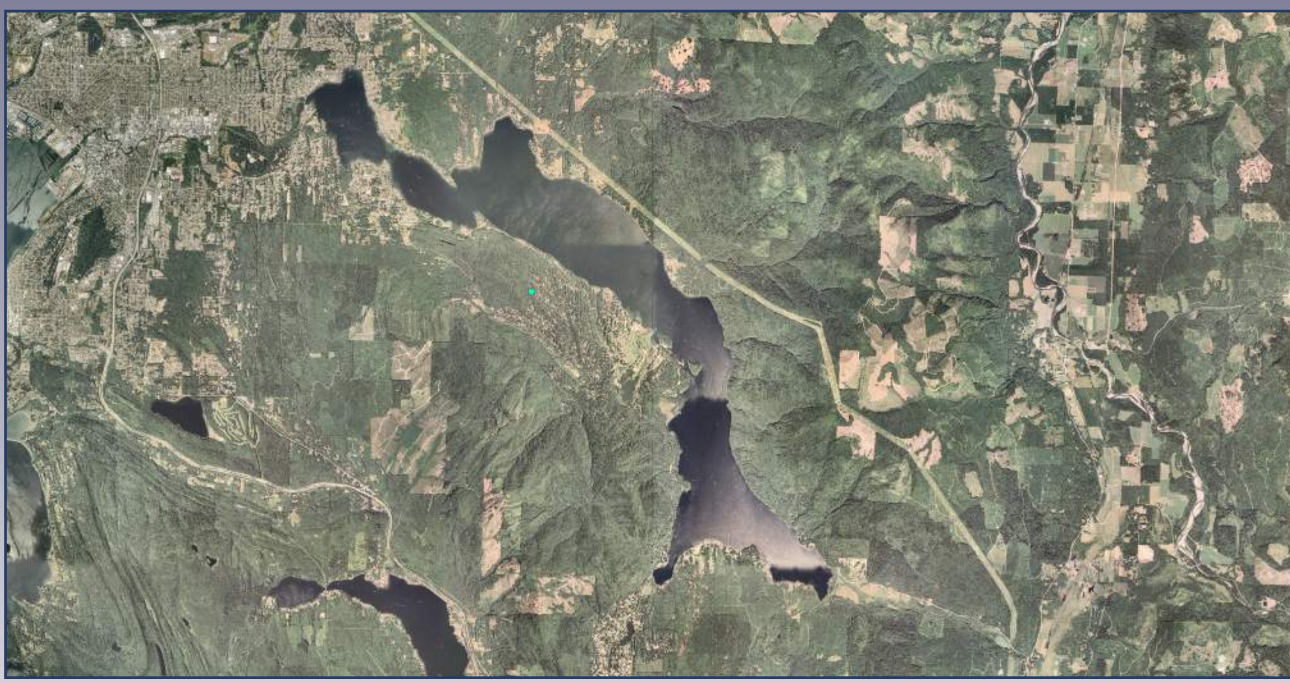
LAKE WHATCOM WATER & SEWER DISTRICT

WHATCOM COUNTY, WASHINGTON

DIVISION 22 RESERVOIR PREDESIGN REPORT

DRAFT



G&O #14456

DRAFT MARCH 2015

Gray & Osborne, Inc.

CONSULTING ENGINEERS

LAKE WHATCOM
WATER & SEWER DISTRICT

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CHAPTER 1

INTRODUCTION

INTRODUCTION

Lake Whatcom Water and Sewer District (District) has contracted with Gray & Osborne, Inc. to provide professional engineering services for predesign and land use permitting of the new Division 22 Reservoir. The District intends to construct a second reservoir next to the existing steel reservoir, which is located in the District's South Shore Water System (DOH Water System ID #95910). The project will include a new reservoir, new site piping, and site improvements. The District has procured funding for this project from the Washington State Public Works Board through the Drinking Water State Revolving Fund (DWSRF).

This report updates storage analyses and demands, considers several alternatives for reservoir dimensions and material, summarizes initial geotechnical findings for the site, analyzes stormwater and drainage needs, and discusses reservoir features. It also provides a summary of permit processes and requirements. Included with this Predesign Report are a planning level cost estimate and a preliminary site plan.

REFERENCES

The following documents are referenced as part of this analysis:

- *Lake Whatcom Water and Sewer District Water System Comprehensive Plan*, October 2010, Wilson Engineering, L.L.C.
- *Sudden Valley - Geneva Reservoir Capacity Analysis*, August 2009, Wilson Engineering, L.L.C.

BACKGROUND

The District's South Shore Water System has an existing storage deficiency that is identified in the District's most recent *2010 Water System Comprehensive Plan* (Water System Plan). The Water System Plan includes a capital project to construct a new reservoir with a volume of approximately 500,000 gallons in the Sudden Valley Division 22 Service Area, which in combination with pressure zone reconfiguration would mitigate storage deficiencies in the Sudden Valley and Geneva Service Areas.

The existing Division 22 Reservoir property contains a previously cleared area suitable for construction of the new reservoir. The District has also identified additional improvements needed for the existing site, including reconfiguration of the drain and overflow sewer discharges and communications improvements.

