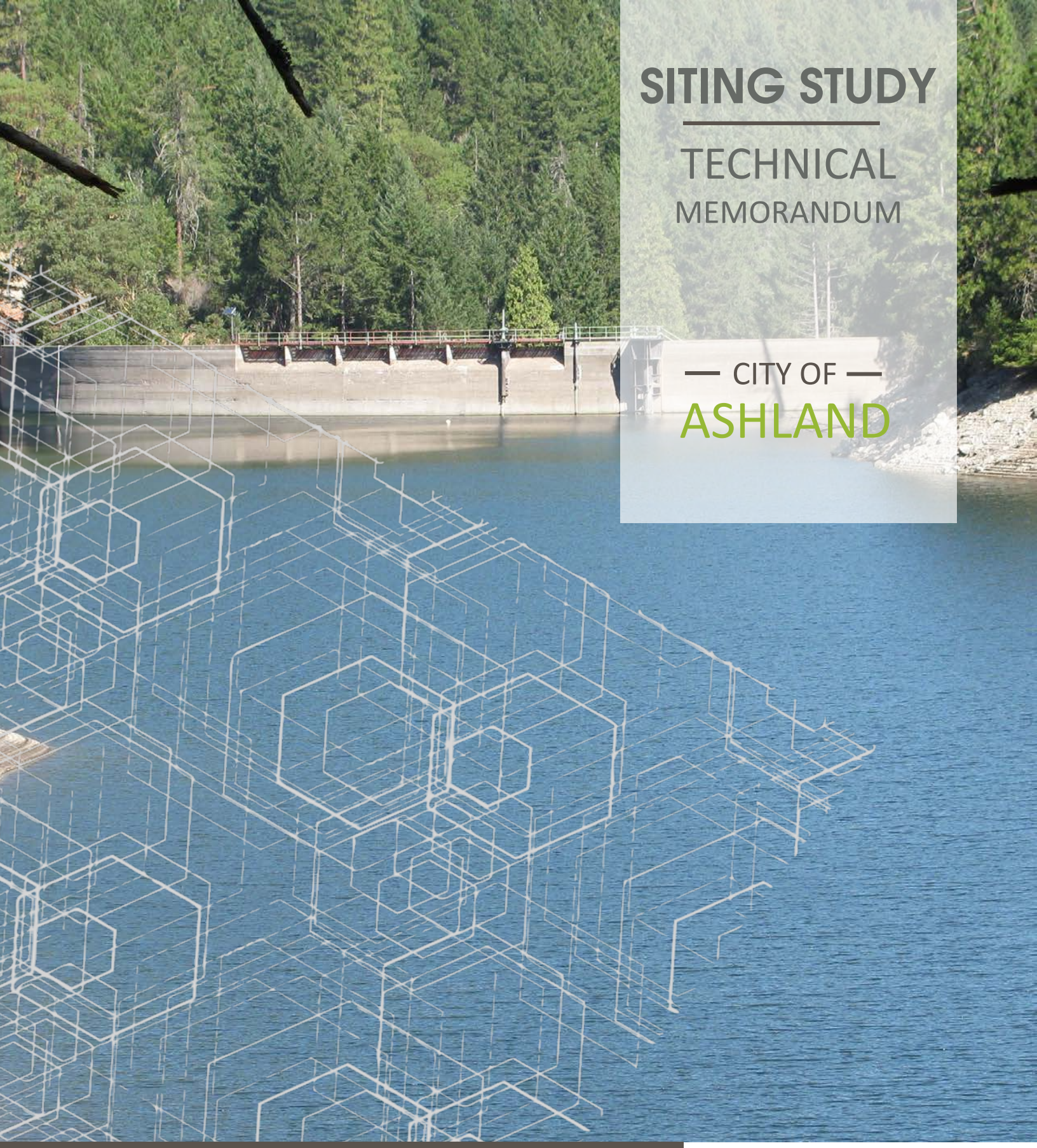

SITING STUDY

TECHNICAL MEMORANDUM

— CITY OF —

ASHLAND

WATER TREATMENT PLANT

SITING STUDY TECHNICAL MEMORANDUM

PROJECT NO. 217002

November | 2017

Get there!

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SITING STUDY

TECHNICAL MEMORANDUM

— CITY OF —

ASHLAND

REGISTERED PROFESSIONAL ENGINEER
45APE
OREGON
JULY 15, 2003
JAMES BLEDSOE

EXPIRATION DATE 12-31-2017

WATER TREATMENT PLANT

SITING STUDY TECHNICAL MEMORANDUM

PROJECT NO. 217002

November | 2017

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Technical Memorandum

TO: Kevin Caldwell
City of Ashland

FROM: James Bledsoe, P.E.
Jeremy Wilson, P.E.

DATE: November 21, 2017

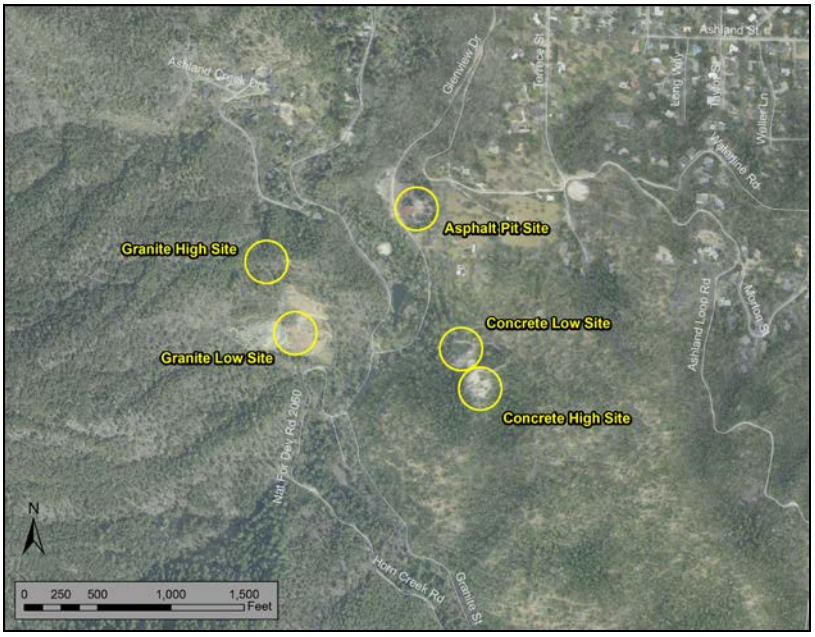
SUBJECT: Ashland Water Treatment Plant Siting Study

SECTION1: BACKGROUND AND OVERVIEW

The City of Ashland intends to construct a new water treatment plant (WTP) with an initial capacity of 2.5 million gallons per day (MGD), expandable to 10-12 MGD, and may construct a new water tank referred to as Crowson II near the same site. The improvements have been spurred by the City's desire to improve capacity, redundancy, and effluent water quality; and relocate the WTP in an area that is less susceptible to damage from periodic flooding, landslides, and wildfire.

A review of potential sites within the Ashland Creek Canyon (south of Ashland) identified five potential treatment sites (Figure 1). Two sites are located on the west side of Ashland Creek and are referred to as the Granite High Site and the Granite Low Site. Two additional sites, referred to as the Concrete Low Site and the Concrete High Site, are located to the east of Ashland Creek. A fifth location, the Asphalt Site, was also identified. All of the sites are on property owned by the City of Ashland.

Figure 1: Potential New WTP Sites



An evaluation of treatment technologies was completed concurrent to this siting evaluation. For this study, it was assumed that the WTP and reservoir will consist of a prestressed

circular concrete reservoir (Crowson II) approximately 3 MG in size, and a treatment plant with a pressurized hydraulic profile including flocculation tank, membrane microfiltration, a granular activated carbon (GAC) contactor, an equalization tank for waste collection, and chlorine disinfection within Crowson II. Site layouts were developed for this treatment process only. Impacts of a conventional treatment process on the site development would likely entail a slightly larger footprint, additional excavation required (for below grade process basins), and additional pumping due to the need to break hydraulic head for open basin filter processes.

The project area is generally characterized as a canyon, containing steep terrain and dense vegetation. Nature trails and canyon roads also run through the area, particularly on the east side of the creek. Existing infrastructure in the area includes various water distribution piping, sanitary sewer piping, underground power lines, nearby natural gas pipelines, underground communications conduits and the existing WTP.

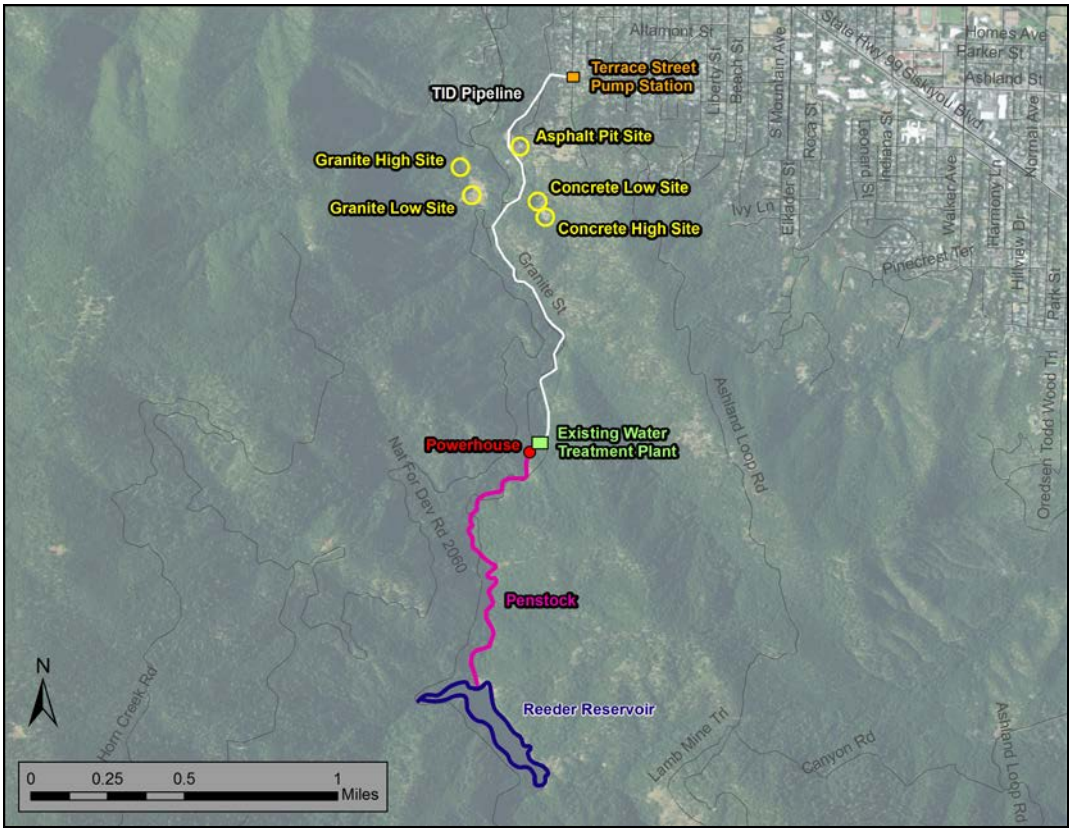
Many factors were considered in evaluating the suitability of each alternative site. These include: costs, geotechnical constraints, site access, flood concerns, offsite piping requirements, hydraulic/pumping requirements and associated power costs expandability, impacts to existing development, and environmental/sustainability impacts. This memorandum contains an evaluation of each of the proposed treatment plant location alternatives.

A preliminary site layout (presented later in this document) was developed for each location through an iterative process with City staff and will need further refining during the predesign phase. These layouts provided the basis for estimating cut and fill volumes used for comparing capital and environmental impacts associated with construction.

SECTION 2: HYDRAULIC CONSIDERATIONS

Raw water is supplied to the existing WTP from Reeder Reservoir on Ashland Creek, located approximately two miles south of the city (Figure 2). Under normal operations, flow is diverted at the reservoir's dam through a 24-inch pipeline, also known as the Penstock. The Penstock feeds the City's hydroelectric powerhouse with water at approximately 170 PSI pressure. The powerhouse is located adjacent to the existing WTP, located approximately one mile downstream of the dam. After flow passes through the powerhouse, it outfalls in the Tailrace (a basin at atmospheric pressure). A portion of the water is then diverted to feed the existing WTP, and all excess flow is returned to Ashland Creek. Raw water from the Talent Irrigation District's (TID) canal can also be used to supply the existing WTP from the Terrace Street Pump Station via a 24-inch pipeline known as the TID pipeline. TID water is only used to supply the WTP during low water conditions in Reeder Reservoir. The new WTP will also be supplied with raw water from the Reeder Reservoir and TID canal. During normal operation of the new WTP (raw water being supplied from Reeder Reservoir), the City's goal will be to gravity feed 60% of the time through the plant, up to Crowson II (if it is built), and over to Crowson I without supplemental pumping unless required by higher headloss conditions of peak flow or fouled process units. Gravity flow is possible with clean membranes and under certain flow conditions, and is estimated to be achievable 60% of the year based on seasonal flow variations.

Figure 2: Raw Water Supply to Existing WTP



As stated previously, the new WTP is planned to include membrane microfiltration (MF) and granular activated carbon (GAC). Membrane microfiltration treatment has a maximum limit of allowable feedwater pressure, which was assumed to be 65 pounds per square inch (psi) for this evaluation. (Note: This is an important factor for some of the sites being considered. All three membrane manufacturers piloted during the fall of 2017 have reported that they can meet or exceed this pressure criteria). Headloss throughout the entire plant will vary depending on frequency of cleanings and flow. Table 1 shows the anticipated headloss for clean, typical, and dirty membranes / GAC at various design flows, and are based on similar reported amounts for the Newport, Oregon plant. Headlosses will be further validated as part of the pilot testing activities currently underway.

Table 1: Expected WTP Headloss (10 MGD Plant Capacity)

	Headloss (ft) @ Various Membrane Conditions		
Flow (MGD)	Clean	Typical	Dirty
2.5	2	3	5
5.0	6	11	19
7.5	14	25	43
10.0	25	44	76

To convey raw reservoir water to the new WTP, a connection to either the Tailrace or Penstock (just upstream of the powerhouse) would be required (Figure 3). Water in the Penstock is pressurized from the Reeder Reservoir surface hundreds of feet above all potential sites being considered. Connecting to the Penstock would provide excess pressure, which would require reducing (with any of the sites) to meet the 65 psi pressure limit for membrane filters. A connection to the Tailrace would not exceed the 65 psi limit at any of the sites, with the exception of the Asphalt Site. The five sites being considered are located at various elevations and distances from the Tailrace of the powerhouse; available pressures and hydraulic grades (HGLs) at various design flows for each site are shown in Table 2.

Figure 3: Penstock/Tailrace Connection

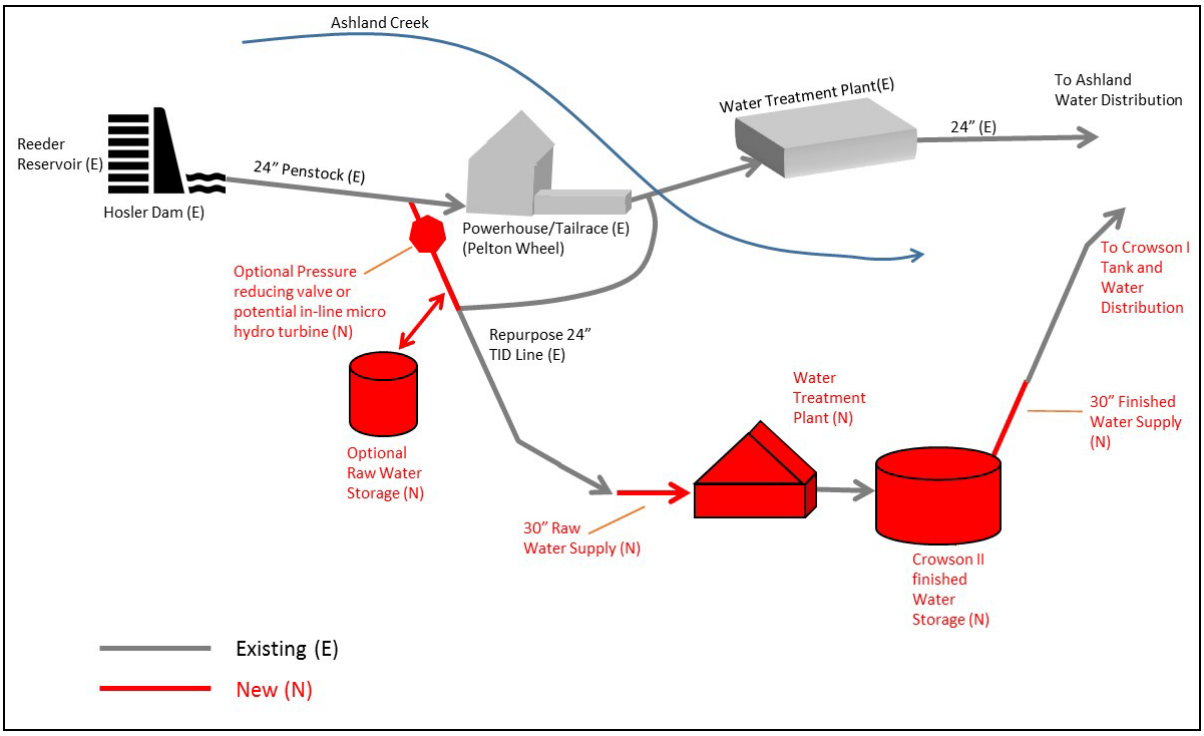


Table 2 – Pressure and HGL Entering New WTP with a Tailrace Connection

Site	Elev. (ft)	2.5 MGD		5 MGD		7.5 MGD[1]		10 MGD[1]	
		psi	HGL	psi	HGL	psi	HGL	psi	HGL
Concrete High	2,366.0	37	2,452.1	36	2,449.2	36	2,449.2	34	2,444.5
Concrete Low	2,280.0	74	2,452.1	72	2,447.2	73	2,448.6	72	2,446.3
Granite High	2,375.0	33	2,452.4	32	2,448.3	32	2,448.9	31	2,446.6
Granite Low	2,316.0	59	2,452.5	57	2,448.6	58	2,450.0	57	2,447.7
Asphalt	2,227.0	97	2,452.1	95	2,447.0	96	2,448.8	95	2,446.5

Assumptions:

1. Transition from a 24-30 inch water supply pipeline from the Trailrace to the new WTP site was assumed for flows of 7.5 and 10 MGD.

After leaving the new WTP, water will flow up to the Crowson II storage tank if it is built. The elevation change and distance between the new plant and Crowson II varies from site

to site, along with the required head for gravity flow. From Crowson II, water will flow across to the existing Crowson I storage tank. In order to obtain 10 MGD of gravity flow for this segment via a 30-inch pipe, Crowson II's overflow will need to be located at an elevation of 2,430 feet – approximately 5 feet above the existing overflow at Crowson I.

Concurrent to this siting study, the City is in the predesign process for upgrading its Terrace Street Pump Station. This upgrade will increase the pumping capacity of TID water to the new WTP. The City is also evaluating whether a portion of the TID pipeline can be repurposed to serve as the reservoir raw water supply to the new WTP until, eventually, the existing plant is abandoned. At that time, the existing 30-inch finished waterline could be repurposed as a raw water feed pipeline from the Tailrace to the new WTP.

2.1 Penstock Connection Power Implications

It should be noted that a connection to the Penstock pipeline would cause flow to the new WTP to bypass the powerhouse, resulting in a loss of power production. Although connecting to the Penstock may reduce or eliminate pumping at the new plant, the resulting costs saved would be insignificant compared to the loss of power production at the powerhouse. It is estimated that with an exclusive Penstock connection, approximately $116,000 of power production revenue would be lost on an annual basis (assuming a rate of $0.08/KW-hr). More detailed calculations are provided in Appendix A. Due to this net loss, along with the added complexities and risks associated with a Penstock connection, the City elected to focus solely on a Tailrace connection.

2.2 Gravity Flow with a Tailrace Connection

A hydraulic analysis was performed to identify potential gravity flow conditions for each site. It was assumed that the raw water supply pipeline from the Tailrace/Penstock to the new WTP will be 30 inches in diameter. Figures 4 to 8 illustrate the gravity flow possible at each site with a Penstock or Tailrace connection for various flows and plant conditions.

In general, approximately 7 MGD could flow at either the Concrete or Granite High Site under typical headloss conditions with a Tailrace connection. Therefore, assuming flows over 70% of the rated capacity and moderately clean membranes / GAC, gravity flow through the plant would be possible. With a Penstock connection, 10 MGD of gravity flow would be allowable under all headloss conditions for these two sites.

At the Concrete Low Site, only about 3 MGD would be allowed to gravity flow with either a Tailrace or Penstock connection (assuming typical WTP headlosses); this is due to the site's low elevation, which requires the pressure to be reduced before entering the plant to meet the maximum allowable pressures of treatment units.. At the Granite Low Site, approximately 7 MGD and 9 MGD of gravity flow under typical headloss conditions would be possible with a Tailrace and Penstock connection, respectively.

Gravity flow would not be possible at the Asphalt Site due to the site's low elevation, which requires the pressure to be reduced significantly before entering the plant to meet the maximum allowable pressures of treatment units.

