an approximately 3 foot by 3 foot trench along the micropile alignment (approximately 55 CY). Approximately half of this material will be used to backfill the excavation. The remaining 25 to 30 CY would either be spread on site or removed. An excavation of this size could be completed using a mini excavator with a track width less than 4 feet. After the trench has been excavated, a small drill rig will drill the holes for the micropiles. A drilling contractor has confirmed that small micropile drill rigs with tracks as narrow as 28 inches wide are available locally. Reinforcing bar is then inserted in the drilled holes and the annular space is filled with pressurized grout. A small skidmounted grout plant would be used to mix the grout at the site and to place the grout into the drilled holes. We anticipate that the grout and reinforcing materials could be delivered to the site using small trailers and off road vehicles.

Because all of the equipment required for construction of the micropiles is available in widths less than 4 feet, minimal improvements will be required along the trail to provide access to the site. However, some work will be required in the areas with wooden steps and trail bridges. Fill could be temporarily placed over the steps to create a flat path for the various pieces of equipment. The trail bridges would need to be analyzed by a structural engineer to determine if the proposed equipment can cross the bridges without causing damage or collapse.

Construction would require vegetation removal and excavation within the micropile footprint shown on Figure 5. The area between the trail and the micropiles will most likely need selective clearing to allow access for the drill rig and materials. It may be possible to work around some of the larger trees that are located in this area; however, at this time we do not have surveyed locations for the trees so it is impossible to determine which trees would need to be removed.

All of the clearing, excavation and micropile installation would occur within the Taylor Creek riparian area which extends from the creek up to the trail. However, no work would occur below the ordinary high water mark.

4.2.2 Schedule

We anticipate that construction of the micropile slope stabilization could be completed within a seven to eight week period. We recommend that Deadhorse Canyon Park be closed to the public during construction.

The work will take place within 200 feet of the ordinary high water mark of Lower Taylor Creek. As a result, a shoreline use permit or exemption will be required. Typically the applications for these permits are processed within 30 days; however, the time period can vary from project to project.

We recommend that construction be completed during the dry season to minimize the potential for erosion and slope instability during construction.

4.2.3 Cost Estimate

Ten percent conceptual design level itemized cost estimates for construction of micropile slope stabilization meeting static and seismic design criteria are provided in Appendix A.

4.2.4 Risk Assessment

The slope reinforcement option has several risks that are present during the design, construction, and post-construction phases of the project.

The design risks are fundamentally the same for the rock buttress and the slope reinforcement options. Both designs are based on three discrete borings completed along one cross-section of the slide. The actual conditions may differ from those observed in the borings and assumed based on available information. In addition, the geometry of the slide is based on visual cues observed at the ground surface, soil properties determined from our subsurface exploration, and our experience with other landslides. As a result, there is some degree of uncertainty in the assumed location of the slip surface and the mass of the slide. If the actual slide is deeper than we have assumed, the designed fix may not be able to retain the sliding mass of soil. The location of the slip surface could be more accurately located with a long term monitoring program that would include slope inclinometers and groundwater monitoring wells.

During construction there is some concern that the increased loads from equipment operating near the top of the slope could decrease the overall stability of the slope, however, the risk is minimal due to the small equipment that would be used. There is also a risk that obstructions such as boulders could be encountered during drilling. The small drill rigs that can access the site may not be able to drill through the obstructions. Based on our interpretation of the subsurface conditions, there is minimal risk of encountering boulders in the upper 15 feet of the soil column, however, there is a potential for encountering boulders in the lower glacial till soil even though cobbles and boulders were not encountered during our subsurface explorations. Another risk associated with drilling is the potential to create subsurface voids if caving soils are present causing too much soil to be removed. The upper 5 to 10 feet of soil at the site have low to moderate potential for caving; the deeper soils have very low potential. The holes can be cased during drilling to prevent caving.

After construction there is a risk that surface erosion in the denuded area could result in increased volumes of sediment in the creek during the first winter. However, the cleared area will be relatively small and a strip of undisturbed vegetation will be left along the creek. As a result, the risk of excessive sediment loads being generated is relatively low. In addition, erosion could be minimized by including an aggressive re-vegetation plan in the contract. We recommend that plants with shallow root systems that extend less than 1 foot below ground surface are selected to re-vegetate the area within a horizontal distance of 5 feet of the embedded concrete beam and micropiles.

The structural reinforcement would stabilize the slope above the reinforcing elements, but the slope below the reinforcing elements could still fail. Failure of the lower slope should not adversely affect stability of the upper slope and sewer. However, additional work would be required to stabilize the failed lower slope surface to prevent the propagation of erosion that could eventually affect stability of the upper slope. It is our opinion that the the risk of the lower slope failing with structural reinforcement in place is low.