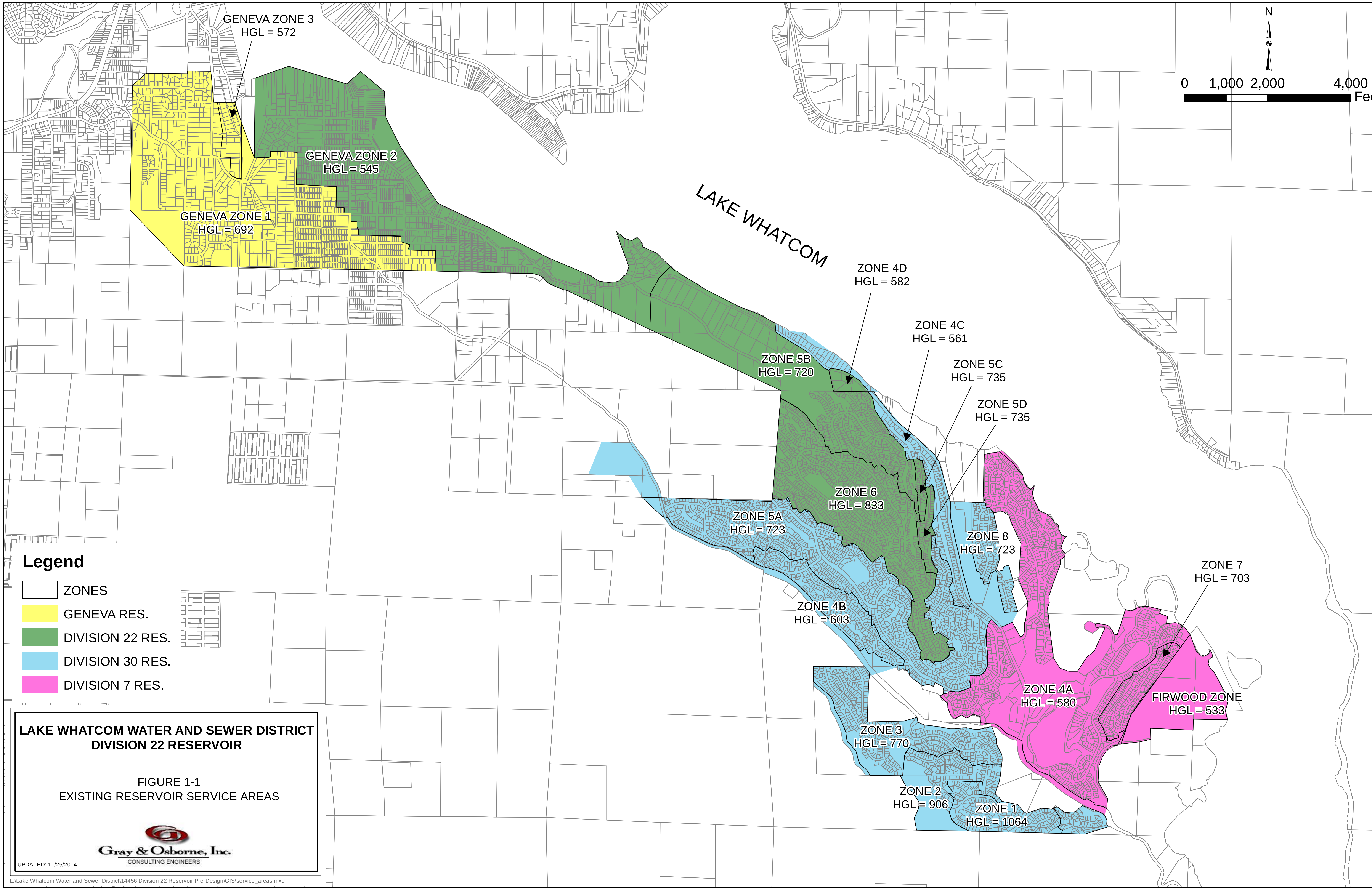
EXISTING FACILITIES

The existing Division 22 Reservoir site is located on a peak in the northwest portion of the Sudden Valley Community, which is located southeast of the City of Bellingham on the southern shore of Lake Whatcom. The existing steel reservoir has a nominal capacity of 500,000 gallons. The existing reservoir property is bordered on the west by the Stimpson Family Nature Reserve and on the north, east, and south by single-family residential properties. The reservoir is located on an easement grated by the Sudden Valley Community Association, which owns the property.

The existing Division 22 Reservoir has a base elevation of approximately 805 feet (NAVD 88 datum), an overflow elevation of 840 feet, and a diameter of approximately 50 feet. The Division 22 Reservoir is fed from the Sudden Valley Treatment Plant via the Division 22 Transmission Pump Station, which contains two pumps with a capacity of 700 gpm at 608 feet TDH. The Division 22 Reservoir currently serves Sudden Valley Zones 5 and 6. The Zone 6 HGL floats on the Division 22 Reservoir level, and Zone 5 is served by PRVs. The Division 22 Reservoir also serves portions of the Geneva Service Area via PRVs and supplies the Geneva Reservoir via the Beecher Booster Pump Station, which has a capacity of 400 gpm. The Geneva Reservoir has a nominal capacity of 500,000 gallons.

The District's other distribution storage facilities include the Division 7 Reservoir and the Division 30 Reservoir. The Division 7 Reservoir has a nominal capacity of 1 million gallons with a maximum water level of 703 feet and is fed from the Sudden Valley Treatment Plant via a transmission pump station. The Division 30 Reservoir has a nominal capacity of 150,000 gallons with a maximum water level of 1,070 feet and is fed from the Division 7 Reservoir via a booster station. Because of the large amount of storage in the Division 7 Reservoir, some areas that could otherwise be served by the Division 22 Reservoir are currently served by the Division 30/Division 7 Reservoirs. However, pumping up to the Division 30 Reservoir consumes additional energy to serve these customers because of the additional 265 feet of head required to fill the Division 30 Reservoir compared to the Division 22 Reservoir. The District has the flexibility to modify the service areas for each reservoir by adjusting the settings of the multiple PRV stations in the Sudden Valley Service Area. Figure 1-1 shows the reservoir service areas with the system's current configuration. Figure 1-2 shows an alternate reservoir service area scenario that would minimize the service area of the Division 30 Reservoir and increase the service area of the Division 22 Reservoir. Both of the reservoir service area schemes will be considered in determining the size of the proposed Division 22 Reservoir. Figure 1-3 shows a hydraulic profile of the pressure zones.

Table 1-1 provides a summary of the characteristics the District's existing reservoirs.