Figure 4: Concrete High Site Gravity Flow Potential

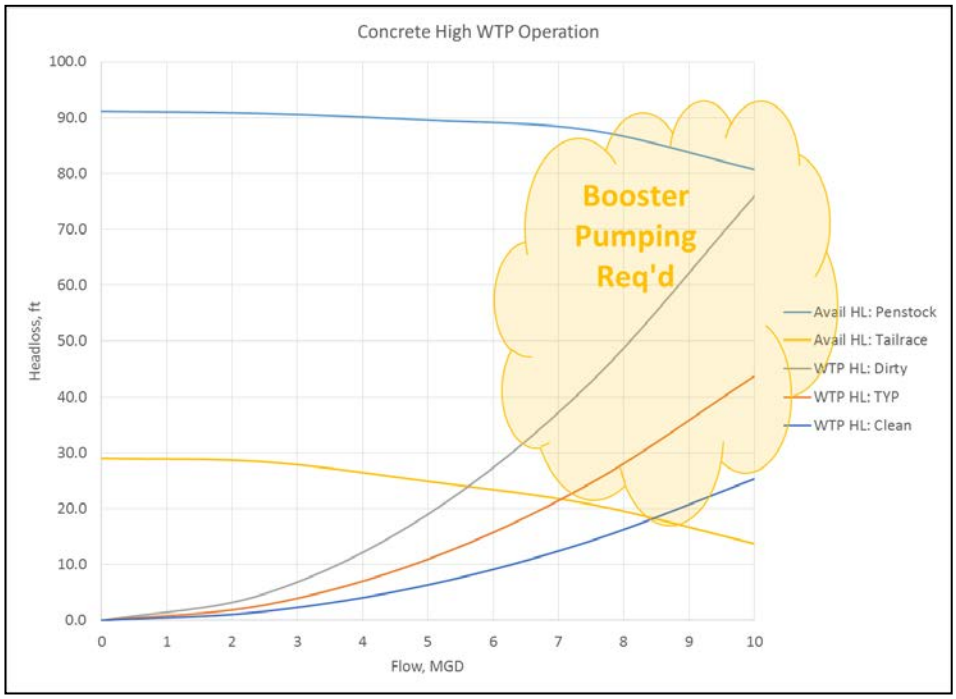


Figure 5: Concrete Low Site Gravity Flow Potential

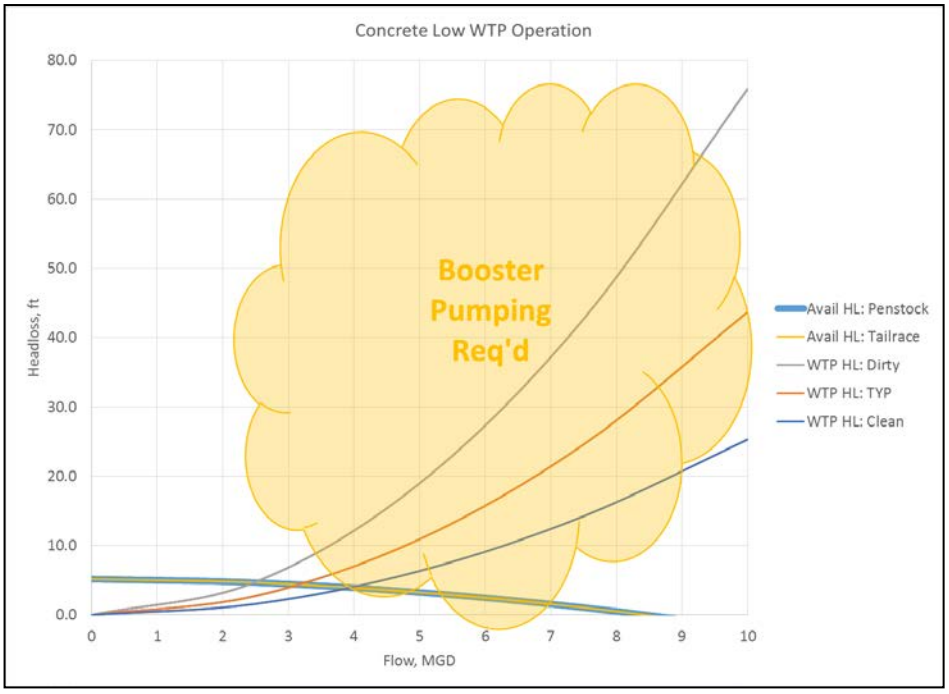


Figure 6: Granite High Site Gravity Flow Potential

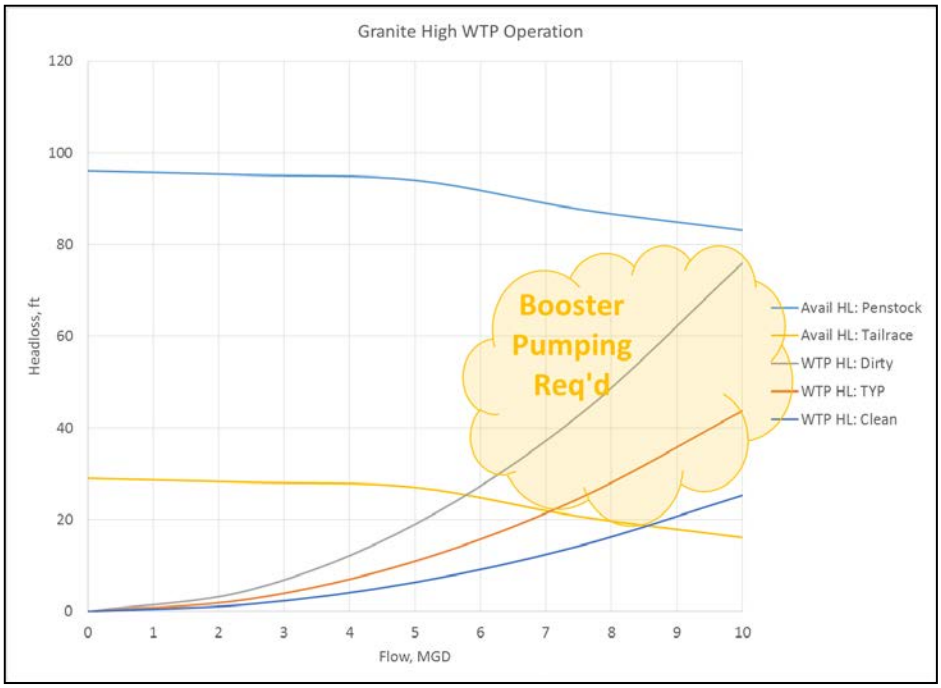


Figure 7 – Granite Low Site Gravity Flow Potential

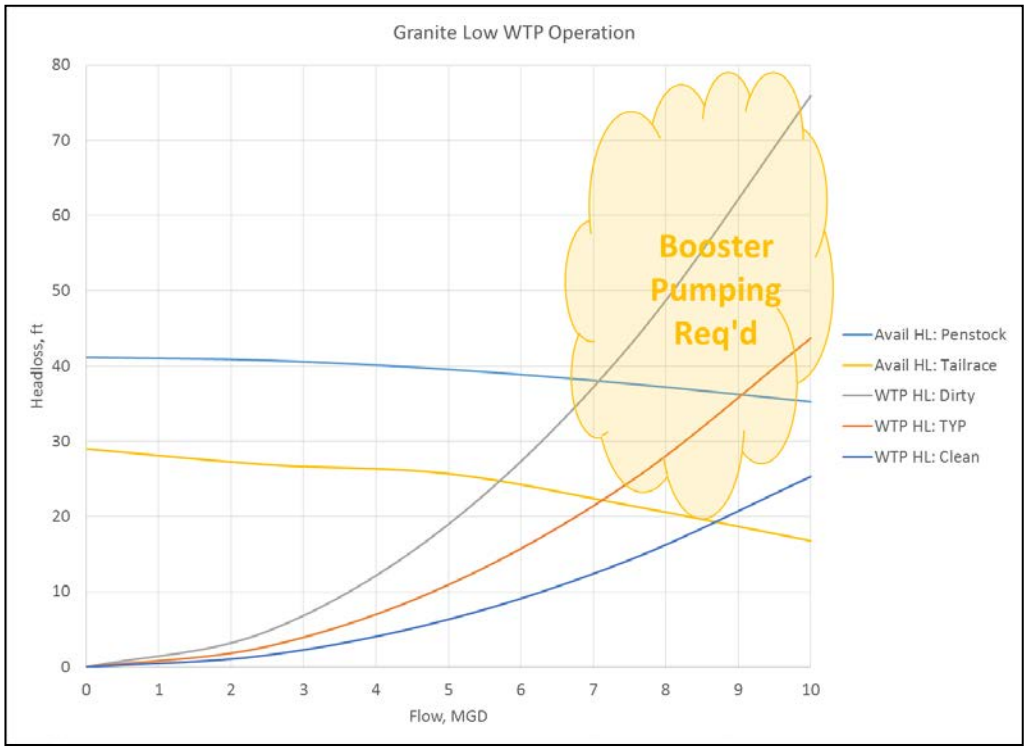
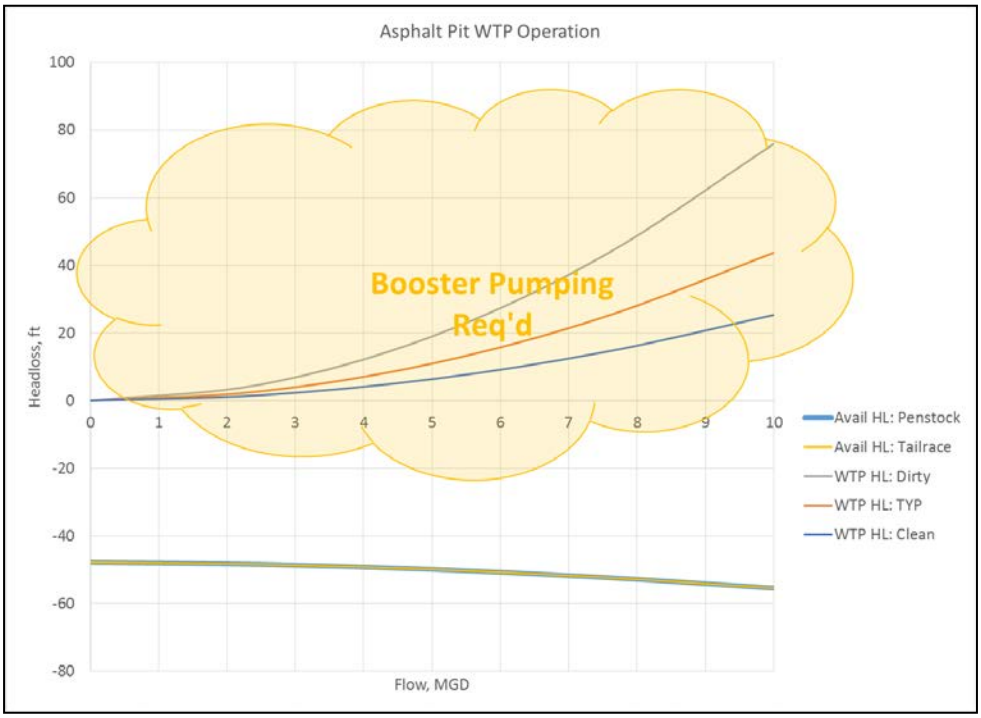


Figure 8: Asphalt Site Gravity Flow Potential



2.3 Dual Supply Connection from Tailrace and Penstock

A dual raw water connection to the Tailrace and the Penstock was also considered. Under this scenario, raw water from the Tailrace would be used to supply the new WTP at times when the membranes are clean and gravity flow to Crowson I and II is possible. Once the membranes at the new plant become dirty and gravity flow to Crowson I and II is not possible, raw water would be supplied from the Penstock. It was found under this scenario, that the power production loss from the Penstock connection was still much larger than the energy savings at the new WTP. A dual connection also would introduce complexities with controls, pressure protection, and regulatory requirements. A dual connection does not provide significant benefit compared to the risks and power production loss. For these reasons, a dual connection scenario was abandoned.

SECTION 3: OFFSITE PIPING AND UTILITY REQUIREMENTS

While each proposed site would use similar piping, both to and from the site, the required length will depend on the site chosen. The evaluation of offsite piping included a layout, associated lengths, and estimated costs for each site. The pipelines included are described below. Refer to Sections 6 and 8 for figures showing the proposed alignment of the offsite piping for the potential sites being considered.

Several utilities will be required at each plant site including: power (City of Ashland), natural gas (Avista) and communications (Century Link). Preliminary connection locations, and proposed utility paths have been identified based on input from each entity. The proposed utility pathing is shown in the figures contained in Appendix G.

3.1 Raw Water

The raw water supply to the plant will be delivered through the existing TID pipeline. The TID pipeline currently delivers raw irrigation water up Ashland Creek Canyon from the Terrace Street Pump Station to the existing WTP. Currently, it is planned to repurpose the existing TID pipeline as a raw water delivery pipeline for the new treatment plant. The TID pipeline will supply raw water to the new water treatment plant from: 1) The Tailrace to deliver Reeder Reservoir water under normal conditions and 2) to deliver TID water from the Terrace Street pump station when supplemental water may be necessary.. A new 30-inch pipe will be constructed between the new treatment plant and the TID pipeline on Granite Street. Ultimately, when the existing WTP is abandoned, the existing 30-inch treated water pipeline from the existing WTP can be repurposed as a raw water supply pipeline from the existing WTP. When this occurs, the new 30-inch pipeline would be connected to the existing 30-inch line and the existing 24-inch TID pipeline would be retained for backup purposes.

3.2 Finished Water

The finished water pipeline is intended to supply treated water from the new WTP, up to the new Crowson II tank (if constructed), then back to the existing 30-inch finished water line that feeds the Crowson I tank. To accomplish this, a 30-inch ductile iron pipe will be constructed from the new WTP to Crowson II, and then down to the existing 30-inch pipeline that runs along Ashland Creek. From the connection point near Ashland Creek, the existing 30-inch pipeline will convey finished water from the existing water treatment plant to Crowson I.

3.3 Sanitary Sewer

The new water treatment plant will require a connection to the existing sanitary sewer infrastructure. The anticipated contents of this pipeline are raw sewage from the treatment plant bathrooms, backwash water from the membranes, neutralized spent chemical cleaning solutions, backwash water from the exisitng filters, and water from the treatment plant floor drains.

There is an existing sanitary sewer pipeline that connects the existing water treatment plant to the rest of the system. From the existing WTP to the intersection of Granite Street and Horn Creek Road the sewer pipeline is 6-inches in diameter. From this intersection to the intersection of N Main Street and Water Street the pipeline is 8-inches in diameter, except for two portions where the pipeline necks back down to 6-inches. These locations are about 760 feet of 6-inch pipe near Ashland Creek Drive on Granite Street, and about 350 feet of 6-inch pipe in Lithia Park near Winburn Way and N Main St. Upsizing these portions of 6-inch sewer pipe will be investigated further in the next phase of design. As part of this investigation, coordinating backwashing of both the new and existing WTPs will be evaluated to determine if upsizing can be avoided. The exact flows in the existing sewer pipeline from the existing WTP are currently unknown. However, the only connection to the pipeline is the existing Water Treatment Plant. Due to the proposed higher recovery water treatment technology being installed at the new treatment plant, the total flow

volume being introduced to the pipeline is anticipated to be less than flows from the existing WTP for the same total plant output.

Membrane backwashing can be programmed to occur in a sequential, ongoing manner to minimize peak discharges to the sewer. However, equalization storage for backwash wasting is recommended for any treatment technology, particularly the GAC units that will backwash at substantial flow rates. Equalization allows for controlled release to the sewer system. This helps mitigate potential downstream impacts and accommodate off peak discharges to the sewer system. Equalization storage would also provide operating flexibility for coordinating releases with those of the existing WTP.

3.4 Tank Overflow

Crowson II will have an emergency overflow pipeline installed. It is anticipated that this will be a 16-inch diameter pipeline that will run from Crowson II and discharge into Ashland creek. This pipeline will only be used in case of an emergency to prevent pressurizing the tank.

3.5 Stormwater

At this time, it is anticipated that all stormwater will be detained to pre-development peak flows, treated with appropriate best management practices, and discharged into existing drainages.

SECTION 4: CROWSON II 2006 SITING STUDY

A separate Siting Study for the Crowson II tank was completed in 2006 by Brown and Caldwell. The Study proposed a site for Crowson II, evaluated three proposed sites for the Ashland Loop Road Reservoir, evaluated several tank types, laid out the required permitting process, and provided cost estimates and proposed schedule.

SECTION 5: GEOTECHNICAL CONSIDERATIONS

A preliminary geotechnical evaluation has been performed for all five sites by Applied Geotechnical Engineering and Geologic Consulting. This evaluation (included in Appendix B) includes a review of the available geotechnical and geologic information for the vicinity of the sites, a ground-level reconnaissance of the sites, and engineering analysis. After a specific site is chosen, a site specific exploration will be performed with associated field work and engineering analysis. A specific summary of the findings is included with each site in Sections 6 through 8 of this document.

SECTION 6: GRANITE SITE ALTERNATIVES

The granite sites are located approximately 1.25 miles up Ashland Creek canyon as measured from Downtown Ashland. The sites are currently accessible by driving up Granite Street, or Glenview Drive to Horn Creek Road. The upper portion is generally covered in shallow dense vegetation, with moderately dense deciduous trees, evergreen trees, scattered brush, and thick ground cover. The lower portion is generally open, and has been

previously cleared. A moderate amount of undocumented fill has been used to level a portion of the sites. There is a small depression in the northern half of the location that can collect rainwater during storm events.

FEMA panels have been acquired to evaluate flood potential near the granite sites. The granite sites are mapped as “Zone X.” Zone X is defined as areas deemed to be outside of the 0.2% floodplain (See Appendix C). However, based on information from the City, Ashland Creek periodically floods and overtops Horn Creek Road where it crosses Ashland Creek. During such flood events, access to the treatment plant would be restricted to foot traffic, and would require passing through private land, unless roadway/crossing improvements were completed at the Ashland Creek crossing location.

6.1 Remediation Requirements

The City will be completing an environmental assessment of the granite site concurrent to this report being created. There are no known environmental concerns for the proposed site.

6.2 Geotechnical Considerations

A preliminary geotechnical evaluation has been performed for all five sites. The evaluation was completed by Applied Geotechnical Engineering and Geologic Consulting and is included in Appendix B.

The Granite High Site is located on the upslope of the granite site, in an area that appears to have had no significant site grading. It is anticipated that competent rock will be encountered at a depth between 10 and 20 feet below the existing native ground. Weathered rock may be encountered above that. It is anticipated that weathered rock could be excavated using conventional excavation methods (excavator with a rock bucket), while competent rock would need to be removed with other means such as blasting.

The Granite Low Site is located in the bowl located in the center of the granite site. However, the proposed development is over an area that has been used as a borrow area. If this location were to be developed, the material would have to be removed. The existing material is believed to be weathered granite. Since the proposed treatment plant location on this site has been filled in, it anticipated that less rock excavation would be required at this location.

6.3 Site Access, Existing Infrastructure

Access to the granite sites is obtained by taking Granite Street to Horn Creek Road, and is generally good. However, during extreme flood events, water has historically overtopped Horn Creek Road at the Ashland Creek crossing. This would limit access to the plant until the flood condition subsides. A proposed solution is to construct a new crossing over Ashland Creek to provide improved access and to mitigate washing out at the culvert during large flood events. During these flood events, access on Granite Street below the crossing would not be possible. However, Glenview Drive would still provide access to the crossing, and the treatment plant.

6.4 Granite High Site

The Granite High Site has the facility located on a knoll with an existing ground elevation of approximately 2,383 feet. Figure 9 shows a conceptual site layout of the Water Treatment Plant at this location. This alternative would require shaving off the top of the knoll to an elevation of approximately 2,360 feet to provide sufficient footprint for the treatment plant. Consequently, this site requires a significant amount of rock excavation in order to obtain the minimum recommended 10-foot setback from the slope edge, and to maintain desired embankment slopes.

Offsite Piping Requirements

The general layout of the anticipated offsite piping for the Granite High Site is shown in Figure 10. A description of the intended purpose of each pipe is included previously in this report.

Both granite sites will require crossing the creek with the 30-inch raw water pipeline and the 8-inch sanitary sewer pipe.

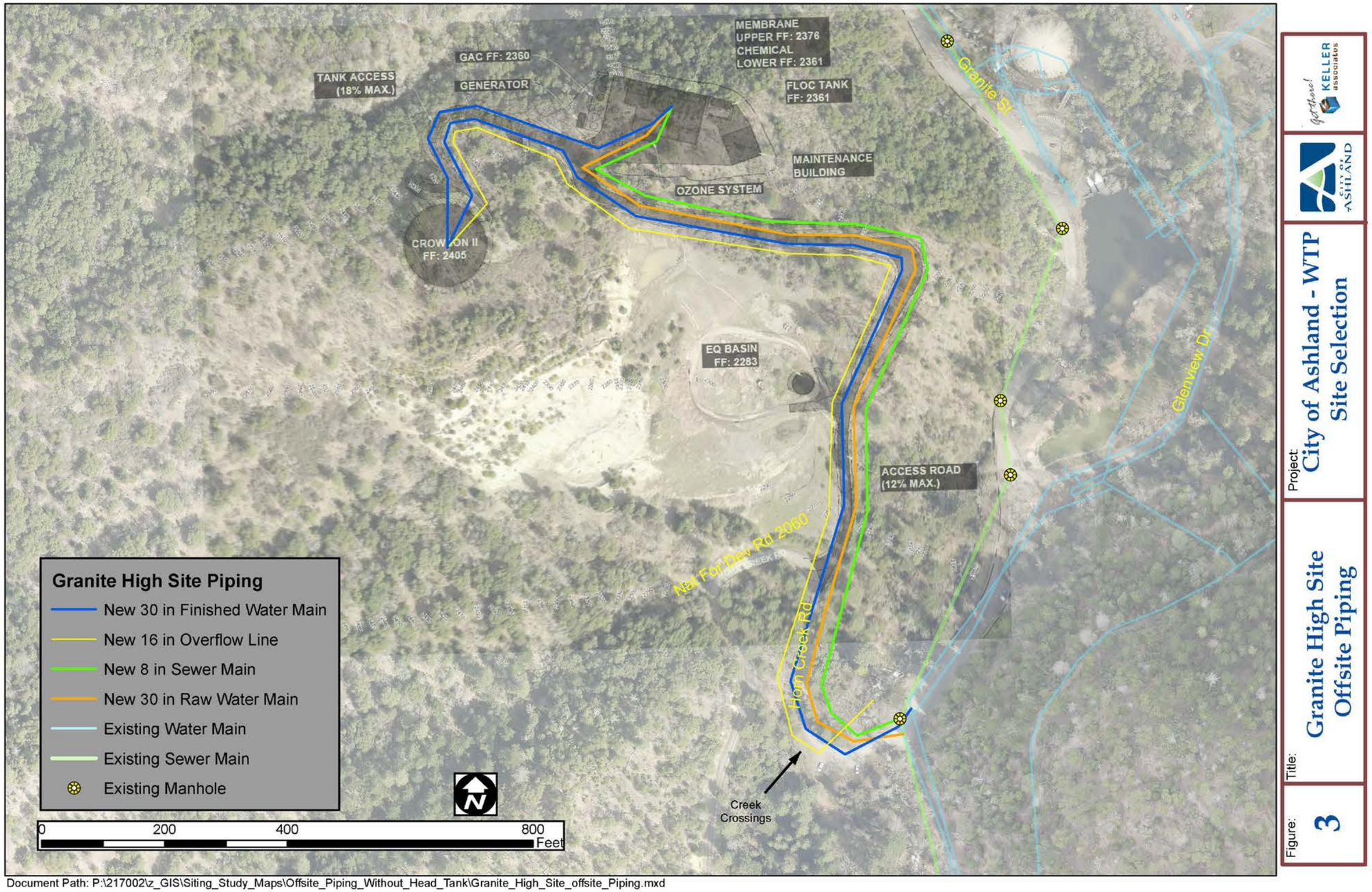
Advantages and Disadvantages

The Granite High Site, similar to the Granite Low Site offers power savings and better solar energy production potential when compared to the concrete sites. However, it has the second highest estimated site development capital cost, and limited expandability due to its location. Since this site is located on top of a knoll, it also has the potential to be one of the most visible locations considered.

Currently access to both granite sites would be cut off during a flood event. To mitigate that concern, it has been proposed to include replacing the culvert where Horn Creek Road crosses Ashland Creek.

Figure 9: Granite High Site Layout

Figure 10: Granite High Site Offsite Piping



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For the Granite Low Site, the facility is located in the bowl just off Horn Creek Road. Figure 11 shows the conceptual site layout of the plant at this location. The majority of the ground at the Granite Low Site has been cleared, and undocumented fill has been used to level the site. This should make excavation much easier in this area, since it is anticipated that much of the rock has already been removed.

However, as mentioned previously, the undocumented fill in this location would have to be completely removed. If it is to be reused, specific processing procedures would have to be followed. Some rock excavation is still anticipated at the plant site, and would likely be required for the access road to Crowson II and for the Crowson II tank itself.

Offsite Piping Requirements

The general layout of the anticipated offsite piping for the Granite Low Site is shown in Figure 12, and is similar to the Granite High Site.

Advantages and Disadvantages

Overall the Granite Low Site is very promising. A water treatment plant in this area would have very good potential for expandability, and high facility configuration flexibility

A lower amount of rock removal is anticipated at this site; largely due to the plant footprint being proposed on an area of fill. This leds to a lower capital cost for the Granite Low Site. The Granite Low Site also has the highest potential for solar power use. The footprint of the proposed plant is in an area that has already been mostly cleared of trees.

The site would provide similar gravity flow capabilities from the tailrace as those afforded the Granite High and Concrete High Sites.

Figure 11: Granite Low Site Layout

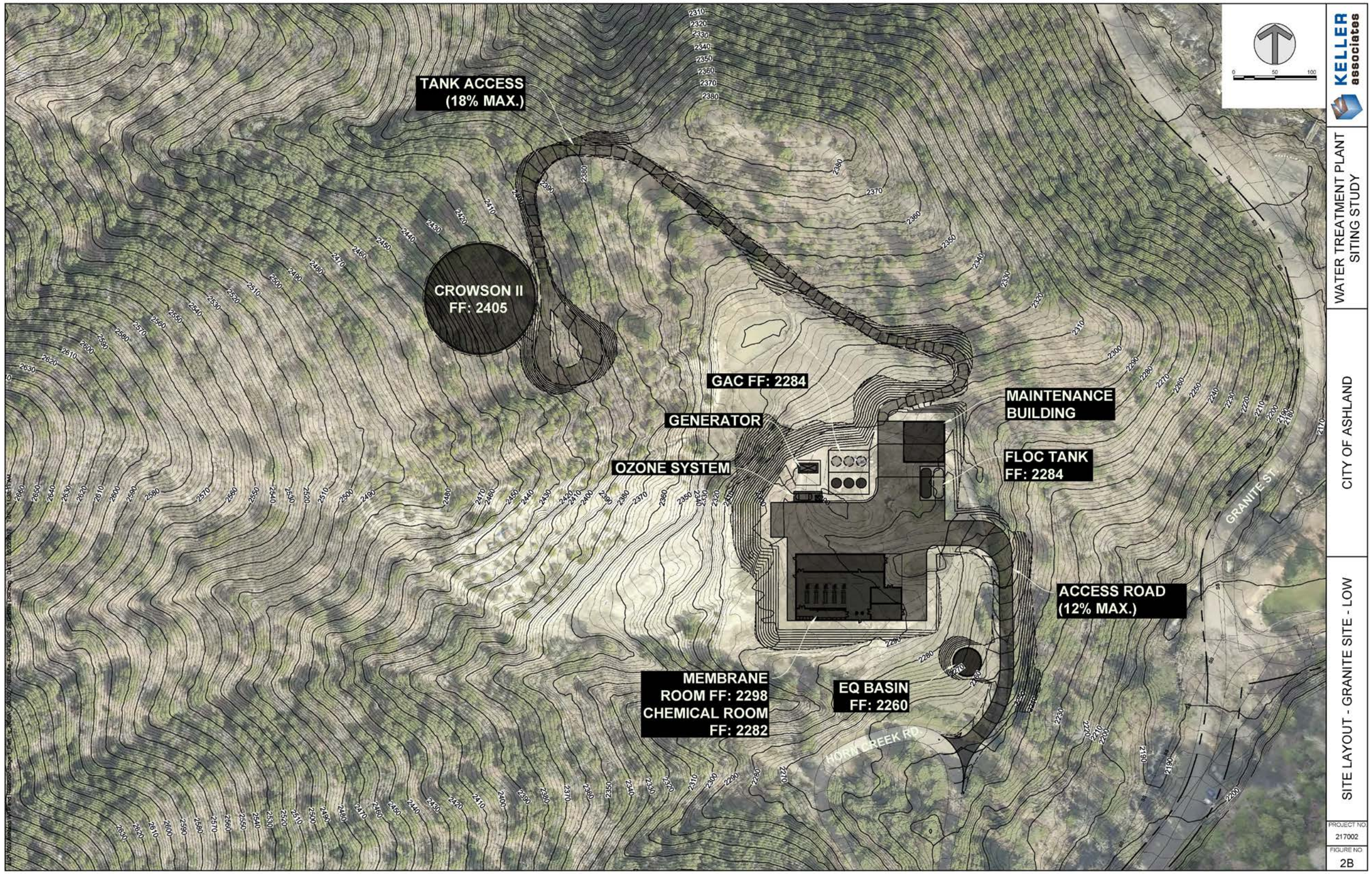
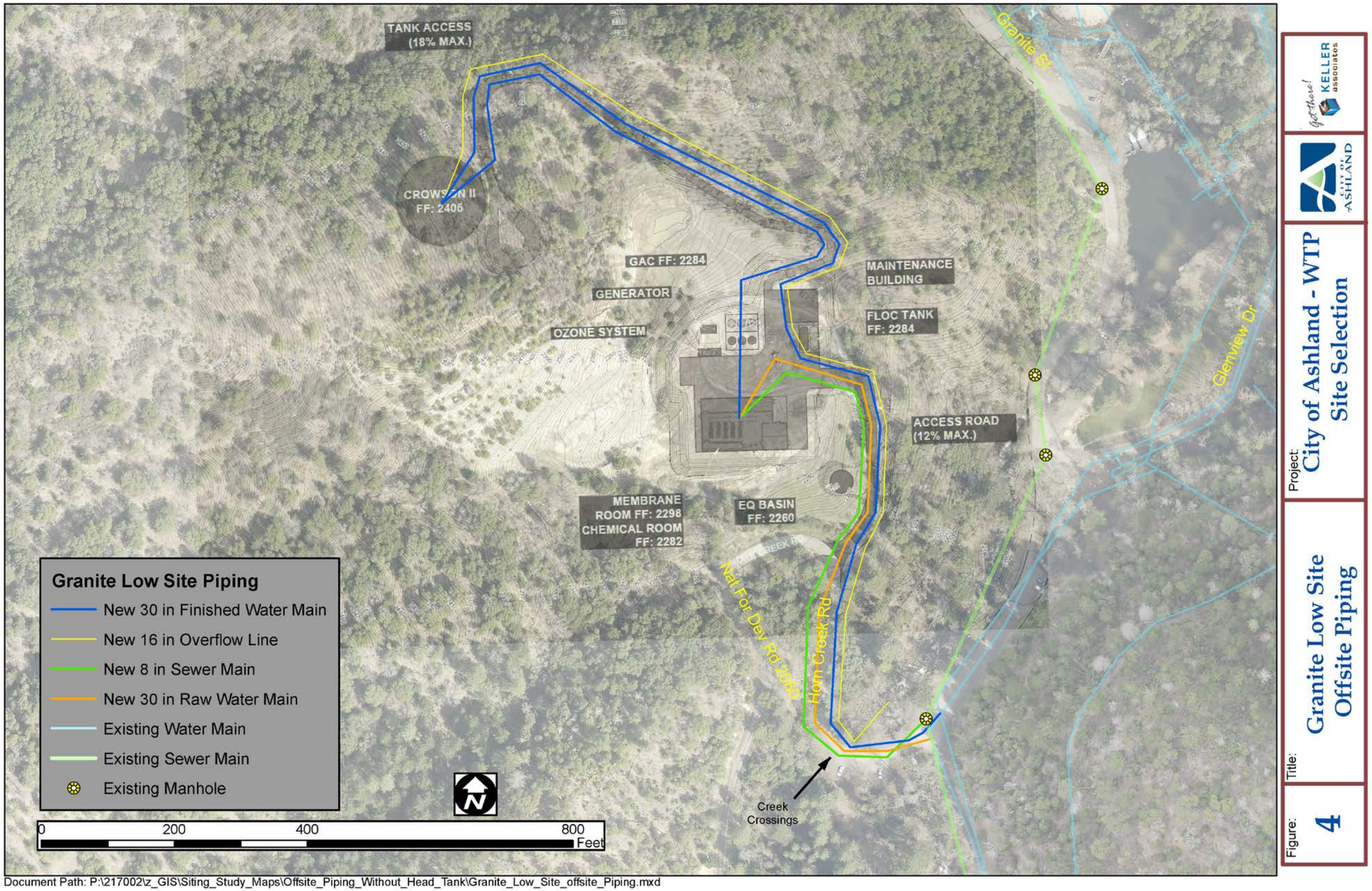


Figure 12: Granite Low Site Off Site Piping



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SECTION 7: CONCRETE SITE ALTERNATIVES

The concrete sites are located approximately 1.25 miles up Ashland Creek Canyon as measured from Downtown Ashland. The sites are located on the eastern side of the canyon nearly directly across Ashland Creek from the granite sites. The sites are currently accessible by driving on Glenview Drive and turning left onto an existing gravel access that is approximately 0.25 miles in length. The sites are generally covered in shallow dense vegetation, and are surrounded by moderately dense deciduous trees, evergreen trees, scattered brush, and thick ground cover. The topography on this site is steep, with limited flat space available for construction.