5.0 LIMITATIONS AND ADDITIONAL SERVICES

This report was prepared accordance with generally accepted professional principles and practices in the field of geotechnical engineering at the time the report was prepared. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site. However, we did not encounter apparent indications of contamination in our explorations

This geotechnical report is intended to provide information and recommendations to support preliminary engineering activities for this project. The conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions.

We recommend that SPU Geotechnical Engineering be retained to review the plans and specifications and verify that our recommendations have been interpreted and implemented as intended. Sufficient geotechnical monitoring, testing, and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations and to verify that the geotechnical aspects of construction comply with the contract plans and specifications. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated.

We appreciate the opportunity to be of service.

Sincerely,

SPU GEOTECHNICAL ENGINEERING

MEGAN K. HIGGINS
50842

09/16/22

Megan Higgins, P.E.
Geotechnical Engineer

6.0 REFERENCES

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Lazarte, C.A., Elias, V., Espinoza, R.D., Sabatini, P.J., 2003. Federal Highway Administration Geotechnical Engineering Circular No. 7: Soil Nail Walls.

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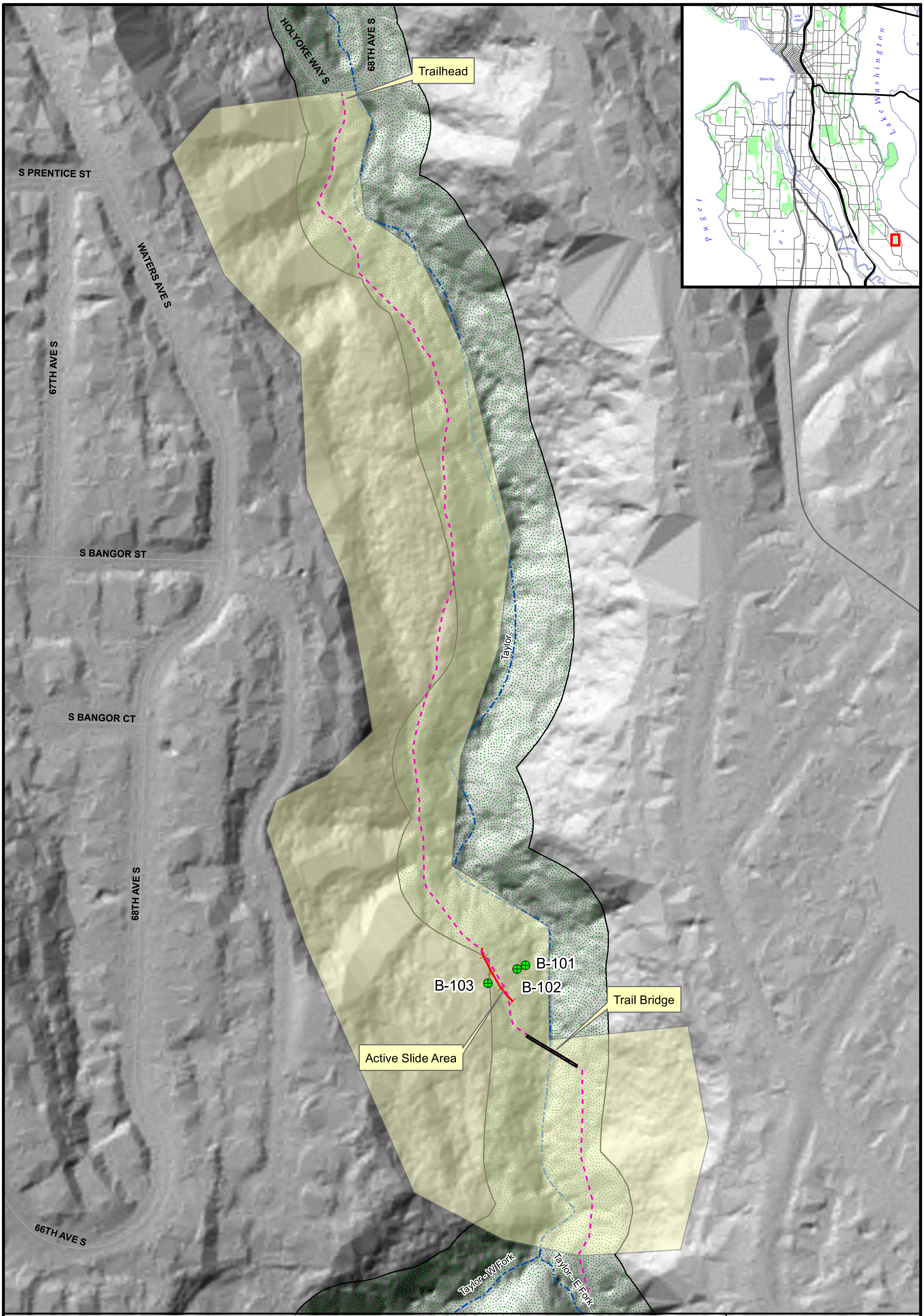