GENEVA ZONE 3
HGL = 572
GENEVA ZONE 2
HGL = 545
GENEVA ZONE 1
HGL = 692
LAKE WHATCOM
ZONE 4D
HGL = 582
ZONE 4C
HGL = 561
ZONE 5C
HGL = 735
ZONE 5D
HGL = 735
ZONE 5B
HGL = 720
ZONE 6
HGL = 833
ZONE 5A
HGL = 723
ZONE 8
HGL = 723
ZONE 7
HGL = 703
ZONE 4B
HGL = 603
ZONE 4A
HGL = 580
FIRWOOD ZONE
HGL = 533
ZONE 3
HGL = 770
ZONE 2
HGL = 906
ZONE 1
HGL = 1064
N
0 1,000 2,000 4,000
Feet
Legend
ZONES
GENEVA RES.
DIVISION 22 RES.
DIVISION 30 RES.
DIVISION 7 RES.
LAKE WHATCOM WATER AND SEWER DISTRICT
DIVISION 22 RESERVOIR
FIGURE 1-1
EXISTING RESERVOIR SERVICE AREAS
Gray & Osborne, Inc.
CONSULTING ENGINEERS
UPDATED: 11/25/2014
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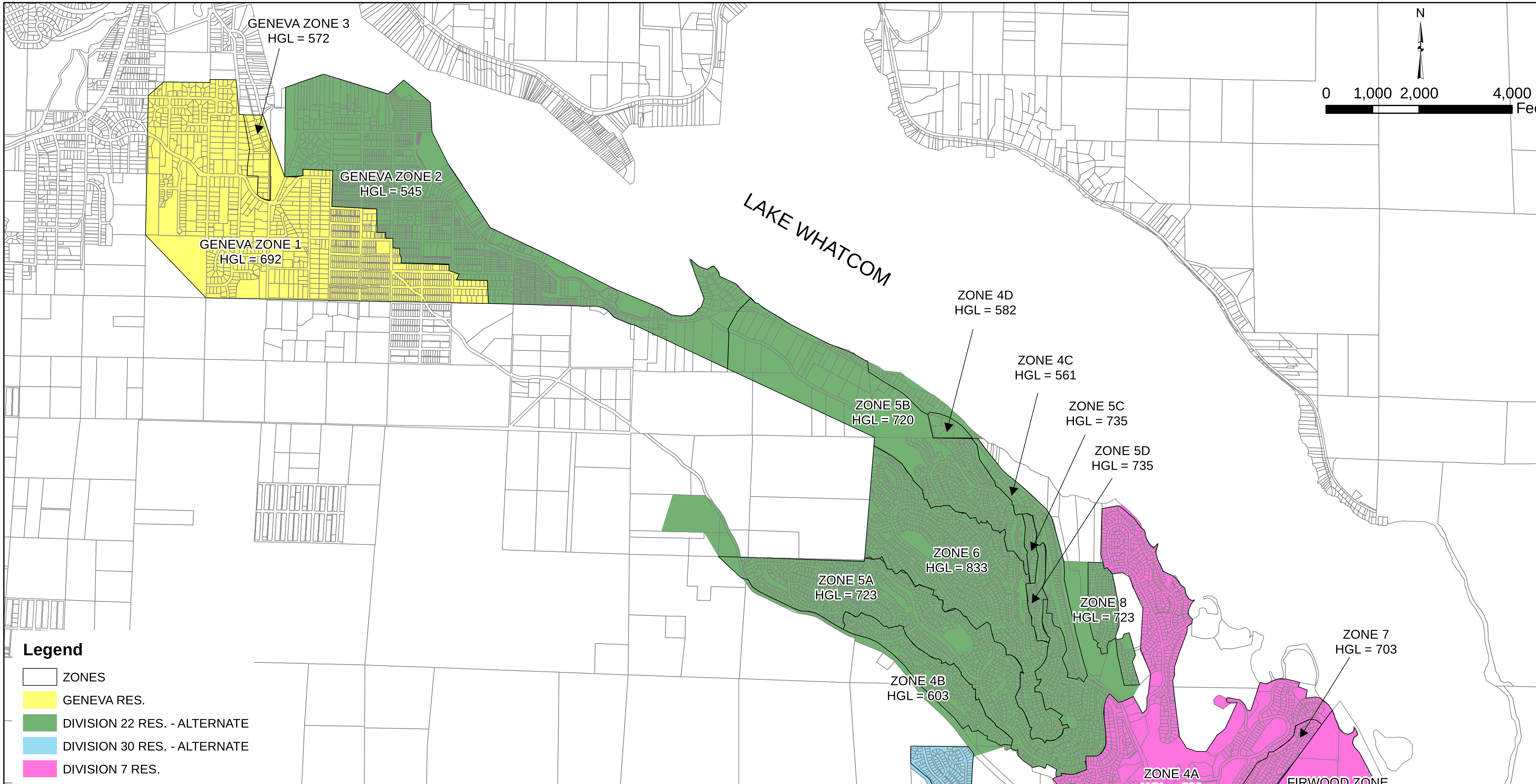

LAKE WHATCOM WATER AND SEWER DISTRICT
DIVISION 22 RESERVOIR

FIGURE 1-2
ALTERNATE RESERVOIR SERVICE AREAS

Gray & Osborne, Inc.
CONSULTING ENGINEERS

UPDATED: 11/25/2014

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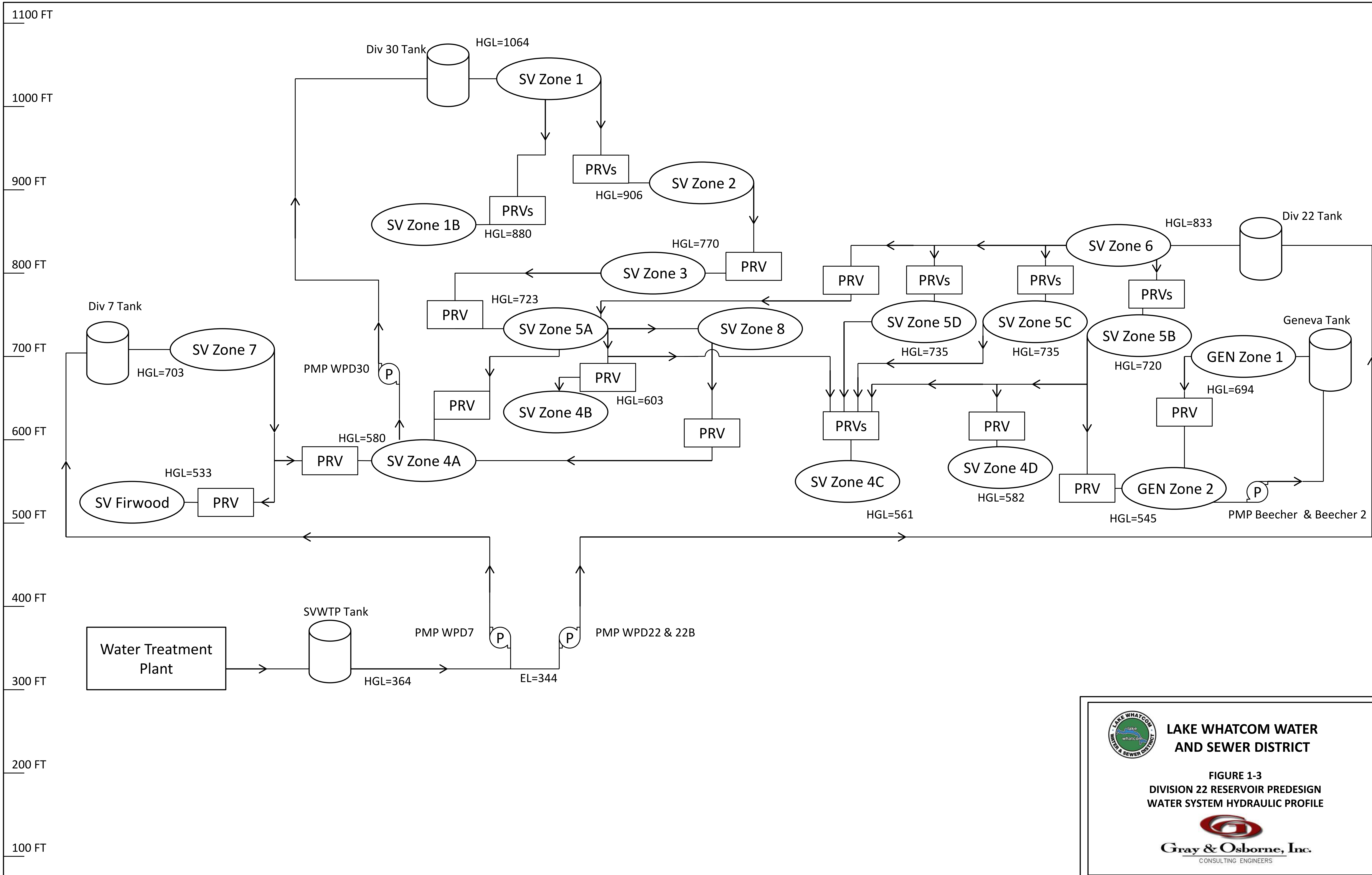