There is a system of public trails in the area. The recreational trails would be impacted by a development at either concrete site. However, it would be possible to maintain access and reroute the trails if desired.

FEMA panels have been acquired to evaluate flood potential at the concrete sites. The concrete sites are mapped as “Zone X.” Zone X is defined as areas deemed to be outside of the 0.2% floodplain. However, based on information from the City, Ashland Creek periodically floods, which would restrict access from Glenview Drive from the south (via Granite Street). However, these sites do have an alternative vehicle access route from the north.

One advantage of the concrete site is that the site could be developed without any apparent short-term impacts to wetlands or waterways.

7.1 Remediation Requirements

A categorical exclusion was previously provided for the site which allowed the City to move forward with construction activities at the site. However, subsequent concerns about potential contaminants that resulted from previous disposal of debris warranted additional investigations completed by the City, and consultant Alpine Environmental

On July 5th, 2017 Oregon Department of Environmental Quality issued a memorandum after exploring potential contamination of hydrocarbons at the site. This report is included in Appendix D. Soil and groundwater samples were collected at the site for testing. The concentrations of hydrocarbons in the samples were well below levels that would pose a risk to human health or the environment.

7.2 Geotechnical Considerations

A preliminary geotechnical evaluation has been performed for all five sites. The evaluation was completed by Applied Geotechnical Engineering and Geologic Consulting and is included in Appendix B. It is anticipated that competent rock will be encountered at a depth of approximately 30 feet below the existing native ground.

The Concrete High Site is located on the upper bench of a quarry previously used by the City of Ashland as a borrow area. Since significant cuts into the hillside are expected at this site, a large amount of rock excavation is expected.

The Concrete Low Site is located on the lower bench of the concrete site. The lower site has been used as a dumping area and has a significant amount of primarily concrete debris that would need to be removed and disposed of prior to developing the site.

7.3 Site Access, Existing Infrastructure

Access to the concrete sites is via Glenview Drive. However, due to steep roadway grades and tight turning radiuses on Glenview, the truck traffic to the concrete site would take Granite Street to Glenview Drive. Access improvements to the entry and access road would be required to accommodate truck traffic to the site. As mentioned previously, the concrete sites have an alternate access route during flood events.

7.4 Concrete High Site

There are two relatively level areas at the concrete sites that would be suitable for constructing the new treatment plant. The Concrete High Site is located on the upper area. For this site, the ground surface would be leveled to an elevation of approximately 2,350 feet to provide adequate space for the facility. A conceptual layout of this site is shown in Figure 13.

Offsite Piping Requirements

The general layout of the anticipated offsite piping for the Concrete High Site is shown in Figure 14. A description of the intended purpose of each pipe is included previously in this report.

Since both concrete sites are located on the eastern side of the canyon, it is not necessary to cross Ashland Creek to connect the 8-inch sanitary sewer and 30-inch finished water pipelines. Additionally, the overflow pipe from Crowson II should be able to shore the final reach of overflow pipe from Crowson I reservoir, minimizing impacts of construction near Ashland Creek.

Advantages and Disadvantages

The Concrete High Site would offer better accessibility during flood times, and has one of the lower overall site development costs. Also, this site is one of three that could utilize gravity flow from the Tailrace during much of the year.

However, plant expandability at this location would be limited because the site is confined by a steep sloping terrain. Limited space would also result in a less favorable site layout (i.e. more challenging delivery access, and the maintenance shop would be located at the lower site.). The site is less conducive to accommodating solar power.

Figure 13: Concrete High Site Layout

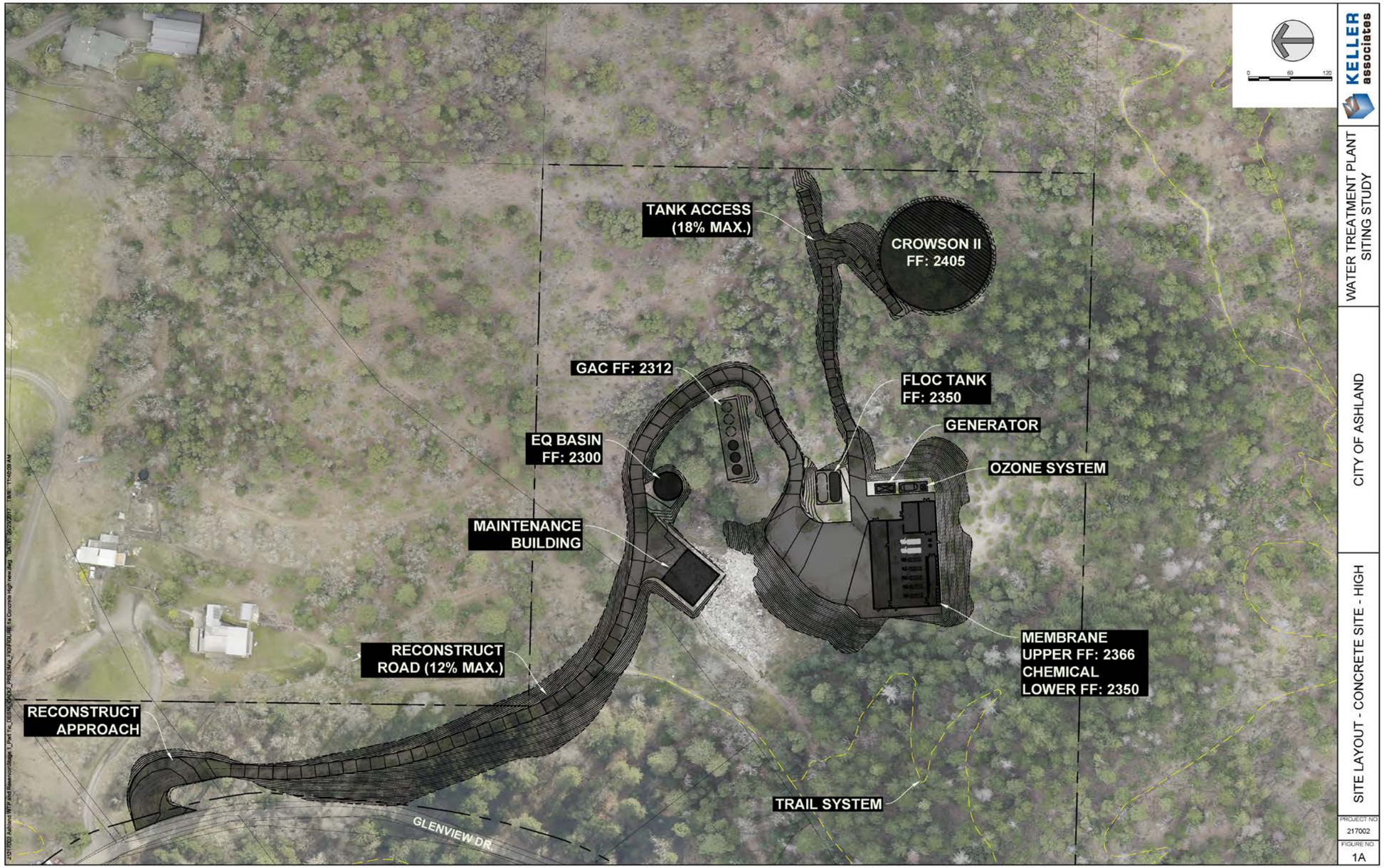
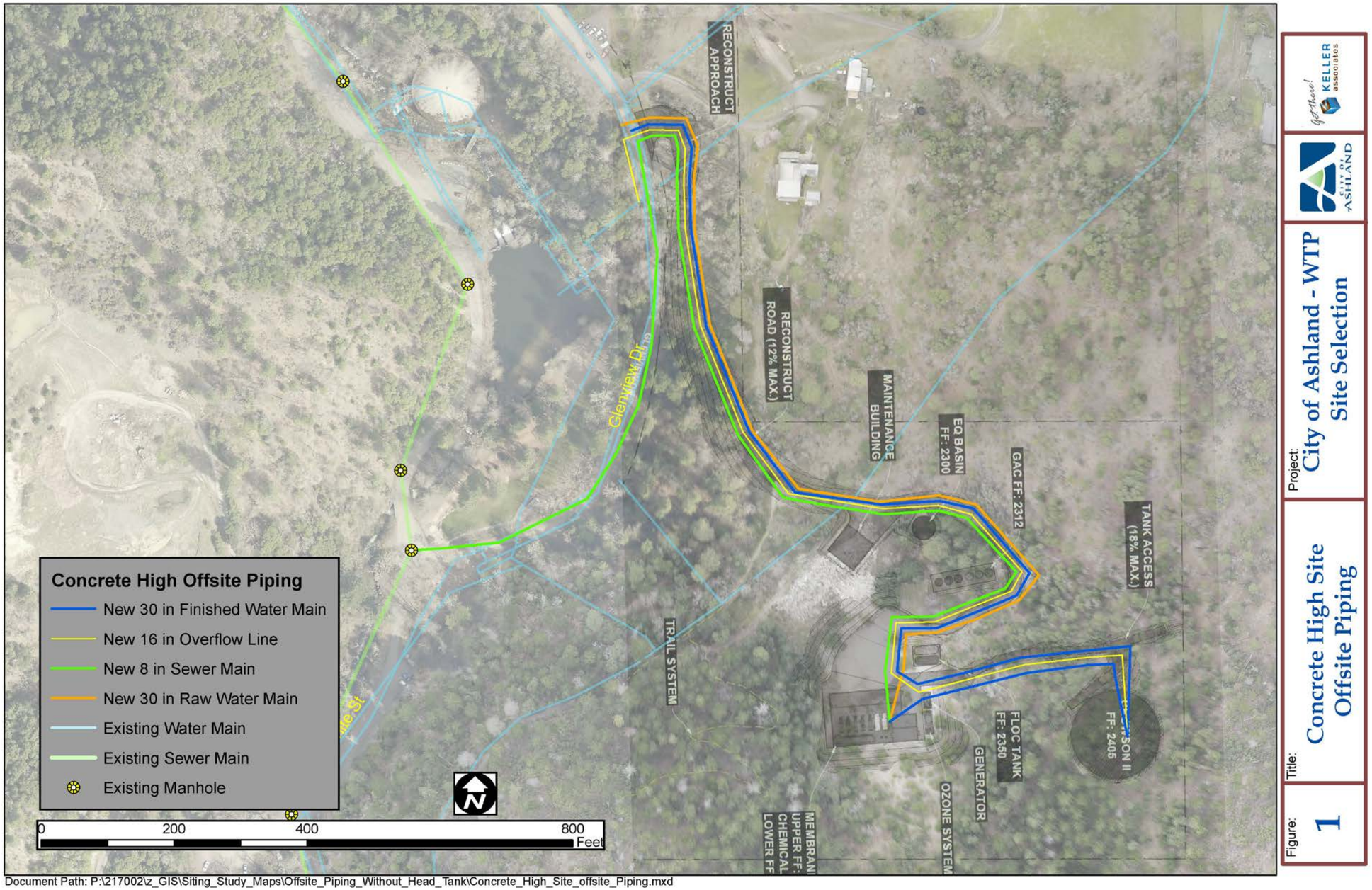


Figure 14: Concrete High Site Off Site Piping



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7.5 Concrete Low Site

The Concrete Low Site is located on the lower level of the concrete sites. A proposed layout of the Concrete Low Site is illustrated in Figure 15. The existing ground at this location has a large amount of debris that has been dumped here over the years. If this location is chosen, all of this material would have to be removed and disposed of. Generally, the existing ground at the proposed facility location ranges from 2,265 to 2,295 feet.

Significant earthwork would be required at this location to provide an adequate footprint for the facility. Concerns about potential contamination of the site were investigated, and findings suggest that environmental remediation would not be required (See Appendix D for report).

Offsite Piping Requirements

The general layout of the anticipated offsite piping for the Concrete Low Site is shown in Figure 16.

Advantages and Disadvantages

The Concrete Low Site has many of the same benefits and drawbacks as the high site. However, the low site would have a higher site development cost than the Concrete High Site. This is largely due to the extensive earthwork required to provide an adequately sized footprint for the new facility. Also, since the area has been used as a dumping ground for some time, there is additional removal and disposal costs at this site. Additionally, the elevation of the lower site is such that gravity flow from the Tailrace to Crowson II is only possible for approximately 30% of the plant's flow capacity as opposed to 70% at the Granite Low Site under typical membrane conditions.

Figure 15: Concrete Low Site Layout

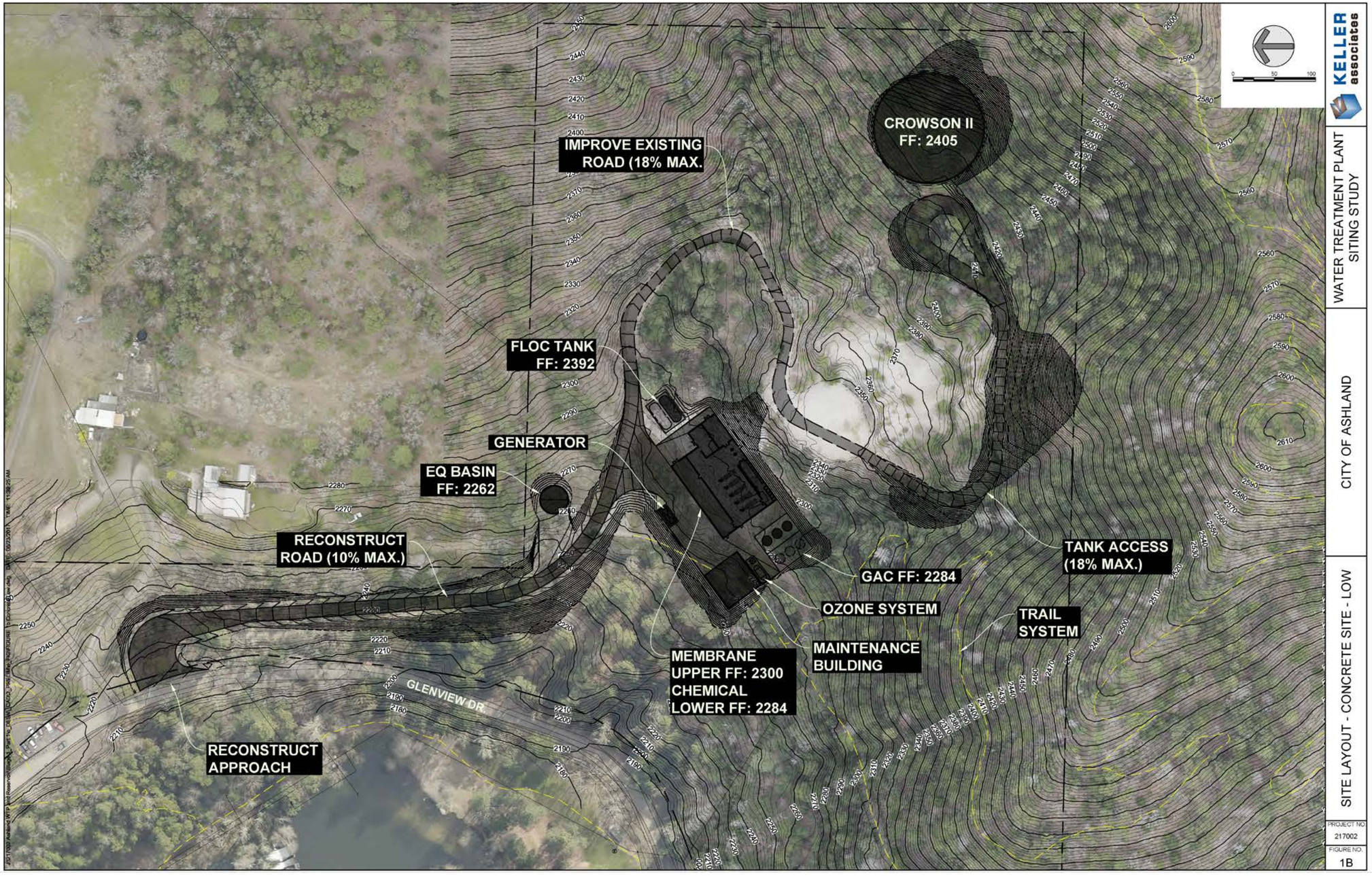
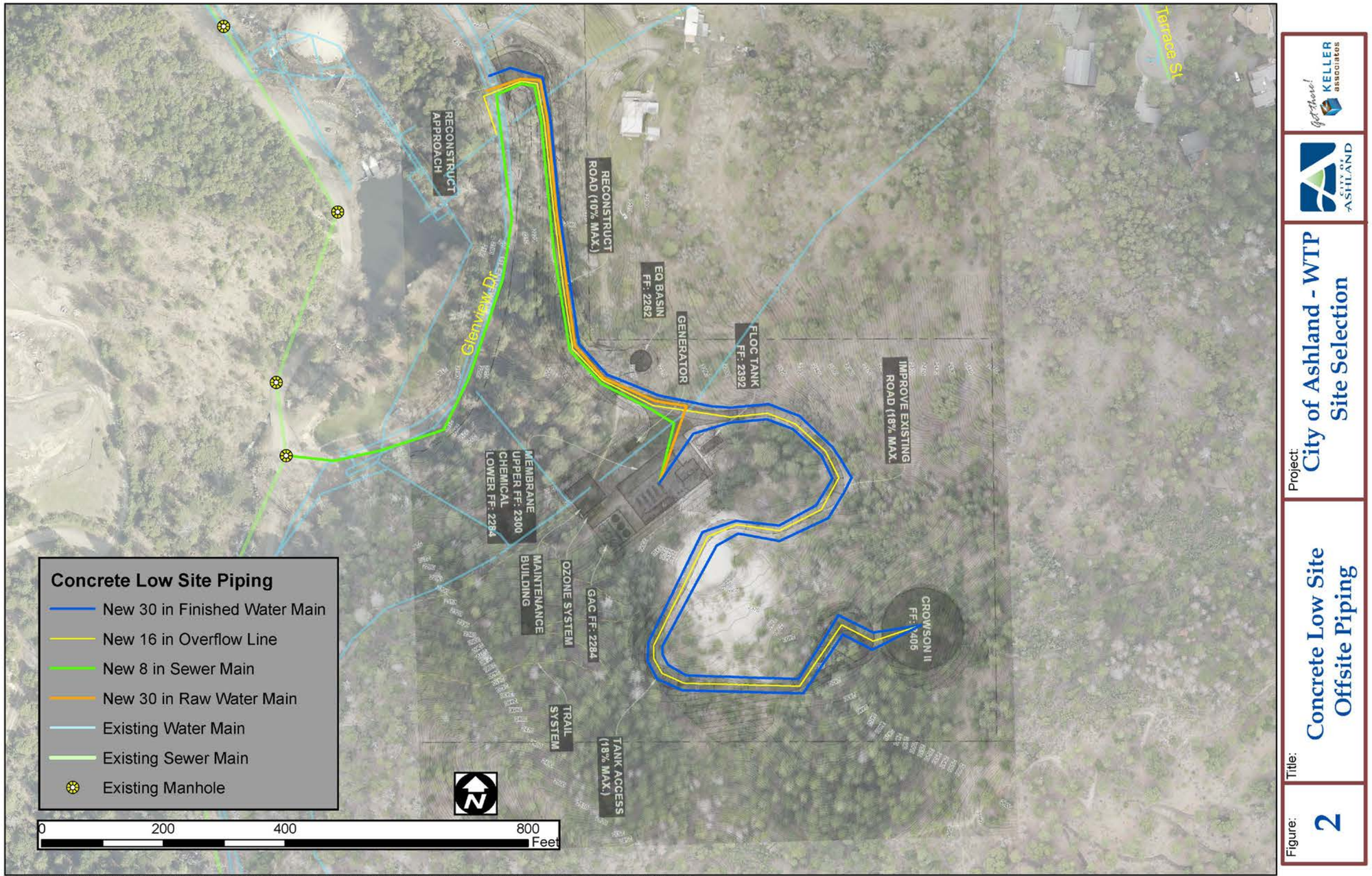


Figure 16: Concrete Low Site Off Site Piping



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SECTION 8: ASPHALT SITE ALTERNATIVE

The Asphalt Site is located adjacent to Glenview Drive and is approximately 1.15 miles measured from Downtown Ashland. A conceptual layout for the Asphalt Site is illustrated in Figure 17. The site contains existing millings spoils, and is a construction material storage area. The site is generally level and is currently covered with material stockpiles, construction equipment piping, and other miscellaneous construction supplies. The site is surrounded on the east and south by steep terrain, and an almost vertical slope to the north. Outside of the currently cleared area, there is dense deciduous trees, evergreen trees, and scattered brush.

FEMA panels have been acquired to evaluate flood potential at the Asphalt Site. The Asphalt Site is mapped as "Zone X." Zone X is defined as areas deemed to be outside of the 0.2% floodplain.

While much of the site is relatively flat and conducive to construction, the overall footprint is small when compared to other sites being considered. This site does not have a suitable location for Crowson II reservoir, and so for the purposes of evaluating this site, Crowson II was assumed to be located at the Concrete Site.

8.1 Remediation Requirements

If the Asphalt Site is selected, an environmental study will need to be performed to determine the suitability of the site.

8.2 Geotechnical Considerations

This site is located on a bench of a quarry previously used by the City as a borrow area, and is currently used to store construction materials (including asphalt millings, pipe, manholes, etc.). The site has been previously excavated to the point that the competent rock is expected at or near the surface of the existing grade. Since that is the case, it is anticipated that significant rock excavation would be required at this site.

8.3 Site Access, Existing Infrastructure

Access to the Asphalt Site is obtained from Glenview Drive. Truck traffic will take Granite Street to Glenview drive. Since there is access available from Glenview Drive, the Asphalt Site will have sufficient access during flood events.

The Asphalt Site is located within a partially developed area completely out of Ashland Creek Canyon. Since that is the case, minimal additional access improvements and paving outside of the parking area would be required. However, the Asphalt Site is not large enough to accommodate the new treatment facility without significant development costs (i.e. retaining walls) and locating Crowson II at an offsite location. An additional access road would be necessary to reach the new Crowson II site.

8.4 Offsite Piping Requirements

The general layout of the anticipated offsite piping for the Asphalt Site is shown in Figure 18.

8.5 Advantages and Disadvantages

Due to its convenient location, the Asphalt Site has the best access of all the considered sites. However, the site is small, and is confined by steep topography and the Glenview Drive. This would make expanding the plant in the future very challenging. The offsite piping costs for this alternative are also relatively high because of the distance from the plant site to the suitable Crowson II future site.