- - - Sewer and Trail Alignment
-+- Scarp - Active Slide
⊕ SPU Exploration Location
▭ Slope Assessment Area
-·- Stream
▨ Riparian Corridor

FIGURE 1 | **AUGUST 2015**

75 37.5 0 75 150
Feet

NAVD88

N
W E
S

Seattle Public Utilities
Geotechnical Engineering

Deadhorse Canyon Landslide Stabilization Alternatives
E314059

Site Map - Study Area

Seattle, Washington

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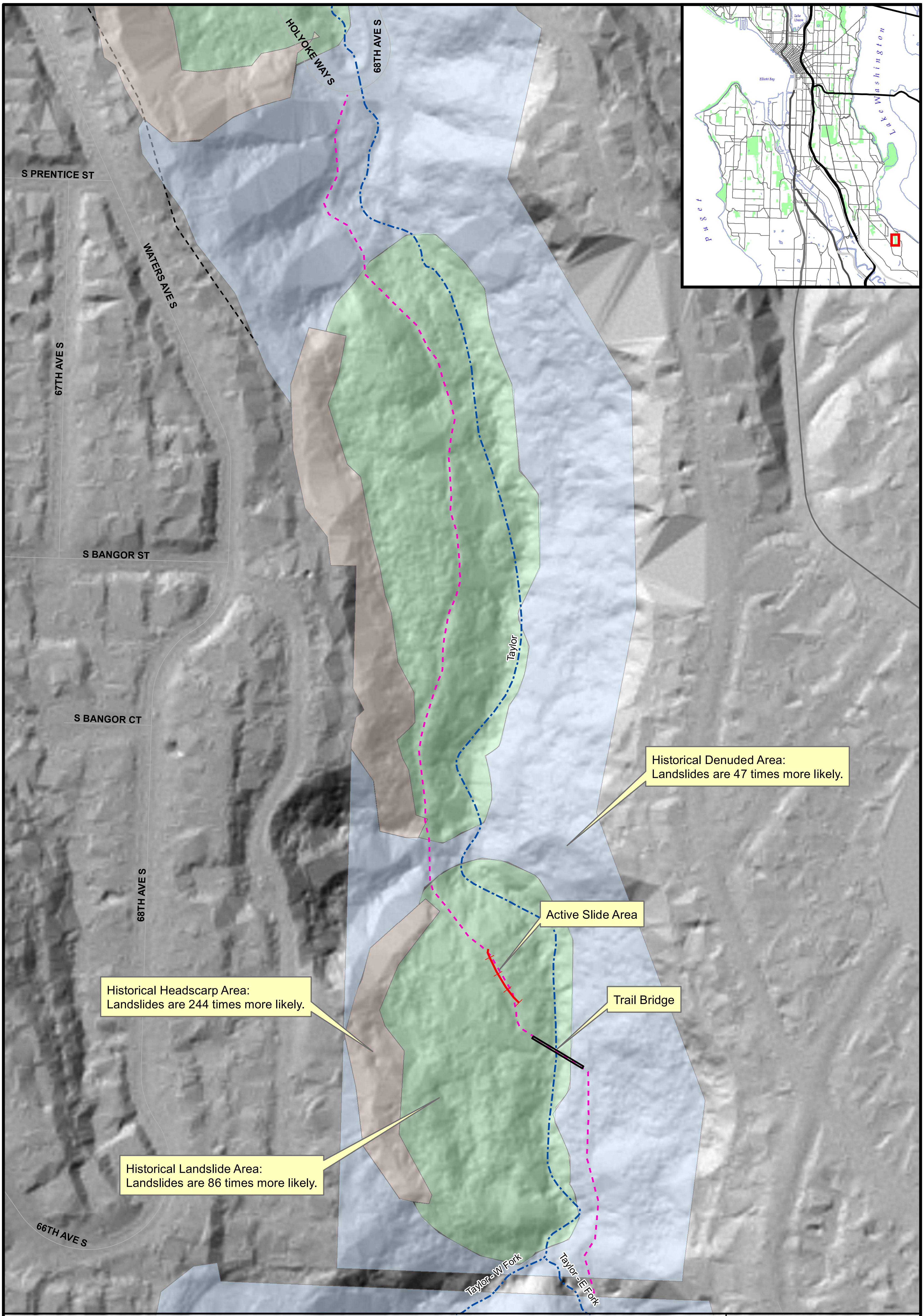

- Scarp - Active Slide
- Sewer and Trail Alignment
- Stream
- denuded
- headscarp
- landslide
- ● Slide Event Inventory
- Scarps