LAKE WHATCOM WATER AND SEWER DISTRICT

FIGURE 1-3
DIVISION 22 RESERVOIR PREDESIGN
WATER SYSTEM HYDRAULIC PROFILE

Gray & Osborne, Inc.
CONSULTING ENGINEERS

TABLE 1-1

Existing Reservoirs

Reservoir[1]	Year Constructed	Material	Volume (gal)	Overflow Elevation (ft)	Diameter (ft)	Height (ft)	Volume/ Foot (gal/ft)	Base Elev. (ft)
SV Div. 7	--	Welded Steel	1,000,000	703	70	34	28,786	669
SV Div. 22	1971	Welded Steel	500,000	840	48	35	13,535	805
SV Div. 30	--	Welded Steel	150,000	1070	25	45	3,672	1025
Geneva	--	Welded Steel	500,000	692	53	30	16,502	662

(1) SV=Sudden Valley.

PROJECT OBJECTIVES

The objectives of the project to construct the new reservoir are as follows:

- Eliminate Storage Deficiencies – The primary objective of the new reservoir is to eliminate the identified storage deficiencies within the South Shore Water System, which are primarily related to standby storage.

- Improve Reliability – The new reservoir will improve reliability in the Division 22 Reservoir Service Area by providing a redundant reservoir in case of emergency or planned maintenance of the existing reservoir.

- Increase Efficiency – Inefficiencies in the existing operational scheme may be reduced with the addition of available storage.

- Improve drainage/overflow capacity – The downstream capacity of the sewer system is limited near the site. The reservoir drain and overflow currently discharge to this sewer system. Rerouting these discharges to other nearby sewer lines will decrease the possibility of sewer overflows in the area.

- Improve communications – The existing remote telemetry unit does not have sufficient capacity for additional input/output and will need to be replaced to accommodate the current project and future upgrades to the existing Reservoir.

The design criteria for these improvements are further discussed in Chapter 2, and the proposed improvements are outlined in Chapter 3.

CHAPTER 2

DESIGN CRITERIA

INTRODUCTION

The new reservoir facility will be designed to meet District and Washington State Department of Health (DOH) standards. This Chapter outlines the basic design criteria for the new facility.

BASIC DESIGN CRITERIA

Storage requirements for the District are based on the sum of storage components laid out in WAC 246-290-235 and Chapter 9 of the *2009 Water System Design Manual* by the Washington State Department of Health, which are comprised of the following:

- Operational Storage
- Equalizing Storage
- Standby Storage
- Fire Suppression Storage
- Dead Storage (if any)

OPERATIONAL STORAGE

According to the DOH *Water System Design Manual*, operational storage is the volume of the reservoir devoted to supplying the water system while, under normal operating conditions, the source(s) of supply are in "off" status. This volume is dependent upon the sensitivity of the reservoir water level sensors and the tank configuration necessary to prevent excessive cycling of source pump motors. Operational storage is in addition to other storage components, thus providing a factor of safety for equalizing, standby, and fire suppression components.

The operational storage for each of the District's reservoirs, based on pump on/off set points, is shown in Table 2-1. The operational storage for the Division 30 and Geneva Reservoirs is determined by booster pump setpoints. The operational storage for the Division 7 and Division 22 Reservoirs is managed manually by the operators to minimize excessive cycling at the Water Treatment Plant.

TABLE 2-1

Operational Storage

Reservoir[1]	Pumps On Level	Pumps Off Level	Operating Range (ft)	Operational Storage (gallons)
SV Div. 7	--[2]	--[2]	5.0	143,932
SV Div. 22	--[2]	--[2]	10.0	135,355
SV Div. 30	35.5	39	3.5	12,851
Geneva	24.5	30.3	5.8	92,135

(1) Additional reservoir information is shown in Table 1-1.
(2) The operational storage in the Division 7 and 22 Reservoirs is managed manually by the operators.

EQUALIZING STORAGE

Equalizing storage is typically used to meet diurnal demands that exceed the average day and maximum day demands. The volume of equalizing storage required depends on maximum system demands, the magnitude of diurnal water system demand variations, the source production rate, and the mode of system operation. Sufficient equalizing storage must be provided in combination with available water sources and pumping facilities such that maximum system demands can be satisfied.

Equalizing storage is calculated using the following equation:

$$V_{ES} = (Q_{PH} - Q_S) \times 150 \text{ minutes}$$

V_{ES} = Equalizing storage component (gallons)
Q_{PH} = Peak hourly demand (gpm)
Q_S = Total source of supply capacity, excluding emergency sources (gpm)

Equalizing storage requirements for each pressure zone are summarized in Table 2-2.

TABLE 2-2

Equalizing Storage

Reservoir	Number of Services		PHD/ERU (gpm/ERU)		PHD (gpm)	Pumped Flows Out[1] (gpm)	Source Capacity[2] (gpm)	Equalizing Storage (gal)
	SV	Geneva	SV	Geneva				
SV Div. 7	714	0	0.42	0.00	300	340	850	0
SV Div. 22	576	470	0.42	0.52	486	400	720	24,948
SV Div. 30	1,104	0	0.42	0.00	464	0	340	18,552
Geneva	0	595	0.00	0.52	309	0	400	0

(1) Includes flows pumped out of each reservoir's service area via booster pump stations.
(2) Includes flows pumped into each reservoir's service are via transfer and booster pump stations.

STANDBY STORAGE

Standby storage is provided in order to meet demands in the event of a system failure such as a power outage, an interruption of supply, or a break in a major transmission line. The amount of emergency storage should be based on the reliability of supply and pumping equipment, standby power sources, and the anticipated length of time the system could be out of service.

Standby storage is calculated using the following equation:

$$SB_{TSS} = (2 \text{ days})(ADD)(N)$$

SB_{TMS}	=	Standby storage component for a single source system (gallons)
ADD	=	Average day demand for the system (gpd/ERU)
N	=	Number of ERUs

Although standby storage volumes are intended to satisfy the requirements imposed by system customers for unusual situations and are addressed by WAC 246-290-420, DOH recommends that standby storage volumes be no less than 200 gallons/ERU. The District's standby storage is calculated based on the greater of 200 gallons/ERU and the equation above.

Standby storage requirements for each pressure zone are presented in Table 2-3.

TABLE 2-3

Standby Storage

Reservoir	Number of Services		ADD/ERU (gpd/ERU)		ADD (gpd)	Standby Storage (gal)
	SV	Geneva	SV	Geneva		
SV Div. 7	655	0	150	0	98,250	196,500
SV Div. 22	576	474	150	175	169,350	338,700
SV Div. 30	1,104	0	150	0	165,600	331,200
Geneva	0	642	0	175	112,350	224,700

FIRE SUPPRESSION STORAGE

Fire suppression storage is provided to ensure that the volume of water required for fighting fires is available when necessary. The amount of water required for firefighting purposes is specified in terms of rate of flow in gallons per minute (gpm) and an associated duration. Fire flows must be provided while maintaining residual water system pressures of at least 20 pounds per square inch (psi) throughout the distribution system as the storage reservoir approaches the lowest level of the fire suppression storage component within the reservoir.