Another disadvantage of this site are the high pumping costs. The site is lower in elevation than all other sites considered. Pressures from the Tailrace would have to be reduced as not to exceed the maximum allowable pressures in a membrane plant. This makes gravity flow through the plant to Crowson II infeasible.

This site had the highest capital and operating costs of all the sites considered.

Figure 17: Asphalt Site Gravity Flow Potential

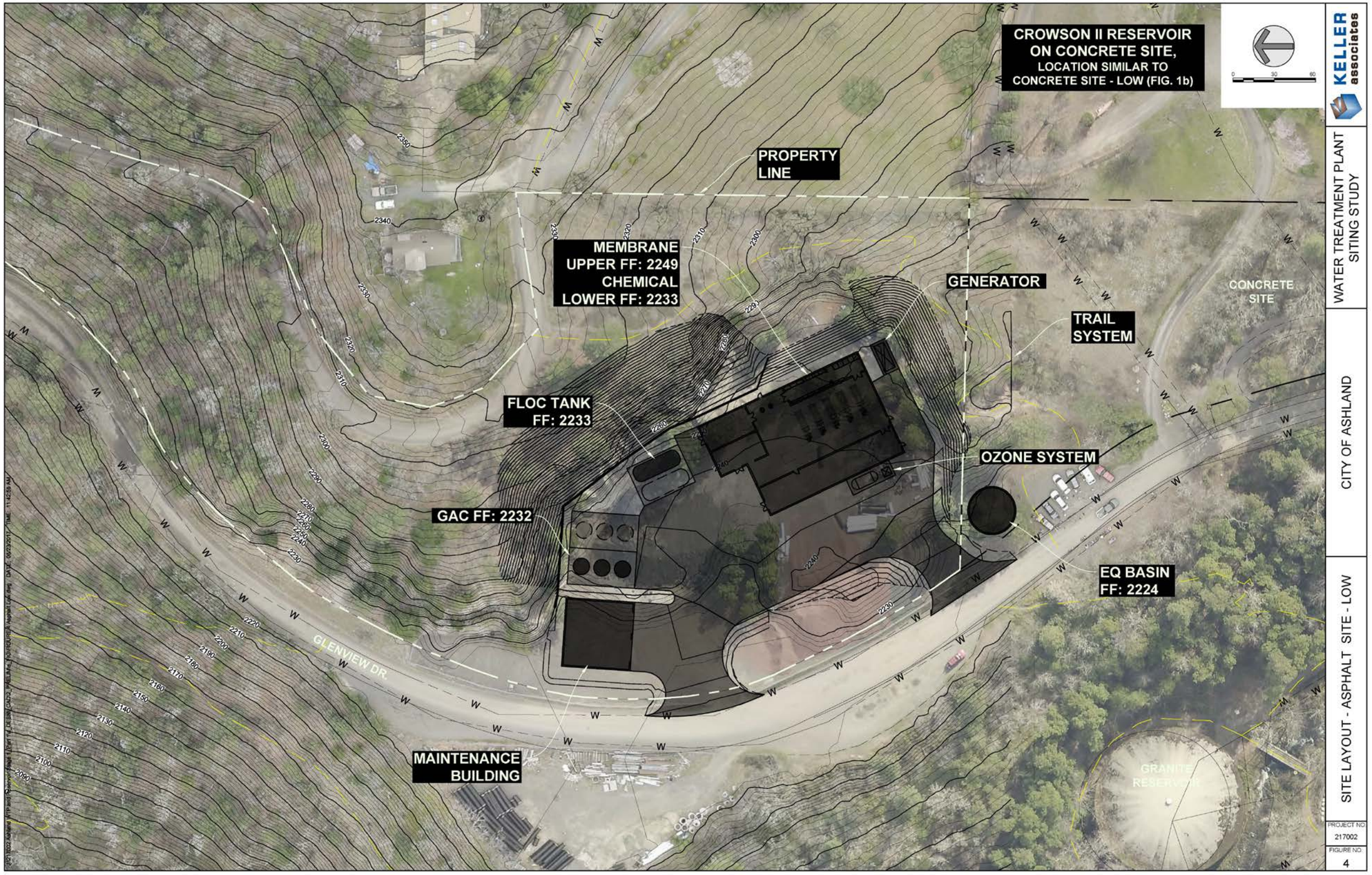


Figure 18: Asphalt Site Off Site Piping

SECTION 9: CONCLUSION AND RECOMMENDATION

After reviewing the suitability of the proposed sites, Keller Associates worked with City staff to identify factors that should be considered in selecting the preferred treatment plant site. These factors included capital costs, O&M (power) costs, environmental/sustainability impacts, expandability, access, and impacts to the public.

Table 3 shows the comparative capital costs and power costs for each site. These costs include major transmission piping (onsite and offsite), access roads, and site development costs for both the treatment plant and the Crowson II reservoir. More detailed cost breakouts are included in Appendix F of this report.

Table 3 – Capital and O&M Costs (Site, Piping, and Power Only)

	Concrete High	Concrete Low	Granite High	Granite Low	Asphalt Pit
Site (Ex/Backfill, Rdwys, Rtng Wall, Site Imp.)	$ 2,951,000	$ 3,248,000	$ 4,140,000	$ 2,704,000	$ 4,625,000
Offsite & Major Yard Piping / Offsite PRV Station[1]	$ 3,399,000	$ 3,727,000	$ 4,194,000	$ 3,692,000	$ 3,405,000
Pumping Station	$ 888,000	$ 1,462,000	$ 888,000	$ 888,000	$ 1,615,000
Subtotal	*$ 7,238,000*	*$ 8,437,000*	*$ 9,222,000*	*$ 7,284,000*	*$ 9,645,000*
Contingency (30%)	$ 2,172,000	$ 2,532,000	$ 2,767,000	$ 2,186,000	$ 2,894,000
Construction Costs	*$ 9,410,000*	*$ 10,969,000*	*$ 11,989,000*	*$ 9,470,000*	*$ 12,539,000*
Engineering (23%)	$ 2,165,000	$ 2,523,000	$ 2,758,000	$ 2,179,000	$ 2,884,000
Total Project	$ 1,575,000	$ 13,492,000	$ 14,747,000	$ 11,649,000	$ 15,423,000
Pumping Power Costs (20 yrs)[2]	$ 104,000	$ 512,000	$ 92,000	$ 98,000	$ 1,052,000
20-Year Comparative Life Cost	$ 11,679,000	$ 14,004,000	$ 14,839,000	$ 11,747,000	$ 16,475,000

Notes:
1. Concrete Low and Asphalt Pit Sities include a small PRV station.
2. Pumping Cost are based on a 4MGD flow 24/7. Discount rate was assumed to be equal to the inflation rate to arrive at the 20-year Comparative Life Cost.
3. Costs are in 2017 dollars.

Power costs assume that the available head from the Tailrace is preserved where feasible. For the Concrete Low and Asphalt Sites, the pressure head would need to be cut to limit pressures to no greater than 65 psi entering the membrane facility. This results in higher power costs for these alternatives. It should be noted that if conventional treatment processes are selected, it would likely be necessary to break head somewhere within the treatment plant. Consequently, the annual power costs over the 20-year life of the facility would increase as shown in Table 4.

Table 4 – Added Power Cost for Conventional Treatment

	Concrete High	Concrete Low	Granite High	Granite Low	Asphalt Pit
Added Pumping Power Costs (20 yrs)[1]	$ 436,000	$ 956,000	$ 448,000	$ 1,018,000	$ 936,000

Notes:
1. Added pumping cost based on a flow of 4MGD. Discount rate was assumed to be equal to the inflation rate to arrive at the 20-Year Comparative Life Cost.
2. Costs are in 2017 dollars.

The two most attractive alternatives from a life cycle cost standpoint appear to be the Concrete High and Granite Low Sites. It should be noted that a new concrete bridge/culvert structure was included in the cost estimate for granite sites to mitigate the City's concern of the culvert historically washing out where Horn Creek Road crosses Ashland Creek in the canyon.

A pairwise comparison process was employed to compare each site in terms of the selection criteria mentioned above. This process included an evaluation of selection criteria to develop overall weighting (see Table 5). A similar comparison process between alternatives was developed for each selection criteria. Scoring in the pairwise comparison is as follows:

1. Alternative is much worse than the one being compared to.
2. Alternative is a little worse than the one being compared to.
3. Alternatives are comparable.
4. Alternative is better than the one being compared to.
5. Alternative is much better than the one being compared to.

<u>For Ashland's capital cost, the ranking went as follows. Similar comparisons of each criteria were developed with input from City staff. (see Capital Costs Row in Table 5):</u>

- Capital Costs compared to O&M Cost – Capital Costs is more important than O&M cost resulting in a score of 4.
- Capital Costs compared to Env./Sust./Carbon – Capital Costs is less important than Env./Sust./Carbon resulting in a score of 2.
- Capital Costs compared to Expandability – Capital Costs is more important than Expandability resulting in a score of 4.
- Capital Costs compared to Access – Capital Costs is more important than Access resulting in a score of 4.
- Capital Costs compared to Impacts to Ex. Develop. – Capital Cost is more important than Impacts to Existing Development; resulting in a score of 4.

Table 5 – Weighting Criteria

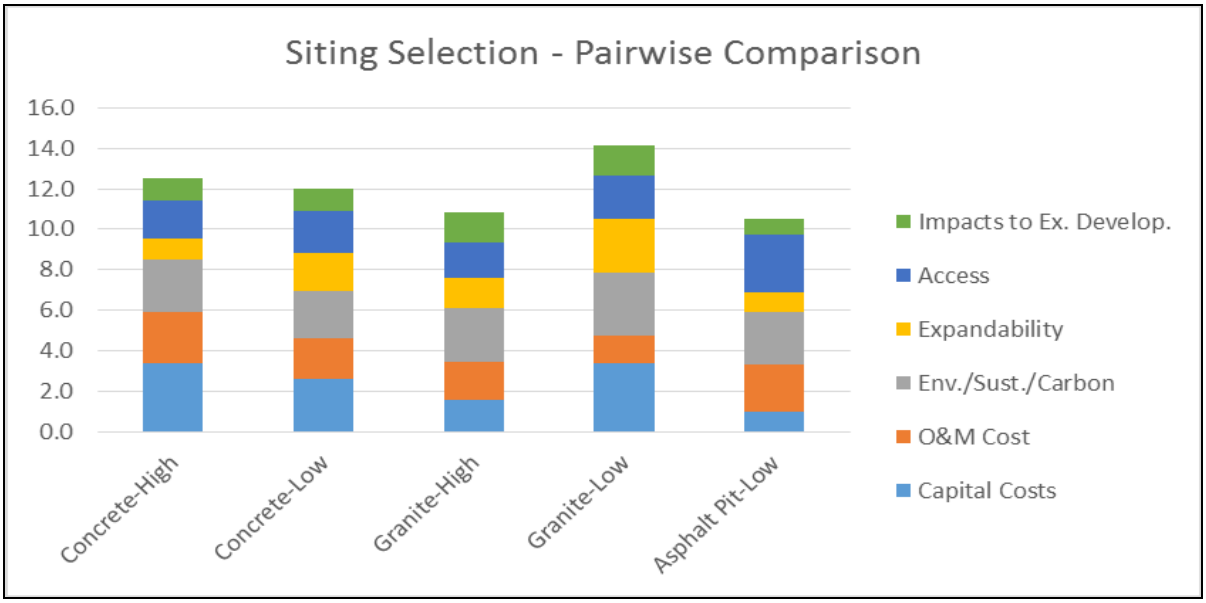
	Capital Costs	O&M Cost	Env./Sust./ Carbon	Expanda-bility	Access	Impacts to Ex. Develop.	Totals	Weights (%)
Capital Costs		4	2	4	4	4	18	20%
O&M Cost	2		2	4	3	4	15	17%
Env./Sust./Carbon	4	4		4	3	5	20	22%
Expandability	2	2	2		2	4	12	13%
Access	2	3	3	4		4	16	18%
Impacts to Ex. Develop.	2	2	1	2	2		9	10%
							90	100%

Appendix E shows the scoring for each selection criteria. Table 6 and Figure 19 show the summary results. **Granite Low scored the highest and is the preferred site**, while Concrete High was ranked second. Both sites have comparable site development costs and retain the available energy from the existing treatment plant Tailrace, allowing for gravity flow conditions for much of the time. Granite Low has a much larger developable area which allows for better site access, more flexibility in building layouts and greater expansion potential. Moving forward, predesign of the new treatment plant and offsite piping will focus on the Granite Low Site.

Table 6 – Summary Scoring

	Capital Costs	O&M Cost	Env./Sust./ Carbon	Expanda-bility	Access	Impacts to Ex. Develop.	Total
Concrete-High	3.40	2.50	2.59	1.07	1.87	1.10	12.5
Concrete-Low	2.60	2.00	2.37	1.87	2.04	1.10	12.0
Granite-High	1.60	1.83	2.67	1.47	1.78	1.50	10.8
Granite-Low	3.40	1.33	3.11	2.67	2.13	1.50	14.1
Asphalt Pit-Low	1.00	2.33	2.59	0.93	2.84	0.80	10.5
							60.0

Figure 19 – Summary Scoring



Crowson II Considerations

In order to get a more complete picture for the costs to develop the sites, the tank access road, piping to and from the tank, and tank excavation costs were also included. For the Granite Low Site, this accounts for approximately 30% of the site development capital costs, or $3,520,000 of the $11,544,000 total. We also understand that the final tank size has not yet been determined and the City is considering a potential future Crowson III reservoir site adjacent to the Crowson II reservoir. A preliminary review of the proposed granite sites' tank location suggests that two 3 MG tanks could be accommodated. Depending on the selected treatment disinfection technology, construction of Crowson II may not be required, which could allow Crowson II improvements to be completed at a later date.

APPENDICES

Appendix A – Power Production Loss Calculations

Appendix B – Preliminary Geotechnical Evaluation

Attachment C – FEMA Panels

Appendix D – Environmental Evaluation

Appendix E – Pairwise Comparison

Appendix F – Cost Estimations

Appendix G – Offsite Utilities

Appendix H – Hydraulic Calculations

APPENDIX A

Power Production and Loss Calculations

KELLER associates

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Ashland

WTP and Power House Flows & Estimated Power Loss from a Penstock Connection

	MONTH	DAYS	Monthly Totals GENERATOR INFLUENT MG	PLANT INFLUENT MG	ASH CREEK BY-PASS MG	Monthly Daily Averages GENERATOR INFLUENT MG	GPM	PLANT INFLUENT MG	ASH CREEK BY-PASS MG	Flow (MGD) WTP	Flow (gpm)	Head ft	Head psi	Kw*Hr	Cost
2016	JAN	31	714.4	54.1	660.3	23.0	16,004	1.7	21.3	1.7	1,212	330.5	143.1	44,943	$ 3,600
	FEB	29	729.3	50.0	679.3	25.1	17,464	1.7	23.4	1.7	1,198	316.2	136.9	39,757	$ 3,200
	MAR	31	1099.7	55.2	1044.5	35.5	24,635	1.8	33.7	1.8	1,235	230.9	100.0	32,008	$ 2,600
	APR	30	638.0	70.1	567.9	21.3	14,769	2.3	18.9	2.3	1,623	341.7	147.9	60,224	$ 4,900
	MAY	31	556.9	98.2	458.7	18.0	12,475	3.2	14.8	3.2	2,199	360.6	156.1	88,974	$ 7,200
	JUN	30	425.0	132.0	293.0	14.2	9,838	4.4	9.8	4.4	3,054	378.9	164.0	125,642	$ 10,100
	JUL	31	203.7	144.3	59.4	6.6	4,563	4.7	1.9	4.7	3,233	404.0	174.9	146,559	$ 11,800
	AUG	31	179.1	160.2	18.9	5.8	4,012	5.2	0.6	5.2	3,589	405.7	175.6	163,356	$ 13,100
	SEP	30	142.9	126.1	16.8	4.8	3,308	4.2	0.6	4.2	2,920	407.6	176.4	129,207	$ 10,400
	OCT	31	278.5	77.6	200.9	9.0	6,239	2.5	6.5	2.5	1,739	397.7	172.2	77,590	$ 6,300
	NOV	30	273.4	58.4	214.9	9.1	6,329	1.9	7.2	1.9	1,353	397.3	172.0	58,352	$ 4,700
	DEC	31	442.2	58.0	384.2	14.3	9,906	1.9	12.4	1.9	1,300	378.4	163.8	55,203	$ 4,500
2015	JAN	31	125.0	58.3	66.7	4.0	2,800	1.9	2.2	1.9	1,306	408.8	177.0	59,894	$ 4,800
	FEB	28	84.0	52.7	31.3	3.0	2,083	1.9	1.1	1.9	1,307	410.1	177.5	54,332	$ 4,400
	MAR	31	93.0	59.5	33.5	3.0	2,083	1.9	1.1	1.9	1,333	410.1	177.5	61,331	$ 5,000
	APR	30	110.0	67.9	42.1	3.7	2,546	2.3	1.4	2.3	1,572	409.3	177.2	69,845	$ 5,600
	MAY	31	132.5	97.6	34.9	4.3	2,968	3.1	1.1	3.1	2,186	408.4	176.8	100,157	$ 8,100
	JUN	30	160.2	129.6	30.6	5.3	3,708	4.3	1.0	4.3	3,000	406.5	176.0	132,444	$ 10,600
	JUL	31	149.6	125.1	24.4	4.8	3,351	4.0	0.8	4.0	2,803	407.5	176.4	128,149	$ 10,300
	AUG	31	116.0	104.8	11.3	3.7	2,599	3.4	0.4	3.4	2,347	409.2	177.1	107,734	$ 8,700
	SEP	30	113.0	101.8	11.3	3.8	2,616	3.4	0.4	3.4	2,356	409.1	177.1	104,650	$ 8,400
	OCT	31	113.4	89.0	24.5	3.7	2,540	2.9	0.8	2.9	1,993	409.3	177.2	91,511	$ 7,400
	NOV	30	127.2	58.6	68.6	4.2	2,944	2.0	2.3	2.0	1,357	408.4	176.8	60,176	$ 4,900
	DEC	31	367.6	54.5	313.0	11.9	8,235	1.8	10.1	1.8	1,221	388.2	168.0	53,187	$ 4,300
2014	JAN	31	102.8	56.0	46.8	3.3	2,303	1.8	1.5	1.8	1,254	409.7	177.4	57,669	$ 4,700
	FEB	28	111.6	51.1	60.5	4.0	2,768	1.8	2.2	1.8	1,266	408.8	177.0	52,454	$ 4,200
	MAR	31	276.8	60.3	216.5	8.9	6,201	1.9	7.0	1.9	1,351	397.9	172.2	60,331	$ 4,900
	APR	30	218.2	72.0	146.0	7.3	5,051	2.4	4.9	2.4	1,666	402.3	174.2	72,757	$ 5,900
	MAY	31	198.3	102.8	96.2	6.4	4,442	3.3	3.1	3.3	2,303	404.4	175.1	104,481	$ 8,400
	JUN	30	132.3	124.2	8.1	4.4	3,063	4.1	0.3	4.1	2,874	408.2	176.7	127,362	$ 10,200
	JUL	31	95.3	144.8	-49.6	3.1	2,135	4.7	-1.6	3.1	2,133	410.0	177.5	98,119	$ 7,900
	AUG	31	80.2	136.7	-56.5	2.6	1,797	4.4	-1.8	2.6	1,796	410.6	177.7	82,748	$ 6,700
	SEP	30	71.4	107.1	-35.7	2.4	1,653	3.6	-1.2	2.4	1,653	410.8	177.8	73,714	$ 5,900
	OCT	31	106.4	88.4	18.0	3.4	2,384	2.9	0.6	2.9	1,980	409.6	177.3	90,992	$ 7,300
	NOV	30	153.4	58.6	94.8	5.1	3,551	2.0	3.2	2.0	1,356	407.0	176.2	59,928	$ 4,800
	DEC	31	212.8	54.4	158.5	6.9	4,767	1.8	5.1	1.8	1,218	403.3	174.6	55,114	$ 4,500
2013	JAN	31	278.5	57.3	221.2	9.0	6,239	1.8	7.1	1.8	1,283	397.7	172.2	57,248	$ 4,600
	FEB	28	289.0	48.2	240.8	10.3	7,168	1.7	8.6	1.7	1,196	393.6	170.4	47,705	$ 3,900
	MAR	31	278.5	55.2	223.3	9.0	6,239	1.8	7.2	1.8	1,236	397.7	172.2	55,139	$ 4,500
	APR	30	360.0	70.5	289.5	12.0	8,333	2.4	9.7	2.4	1,632	387.6	167.8	68,692	$ 5,500
	MAY	31	372.0	114.6	257.4	12.0	8,333	3.7	8.3	3.7	2,568	387.6	167.8	111,684	$ 9,000
	JUN	30	262.5	140.8	121.7	8.8	6,076	4.7	4.1	4.7	3,259	398.4	172.5	140,967	$ 11,300
	JUL	31	203.0	176.5	26.5	6.5	4,547	5.7	0.9	5.7	3,954	404.0	174.9	179,247	$ 14,400
	AUG	31	161.7	164.4	23.4	5.2	3,622	5.3	0.8	5.3	3,683	406.8	176.1	168,067	$ 13,500
	SEP	30	112.8	124.6	28.5	3.8	2,611	4.2	1.0	4.2	2,883	409.2	177.1	128,088	$ 10,300
	OCT	31	154.0	114.4	39.7	5.0	3,450	3.7	1.3	3.7	2,562	407.2	176.3	117,037	$ 9,400
	NOV	30	49.0	97.6	27.5	1.6	1,134	3.3	0.9	3.3	2,259	411.4	178.1	100,904	$ 8,100
	DEC	31	119.3	67.2	52.1	3.8	2,672	2.2	1.7	2.2	1,506	409.0	177.1	69,123	$ 5,600

$$HP = (h_a * Q * SG) / 3956$$

h_a = added head in feet

Q = flow in gpm

SG = Specific Gravity (water =1)

Conversion Factors

HP to KW - multiply by:

0.7457

Assumptions

Turbine Efficiency	80%	
KW-hr	$ 0.08	
% Pumping @ Concrete High	15%	
Reeder Elevation	2,876	ft
Power Plant Elevation	2,464	ft
Penstock Pipe:		
Assume C=	120	
Assume L =	4,750	ft
Diameter =	24	inches

Typical Monthly Production Loss at 3 and 4 MGD

																Month	Annual	20 Year
Typical Month @ 3 MGD	30					8.1	5,595				3	2,083	400.3	173.3	90,555	$ 7,300	$ 87,600	$ 1,752,000
Typical Month @ 4 MGD	30					8.1	5,595				4	2,778	400.3	173.3	120,740	$ 9,700	$ 116,400	$ 2,328,000
Typical Month @ 4 MGD With 54% of the flow from Penstock	30					6.7	4,653				4	2,778	403.7	174.8	66,205	$ 5,300	$ 63,600	$ 1,272,000

APPENDIX B

Preliminary Geotech Evaluation

KELLER associates

Get there!

217002/b/17-131

Applied
Geotechnical
Engineering
& Geologic Consulting

July 10, 2017

Keller Associates, Inc.
131 SW 5th Avenue, Suite A
Meridian, ID 83642

SUBJECT: **Preliminary Geotechnical Evaluation**
City of Ashland's Proposed New Water Treatment Plant
Ashland, Oregon

At your request, Applied Geotechnical Engineering and Geologic Consulting LLC (AGEGC) has performed preliminary geotechnical evaluation of five alternative sites for development with the new water treatment plant and associated water reservoir. The geotechnical evaluation included a review of available geotechnical and geologic information for the vicinity of the sites, a ground-level reconnaissance of the sites by a geotechnical engineer from AGEGC, and engineering analysis. This report describes the work accomplished and provides our preliminary conclusions and recommendations regarding considerations for development of each of the five sites. Site-specific explorations will need to be completed once the site location has been determined.

The five considered sites consist of the Concrete Site – High, the Concrete Site – Low, the Granite Site – High, the Granite Site – Low, and the Asphalt Site - Low. All five sites have previously been partially developed by the City of Ashland as granite borrow sites (quarries). A licensed geotechnical engineer and geologist from AGEGC completed site visits to all five sites.

CONCLUSIONS AND RECOMMENDATIONS

The following sections provide a summary of our observations and conclusions for each of the five evaluated sites.

Concrete Site – High

The Concrete Site - High is located on the upper bench of a quarry previously used by the City of Ashland as a borrow area. The rock exposed in the cut slopes at the site can be excavated with a moderate to large trackhoe equipped with a rock bucket.

The site is underlain by granite that is part of the Ashland Pluton. The depth of weathering and hardness of the granite can change significantly over short distances. In general, the deeper the excavation into the rock, the greater the risk of encountering hard granite that will need rock excavation methods to remove the material. Hard, intact granite has a high unconfined compressive strength and is highly abrasive to drilling equipment.

We anticipate that deeper cuts (such as for the water reservoir) will encounter hard rock at a depth of less than 30 ft. Given the location of the proposed water reservoir on a steep slope, we anticipate that significant rock excavation will be required to provide a level surface for support of the tank.

Geologic mapping of the area indicates no active faults near the proposed site.

Recent test pits completed at the site indicate the upper bench at the Concrete Site has been graded with minor fill (typically less than 2 ft thick). The fill consists of granitic soils but we anticipate the fill was imported from off-site. These soils are not structural fill and will need to be removed or compacted as structural fill in all building and roadway areas.

Weathered granitic rock (free of organics and other deleterious materials) can be used as structural fill for most structures if properly placed and compacted.

There are moderate to steep slopes east of the upper bench at the Concrete Site. The granite typically does not have continuous shears zones or joints that can result in a large rock fall. Indications of large-scale slope failures were not observed in the vicinity of the site. In our opinion, there is a low risk of slope instability at the site due to a seismic event.

Perched groundwater will occur in the surficial soils at the top of the granite rock. During a large storm event, the surficial weathered granite (silty sand to sandy silt soils) may become super-saturated, resulting in a mud flow. In our opinion, this risk may be minimized by appropriate surface (ditching upslope of the site) and/or subsurface drainage (French drains) to reduce the amount of saturation of the surficial soils.