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Feet

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N W E S

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FIGURE 2

AUGUST 2015

Deadhorse Canyon Landslide Stabilization Alternatives E314059

Historical Landslide Map with Reoccurrence Potential

Seattle, Washington

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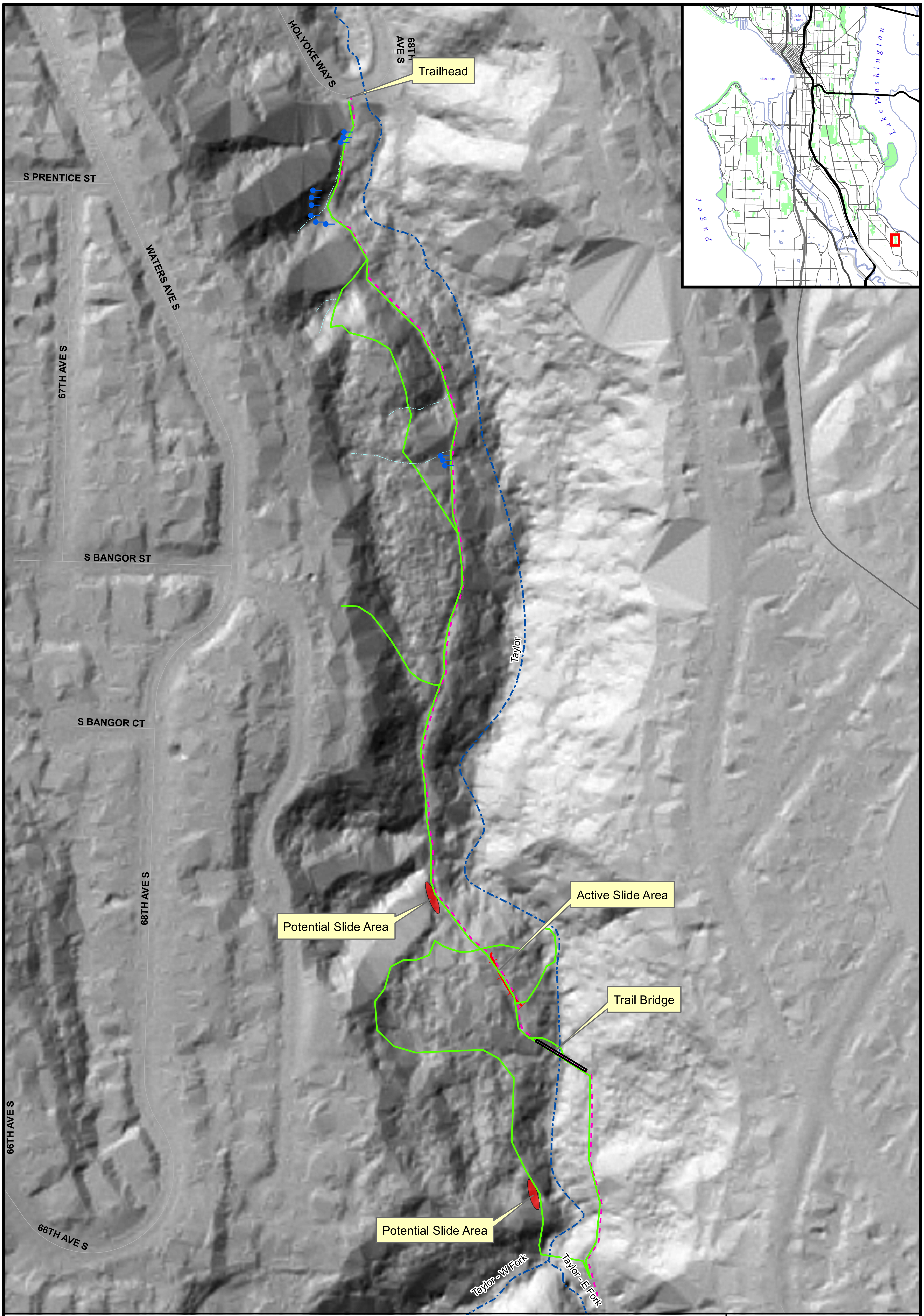

- ● Seep
- Intermittent Stream
- Reconnaissance Route
- Scarp - Active Slide
- Sewer and Trail Alignment
- 🟥 Observed Potential Active Slide Areas
- Stream

75 37.5 0 75 150
Feet

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Deadhorse Canyon Landslide Stabilization Alternatives E314059