Fire suppression storage is calculated using the following equation:

$$FSS = (FF)(t_m)$$

FSS = Required fire suppression storage component (gallons)
FF = Required fire flow rate, as specified by fire protection authority (gpm)
t_m = Duration of FF rate, as specified by fire protection authority (minutes)

Per WAC 246-290-235(4), standby and fire suppression storage volumes may be "nested," with the larger of the two volumes being the minimum available, provided that such practice is not prohibited by: (1) a locally developed and adopted Coordinated Water System Plan, (2) local ordinance, or (3) the local fire protection authority or County Fire Marshal. The District policy is to nest fire suppression storage volumes in the standby storage volumes, which are much greater.

The fire suppression storage for each pressure zone is shown in Table 2-4.

TABLE 2-4

Fire Suppression Storage

Reservoir	Max. FF Required (gpm)	FF Duration Required (min)	Fire Suppression Storage (gal)
SV Div. 7	750	60	45,000
SV Div. 22	750	60	45,000
SV Div. 30	500	60	30,000
Geneva	750	60	45,000

DEAD STORAGE

Dead storage is the volume of stored water in a reservoir that is not available for service to customers while maintaining the minimum system design pressures in accordance with WAC 246-290-230(5) and (6). Dead storage is excluded from the volumes provided to meet the other storage requirements.

The service connections with the highest elevation for each pressure zone were compared to the base elevations of the reservoirs. A minimum pressure of 20 psi is to be maintained at all times throughout the distribution system. Based on available LIDAR elevation data and previous analyses, the Sudden Valley Division 7, Sudden Valley Division 30, and Geneva Reservoirs can maintain a minimum pressure of 20 psi at all or nearly all services at the base elevation of the reservoirs. Thus, these reservoirs have no dead storage component.

Based on LIDAR elevation data, developer drawings, and a recent survey of the site, the Division 22 Reservoir cannot serve approximately a dozen services in its immediate vicinity at a pressure of 20 psi at the base elevation of the tank. The pressure at these services is boosted by individual booster pumps. For the purposes of this analysis, dead storage to meet the 20 psi requirement at these services is not included.

OVERALL SYSTEM ANALYSIS

The storage analysis for the District's South Shore Water System Reservoirs is given in Table 2-5. As shown in the table, there is an existing storage deficiency for the Division 22 and Division 30 Reservoirs. The largest storage component for the South Shore Reservoirs is standby storage. Based on the projected buildout demands in the District's Water System Plan, the buildout storage analysis for the South Shore Reservoirs is given in Table 2-6. As shown in the table, the projected increase in demands will increase the storage deficits for the Division 22 and Division 30 Reservoirs.

In a standby scenario, the storage surplus in the Division 7 Reservoir could be used to supply the Division 30 Reservoir. The Division 30 Reservoir is fed from the Division 7 Reservoir via a booster station with a redundant pump and an on-site generator. This level of reliability would be adequate to transfer standby storage in the majority of standby situations, including a prolonged power outage.

The proposed second Division 22 Reservoir will be constructed to eliminate the storage deficiency for the existing Division 22 Reservoir. At least 150,000 gallons of storage would be needed to eliminate this deficiency at projected buildout demands.

ALTERNATIVE OPERATION ANALYSIS

As discussed in Chapter 1, because of the large amount of storage in the Division 7 Reservoir, some areas that could otherwise be served by the Division 22 Reservoir are currently served by the Division 30/Division 7 Reservoirs. Pumping up to the Division 30 Reservoir requires an additional 265 feet of head compared to the Division 22 Reservoir. The District has the flexibility to modify the service areas for each reservoir by adjusting the settings of the multiple PRV stations in the Sudden Valley Service Area. This modification could also offset storage deficiencies for the Division 30 Reservoir by reducing the equalizing and standby storage requirements. The potential benefits of this operational change must be weighed against the potential for water quality problems associated with longer turnover times for the Division 7 Reservoir. Demands in the area served by the Division 30/Division 7 Reservoirs would decrease by approximately one third in the alternate scheme, which would increase turnover times for the Division 7 Reservoirs by 50 percent.

The buildout storage analysis for the alternative operational scenario is shown in Table 2-7. As shown in the Table, approximately 550,000 gallons of storage would be needed to eliminate the storage deficiency for the existing Division 22 Reservoir.

TABLE 2-5

Existing Storage Analysis

Reservoir	Number of Services		Operational Storage (gallons)	Equalizing Storage (gallons)	Standby Storage (gallons)	Fire Suppression Storage (gallons)	Total Required Storage[(1)] (gallons)	Available Storage Volume (gallons)	Storage Surplus/ (Deficit)
	SV	Geneva							
SV Div. 7	714	0	143,932	0	196,500	45,000	340,432	1,000,000	659,568
SV Div. 22	576	470	135,355	24,948	338,700	45,000	499,003	500,000	997
SV Div. 30	1,104	0	12,851	18,552	331,200	30,000	362,603	150,000	-212,603
Geneva	0	595	92,135	0	224,700	45,000	316,835	500,000	183,165

(1) The total required storage is based on the sum of operational storage, equalizing storage, and the larger of the two of standby or fire suppression storage.

TABLE 2-6

Buildout Storage Analysis

Reservoir	Number of Services		Operational Storage (gallons)	Equalizing Storage (gallons)	Standby Storage (gallons)	Fire Suppression Storage (gallons)	Total Required Storage[(1)] (gallons)	Available Storage Volume (gallons)	Storage Surplus/ (Deficit)
	SV	Geneva							
SV Div. 7	843	0	143,932	0	245,400	45,000	389,332	1,000,000	610,668
SV Div. 22	833	527	135,355	45,585	473,200	45,000	654,140	500,000	-154,140
SV Div. 30	1,468	0	12,851	41,484	440,400	30,000	494,735	150,000	-344,735
Geneva	0	651	92,135	0	301,700	45,000	393,835	500,000	106,165

(1) The total required storage is based on the sum of operational storage, equalizing storage, and the larger of the two of standby or fire suppression storage.

TABLE 2-7

Alternative Buildout Storage Analysis

Reservoir	Number of Services		Operational Storage (gallons)	Equalizing Storage (gallons)	Standby Storage (gallons)	Fire Suppression Storage (gallons)	Total Required Storage[(1)] (gallons)	Available Storage Volume (gallons)	Storage Surplus/ (Deficit)
	SV	Geneva							
SV Div. 7	843	0	143,932	0	245,400	45,000	389,332	1,000,000	610,668
SV Div. 22	1,912	527	135,355	113,562	796,900	45,000	1,045,817	500,000	-545,817
SV Div. 30	389	0	12,851	0	116,700	30,000	129,551	150,000	20,449
Geneva	0	651	92,135	0	301,700	45,000	393,835	500,000	106,165

(1) The total required storage is based on the sum of operational storage, equalizing storage, and the larger of the two of standby or fire suppression storage.