Concrete Site – Low

This site is located on the lower bench of a quarry previously used by the City of Ashland as a borrow area and currently used to store of construction debris (primarily concrete debris). The rock exposed in the cut slopes at the site can be excavated with a moderate to large trackhoe equipped with a rock bucket.

The site is underlain by granite that is part of the Ashland Pluton. The depth of weathering and hardness of the granite can change significantly over short distances. In general, the deeper the excavation into the rock, the greater the risk of encountering hard granite that will need rock excavation methods to remove the material. Hard, intact granite has a high unconfined compressive strength and is highly abrasive to drilling equipment.

We anticipate that deeper cuts (such as for the water reservoir) will encounter hard rock at a depth of less than 30 ft. Given the location of the proposed water reservoir on a steep slope, we anticipate that significant rock excavation will be required to provide a level surface for support of the tank.

Geologic mapping of the area indicates no active faults near the proposed site.

The Concrete Site has been graded with cut. A near-vertical cut is located along the uphill side of the site.

Recent test pits completed at the site indicate the lower bench at the Concrete Site has been graded with fill, typically less than 2 ft thick. The fill consists of granitic soils. The fill soils are not structural fill and will need to be removed or compacted as structural fill in all building and roadway areas. The fill is thicker along the downhill (southwestern) edge of the site. Depending on foundation loads, a significant portion of these soils may need to be overexcavated and

replaced with structural fill. In addition, the lower bench has been used to store concrete debris from various projects. The construction debris will need to be removed as part of site development.

Excavated granitic rock (free of organics and other deleterious materials) can be used as structural fill for most structures if properly placed and compacted.

Indications of large rock falls were not observed in the cut slope along the northeastern side of the proposed main site. The granite typically does not have continuous shears zones or joints that can result in a large rock fall. In our opinion, there is a low risk of slope instability at the site due to a seismic event.

Perched groundwater will occur in the surficial soils at the top of the granite rock. During a large storm event, the surficial weathered granite (silty sand to sandy silt soils) may become super-saturated, resulting in a mud flow. In our opinion, this risk may be minimized by appropriate surface (ditching upslope of the site) and/or subsurface drainage (French drains) to reduce the amount of saturation of the surficial soils.

Granite Site – High

This site is located north (upslope) of the Granite Street quarry, in an area that appears to have had no significant site grading.

The site is underlain by granite that is part of the Ashland Pluton. The depth of weathering and hardness of the granite can change significantly over short distances. In general, the deeper the excavation into the rock, the greater the risk of encountering hard granite that will need rock excavation methods to remove the material. Hard, intact granite has a high unconfined compressive strength and is highly abrasive to drilling equipment.

We anticipate that cuts for the treatment plant and water reservoir will encounter hard rock at a depth of less than 20 ft. Rock excavation may require blasting to remove the harder granite where encountered.

Geologic mapping of the area indicates no active faults near the proposed site.

The excavated granitic rock (free of organics and other deleterious materials) can be used as structural fill for most structures if properly placed and compacted. We do not recommend use of untreated granitic fills for support of the reservoir; however, cement treated granite may be suitable for support of the reservoir if properly placed and compacted.

The granite typically does not have continuous shears zones or joints that can result in a large rock fall. The majority of the site is located on moderately sloping ground. In our opinion, there is a low risk of slope instability at the site due to a seismic event.

Perched groundwater will occur in the surficial soils at the top of the granite rock. During a large storm event, the surficial weathered granite (silty sand to sandy silt soils) may become super-saturated, resulting in a mud flow. In our opinion, this risk may be minimized by appropriate surface (ditching upslope of the site) and/or subsurface drainage (French drains) to reduce the amount of saturation of the surficial soils. Small retaining walls (we estimate the walls should be

less than about 5 ft high) may be required along the top of cut slopes above structures to retain the surficial silty sand/sandy silt soils.

Granite Site – Low
This site is located on the north side of the Granite Street quarry, in an area that has been used as a borrow area (cut).

The site is underlain by granite that is part of the Ashland Pluton. The depth of weathering and hardness of the granite can change significantly over short distances. Hard granite was locally observed at the ground surface in deeper cuts in this area. In general, the deeper the excavation into the rock, the greater the risk of encountering hard granite that will need rock excavation methods to remove the material. Hard, intact granite has a high unconfined compressive strength and is highly abrasive to drilling equipment.

We anticipate that cuts for the treatment plant and water reservoir will encounter hard rock at a depth of less than 10 ft. Rock excavation may require blasting to remove the harder granite where encountered.

Geologic mapping of the area indicates no active faults near the proposed site.

The excavated granitic rock (free of organics and other deleterious materials) can be used as structural fill for most structures if properly placed and compacted. We do not recommend use of untreated granitic fills for support of the reservoir; however, cement treated granite may be suitable for support of the reservoir if properly placed and compacted.

The granite typically does not have continuous shears zones or joints that can result in a large rock fall. The majority of the site is located on moderately sloping ground. In our opinion, there is a low risk of slope instability at the site due to a seismic event.

During our site visit in February 2017, perched groundwater was observed in the surficial soils at the top of the granite rock. During a large storm event, the surficial weathered granite (silty sand to sandy silt soils) may become super-saturated, resulting in a mud flow. In our opinion, this risk may be minimized by appropriate surface drainage (ditching upslope of the site) and/or subsurface drainage (French drains) to reduce the amount of saturation of the surficial soils. Small retaining walls (we estimate the walls should be less than about 5 ft high) may be required along the top of cut slopes above structures to retain the surficial silty sand/sandy silt soils.

Asphalt Site – Low
This site is located on a bench of a quarry previously used by the City of Ashland as a borrow area and is currently used to store construction materials (including ground asphaltic pavement). The rock exposed in the cut slope on the east (uphill) side of the site is relatively hard and may be difficult to excavate with a large trackhoe equipped with a rock bucket.

The site is underlain by granite that is part of the Ashland Pluton. The depth of weathering and hardness of the granite can change significantly over short distances. In general, the deeper the excavation into the rock, the greater the risk of encountering hard granite that will need rock excavation methods to remove the material. Hard, intact granite has a high unconfined compressive strength and is highly abrasive to drilling equipment.

We anticipate that significant cuts will be required for the treatment plant and that significant rock excavation will be required for site grading.

Geologic mapping of the area indicates no active faults near the proposed site.

The Asphalt Site has been graded with cut. The area has historically been used as a borrow pit for granite materials used as fills within the City of Ashland. A near-vertical cut is located along the uphill side of the site.

Weathered granitic rock (free of organics and other deleterious materials) can be used as structural fill for most structures if properly placed and compacted.

Indications of large rock falls were not observed in the cut slope along the eastern side of the proposed main site; however, significant erosion of the cut slope has occurred, exposing harder, less erodible granitic rock. In our opinion, there is a low risk of slope instability at the site due to a seismic event.

LIMITATIONS

This report has been prepared to aid in the City of Ashland in evaluation of the five alternate sites for the new water treatment plant and water reservoir. The scope is limited to the specific project and locations described herein. Final design of the chosen site will need to include site-specific explorations including borings and test pits and site-specific geotechnical design recommendations based on these explorations.

Please contact the undersigned if you have any questions.

Submitted for Applied Geotechnical Engineering and Geologic Consulting LLC,

Robin L. Warren, P.E., G.E., R.G.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
15045PE
OREGON
JULY 25, 1990
ROBIN L. WARREN

Renewal: June 2018

REFERENCES

DOGAMI; Oregon Department of Geology and Mineral Resources; Interpretive Map Series IMS-009; Relative earthquake hazard maps for selected urban areas in western Oregon: Ashland, Cottage Grove, Grants Pass, Roseburg, Sutherland-Oakland; 1999.

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State of Oregon Structural Specialty Code Amendments; International Code Council.

APPENDIX C

FEMA Panels

KELLER associates

Get there!

217002/b/17-131

NOTES TO USERS

This map is for use in administering the National Flood Insurance Program. It does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size. The **community map repository** should be consulted for possible updated or additional flood hazard information.

To obtain more detailed information in areas where **Base Flood Elevations** (BFEs) and/or **floodways** have been determined, users are encouraged to consult the Flood Profiles and Floodway Data and/or Summary of Stillwater Elevations tables contained within the Flood Insurance Study (FIS) report that accompanies this FIRM. Users should be aware that BFEs shown on the FIRM represent rounded whole-foot elevations. These BFEs are intended for flood insurance rating purposes only and should not be used as the sole source of flood elevation information. Accordingly, flood elevation data presented in the FIS report should be utilized in conjunction with the FIRM for purposes of construction and/or floodplain management.

Coastal Base Flood Elevations shown on this map apply only landward of 0.0' North American Vertical Datum of 1988 (NAVD 88). Users of this FIRM should be aware that coastal flood elevations are also provided in the Summary of Stillwater Elevations table in the Flood Insurance Study report for this jurisdiction. Elevations shown in the Summary of Stillwater Elevations table should be used for construction and/or floodplain management purposes when they are higher than the elevations shown on this FIRM.

Boundaries of the **floodways** were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the National Flood Insurance Program. Floodway widths and other pertinent floodway data are provided in the Flood Insurance Study report for this jurisdiction.

Certain areas not in Special Flood Hazard Areas may be protected by **flood control structures.** Refer to Section 2.4 "Flood Protection Measures" of the Flood Insurance Study report for information on flood control structures for this jurisdiction.

The **projection** used in the preparation of this map was Universal Transverse Mercator (UTM) zone 10. The **horizontal datum** was NAD83, GRS1980 spheroid. Differences in datum, spheroid, projection or UTM zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of this FIRM.

Flood elevations on this map are referenced to the North American Vertical Datum of 1988. These flood elevations must be compared to structure and ground elevations referenced to the same **vertical datum.** For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, visit the National Geodetic Survey website at http://www.ngs.noaa.gov/ or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, N/NGS12
National Geodetic Survey
SSMC- 3, #9202
1315 East- West Highway
Silver Spring, MD 20910- 3282

To obtain current elevation, description, and/or location information for **bench marks** shown on this map, please contact the Information Services Branch of the National Geodetic Survey at **(301) 713- 3242,** or visit its website at http://www.ngs.noaa.gov/.

Base map information shown on this FIRM was derived from multiple sources. Base map files were provided in digital format by Jackson County GIS Services, State of Oregon OLCD, and the National Geodetic Survey. This information was compiled at various map scales during the time period 2003-2006.

This map reflects more detailed and up- to- date **stream channel configurations** than those shown on the previous FIRM for this jurisdiction. The floodplains and floodways that were transferred from the previous FIRM may have been adjusted to conform to these new stream channel configurations. As a result, the Flood Profiles and Floodway Data tables *in the Flood Insurance Study report (which contains authoritative hydraulic data)* may reflect stream channel distances that differ from what is shown on this map.

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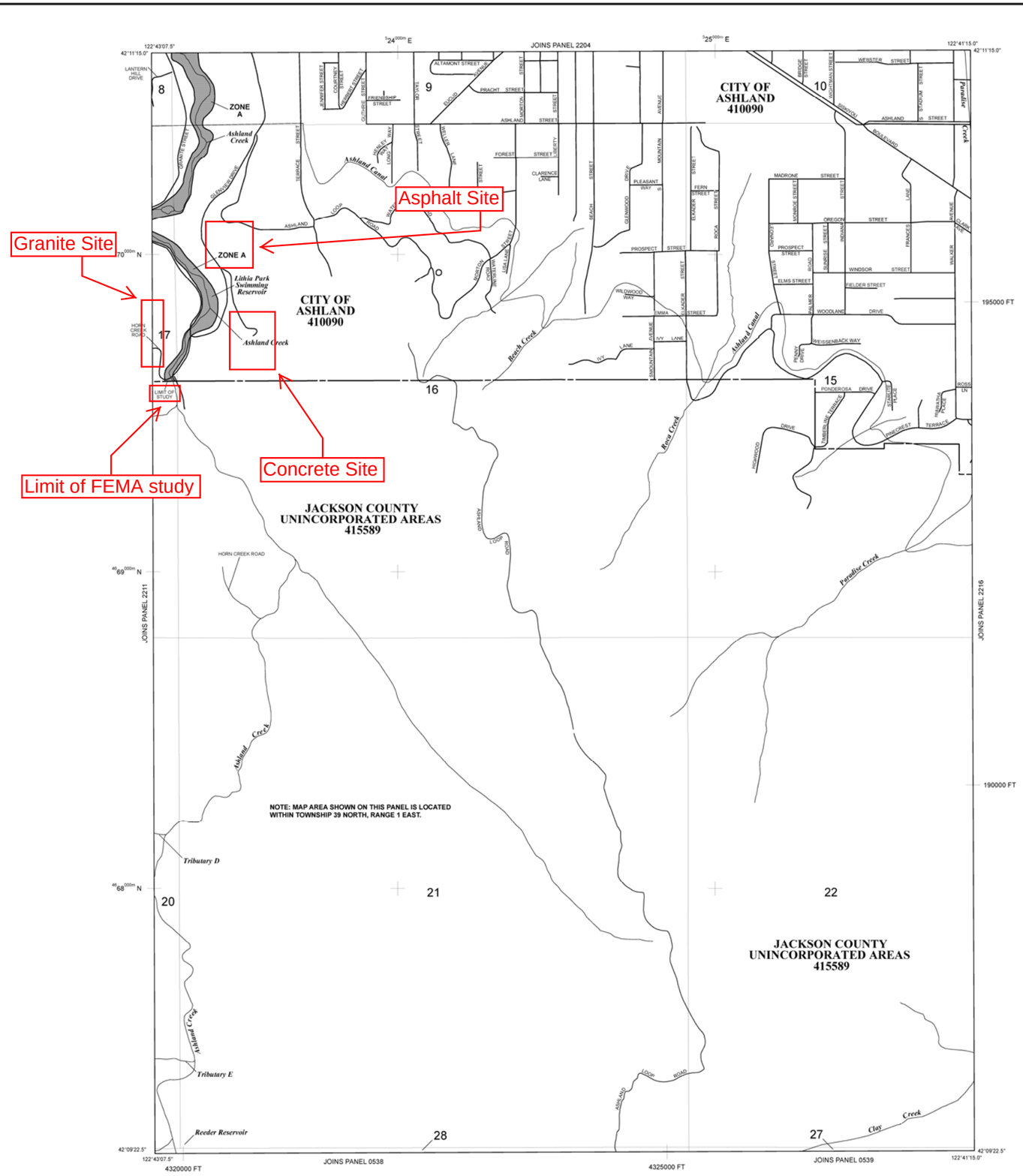

LEGEND

SPECIAL FLOOD HAZARD AREAS (SFHAs) SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

	Floodplain boundary
	Floodway boundary
	Zone D boundary
	CBRS and OPA boundary
	Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.
513	Base Flood Elevation line and value; elevation in feet*
(EL 987)	Base Flood Elevation value where uniform within zone; elevation in feet*

* Referenced to the North American Vertical Datum of 1988 (NAVD 88)

A — A	Cross section line
23 - - - - - 23	Transect line
97°07'30", 32°22'30"	Geographic coordinates referenced to the North American Datum of 1983 (NAD 83)
$^{42}75^{000m}$N	1000-meter Universal Transverse Mercator grid ticks, zone 10
6000000 FT	5000-foot grid ticks: Oregon State Plane coordinate system, south zone (FIPSZONE 3602), Lambert Conformal Conic
DX5510	Bench mark (see explanation in Notes to Users section of this FIRM panel)
• M1.5	River Mile

MAP REPOSITORIES
Refer to Map Repositories list on Map Index

EFFECTIVE DATE OF COUNTYWIDE
FLOOD INSURANCE RATE MAP
May 3, 2011
EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at 1- 800- 638- 6620.

MAP SCALE 1" = 500'
250 0 500 1000 FEET
150 0 150 300 METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 2212F

FIRM
FLOOD INSURANCE RATE MAP
JACKSON COUNTY, OREGON
AND INCORPORATED AREAS

PANEL 2212 OF 2327
(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
JACKSON COUNTY	415589	2212	F
ASHLAND, CITY OF	410090	2212	F

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
41029C2212F

EFFECTIVE DATE
MAY 3, 2011

Federal Emergency Management Agency

NOTES TO USERS

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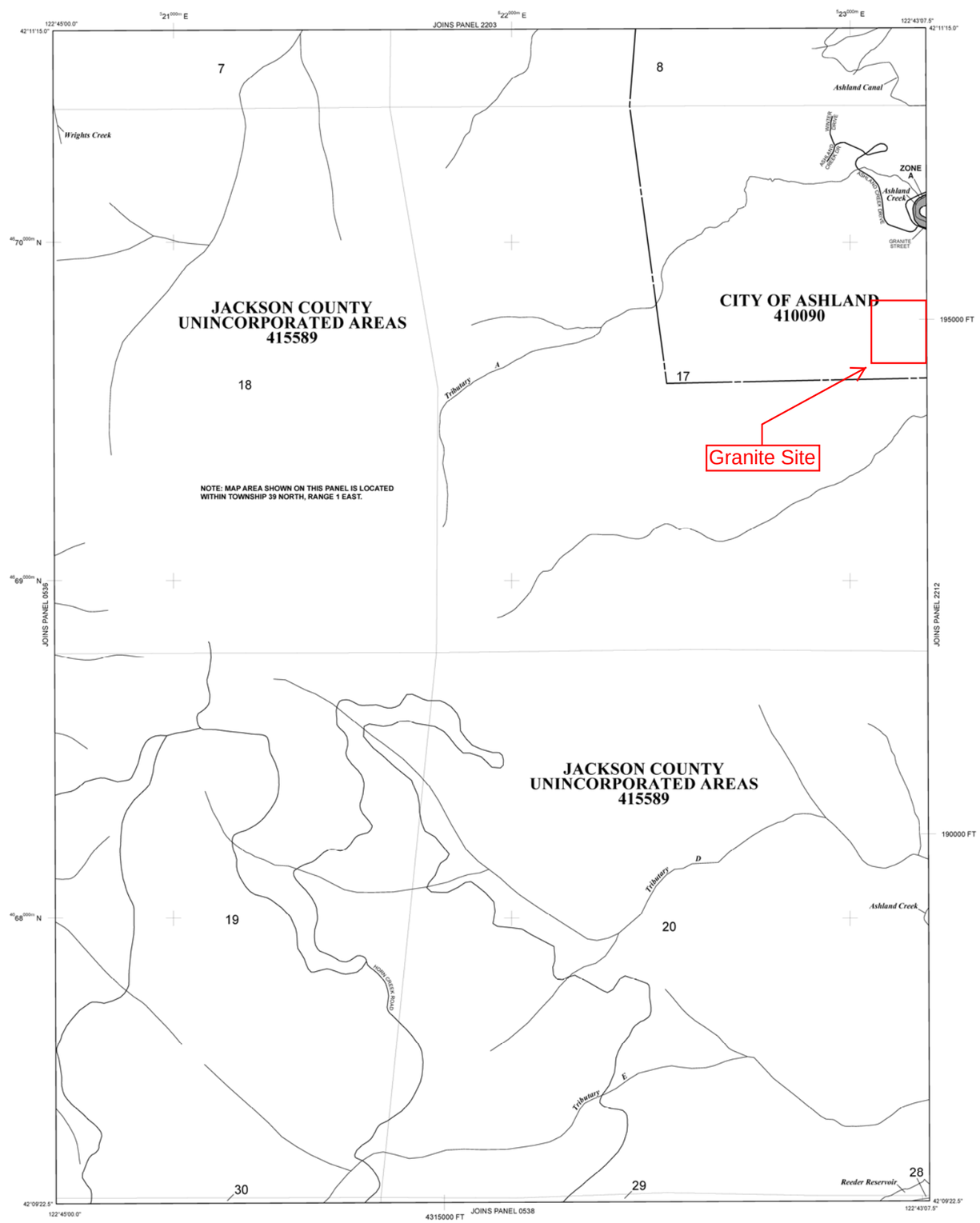

LEGEND

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ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

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OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

	Floodplain boundary
	Floodway boundary
	Zone D boundary
	CBRS and OPA boundary
	Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.
513	Base Flood Elevation line and value; elevation in feet*
(EL 987)	Base Flood Elevation value where uniform within zone; elevation in feet*

* Referenced to the North American Vertical Datum of 1988 (NAVD 88)

A — A	Cross section line
23 - - - - - - 23	Transect line
97°07'30", 32°22'30"	Geographic coordinates referenced to the North American Datum of 1983 (NAD 83)
^{4275000m}N	1000-meter Universal Transverse Mercator grid ticks, zone 10
6000000 FT	5000-foot grid ticks: Oregon State Plane coordinate system, south zone (FIPSZONE 3602), Lambert Conformal Conic
DX5510	Bench mark (see explanation in Notes to Users section of this FIRM panel)
• M1.5	River Mile

MAP REPOSITORIES
Refer to Map Repositories list on Map Index

EFFECTIVE DATE OF COUNTYWIDE
FLOOD INSURANCE RATE MAP
May 3, 2011
EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at 1- 800- 638- 6620.

MAP SCALE 1" = 500'

250 0 500 1000 FEET

150 0 150 300 METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 2211F

FIRM

FLOOD INSURANCE RATE MAP

JACKSON COUNTY, OREGON

AND INCORPORATED AREAS

PANEL 2211 OF 2327

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
JACKSON COUNTY	415589	2211	F
ASHLAND, CITY OF	410090	2211	F

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
41029C2211F

EFFECTIVE DATE
MAY 3, 2011

Federal Emergency Management Agency

APPENDIX D

Environmental Evaluation

KELLER associates

Get there!

217002/b/17-131

State of Oregon

Department of Environmental Quality Memorandum

Date: **July 5, 2017**

To: **Mike Faught**
Ashland Public Works Department

From: **Greg Aitken, DEQ Cleanup Program**

Subject: **Environmental Review for Alleged Contamination**
Former Ashland Quarry (aka Ashland Concrete Pit)
About 500 feet east of the Granite Street – Glenview Drive intersection
Ashland, Oregon

This memorandum addresses a report[1] prepared on behalf of the City of Ashland to address potential residual contamination of groundwater and soil after they received a complaint from a former city employee that containers of diesel and pesticide were dumped at the former quarry property owned and operated by Ashland. The property was used in the past as a rock quarry and appears to have also been used to stockpile construction debris.

In 2016, a former city employee alleged that containers of diesel fuel and pesticides were dumped at the former quarry. The city was initially concerned about impacts to nearby Ashland Creek and the Swimming Reservoir dam, and city staff collected two surface water samples in November 2016 for laboratory analysis of herbicides and petroleum hydrocarbons. None were detected.

In January 2017, the City of Ashland requested participation in DEQ's Independent Cleanup Program for investigation of potential site contamination from the alleged dumping that could pose a potential health risks to future site workers if the site is developed for a drinking water treatment plant. The city contracted with Alpine Environmental Consultants LLC to investigate for potential dumping and environmental impacts at the site. DEQ approved Alpine's February 14, 2017 workplan submittal for collection and laboratory analysis of soil and groundwater samples collected from test pits dug at the quarry.

Soil samples collected in March 2017 by Alpine from test pits were analyzed for diesel, gasoline, oil, volatile organic hydrocarbons, polynuclear aromatic hydrocarbons, metals, organochlorine pesticides, organophosphate pesticides, and chlorinated herbicides. Several organic constituents were detected in one or more samples, including 4,4-DDT, but concentration levels were well below DEQ's risk-based concentrations for the exposure pathways that apply to this property. Metal concentrations were detected below DEQ's regional background concentrations.

The groundwater samples collected by Alpine from the two test pits that yielded water were analyzed for the same list of analytes as soil samples using appropriate laboratory methods. Several chemical constituents were detected, but none exceeded DEQ's applicable risk-based concentrations for the site. Elevated levels of arsenic and lead reported in groundwater samples are likely biased high given the turbidity observed in the samples.

---

[1] May 17, 2017, Alpine Environmental Consultants LLC. Concrete Pit Environmental Investigation, City of Ashland.

Available at: https://tinyurl.com/AshlandQuarryReport

In summary, DEQ agrees with Alpine's (2017) finding that there is no evidence that containers of hazardous substances were dumped at the Ashland quarry. Alpine's observations of visqueen plastic and low levels of certain petroleum hydrocarbon constituents at the property suggest that petroleum-contaminated soil may have been imported to the quarry for treatment in the past. Regardless, the concentrations detected in one or more soil samples are well below levels that could pose a risk to human health or the environment.

DEQ concludes that there are no unacceptable environmental risks evident at the Ashland quarry, and that future use of property by city employees for a municipal drinking water treatment plant would not pose a human health risk. DEQ will update its Environmental Cleanup Site Information database to reflect this finding.