Field Reconnaissance Observations

Seattle, Washington

FIGURE 3 **AUGUST 2015**

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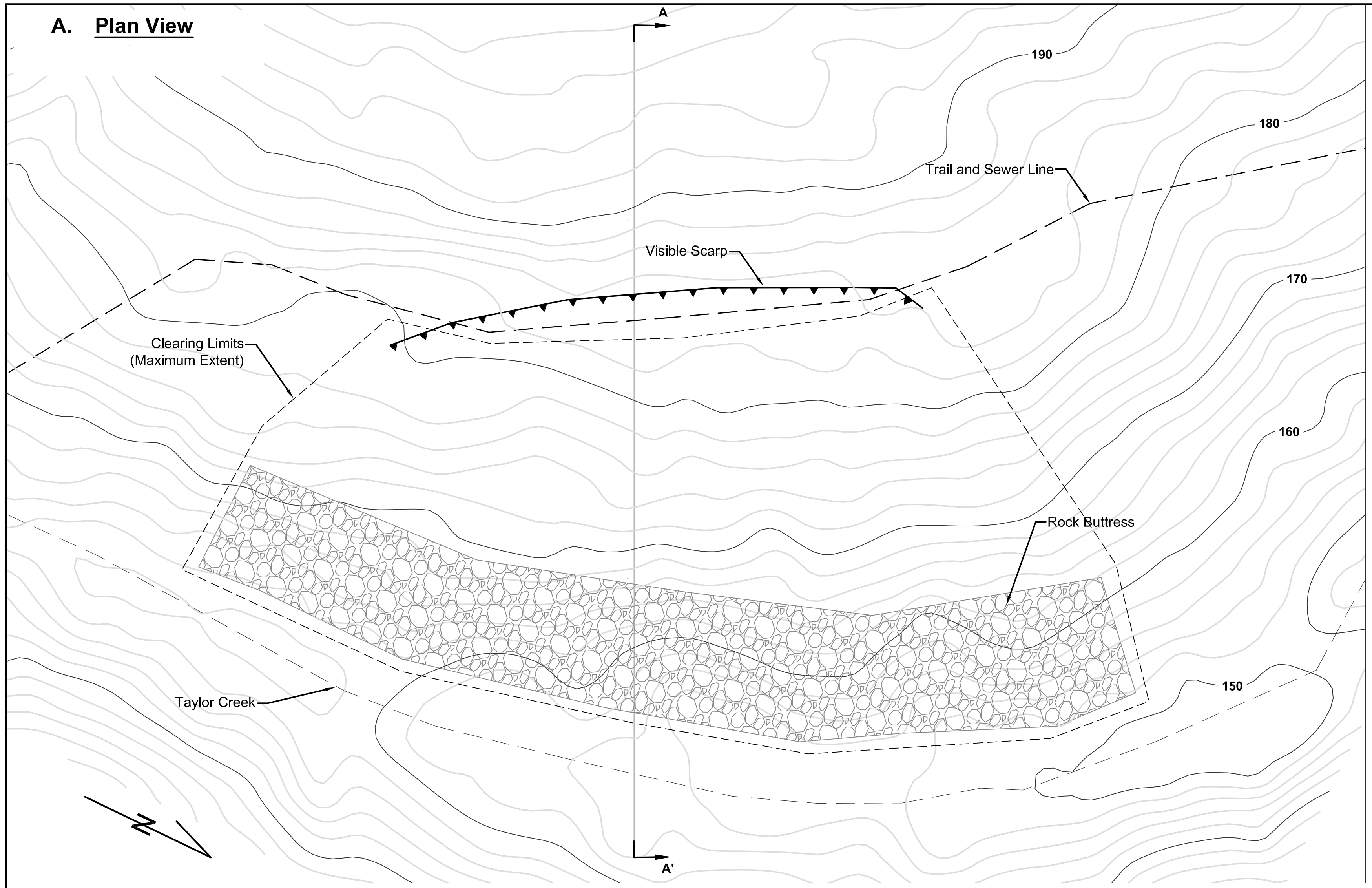
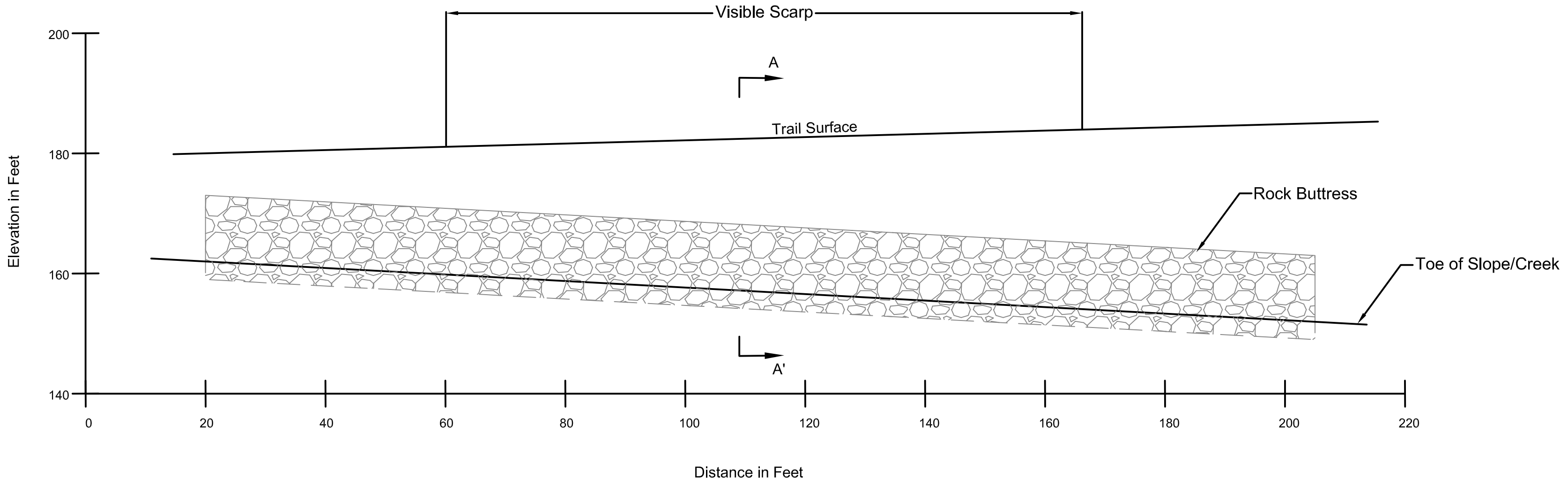
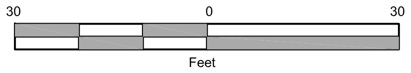
A. Plan View