RESERVOIR SIZING CRITERIA

Based on the storage analyses, a reservoir capacity of at least 550,000 gallons will be sufficient to meet buildout storage requirements for both the current and alternative operational scenarios. The reservoir will be designed with an overflow level to match the existing reservoir overflow level at approximately 840 feet. To accommodate the constraints of the existing site, the reservoir will be approximately the same diameter and height as the existing reservoir. The reservoir design criteria are summarized in Table 2-8.

TABLE 2-8

Reservoir Design Criteria

Parameter	Value
Volume	630,000 gallons
Overflow Level	841 feet
Diameter	56 feet
Max. Water Level	35 feet

CHAPTER 3

RESERVOIR MATERIAL COMPARISON

RESERVOIR MATERIAL COMPARISON

Both steel and concrete are common construction materials for water storage reservoirs. Each material offers distinct advantages and disadvantages depending on the application for which it will be used. The following sections provide a discussion of the construction methods for steel and concrete and summarize the advantages and disadvantages of each material.

STEEL

Welded or bolted steel storage tanks are common in municipal water storage reservoir applications and are compared in the following sections.

Welded Steel

Welded steel construction provides for versatile reservoir size and low construction costs. Welded steel tanks are comprised of steel panels welded together in the field to form the walls and roof of the tank. The entire tank structure is coated after construction to provide protection against weather and corrosion. Welded steel reservoirs can be constructed as either ground-level reservoirs or elevated storage tanks. There are no height requirements when constructing steel reservoirs. However, if the diameter-to-height ratio is less than 1.5, anchorage may be required to counteract uplift during a seismic event.

Although welded steel reservoirs generally have lower capital costs than bolted steel or concrete reservoirs, one of the arguments against welded steel reservoirs is that they have potentially higher life cycle costs due to maintenance of interior and exterior coatings. Table 3-1 summarizes the advantages and disadvantages of welded steel reservoirs.

TABLE 3-1

Welded Steel Reservoir Advantages and Disadvantages

Advantages	Disadvantages
• Lower capital costs • Three or more local bidders ensure competitive quotes • Negligible leakage • Smooth surface facilitates disinfection • Can accommodate changes in piping configuration • Easy to repair	• Higher ongoing maintenance costs to maintain coatings. • Susceptible to corrosion if coatings not maintained • Cannot be backfilled or buried • Cathodic protection is an additional cost • Must be taken out of service for painting

Bolted Steel

Glass-fused-to-steel (GFS) bolted tanks are a competitive alternative to welded steel tanks. GFS bolted tanks are comprised of steel panels with fused glass coatings on the interior and exterior that are bolted together to form the walls of the tank. The glass coating provides an exterior and interior barrier against weather and corrosion that replaces the coating systems required for welded steel tanks. Similar to welded steel tanks, there are no height requirements for bolted steel tanks. The roof structure can be domed aluminum or a GFS paneled flat roof supported by columns. Both of these roof structures are lightly constructed and are vulnerable to damage caused by tree branches or other windblown debris. Tanks must be above grade in order to access panels and joints for maintenance.

There are currently two commercially viable manufacturers of GFS bolted steel tanks; one is located in the United States (supplied by Aquastore) and the other in England (supplied by Shearer Tanks). Due to the required competitive bidding process, either manufacturer could win the low bid. Lead time for the panels is significantly increased when they must be shipped from England.

Bolted steel tanks have higher capital costs than welded steel, but have approximately one-third the life cycle costs, as they do not require post-factory coatings. However, bolted steel tanks are most common and most competitively priced at volumes less than 500,000 gallons. Table 3-2 summarizes the advantages and disadvantages of bolted steel reservoirs.

TABLE 3-2

Bolted Steel Reservoir Advantages and Disadvantages

Advantages	Disadvantages
• Lower ongoing maintenance costs • Requires less maintenance • Negligible leakage typical • Smooth surface facilitates disinfection • Can accommodate changes in piping configuration	• Higher capital costs • Joints have the potential to leak • Fewer bidders • Repairs costly, by manufacturer only • Cathodic protection is an additional cost • Light roof structure is more prone to damage from tree limbs • Long lead time for panels

CONCRETE

Generally, concrete reservoirs consist of a concrete floor, concrete walls, and a concrete slab roof supported by a system of columns. The concrete walls are generally installed in sections, with angled reinforcement for seismic stability and vertical tendons. After the sections are installed and the reservoir wall ring is complete, the vertical tendons are post-tensioned to counteract the hydraulic load. A shotcrete layer is then applied as a final protective skin for the structure. Concrete reservoirs can be aboveground or can be partially or completely buried.

Concrete reservoirs generally have higher capital costs than steel reservoirs, but they usually require less maintenance because no interior or exterior coating system is needed. Concrete reservoirs are typically not cost competitive with steel for reservoirs with a capacity of less than 2 million gallons. Table 3-3 summarizes the advantages and disadvantages of concrete reservoirs.

TABLE 3-3

Concrete Reservoir Advantages and Disadvantages

Advantages	Disadvantages
• Lower life cycle costs • Requires less maintenance • Can be partially or completely buried • Higher percentage of construction costs expended within community	• Higher capital cost • Repairs can be difficult and costly • Difficult to prevent leakage entirely • Fewer bidders are qualified to bid for prestressed design • Not cost competitive for reservoirs smaller than 2 MG

ACCESS, SECURITY, AND SAFETY

Access to inside the reservoir does not differ much for steel and concrete tanks. Manways in the sidewalls are possible for all three types, along with roof hatches. Welded steel tank roofs can have ladder or stair access, with stairs welded directly to the tank. Bolted steel tanks have an aluminum domed or flat roof, which limits access to the roof. Reservoir accessories, such as ladders, drains, and conduits, cannot be welded directly to the exterior of the tank either, limiting design and repair flexibility. Concrete tank roof access is typically a ladder bolted to the side of the tank, although stairs could also be installed.

Steel tanks and concrete tanks have minimal security and safety differences.

MAINTENANCE

All three types of tanks require similar periodic inspection and cleaning, although major maintenance needs differ considerably.

Welded steel tanks require significant maintenance throughout the life of the tank. Corrosion is the most common type of deterioration in welded steel reservoirs. Steel reservoirs are susceptible to corrosion from both the atmosphere and the water stored inside. Corrosion in coastal environments can be particularly aggressive because of the higher salt content in the atmosphere. Protective coatings are necessary to prevent corrosion and extend the life of a steel reservoir. Cathodic protection will reduce corrosion on the wetted surfaces of the reservoir in areas where the coating has failed. Properly installed cathodic protection can extend the recoat interval for the reservoir from 20 years up to 30+ years.