APPENDIX E

Pairwise Comparison

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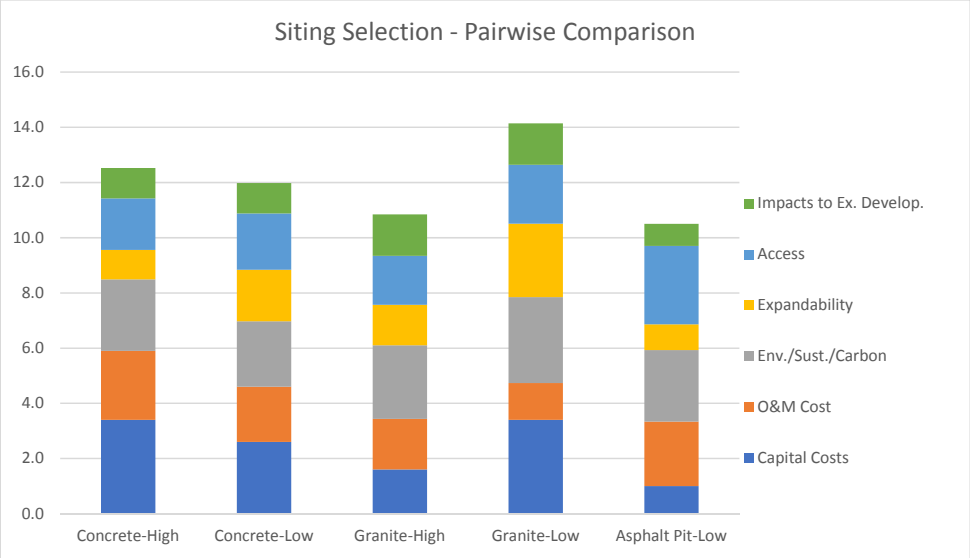
217002/b/17-131

Pairwise Comparison

Results

	Capital Costs	O&M Cost	Env./Sust./Carbon	Expandability	Access	Impacts to Ex. Develop.	**Total**
Concrete-High	3.40	2.50	2.59	1.07	1.87	1.10	**12.53**
Concrete-Low	2.60	2.00	2.37	1.87	2.04	1.10	**11.98**
Granite-High	1.60	1.83	2.67	1.47	1.78	1.50	**10.84**
Granite-Low	3.40	1.33	3.11	2.67	2.13	1.50	**14.14**
Asphalt Pit-Low	1.00	2.33	2.59	0.93	2.84	0.80	**10.50**
							60.00



Criteria Weighting

	Capital Costs	O&M Cost	Env./Sust./Carbon	Expandability	Access	Impacts to Ex. Develop.	Totals	**Weights (%)**
Capital Costs	0	4	2	4	4	4	18	**20%**
O&M Cost	2	0	2	4	3	4	15	**17%**
Env./Sust./Carbon	4	4	0	4	3	5	20	**22%**
Expandability	2	2	2	0	2	4	12	**13%**
Access	2	3	3	4	0	4	16	**18%**
Impacts to Ex. Develop.	2	2	1	2	2	0	9	**10%**
							90	100%

Criteria Results

Capital Costs

	Concrete-High	Concrete-Low	Granite-High	Granite-Low	Asphalt Pit-Low	**Totals**
Concrete-High	0.0	4.0	5.0	3.0	5.0	**17.0**
Concrete-Low	2.0	0.0	4.0	2.0	5.0	**13.0**
Granite-High	1.0	2.0	0.0	1.0	4.0	**8.0**
Granite-Low	3.0	4.0	5.0	0.0	5.0	**17.0**
Asphalt Pit-Low	1.0	1.0	2.0	1.0	0.0	**5.0**
						60.0

Expandability

	Concrete-High	Concrete-Low	Granite-High	Granite-Low	Asphalt Pit-Low	**Totals**
Concrete-High	0.0	1.0	2.0	1.0	4.0	**8.0**
Concrete-Low	5.0	0.0	4.0	1.0	4.0	**14.0**
Granite-High	4.0	2.0	0.0	1.0	4.0	**11.0**
Granite-Low	5.0	5.0	5.0	0.0	5.0	**20.0**
Asphalt Pit-Low	2.0	2.0	2.0	1.0	0.0	**7.0**
						60.0

O&M Costs

	Concrete-High	Concrete-Low	Granite-High	Granite-Low	Asphalt Pit-Low	**Totals**
Concrete-High	0.0	4.0	3.0	5.0	3.0	**15.0**
Concrete-Low	2.0	0.0	2.0	5.0	3.0	**12.0**
Granite-High	3.0	4.0	0.0	1.0	3.0	**11.0**
Granite-Low	1.0	1.0	5.0	0.0	1.0	**8.0**
Asphalt Pit-Low	3.0	3.0	3.0	5.0	0.0	**14.0**
						60.0

Access

	Concrete-High	Concrete-Low	Granite-High	Granite-Low	Asphalt Pit-Low	**Totals**
Concrete-High	0.0	2.5	3.5	2.5	2.0	**10.5**
Concrete-Low	3.5	0.0	3.0	3.0	2.0	**11.5**
Granite-High	2.5	3.0	0.0	2.5	2.0	**10.0**
Granite-Low	3.5	3.0	3.5	0.0	2.0	**12.0**
Asphalt Pit-Low	4.0	4.0	4.0	4.0	0.0	**16.0**
						60.0

Env. Sust. Carbon

	Concrete-High	Concrete-Low	Granite-High	Granite-Low	Asphalt Pit-Low	**Totals**
Concrete-High	0.0	3.3	2.7	2.7	3.0	**11.7**
Concrete-Low	2.7	0.0	2.3	2.7	3.0	**10.7**
Granite-High	3.3	3.7	0.0	1.7	3.3	**12.0**
Granite-Low	3.3	3.3	4.3	0.0	3.0	**14.0**
Asphalt Pit-Low	3.0	3.0	2.7	3.0	0.0	**11.7**
						60.0

Impacts to Existing Development

	Concrete-High	Concrete-Low	Granite-High	Granite-Low	Asphalt Pit-Low	Totals
Concrete-High	0.0	3.0	2.0	2.0	4.0	**11.0**
Concrete-Low	3.0	0.0	2.0	2.0	4.0	**11.0**
Granite-High	4.0	4.0	0.0	3.0	4.0	**15.0**
Granite-Low	4.0	4.0	3.0	0.0	4.0	**15.0**
Asphalt Pit-Low	2.0	2.0	2.0	2.0	0.0	**8.0**
						60.0

APPENDIX F

Cost Estimations

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217002/b/17-131

Last Updated: 7/13/2017

City of Ashland, Siting Study, Comparitive Cost Estimate

Tailrace Connection

	Concrete High	Concrete Low	Granite High	Granite Low	Asphalt Pit
Site (Ex/Backfill, Rdwys, Rtng Wall, Site Imp.)	$ 2,951,000	$ 3,248,000	$ 4,140,000	$ 2,704,000	$ 4,625,000
Offsite & Major Yard Piping / Offsite PRV Station[1]	$ 3,399,000	$ 3,727,000	$ 4,194,000	$ 3,692,000	$ 3,405,000
Pumping Station	$ 888,000	$ 1,462,000	$ 888,000	$ 888,000	$ 1,615,000
Subtotal	***$ 7,238,000***	***$ 8,437,000***	***$ 9,222,000***	***$ 7,284,000***	***$ 9,645,000***
Contingency (30%)	$ 2,172,000	$ 2,532,000	$ 2,767,000	$ 2,186,000	$ 2,894,000
Construction Costs	***$ 9,410,000***	***$ 10,969,000***	***$ 11,989,000***	***$ 9,470,000***	***$ 12,539,000***
Engineering (23%)	$ 2,165,000	$ 2,523,000	$ 2,758,000	$ 2,179,000	$ 2,884,000
Total Project	**$ 11,575,000**	**$ 13,492,000**	**$ 14,747,000**	**$ 11,649,000**	**$ 15,423,000**
Pumping Power Costs (20 yrs)[2]	$ 104,000	$ 512,000	$ 92,000	$ 98,000	$ 1,052,000
20-Year Comparative Life Cost	**$ 11,679,000**	**$ 14,004,000**	**$ 14,839,000**	**$ 11,747,000**	**$ 16,475,000**

1. Concrete Low and Asphalt Pit sites include small PRV station.

2.Pumping cost based on a 4 MGD flow 24/7.

Additional Penstock Connection

	Concrete High		Granite High		
PRV & Controls	$ 383,760		$ 383,760		
Additional Headtank	$ 388,557		$ 388,557		
Reduced Pump Station Cost	$ (1,419,912)	Capital (Savings)	$ (1,419,912)		
Pumping Power Costs (20 yrs)[1]	$ -	$ (647,595)	$ -		
Pumping Power Savings	$ (104,000)		$ (92,000)		
Pump Replacement Savings (year 20)	$ (285,000)		$ (285,000)		
Lost Production[2] (20 yrs)	$ 1,272,000	Added Power	$ 1,272,000		
Total	**$ 235,405**	$ 883,000	**$ 247,405**		
	$ 11,914,405				

1. Assumes pumping is required 15% of the year.

2. Loss Energy Production is based on 4 MGD flow to Plant with Average flows through the Penstock (8.4 MGD).
It is also based on an assumed level in Reeder of 2,876 feet and a power generation eff. of 80%.

Conventional Plant

	Concrete High	Concrete Low	Granite High	Granite Low	Asphalt Pit
Added Pumping Power Costs (20 yrs)[1]	$ 436,000	$ 956,000	$ 448,000	$ 1,018,000	$ 936,000

1. Added pumping cost based on a flow of 4 MGD.

Note -- These costs do not include added cost for disinfection volume or site development

KELLER associates

131 SW 5th Avenue, Suite A Meridian, Idaho 83642
Phone: 208.288.1992 - Fax: 509.295.6104

Project: Ashland Water Treatment Plant
Site Concrete High
Engineer: Jeremy Wilson
Client: City of Ashland
Printed: July 14, 2017

ENGINEER'S OPINION OF MOST PROBABLE CONSTRUCTION COST

Site

ITEM	QUANTITY	UNITS	UNIT PRICE	COST
Excavation soil	4416	CY	$ 50.00	$ 220,800
Excavation rock	10305	CY	$ 130.00	$ 1,339,650
Fill	27919	CY	$ 35.00	$ 977,165
Clearing and Grubbing	2	Acres	$ 25,000.00	$ 60,000
Concrete Debris Removal (2370 CY)	1	LS	$ 50,000.00	$ 50,000
Asphalt	686	Ton	$ 125.00	$ 85,750
Aggregate	1358	CY	$ 35.00	$ 47,530
Concrete	232	CY	$ 300.00	$ 69,600
Retaining Wall (12' tall)	0	LF	$ 900.00	$ -
Utilities (Electrical, communications)	1	LS	$100,000.00	$ 100,000
			subtotal	***$ 2,951,000***

Yard Piping

ITEM	QUANTITY	UNITS	UNIT PRICE	COST
Waste Line (8")	2,245	LF	$ 110	$ 247,000
Raw Water Line (30")	1,475	LF	$ 505	$ 745,000
Finished Water Line (30")	2,315	LF	$ 505	$ 1,170,000
Crowson II Overflow (16")	1,960	LF	$ 170	$ 334,000
Rock Excavation	5,655	CY	$ 130	$ 736,000
Natual Gas	1	LS	$ 167,000	$ 167,000
			subtotal	***$ 3,399,000***

CONCRETE HIGH TOTAL = $ 6,350,000

Assumptions

1 Cost estimate is not all inclusive. This is intended to illustrate the differences between the sites.

The opinion of most probable cost herein is based on our perception of current conditions at the project location. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. Keller Associates has no control over variances in the cost of labor, materials, equipment, services provided by others, contractor's methods of determining prices, competitive bidding or market conditions, practices or bidding strategies. Keller Associates cannot and does not warrant or guarantee that proposals, bids or actual construction costs will not vary from the costs presented herein.

KELLER associates

131 SW 5th Avenue, Suite A Meridian, Idaho 83642
Phone: 208.288.1992 - Fax: 509.295.6104

Project: Ashland Water Treatment Plant
Site Concrete Low
Engineer: Jeremy Wilson
Client: City of Ashland
Printed: July 14, 2017

ENGINEER'S OPINION OF MOST PROBABLE CONSTRUCTION COST

Site

ITEM	QUANTITY	UNITS	UNIT PRICE	COST
Excavation soil	6156	CY	$ 50.00	$ 307,815
Excavation rock	14364	CY	$ 130.00	$ 1,867,320
Fill	21062	CY	$ 35.00	$ 737,170
Clearing and Grubbing	2.7	Acres	$ 25,000.00	$ 67,500
Concrete Debris Removal (2370 CY)	1	LS	$ 50,000.00	$ 50,000
Asphalt	567	Ton	$ 125.00	$ 70,875
Aggregate	1336	CY	$ 35.00	$ 46,760
Concrete	0	CY	$ 300.00	$ -
Retaining Wall (12' tall)	0	LF	$ 900.00	$ -
Utilities (Electrical, communications)	1	LS	$ 100,000.00	$ 100,000
			subtotal	***$ 3,248,000***

Yard Piping

ITEM	QUANTITY	UNITS	UNIT PRICE	COST
Waste Line (8")	1,660	LF	$ 110.00	$ 183,000
Raw Water Line (30")	885	LF	$ 505.00	$ 447,000
Finished Water Line (30")	3,475	LF	$ 505.00	$ 1,755,000
Crowson II Overflow (16")	2,135	LF	$ 170.00	$ 363,000
Rock Excavation	5,475	CY	$ 130.00	$ 712,000
Natual Gas	1	LS	$ 167,000.00	$ 167,000
Raw Water PRV Station	1	LS	$ 100,000.00	$ 100,000
			subtotal	***$ 3,727,000***

CONCRETE LOW TOTAL = $ 6,975,000

Assumptions

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KELLER associates

131 SW 5th Avenue, Suite A Meridian, Idaho 83642
Phone: 208.288.1992 - Fax: 509.295.6104

Project: Ashland Water Treatment Plant
Site Granite High
Engineer: Jeremy Wilson
Client: City of Ashland
Printed: July 14, 2017

ENGINEER'S OPINION OF MOST PROBABLE CONSTRUCTION COST

Site

ITEM	QUANTITY	UNITS	UNIT PRICE	COST
Excavation soil	9048	CY	$ 50.00	$ 452,400
Excavation rock	21112	CY	$ 130.00	$ 2,744,560
Fill	6830	CY	$ 35.00	$ 239,050
Clearing and Grubbing	3.2	Acres	$ 25,000.00	$ 80,000
Concrete Debris Removal (2370 CY)	0	LS	$ 50,000.00	$ -
Asphalt	686	Ton	$ 125.00	$ 85,750
Aggregate	1087	CY	$ 35.00	$ 38,045
Concrete	0	CY	$ 300.00	$ -
Retaining Wall (12' tall)	0	LF	$ 900.00	$ -
Utilities (Electrical, Communications)	1	LS	$ 100,000.00	$ 100,000
Culvert Crossing	1	LS	$ 400,000.00	$ 400,000
			subtotal	***$ 4,140,000***

Yard Piping

ITEM	QUANTITY	UNITS	UNIT PRICE	COST
Waste Line (8")	1,655	LF	$ 110	$ 183,000
Raw Water Line (30")	1,690	LF	$ 505	$ 854,000
Finished Water Line (30")	2,680	LF	$ 505	$ 1,354,000
Crowson II Overflow (16")	1,980	LF	$ 170	$ 337,000
Rock Excavation	9,775	CY	$ 130	$ 1,271,000
Natual Gas	1	LS	$ 135,000	$ 135,000
Ashland Creek Crossing (30")	2	EA	$ 24,000	$ 48,000
Ashland Creek Crossing (8")	1	EA	$ 12,000	$ 12,000
			subtotal	***$ 4,194,000***

GRANITE HIGH TOTAL = $ 8,334,000

Assumptions

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KELLER associates

131 SW 5th Avenue, Suite A Meridian, Idaho 83642
Phone: 208.288.1992 - Fax: 509.295.6104

Project: Ashland Water Treatment Plant
Site Granite Low
Engineer: Jeremy Wilson
Client: City of Ashland
Printed: July 14, 2017

ENGINEER'S OPINION OF MOST PROBABLE CONSTRUCTION COST

Site

ITEM	QUANTITY	UNITS	UNIT PRICE	COST
Excavation soil	21131	CY	$ 50.00	$ 1,057,000
Excavation rock	7044	CY	$ 130.00	$ 916,000
Fill	2702	CY	$ 35.00	$ 95,000
Clearing and Grubbing	1.6	Acres	$ 25,000.00	$ 40,000
Concrete Debris Removal (2370 CY)	0	LS	$ 50,000.00	$ -
Asphalt	480	Ton	$ 125.00	$ 60,000
Aggregate	1019	CY	$ 35.00	$ 36,000
Concrete	0	CY	$ 300.00	$ -
Retaining Wall (12' tall)	0	LF	$ 900.00	$ -
Utilities (Electrical, Communications)	1	LS	$100,000.00	$ 100,000
Culvert Crossing	1	LS	$400,000.00	$ 400,000
			subtotal	***$ 2,704,000***

Yard Piping

ITEM	QUANTITY	UNITS	UNIT PRICE	COST
Waste Line (8")	955	LF	$ 110	$ 105,000
Raw Water Line (30")	950	LF	$ 505	$ 480,000
Finished Water Line (30")	3,035	LF	$ 505	$ 1,533,000
Crowson II Overflow (16")	1,855	LF	$ 170	$ 315,000
Rock Excavation	8,185	CY	$ 130	$ 1,064,000
Natual Gas	1	LS	$ 135,000	$ 135,000
Ashland Creek Crossing (30")	2	EA	$ 24,000	$ 48,000
Ashland Creek Crossing (8")	1	EA	$ 12,000	$ 12,000
			subtotal	***$ 3,692,000***

GRANITE LOW TOTAL = $ 6,396,000

Assumptions

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KELLER associates

131 SW 5th Avenue, Suite A Meridian, Idaho 83642
Phone: 208.288.1992 - Fax: 509.295.6104

Project: Ashland Water Treatment Plant
Site Asphalt Pit
Engineer: Jeremy Wilson
Client: City of Ashland
Printed: July 14, 2017

ENGINEER'S OPINION OF MOST PROBABLE CONSTRUCTION COST

Site

ITEM	QUANTITY	UNITS	UNIT PRICE	COST
Excavation soil	10683	CY	$ 50.00	$ 534,150
Excavation rock	24927	CY	$ 130.00	$ 3,240,510
Fill	4252	CY	$ 35.00	$ 148,820
Clearing and Grubbing	2.4	Acres	$ 25,000.00	$ 60,000
Concrete Debris Removal (2370 CY)	0	LS	$ 50,000.00	$ -
Asphalt	470	Ton	$ 125.00	$ 58,750
Aggregate	914	CY	$ 35.00	$ 31,990
Concrete	0	CY	$ 300.00	$ -
Retaining Wall (20' tall)	265	LF	$ 1,700.00	$ 450,500
Utilities (Electrical, communications)	1	LS	$ 100,000.00	$ 100,000
			subtotal	***$ 4,625,000***

Yard Piping

ITEM	QUANTITY	UNITS	UNIT PRICE	COST
Waste Line (8")	1,370	LF	$ 110.00	$ 151,000
Raw Water Line (30")	120	LF	$ 505.00	$ 61,000
Finished Water Line (30")	4,185	LF	$ 505.00	$ 2,114,000
Crowson II Overflow (16")	1,770	LF	$ 170.00	$ 301,000
Rock Excavation	5,045	CY	$ 130.00	$ 656,000
Natual Gas	1	LS	$ 22,000.00	$ 22,000
Raw Water PRV Station	1	LS	$ 100,000.00	$ 100,000
			subtotal	***$ 3,405,000***

ASPHALT PIT TOTAL = $ 8,030,000

Assumptions

1 Cost estimate is not all inclusive. This is intended to illustrate the differences between the sites.

The opinion of most probable cost herein is based on our perception of current conditions at the project location. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. Keller Associates has no control over variances in the cost of labor, materials, equipment, services provided by others, contractor's methods of determining prices, competitive bidding or market conditions, practices or bidding strategies. Keller Associates cannot and does not warrant or guarantee that proposals, bids or actual construction costs will not vary from the costs presented herein.

City of Ashland
Estimated Capital Cost for Intermitent Pumping Station

High Plants and Granite Low				
Item	**Quantity**	**Units**	**Unit Price**	**Total**
Additional Building Space	600	SF	$ 300	$ 180,000
Pumps	3	EA	$ 95,000	$ 285,000
VFD	3	EA	$ 14,000	$ 42,000
Mechanical piping	1	LS	$ 75,000	$ 75,000
HVAC	1	LS	$ 10,000	$ 10,000
Plumbing	1	LS	$ 10,000	$ 10,000
Pump Station Electrical & Controls	1	LS	$ 121,000	$ 121,000
Connection to Penstock				
PRVs		EA	$ 50,000	$ -
Isolation Butterfly Valves		EA	$ 20,000	$ -
Vault		EA	$ 20,000	$ -
Electrical/Controls		LS	$ 25,000	$ -
Pressure Relief & Piping		LS	$ 15,000	$ -
Subtotal				***$ 723,000***
Mobilization	10%			$ 73,000
Contractor Overhead and Profit	11.5%			$ 92,000
Contingency	0%			$ -
Total Pre-Contingency Construction Cost				**$ 888,000**

Concrete Low WTP				
Item	**Quantity**	**Units**	**Unit Price**	**Total**
Additional Building Space	600	SF	$ 300	$ 180,000
Pumps	4	EA	$ 110,000	$ 440,000
VFD	4	EA	$ 11,000	$ 44,000
Mechanical piping	1	LS	$ 100,000	$ 100,000
HVAC	1	LS	$ 10,000	$ 10,000
Plumbing	1	LS	$ 10,000	$ 10,000
Pump Station Electrical & Controls	1	LS	$ 157,000	$ 157,000
PRVs	1	LS	$ 100,000	$ 100,000
Additional Standpipe Cost	1	LS	$ 150,000	$ 150,000
Subtotal				***$ 1,191,000***
Mobilization	10%			$ 120,000
Contractor Overhead and Profit	11.5%			$ 151,000
Contingency	0%			$ -
Total Pre-Contingency Construction Cost				**$ 1,462,000**

Asphalt Pit WTP				
Item	**Quantity**	**Units**	**Unit Price**	**Total**
Total Pre-Contingency Construction Cost	600	SF	$ 300	$ 180,000
Pumps	4	EA	$ 130,000	$ 520,000
VFD	4	EA	$ 17,000	$ 68,000
Mechanical piping	1	LS	$ 100,000	$ 100,000
HVAC	1	LS	$ 10,000	$ 10,000
Plumbing	1	LS	$ 10,000	$ 10,000
Pump Station Electrical & Controls	1	LS	$ 178,000	$ 178,000
PRVs	1	LS	$ 100,000	$ 100,000
Additional Standpipe Cost	1	LS	$ 150,000	$ 150,000
Subtotal				***$ 1,316,000***
Mobilization	10%			$ 132,000
Contractor Overhead and Profit	11.5%			$ 167,000
Contingency	0%			$ -
Total Pre-Contingency Construction Cost				**$ 1,615,000**

Ashland WTP

Estimated Pumping/Energy Costs at Each Site With Tailrace Connection & 24" TID Line

Assumes on Low Plants Head is not Broken, and the 30" Finish line is being used as the new raw water supply.