Cross-Section A-A'

B. Elevation View





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DEADHORSE CANYON SEWER
SEATTLE, WA

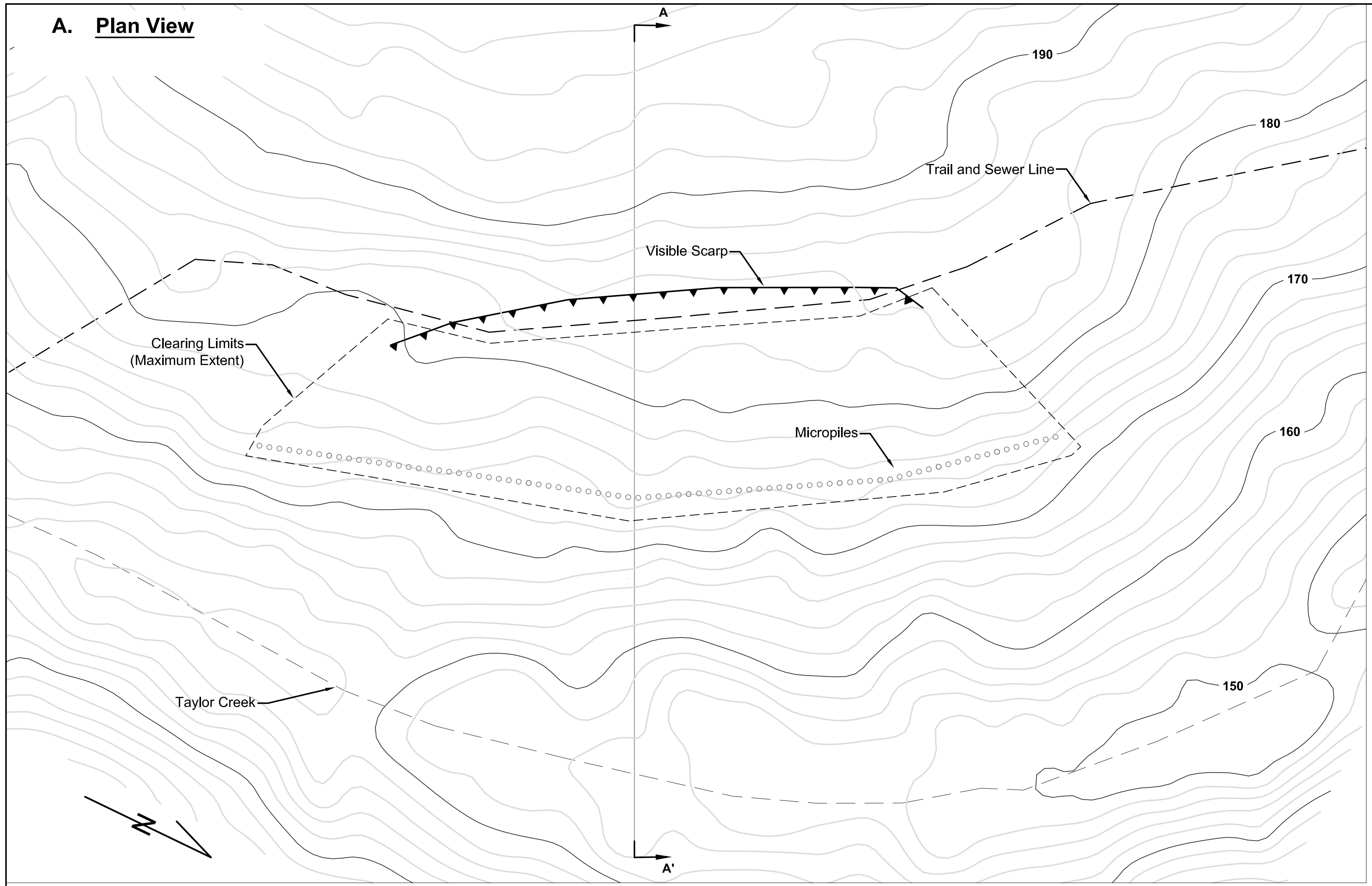
RIP RAP BUTTRESS SLOPE STABILIZATION
CONCEPTUAL PLAN, PROFILE, AND CROSS-SECTION