Surface preparation is required prior to the application of protective coatings for welded steel tanks. The surface preparation required for different types of coating systems depends on the type of coating as well as the service environment. Typically, the interior of a steel reservoir is coated with a three-coat epoxy-polymide, with a total dry film thickness of a minimum of 12 mils. The exterior of a steel reservoir is coated with a High-Build Acrylic Polyurethane, with a total DFT of a minimum of 10 mils. Recoating systems are typically much cheaper than initial coatings, due to different surface preparation requirements; however, recoats are still a significant maintenance cost.

Bolted GFS tanks require far less ongoing maintenance than welded steel tanks, due to the glass and steel fused surface. Steel sheets are fabricated for uniformity in size and surface, and a glass formulation is applied and fired at extremely high temperatures, creating a surface that is resistant to corrosion. Additional finishes are applied by the manufacturer, with no additional coatings required post construction.

Maintenance of GFS tanks is limited, and generally consists of replacing surface seals between panels every 15 to 20 years. If a panel becomes damaged, such as dented or gouged, single panels can be replaced. The installation of a replacement panel requires that the tank be drained, seals removed surrounding the damaged panel, and the new panel installed.

Ongoing maintenance of a concrete tank is minimal, as it does not require interior or exterior coatings. If minor exterior damage occurs, the surface can easily be patched. Additional coatings may be applied to the exterior for aesthetic purposes.

Periodic cleaning is needed for all types of reservoirs to limit sediment buildup or growth of biological organisms inside the reservoir.

WATER QUALITY

Water quality will not differ significantly between a welded steel, bolted steel, or concrete tank and water quality monitors are needed regardless of material. However, a partially buried concrete reservoir will have lower water temperature than an above-grade steel reservoir during summer months. Higher water temperature can result in increased potential for organism growth and increased disinfection by-products formation. If water temperature remains lower during the summer in a concrete reservoir and thus more stable year round, water quality could be improved compared to a steel reservoir.

SITE ISSUES AND AESTHETICS

Site issues vary for steel and concrete reservoirs at different stages of construction. The primary difference between steel and concrete tanks in regard to site issues and aesthetics is the ability to bury a concrete tank. Since a steel tank must be completely above grade, grading and/or retaining walls may be needed if constructed in a location where the existing ground elevation is greater than the reservoir base elevation.

Both welded steel and concrete tanks can be coated with murals or a solid color. If highly visible within the site, the District may opt to finish the reservoir with a mural such as trees or graphics more specific to the District to enhance the appearance or blend it in with the existing surroundings. Bolted steel tanks are generally available in two standard factory color finishes. An additional five to six color options are generally available at an added cost. Painting of bolted steel tanks is neither necessary nor desirable.

Figure 3-1 provides typical appearances for each material type.

PRELIMINARY COST COMPARISON

RESERVOIR COST

Reservoir cost increases with size, although the cost per gallon decreases with size. Preliminary budgetary costs for a range of reservoir sizes have been provided by

DN Tanks for concrete reservoirs and Shearer Tanks for bolted steel reservoirs. The cost for bolted steel reservoirs is also based on one recently bid bolted steel reservoir project of a similar size. Welded steel reservoir costs are based on actual bids for projects designed by Gray & Osborne over the past 20 years. Table 3-4 summarizes these costs.

MAINTENANCE COSTS

The ongoing operational and maintenance costs of a reservoir include costs associated with cleaning, utility personnel labor, and recoating. Cleaning and utility staff time will be similar for all three materials, and are thus not factored into the maintenance cost analysis. The costs compared in this analysis include recoating a welded steel tank and resealing joints on a bolted steel tank. A concrete tank will not require significant maintenance throughout its life, thus a maintenance cost is not calculated.

The exterior of a welded steel reservoir typically needs an overcoat every 10 years. Interior surface recoating is required every 25 years for a welded steel tank, which entails removing existing paint and recoating all surfaces including the roof. Recoating costs are estimated to be approximately $9 per square foot for the interior and $3 per square foot for the exterior.

All exposed joints on a bolted steel reservoir must be stripped and resealed approximately every 20 years. Costs for this task are difficult to estimate since the tank supplier typically performs the work. However, GFS tanks are advertised as having maintenance costs equal to approximately one-third of that for a welded steel tank.

COST COMPARISON SUMMARY

Table 3-4 summarizes maintenance costs over a 30-year life cycle in 2014 dollars. These costs do not include costs for other work required to complete the project that would be similar for all reservoir materials, such as site work and piping.

TABLE 3-4

Reservoir Material Cost Comparison

	Material		
Estimated Costs	**Welded Steel**	**Bolted Steel**	**Concrete**
Capital Costs(1)	$490,000	$630,000	$950,000
Maintenance Costs(2)	$155,000	$52,000	-
Life Cycle Costs	$645,000	$682,000	$950,000

(1) Capital costs for reservoir only, not including piping, site work, etc.
(2) Maintenance costs for welded steel based on three exterior overcoats and one interior surface recoating. Maintenance costs for bolted steel are estimated to be approximately one third the maintenance costs for welded steel.

BOLTED STEEL RESERVOIR
PHOTO SOURCE: GRAY & OSBORNE

PRE-STRESSED CONCRETE RESERVOIR
PHOTO SOURCE: DN TANKS

WELDED STEEL RESERVOIR
PHOTO SOURCE: GRAY & OSBORNE

LAKE WHATCOM
WATER & SEWER DISTRICT
DIVISION 22 RESERVOIR
PRE-DESIGN REPORT
FIGURE 3-1 - RESERVOIR MATERIALS

Gray & Osborne, Inc.
CONSULTING ENGINEERS

PROPOSED RESERVOIR MATERIAL SELECTION

The District has selected welded steel construction for the proposed reservoir based on factors such as reservoir capital cost, maintenance needs and costs, aesthetic options, and overall site considerations.

CHAPTER 4

RESERVOIR FEATURES

RESERVOIR FEATURES

The following sections provide information regarding specific reservoir design features. The design features include the type of construction, roof and venting, inlet and outlet, and flexible pipe connections. Figure 4-1 shows an elevation of the proposed reservoir.

STEEL RESERVOIR CONSTRUCTION

The proposed reservoir will be an aboveground steel reservoir designed to meet the 2012 International Building Code (IBC) criteria and the AWWA D100-11 Standard.

COATING SYSTEM

Welded steel reservoirs must have an interior and exterior coating system to protect the steel from corrosion. Steel reservoirs are susceptible to corrosion from both the atmosphere and the water stored inside. Protective coatings are necessary to prevent corrosion and extend the life of a steel reservoir. If desired by the District, cathodic protection can further reduce corrosion on the wetted surfaces of the reservoir in areas where the coating has failed. Properly installed cathodic protection can extend the recoat interval for the reservoir from 20 years up to 30-plus years.