Location	Elevation (ft)
Elevation of Tailrace	2,454.0
Concrete High Plant	2,375.0
Concrete Low Plant	2,280.0
Granite Knoll Plant	2,375.0
Granite Low Plant	2,316.0
Asphalt Low Plant	2,227.0

Power Cost

KW-hr $ 0.08

	Concrete High Site				
Flow (MGD)	**2.5**	**4.0**	**7.5**	**10.0**	
Arriving Pressure (psi)	34.0	33.0	31.0	30.0	
Arriving HGL (ft)	2,454	2,451	2,447	2,444	**Max HL (dirty) - 10 MGD**
Average Headloss through Plant (ft)	44				76
Typ. HGL Leaving the Plant (ft)	2,410	2,407	2,403	2,400	2,368
Crowson II HGL (ft)	2,425.4	2,426.6	2,428.4	2,430.8	2,430.8
Needed Pumping Head (ft)	16	19	26	31	63
Calculated HP (add'l lift)	7	14	34	54	110
Wire to Water Eff.	75%				**Max HP (pump Sizing)**
Actual HP (typical)	9	18	45	71	146
Actual KW	7	14	34	53	
Annual KW-hr	60,622	118,462	295,734	466,324	
Annual Cost	**$ 4,900**	**$ 9,500**	**$ 23,700**	**$ 37,400**	
% volume delivered above 50% capacity	54%	54%	54%	54%	
Power Cost Tailrace Connection	**$ 2,664**	**$ 5,200**	**$ 12,887**	**$ 20,336**	
Dual Connection - Power loss at Power Plant		$ 63,600			

	Granite Low Site				
Flow (MGD)	**2.5**	**4.0**	**7.5**	**10.0**	
Arriving Pressure (psi)	59.0	59.0	58.0	57.0	
Arriving HGL (ft)	2,452	2,452	2,450	2,448	**Max HL (dirty) - 10 MGD**
Average Headloss through	44				76
HGL Leaving the Plant (ft)	2,408	2,408	2,406	2,404	2,372
Crowson II HGL (ft)	2,425.4	2,426.6	2,428.5	2,430.9	2,431
Needed Pumping Head (ft)	17	18	23	27	59
Calculated HP (add'l lift)	8	13	30	48	104
Wire to Water Eff.	75%				**Max HP (pump Sizing)**
Actual HP	10	17	40	64	139
Actual KW	7	13	29	48	
Annual KW-hr	65,400	111,979	258,237	416,328	
Annual Cost	**$ 5,300**	**$ 9,000**	**$ 20,700**	**$ 33,400**	
% volume delivered above	54%	54%	54%	54%	
Power Cost Tailrace Conne	**$ 2,882**	**$ 4,900**	**$ 11,256**	**$ 18,161**	

	Concrete Low Site				
Flow (MGD)	**2.5**	**4.0**	**7.5**	**10.0**	
Arriving Pressure (psi)	75.0	74.0	73.0	71.0	
Arriving HGL (ft)	2,453	2,451	2,449	2,444	
Head Cut by PRV (ft)	35	32	30	25	
HGL at Membranes (ft) (60 psi at membranes)	2,419	2,419	2,419	2,419	**Max HL (dirty) - 10 MGD**
Average Headloss through Plant (ft)	44				76
HGL Leaving the Plant (ft)	2,375	2,375	2,375	2,375	2,343
Crowson II HGL (ft)	2,425.5	2,426.9	2,429.1	2,432.0	2,432
Needed Pumping Head (ft)	51	52	55	57	89
Calculated HP (add'l lift)	22	37	72	101	157
Wire to Water Eff.	75%				**Max HP (pump Sizing)**
Actual HP	30	49	96	134	209
Actual KW	22	37	71	100	
Annual KW-hr	194,557	319,853	624,951	877,607	
Annual Cost	**$ 15,600**	**$ 25,600**	**$ 50,000**	**$ 70,300**	
% volume delivered above 50% capacity	100%	100%	100%	100%	
Power Cost Tailrace Connection	**$ 15,600**	**$ 25,600**	**$ 50,000**	**$ 70,300**	

	Asphalt Pit Site				
Flow (MGD)	**2.5**	**4.0**	**7.5**	**10.0**	
Arriving Pressure (psi)	98.0	97.0	95.0	94.0	
Arriving HGL (ft)	2,453	2,451	2,446	2,444	
Head Cut by PRV (ft)	88	85	81	79	
HGL at Membranes (ft) (60 p	2,366	2,366	2,366	2,366	**Max HL (dirty) - 10 MGD**
Average Headloss through	44				76
HGL Leaving the Plant (ft)	2,322	2,322	2,322	2,322	2,290
Crowson II HGL (ft)	2,429				2,429
Needed Pumping Head (ft)	107	107	107	107	139
Calculated HP (add'l lift)	47	75	141	189	245
Wire to Water Eff.	75%				**Max HP (pump Sizing)**
Actual HP	63	101	189	251	326
Actual KW	47	75	141	187	
Annual KW-hr	410,518	656,829	1,231,555	1,642,073	
Annual Cost	**$ 32,900**	**$ 52,600**	**$ 98,600**	**$ 131,400**	
% volume delivered above	100%	100%	100%	100%	
Power Cost Tailrace Conne	**$ 32,900**	**$ 52,600**	**$ 98,600**	**$ 131,400**	

	Granite High Site				
Flow (MGD)	**2.5**	**4.0**	**7.5**	**10.0**	
Arriving Pressure (psi)	34.0	34.0	32.0	31.0	
Arriving HGL (ft)	2,454	2,454	2,449	2,447	**Max HL (dirty) - 10 MGD**
Average Headloss through Plant (ft)	44				76
Typical HGL Leaving the Plant (ft)	2,410	2,410	2,405	2,403	2,371
Crowson II HGL (ft)	2,425.4	2,426.5	2,428.1	2,430.4	2,430
Needed Pumping Head (ft)	16	17	23	28	60
Calculated HP (add'l lift)	7	12	31	49	105
Wire to Water Eff.	75%				**Max HP (pump Sizing)**
Actual HP	9	16	41	65	140
Actual KW	7	12	30	48	
Annual KW-hr	60,622	103,478	266,034	424,126	
Annual Cost	**$ 4,900**	**$ 8,300**	**$ 21,300**	**$ 34,000**	
% volume delivered above 50% capacity	54%	54%	54%	54%	
Power Cost Tailrace Connection	**$ 2,664**	**$ 4,600**	**$ 11,582**	**$ 18,487**	
Dual Connection - Loss Power at Power plant		**$ 63,600**			

Ashland WTP

Estimated Pumping/Energy Costs at Each Site with Connection to Penstock for High Plants and Conventional Plant at all Plants

Site	Concrete High & Granite Knoll	Concrete Low Convetional	Granite Low Convetional	Asphalt Pit Convent	Concrete High & Granite Knoll Conventional
Max Pumping Req'd (ft)	26.0	150	114	203	55
Max Pumping Req'd (psi)	11.3	64.9	49.4	87.9	23.8
% of Time Pumping is Req'd	15%	100%	100%	100%	100%

	Concrete High Site / Granite Knoll Site			
Flow (MGD)	2	3.5	4	10
Max Calculated HP (add'l lift)	9	16	18	46
Wire to Water Eff.	75%			
Actual HP	12	21	24	61
Max Actual KW	9	16	18	45
Est. Annual KW-hr	11,943	20,901	23,887	59,717
Annual Cost	$ 1,000	$ 1,700	$ 2,000	$ 4,800

	Granite Low Site Conventional Plant			
Flow (MGD)	2	3.5	4	10
Calculated HP	40	70	80	200
Wire to Water Eff.	75%			
Actual HP	53	93	107	267
Actual KW	40	70	80	199
Annual KW-hr	348597	610044	697193	1742983
Annual Cost	$ 27,900	$ 48,900	$ 55,800	$ 139,500

	Concrete High Site / Granite Knoll Site Conventional Plant			
Flow (MGD)	2	3.5	4	10
Max Calculated HP (add'l lift)	19	34	39	97
Wire to Water Eff.	75%			
Actual HP	26	45	51	129
Max Actual KW	19	34	38	96
Est. Annual KW-hr	168,183	294,319	336,365	840,913
Annual Cost	$ 13,500	$ 23,600	$ 27,000	$ 67,300

	Asphalt Pit Conventional Plant			
Flow (MGD)	2	3.5	4	10
Calculated HP	71	125	143	356
Wire to Water Eff.	75%			
Actual HP	95	166	190	475
Actual KW	71	124	142	354
Annual KW-hr	620746	1086306	1241493	3103732
Annual Cost	$ 49,700	$ 87,000	$ 99,400	$ 248,300

	Concrete Low Site Conventional Plant			
Flow (MGD)	2	3.5	4	10
Calculated HP	53	92	105	263
Wire to Water Eff.	75%			
Actual HP	70	123	140	351
Actual KW	52	92	105	262
Annual KW-hr	458,680	802,689	917,359	2,293,398
Annual Cost	$ 36,700	$ 64,300	$ 73,400	$ 183,500

Site	Annual Cost @ 4 MGD Conventional Plant	20 Year Cost @ 4 MGD Conventional Plant
Concrete and Granite Knoll	$ 2,000	$ 40,000
Concrete Low Conventional	$ 73,400	$ 1,468,000
Granite Low Conventionl	$ 55,800	$ 1,116,000
Asphalt Conventional	$ 99,400	$ 1,988,000
Concrete & Granite Knoll Sit	$ 27,000	$ 540,000

APPENDIX G

Offsite Utilities

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217002/b/17-131

Document Path: P:\217002\z_GIS\Siting_Study_Maps\Concrete_High_Site_offsite_UtilitiesA.mxd

Project: City of Ashland - WTP Site Selection

Title: Concrete High Site Utilities

Figure: 7

Document Path: P:\217002\z_GIS\Siting_Study_Maps\Concrete_Low_Site_offsite_UtilitiesA.mxd

Project: City of Ashland - WTP Site Selection

Title: Concrete Low Site Utilities

Figure: 8

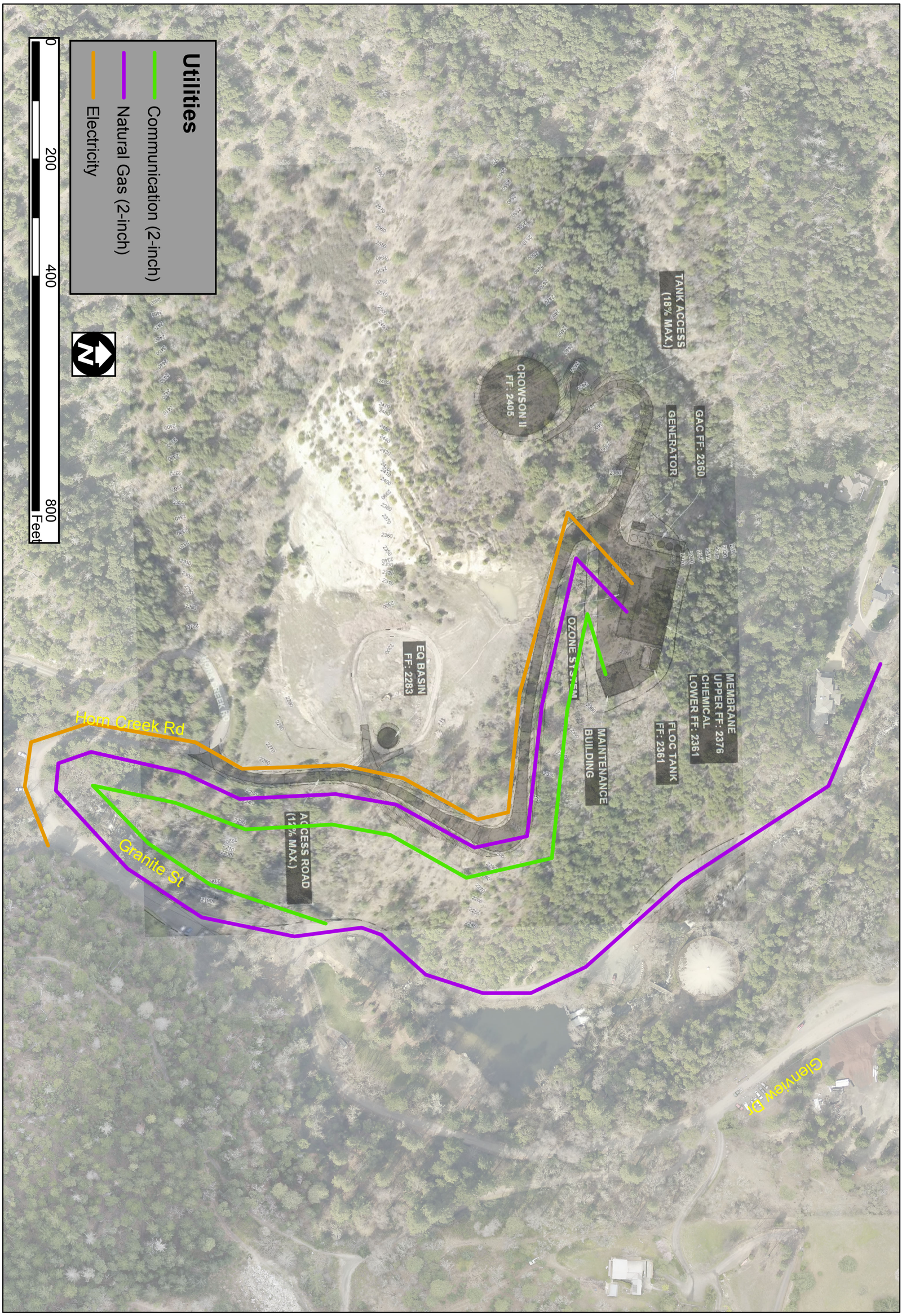
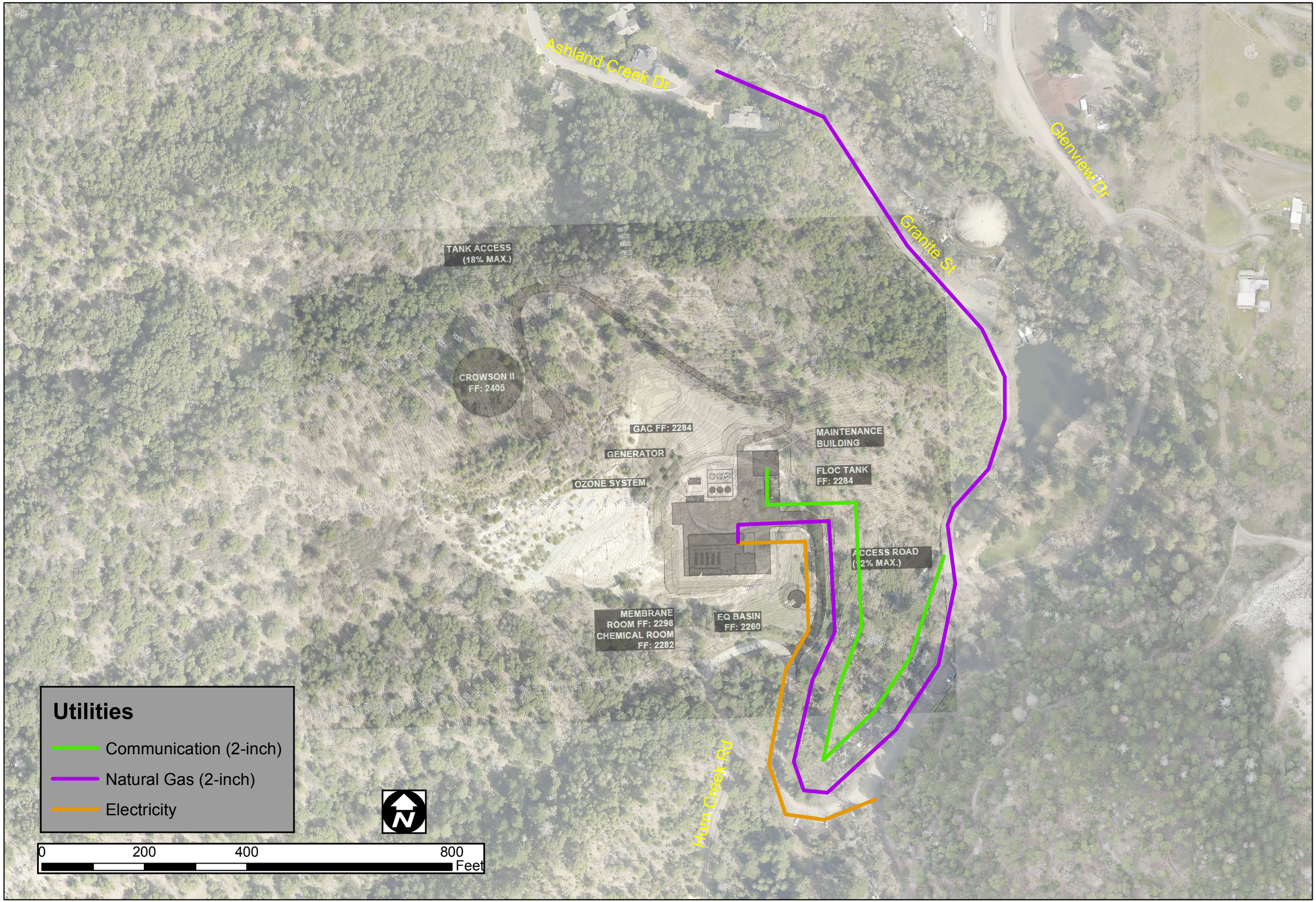


Figure:	Title:	Project:
9	Granite High Site Utilities	City of Ashland - WTP Site Selection



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Project: City of Ashland - WTP Site Selection

Title: Granite Low Site Utilities

Figure: 10

Document Path: P:\217002\z_GIS\Siting_Study_Maps\Asphalt_Pit_Offsite_UtilitiesA.mxd

Project: City of Ashland - WTP Site Selection

Title: Asphalt Pit Utilities

Figure: 6

APPENDIX H

Hydraulic Calculations

KELLER associates

Get there!

217002/b/17-131

Ashland - New WTP Hydraulic Analysis

Elevations

Tailrace	2,454.0
Crowson I Overflow	2,425.0
Terrace St Pump Station	2,308.2
Existing WTP	2,460.0
Penstock Head tank	2,521.0
Concrete High WTP Membranes	2,366.0
Concrete Low WTP Membranes	2,280.0
Granite High WTP Membranes	2,356.0
Granite Low WTP Membranes	2,316.0
Granite Knoll WTP Membranes	2,375.0
Asphalt Pit WTP	2,227.0

Ashland - New WTP Hydraulic Analysis

Headloss Through New WTP

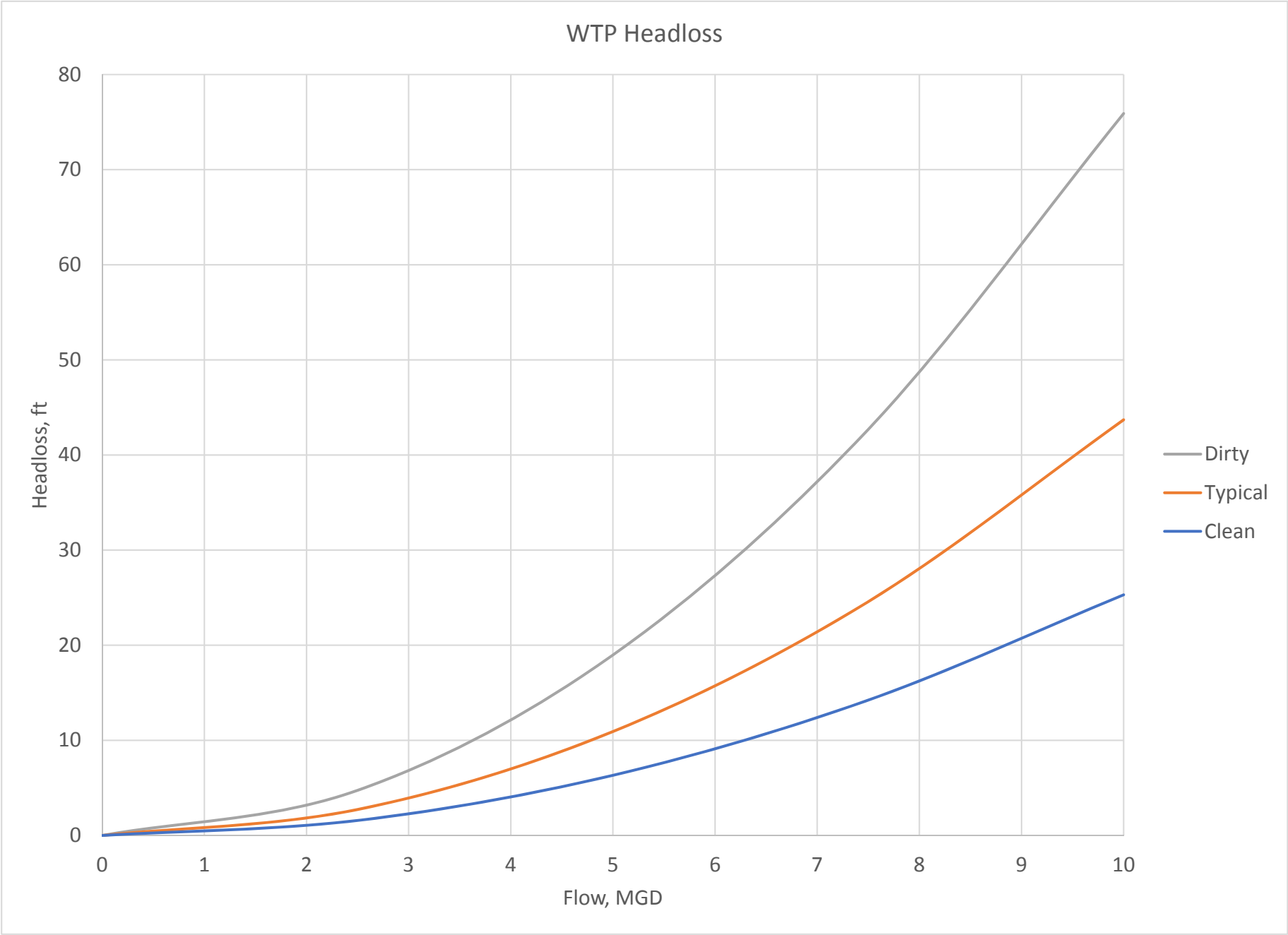
		Headloss		Clean	Typ	Dirty	
	Elev (ft)	ft	psi	psi	psi	psi	
Elevation of Tailrace	2,454.0						
Strainer HL		4.6	2	1	2	3	
Membrane HL		27.6	12	7	12	22	
GAC HL		11.5	5	3	5	8	
			subtotal	11	19	33	psi
			subtotal	25	44	76	head ft

Elev Tailwater 2,410.3

Crowson 1 Overflow 2,425.0

Note: Crowson 1 inlet would have motor actuated butterfly valve to keep it from spilling (not altitude valve)

New WTP Headloss (ft) at Various Flows and Plant Conditions					
Flow (MGD)	**0**	**2.5**	**5**	**7.5**	**10**
Clean	0	2	6	14	25
Typical	0	3	11	25	44
Dirty	0	5	19	43	76


WTP Headloss
Headloss, ft
80
70
60
50
40
30
20
10
0
0
1
2
3
4
5
6
7
8
9
10
Flow, MGD
Dirty
Typical
Clean

Ashland - New WTP Hydraulic Analysis

Concrete High Site Hydraulic Analysis

	TAILRACE					PENSTOCK				
Flow (MGD)	**0**	**2.5**	**5**	**7.5**	**10**	**0**	**2.5**	**5**	**7.5**	**10**
HL in RW supply (ft) (assumes 30" pipe)	0	0.2	2.5	4.8	9.5	0	0.2	2.5	4.8	9.5
HGL U/S WTP (ft)	2,454	2,454	2,451	2,449	2,445	2,521	2,521	2,518	2,516	2,512
Head U/S WTP (ft)	88	88	85	83	79	155	155	152	150	146
Pressure U/S WTP (psi)	38.1	38.0	37.0	36.0	34.0	67.1	67.0	66.0	65.0	63.0
HGL U/S WTP (ft)	2,454	2,454	2,451	2,449	2,445	2,521	2,521	2,518	2,516	2,512
Clipped pressure U/S WTP (psi)	38.1	38.0	37.0	36.0	34.0	65.0	65.0	65.0	65.0	63.0
Clipped HGL U/S WTP (ft)	2,454	2,454	2,451	2,449	2,445	2,516	2,516	2,516	2,516	2,512
Required Finished Water HGL (ft)	2,425	2,425.4	2,426.6	2,428.4	2,430.8	2,425	2,425.4	2,426.6	2,428.4	2,430.8
Available HL at Plant for GravFlow (ft)	29	28.4	24.9	20.8	13.7	91.2	90.8	89.6	87.8	80.7
Available HL at Plant for GravFlow (psi)	13	12	11	9	6	39	39	39	38	35

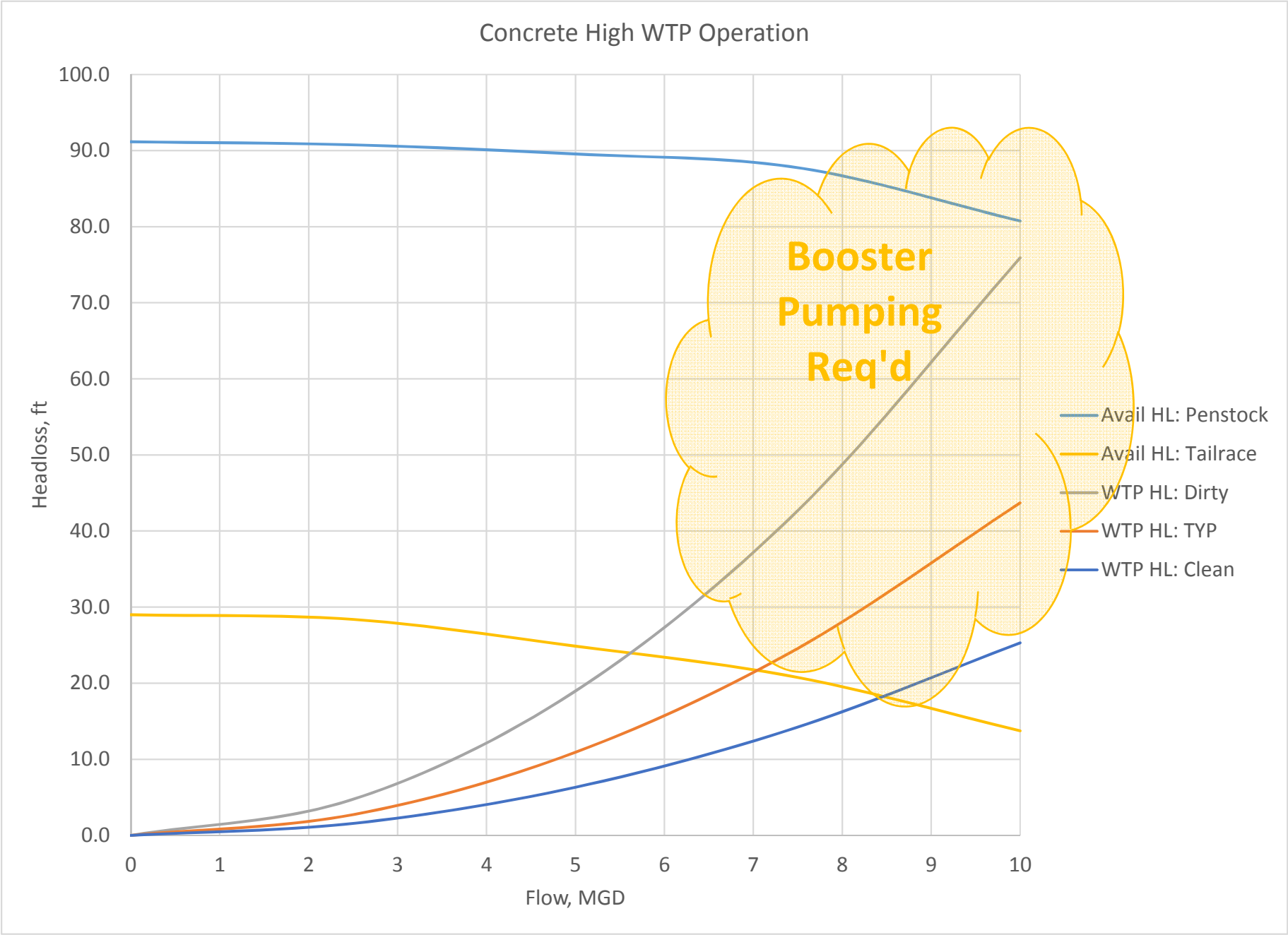
Supply Line Headloss				
Flow (MGD)	2.5	5	7.5	10
headlosses for 24" TID	1.9	7.0	15.0	25.6
For a 30" Line	0.2	2.5	4.8	9.5

Headloss - WTP to Crowson I and Required HGL leaving WTP				
Flow (MGD)	2.5	5	7.5	10
Headloss from Crowson I to WTP	0.4	1.6	3.4	5.8
Required HGL	2,425.4	2,426.6	2,428.4	2,430.8

65 psi max pressure allowed at Membranes

Elevations

Concrete High Membranes	2,366.0	ft
Penstock Headtank	2,521.0	ft
Tailrace	2,454.0	ft


Concrete High WTP Operation
Headloss, ft
100.0
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
0
1
2
3
4
5
6
7
8
9
10
Flow, MGD
Booster Pumping Req'd
Avail HL: Penstock
Avail HL: Tailrace
WTP HL: Dirty
WTP HL: TYP
WTP HL: Clean