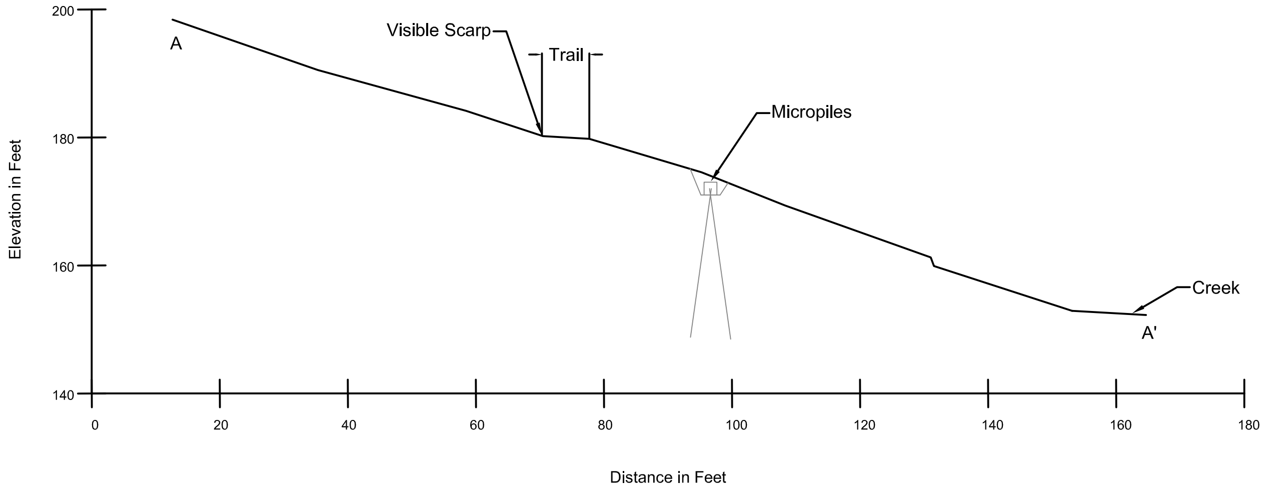
AUGUST 2015

FIGURE 4

MKH 07/30/15 Deadhorse Large Buttress.dwg

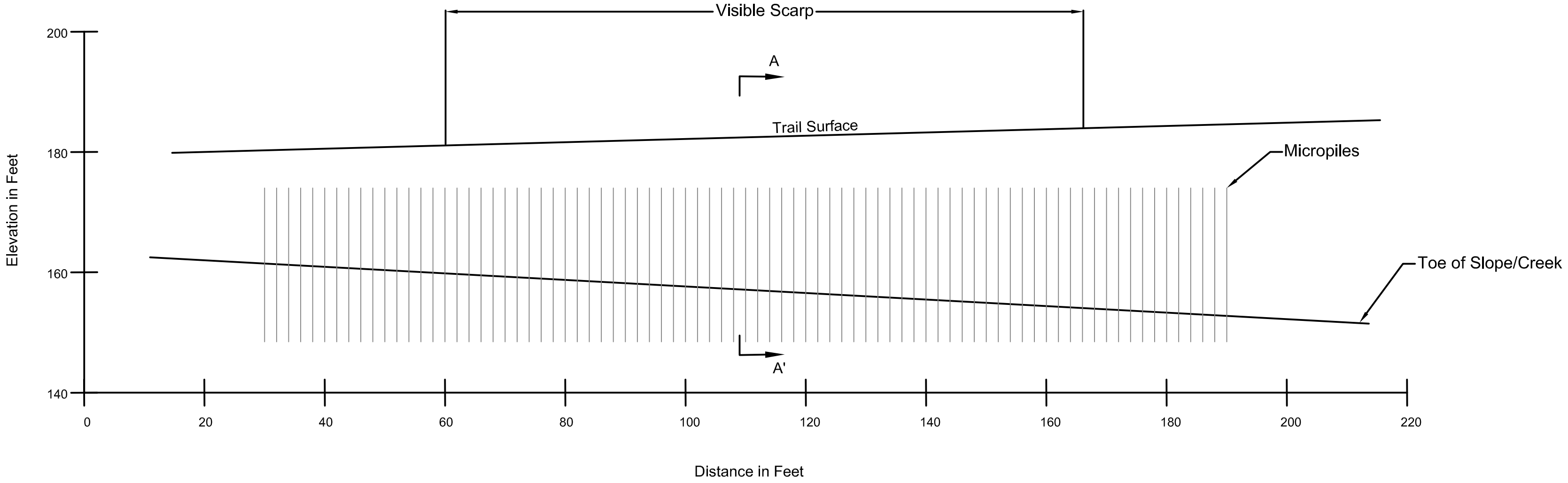
A. Plan View

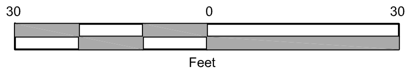




Cross-Section A-A'

B. Elevation View





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DEADHORSE CANYON SEWER
SEATTLE, WA

MICROPILE SLOPE REPAIR
CONCEPTUAL PLAN, PROFILE, AND CROSS-SECTION

AUGUST 2015 **FIGURE 5**

MKH 07/30/15 Deadhorse Large Buttress.dwg

APPENDIX A

ITEMIZED COST ESTIMATES

Static Design

Item	Estimated Quantity	Pay Unit	Unit Cost	Total Cost
Mobilization (15 %)	1	Lump Sum	$ 41,587.50	$ 41,588
Clearing and Grubbing (Access Route)	33,000	Square Foot	$ 1.00	$ 33,000
Clearing and Grubbing (Site)	12,500	Square Foot	$ 1.00	$ 12,500
Excavation (Access Route)	850	Cubic Yard	$ 50.00	$ 42,500
Excavation (Site)	700	Cubic Yard	$ 50.00	$ 35,000
Creek Diversion	1	Lump Sum	$ 12,000.00	$ 12,000
Rip Rap Placement	1730	Ton	$ 40.00	$ 69,200
Obliterate Access Route	0.30	Mile	$ 4,300.00	$ 1,290
Revegetating (Access Route)	0.75	Acre	$ 78,000.00	$ 58,500
Revegetating (Site)	0.17	Acre	$ 78,000.00	$ 13,260
				$ 305,578

Seismic Design

Item	Estimated Quantity	Pay Unit	Unit Cost	Total Cost
Mobilization (15 %)	1	Lump Sum	$ 44,737.50	$ 44,738
Clearing and Grubbing (Access Route)	33,000	Square Foot	$ 1.00	$ 33,000
Clearing and Grubbing (Site)	12,500	Square Foot	$ 1.00	$ 12,500
Excavation (Access Route)	850	Cubic Yard	$ 50.00	$ 42,500
Excavation (Site)	800	Cubic Yard	$ 50.00	$ 40,000
Creek Diversion	1	Lump Sum	$ 12,000.00	$ 12,000
Rip Rap Placement	2130	Ton	$ 40.00	$ 85,200
Obliterate Access Route	0.30	Mile	$ 4,300.00	$ 1,290
Revegetating (Access Route)	0.75	Acre	$ 78,000.00	$ 58,500
Revegetating (Site)	0.17	Acre	$ 78,000.00	$ 13,260
				$ 329,728

Notes:

1. Access route excavation quantitiy is based on typical excavation volumes for a 12 foot wide road without a ditch and with a 3/4:1 cutslope constructed across 40% sideslopes.
2. Access route obliteration includes re-contouring the area to its original condition.
3. Revegatation includes seeding, mulching and planting 1 tree or shrub every 100 square feet.

Static or Seismic Design

Item	Estimated Quantity	Pay Unit	Unit Cost	Total Cost
Mobilization (15 %)	1	Lump Sum	$ 47,037.00	$ 47,037
Clearing and Grubbing (Site)	5,000	Square Feet	$ 1.00	$ 5,000
Micropile Installation	1	Lump Sum	$ 300,000.00	$ 300,000
Revegetating (Site)	0.11	Acre	$ 78,000.00	$ 8,580
				$ 360,617

Notes:

1. Micropile installation cost is based on conversations with Tom Armour (DBM Contractors, Inc.). There is not a significant difference in cost for static and seismic design.
2. Revegatation includes seeding, mulching and planting 1 tree or shrub every 100 square feet.