Ashland - New WTP Hydraulic Analysis

Concrete Low Site Hydraulic Analysis

	TAILRACE					PENSTOCK				
Flow (MGD)	**0**	**2.5**	**5**	**7.5**	**10**	**0**	**2.5**	**5**	**7.5**	**10**
HL in RW supply (ft) (assumes 30" pipe)	0	0.8	3.1	5.4	7.7	0	0.8	3.1	5.4	7.7
HGL U/S WTP (ft)	2,454	2,453	2,451	2,449	2,446	2,521	2,520	2,518	2,516	2,513
Head U/S WTP (ft)	174	173	171	169	166	241	240	238	236	233
Pressure U/S WTP (psi)	75.3	75.0	74.0	73.0	72.0	104.3	104.0	103.0	102.0	101.0
HGL U/S WTP (ft)	2,454	2,453	2,451	2,449	2,446	2,521	2,520	2,518	2,516	2,513
Clipped pressure U/S WTP (psi)	65.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0
Clipped HGL U/S WTP (ft)	2,430	2,430	2,430	2,430	2,430	2,430	2,430	2,430	2,430	2,430
Required Finished Water HGL (ft)	2,425	2,426	2,427	2,429	2,432	2,425	2,426	2,427	2,429	2,432
Available HL at Plant for GravFlow (ft)	5.2	4.7	3.3	1.1	-1.8	5.2	4.7	3.3	1.1	-1.8
Available HL at Plant for GravFlow (psi)	2	2	1	0	-1	2	2	1	0	-1

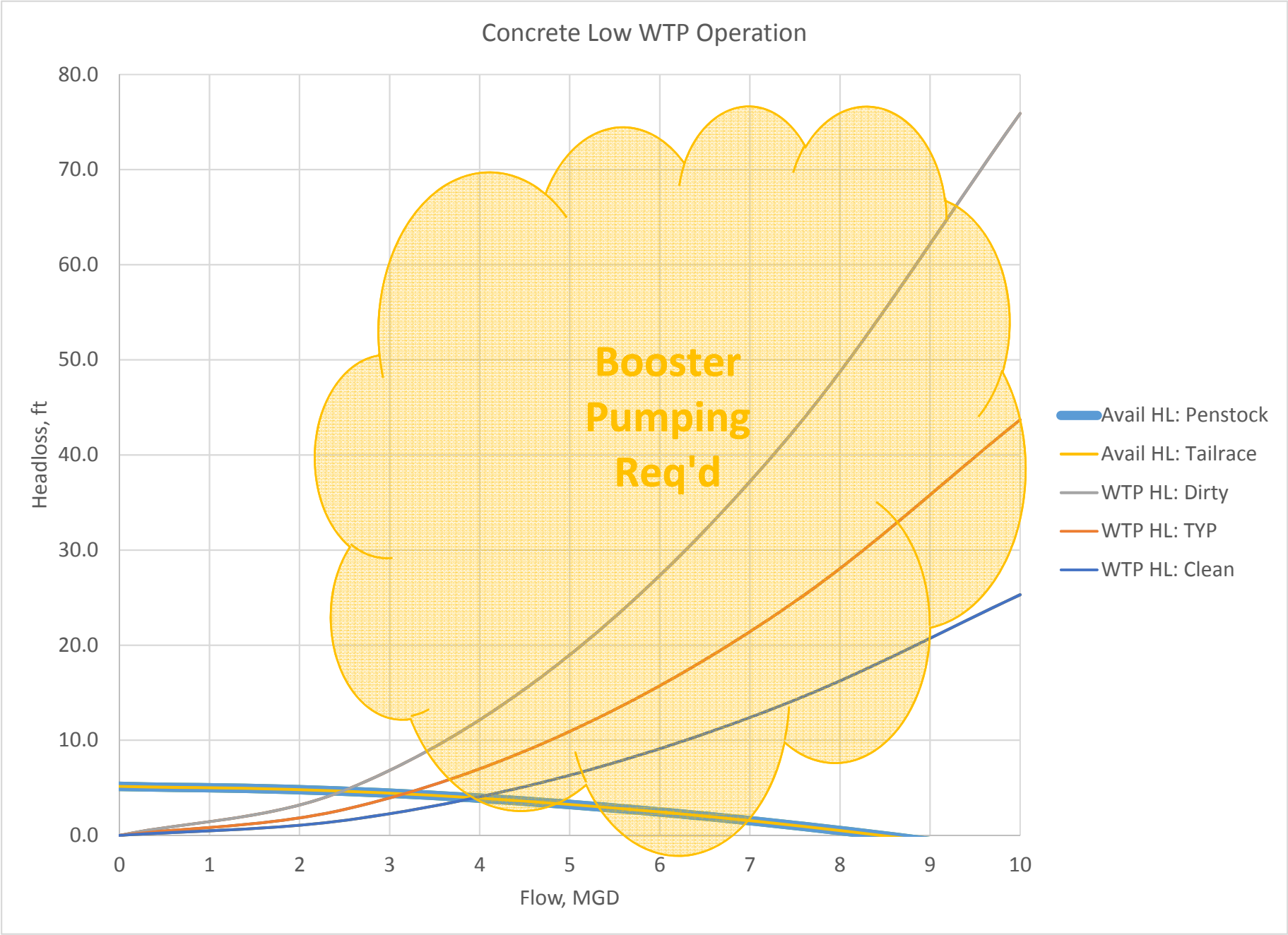
Supply Line Headloss				
Flow (MGD)	2.5	5	7.5	10
headlosses for 24" TID	1.9	6.8	14.4	24.7
For a 30" Line	0.8	3.1	5.4	7.7

Headloss - WTP to Crowson I and Required HGL leaving WTP				
Flow (MGD)	2.5	5	7.5	10
Headloss from WTP To Crowson II	0.5	1.9	4.1	7
Required Finish HGL	2,426	2,427	2,429	2,432

65 psi max pressure allowed at Membranes

Elevations

Concrete Low Membranes	2,280.0	ft
Penstock Headtank	2,521.0	ft
Tailrace	2,454.0	ft


Concrete Low WTP Operation
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
Headloss, ft
0
1
2
3
4
5
6
7
8
9
10
Flow, MGD
Booster Pumping Req'd
Avail HL: Penstock
Avail HL: Tailrace
WTP HL: Dirty
WTP HL: TYP
WTP HL: Clean

Ashland - New WTP Hydraulic Analysis

Granite High Site Hydraulic Analysis

	TAILRACE					PENSTOCK				
Flow (MGD)	**0**	**2.5**	**5**	**7.5**	**10**	**0**	**2.5**	**5**	**7.5**	**10**
HL in RW supply (ft) (assumes a 30" pipe)	0	0.5	0.5	5.1	7.4	0	0.5	0.5	5.1	7.4
HGL U/S WTP (ft)	2,454.0	2,453.5	2,453.5	2,448.9	2,446.6	2,521.0	2,520.5	2,520.5	2,515.9	2,513.6
Head U/S WTP (ft)	79.0	78.5	78.5	73.9	71.6	146.0	145.5	145.5	140.9	138.6
Pressure U/S WTP (psi)	34.2	34.0	34.0	32.0	31.0	63.2	63.0	63.0	61.0	60.0
HGL U/S WTP (ft)	2,454.0	2,453.5	2,453.5	2,448.9	2,446.6	2,521.0	2,520.5	2,520.5	2,515.9	2,513.6
Clipped pressure U/S WTP (psi)	34.2	34.0	34.0	32.0	31.0	63.2	63.0	63.0	61.0	60.0
Clipped HGL U/S WTP (ft)	2,454.0	2,453.5	2,453.5	2,448.9	2,446.6	2,521.0	2,520.5	2,520.5	2,515.9	2,513.6
Required Finished Water HGL (ft)	2,425.0	2,425.4	2,426.5	2,428.2	2,430.5	2,425.0	2,425.4	2,426.5	2,428.2	2,430.5
Available HL at Plant for GravFlow (ft)	29.0	28.14	27.04	20.72	16.11	96	95.14	94.04	87.72	83.1
Available HL at Plant for GravFlow (psi)	13	12	12	9	7	42	41	41	38	36

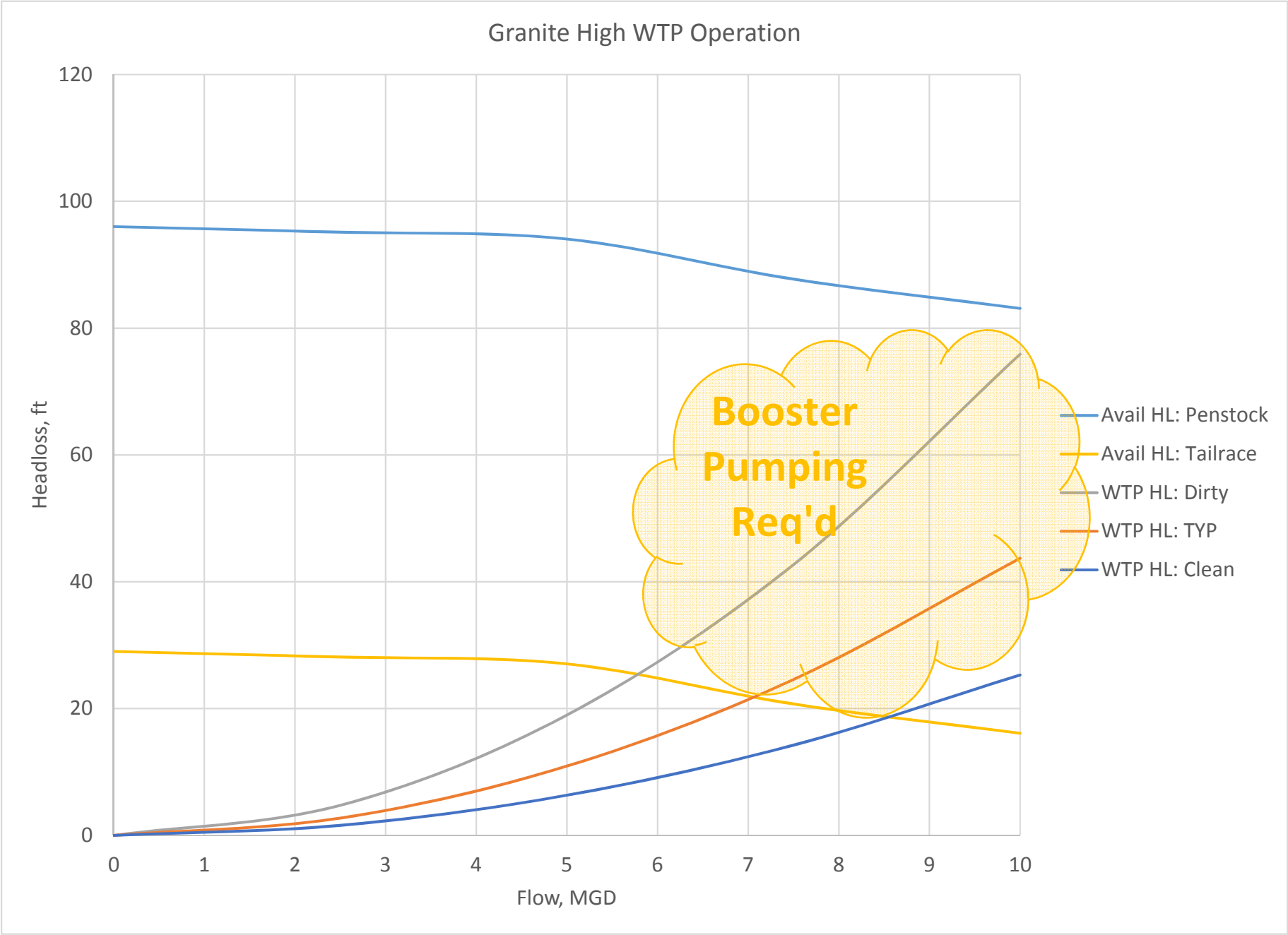
Supply Line Headloss				
Flow (MGD)	2.5	5	7.5	10
For TID (24")	1.6	5.7	12.1	20.7
For a 30" Raw water line	0.5	0.5	5.1	7.4

Headloss - WTP to Crowson I and Required HGL leaving WTP				
Flow (MGD)	2.5	5	7.5	10
Headloss from Crowson I to WTP	0.4	1.5	3.2	5.5
Required HGL	2,425.4	2,426.5	2,428.2	2,430.5

65 psi max pressure allowed at Membranes

Elevations

Granite High Elevation	2,375.0 ft
Penstock Headtank Elevation	2,521.0 ft
Tailrace Elevation	2,454.0 ft


Granite High WTP Operation
Headloss, ft
120
100
80
60
40
20
0
0
1
2
3
4
5
6
7
8
9
10
Flow, MGD
Booster Pumping Req'd
Avail HL: Penstock
Avail HL: Tailrace
WTP HL: Dirty
WTP HL: TYP
WTP HL: Clean

Ashland - New WTP Hydraulic Analysis

Granite Low Site Hydraulic Analysis

	TAILRACE					PENSTOCK				
	0	**2.5**	**5**	**7.5**	**10**	**0**	**2.5**	**5**	**7.5**	**10**
HL in RW supply (ft) (assumes a 30" pipe)	0	1.7	1.7	4.0	6.3	0	1.7	1.7	4.0	6.3
HGL U/S WTP (ft)	2,454.0	2,452.3	2,452.3	2,450.0	2,447.7	2,521.0	2,519.3	2,519.3	2,517.0	2,514.7
Head U/S WTP (ft)	138.0	136.3	136.3	134.0	131.7	205.0	203.3	203.3	201.0	198.7
Pressure U/S WTP (psi)	59.7	59.0	59.0	58.0	57.0	88.7	88.0	88.0	87.0	86.0
HGL U/S WTP (ft)	2,454.0	2,452.3	2,452.3	2,450.0	2,447.7	2,521.0	2,519.3	2,519.3	2,517.0	2,514.7
Clipped pressure U/S WTP (psi)	59.7	59.0	59.0	58.0	57.0	65.0	65.0	65.0	65.0	65.0
Clipped HGL U/S WTP (ft)	2,454.0	2,452.3	2,452.3	2,450.0	2,447.7	2,466.2	2,466.2	2,466.2	2,466.2	2,466.2
Required Finished Water HGL (ft)	2,425.0	2,425.4	2,426.6	2,428.5	2,430.9	2,425.0	2,425.4	2,426.6	2,428.5	2,430.9
Available HL at Plant for GravFlow (ft)	29	26.89	25.69	21.48	16.77	41.15	40.75	39.55	37.65	35.25
Available HL at Plant for GravFlow (psi)	13	12	11	9	7	18	18	17	16	15

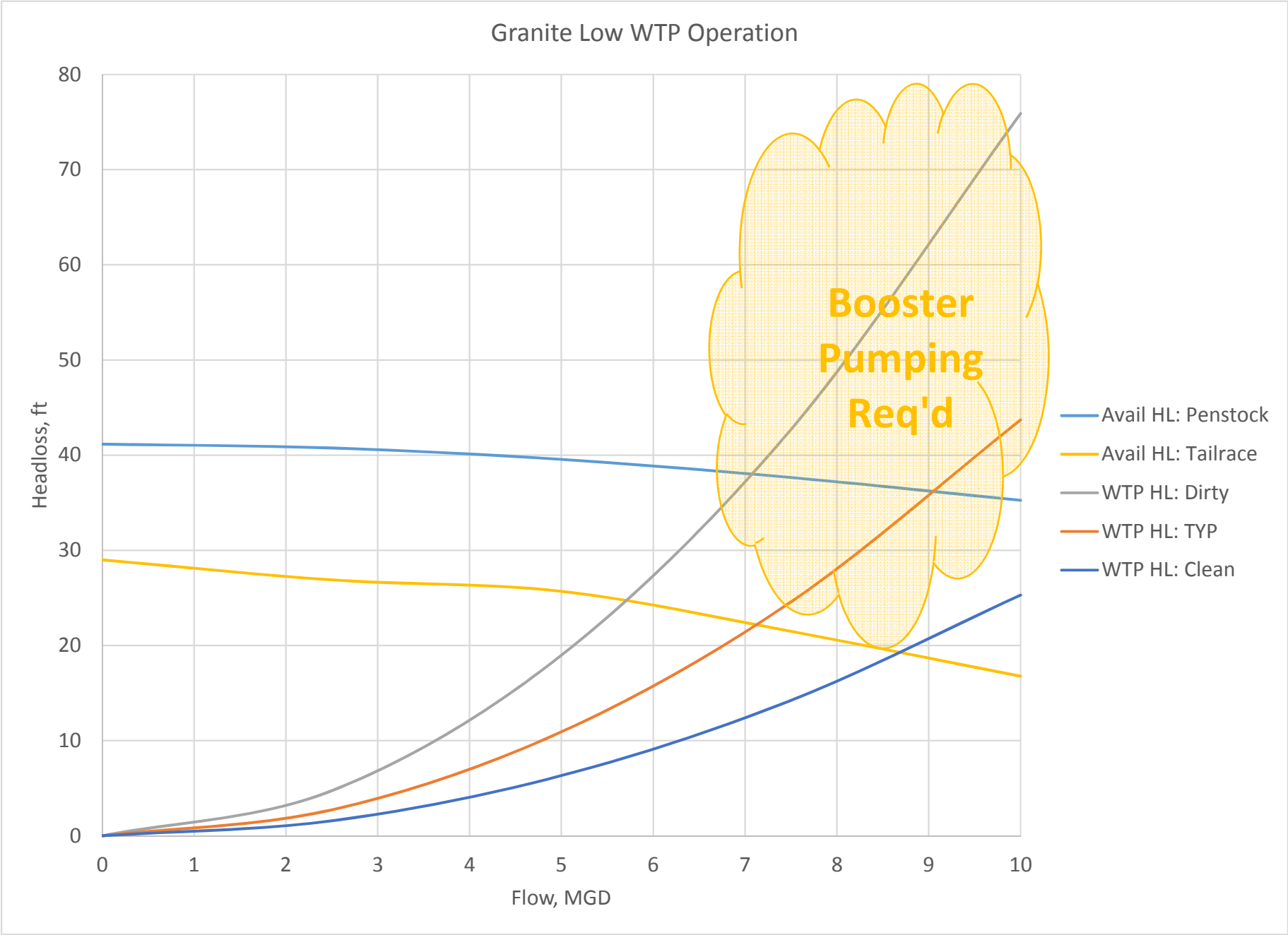
Supply Line Headloss				
Flow (MGD)	2.5	5	7.5	10
For TID (24")	1.5	5.4	11.5	19.7
For a 30" Raw water line	1.7	1.7	4.0	6.3

Headloss - WTP to Crowson I and Required HGL leaving WTP				
Flow (MGD)	2.5	5	7.5	10
Headloss from Crowson I to WTP	0.4	1.6	3.5	5.9
Required HGL	2425.4	2426.6	2428.5	2430.9

65 psi max pressure allowed at Membranes

Elevations

Granite Low Membranes	2,316.0 ft
Penstock Headtank	2,521.0 ft
Tailrace	2,454.0 ft


Granite Low WTP Operation
Headloss, ft
80
70
60
50
40
30
20
10
0
0
1
2
3
4
5
6
7
8
9
10
Flow, MGD
Booster Pumping Req'd
Avail HL: Penstock
Avail HL: Tailrace
WTP HL: Dirty
WTP HL: TYP
WTP HL: Clean

Ashland - New WTP Hydraulic Analysis

Asphalt Site Hydraulic Analysis

	TAILRACE					PENSTOCK				
Flow (MGD)	**0**	**2.5**	**5**	**7.5**	**10**	**0**	**2.5**	**5**	**7.5**	**10**
HL in RW supply (ft) (assumes a 30" pipe)	0	0.6	0.6	5.2	7.6	0	0.6	0.6	5.2	7.6
HGL U/S WTP (ft)	2,454.0	2,453.4	2,453.4	2,448.8	2,446.5	2,521.0	2,520.4	2,520.4	2,515.8	2,513.5
Head U/S WTP (ft)	227.0	226.4	226.4	221.8	219.5	294.0	293.4	293.4	288.8	286.5
Pressure U/S WTP (psi)	98.3	98.0	98.0	96.0	95.0	127.3	127.0	127.0	125.0	124.0
HGL U/S WTP (ft)	2,454.0	2,453.4	2,453.4	2,448.8	2,446.5	2,521.0	2,520.4	2,520.4	2,515.8	2,513.5
Clipped pressure U/S WTP (psi)	65.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0
Clipped HGL U/S WTP (ft)	2,377.2	2,377.2	2,377.2	2,377.2	2,377.2	2,377.2	2,377.2	2,377.2	2,377.2	2,377.2
Required Finished Water HGL (ft)	2,425.0	2,425.6	2,427.1	2,429.5	2,432.6	2,425.0	2,425.6	2,427.1	2,429.5	2,432.6
Available HL at Plant for GravFlow (ft)	-47.85	-48.47	-49.94	-52.31	-55.48	-47.85	-48.47	-49.94	-52.31	-55.48
Available HL at Plant for GravFlow (psi)	-21	-21	-22	-23	-24	-21	-21	-22	-23	-24

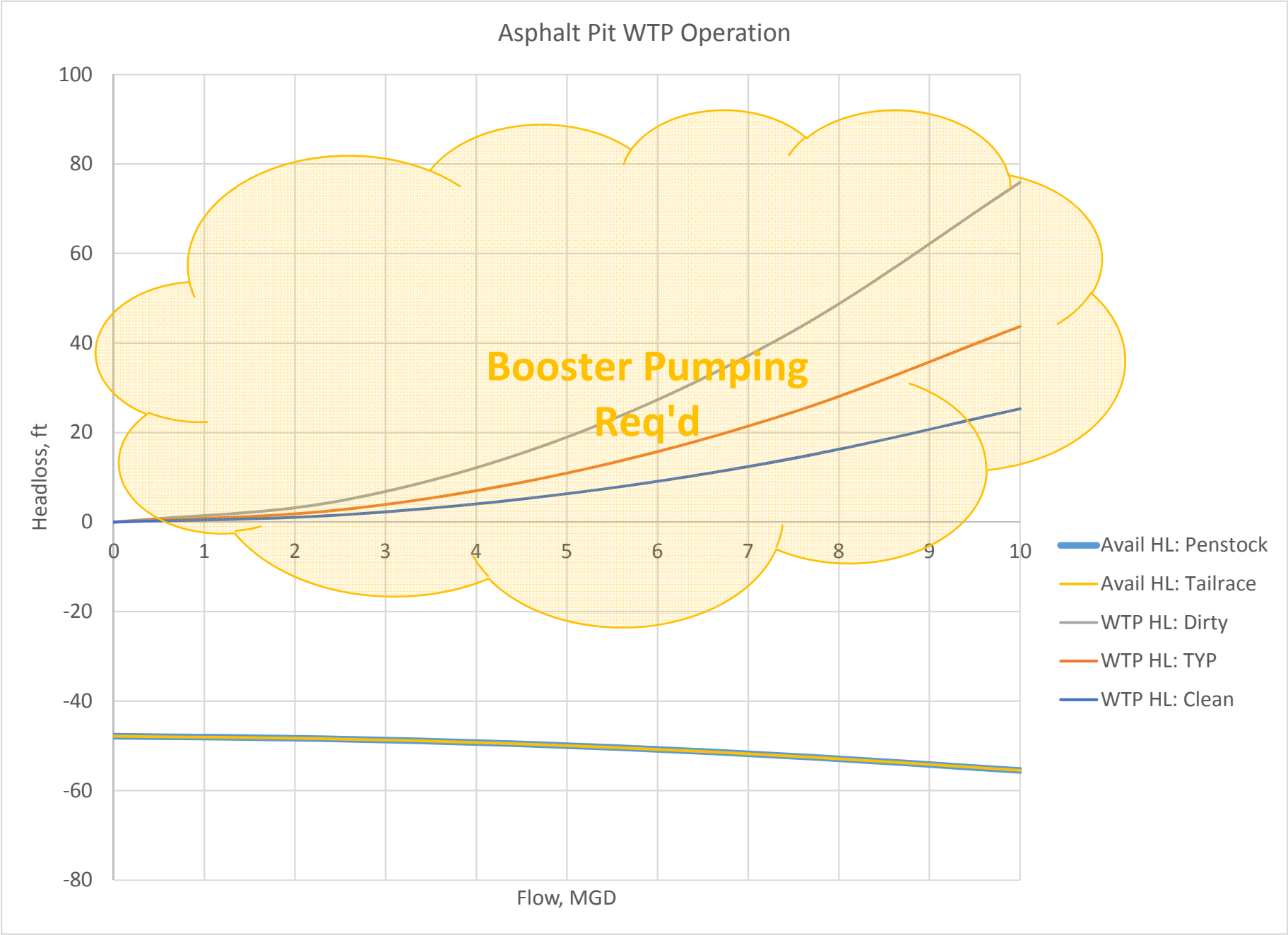
Supply Line Headloss				
Flow (MGD)	2.5	5	7.5	10
For TID (24")	1.9	7.0	14.9	25.5
For a 30" Raw water line	0.6	0.6	5.2	7.6
Headloss - WTP to Crowson I and Required HGL leaving WTP				
Flow (MGD)	2.5	5	7.5	10
Headloss from Crowson I to WTP[1]	0.6	2.1	4.5	7.6
Required HGL	2425.62	2427.09	2429.46	2432.63

1. Headloss estimated.

65 psi max pressure allowed at Membranes

Elevations

Asphalt Membranes	2,227.0	ft
Penstock Headtank	2,521.0	ft
Tailrace	2,454.0	ft


Asphalt Pit WTP Operation
Headloss, ft
100
80
60
40
20
0
-20
-40
-60
-80
0
1
2
3
4
5
6
7
8
9
10
Flow, MGD
Booster Pumping Req'd
Avail HL: Penstock
Avail HL: Tailrace
WTP HL: Dirty
WTP HL: TYP
WTP HL: Clean