Typically, the interior of a steel reservoir is coated with a three-coat zinc/epoxy/epoxy, with a minimum total dry film thickness (DFT) of 12 mils. Interior coating will be NSF approved and will include a zinc rich primer for cathodic protection. The exterior of a steel reservoir is coated with a zinc/epoxy/polyurethane system, with a total DFT of a minimum of 10 mils. These systems are in accordance with AWWA D102-11.

RESERVOIR ACCESS

One access hatch will be installed on the roof, and two 36-inch manways will be installed in the reservoir wall to provide additional access during construction and future maintenance. At minimum, a ladder will be installed for roof access. The ladder will include security features to limit access. If funding allows, a stairway may be considered in place of the ladder. Railing will be provided to improve roof access safety. At a minimum, the railing will extend from the ladder to stairway to the vent and hatch openings. If funding allows, the railing may be installed around the full circumference of the reservoir.

An internal platform will be provided under the roof access hatch to improve access inside the reservoir.

SECURITY AND SAFETY

Intrusion alarms will be installed on all hatches and ladders for security.

Reservoir safety focuses primarily on safety measures for the water system staff. Harness mechanisms along the ladder and on the roof will be provided.

ROOF AND VENTING

The reservoir roof will be conical and slightly sloped at a minimum of 3/4-inch per foot to shed water. The roof structure will be seal welded on its interior to minimize corrosion.

A protected and screened center roof vent will be provided to allow air exchanges upon filling and drawing down the reservoir. The vent will be sized to prevent roof collapse during rapid withdrawal.

The roof to wall connection will be a chine. A chine is a sharp angled connection that is simpler to construct and easier to maintain.

RESERVOIR INLET, OUTLET, AND OVERFLOW

The reservoir will have a separate inlet and outlet that branch off from a single 12-inch pipe. The inlet and the outlet will be located on opposite ends of the tank. This will promote mixing within the reservoir and will minimize the potential for water quality issues. The reservoir inlet will be a duckbill check valve on a riser.

The overflow pipe will be 12 inch to match the inlet piping. It will discharge to the new sewer connection as discussed in the site design section. The bottom outlet end will be equipped with a rubber check valve to prevent animal or insect intrusion and to provide an air gap for backflow prevention.

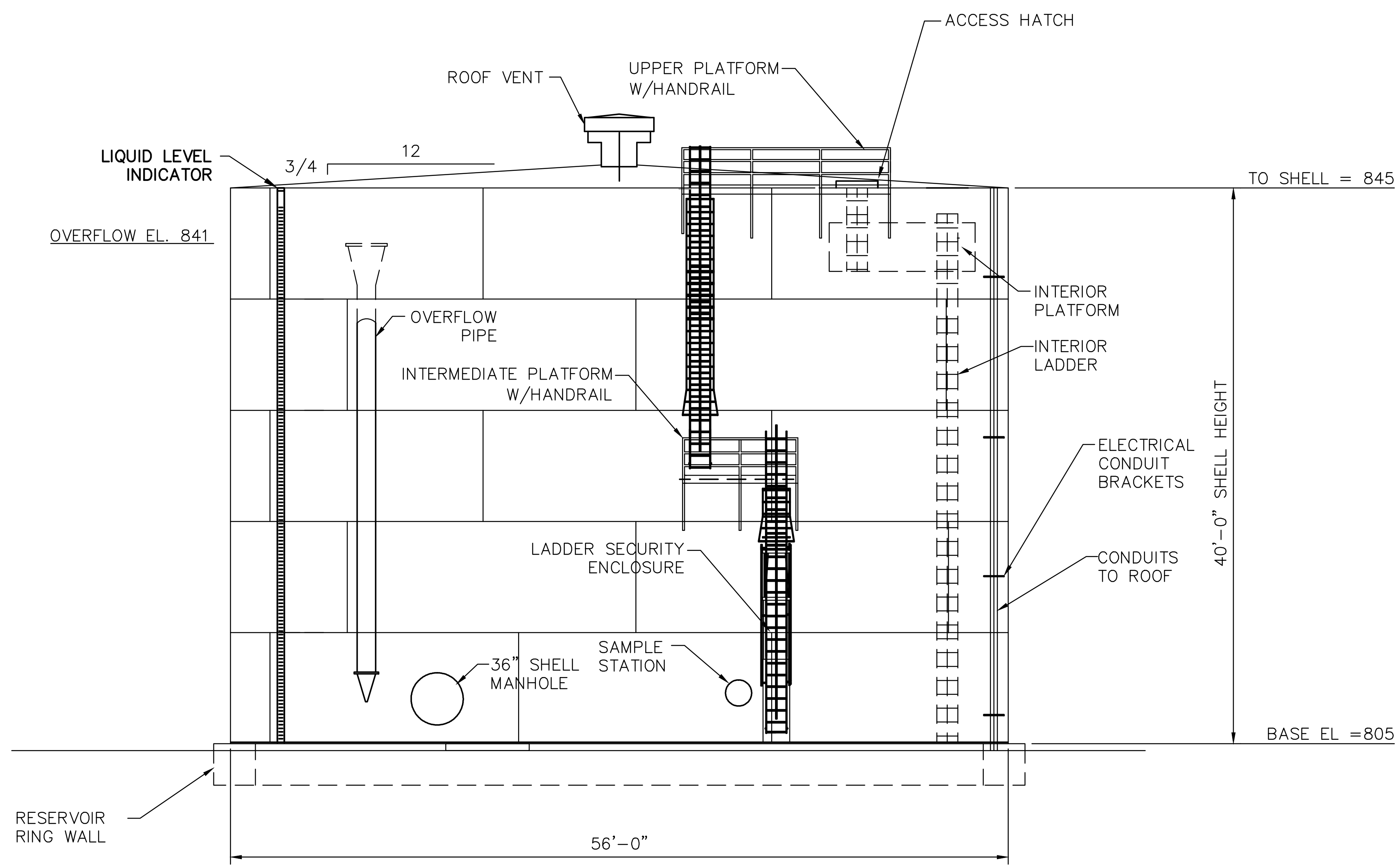
SEISMIC ISSUES

Several design features will help minimize damage and ensure that the reservoir remains fully operational during a seismic event.

FOUNDATION AND ANCHORAGE

All reservoirs that exceed the height to diameter ratio of approximately 1H:2D require anchorage to the foundation to counteract uplift during a seismic event. When a reservoir exceeds this ratio, the base cannot withstand the potential moment that results from a taller reservoir during a seismic event by gravity alone.

RESERVOIR DESIGN SUMMARY	
MATERIAL	WELDED STEEL
DIAMETER	56'–0"
SHELL HEIGHT	40'–0"
OVERFLOW	EL 841
WATER LEVEL	EL 840
FOUNDATION ELEVATION	805
NOMINAL CAPACITY	630,000 GAL



RESERVOIR ELEVATION

SCALE: 3/32"=1'–0"

NOTE:

FOR CLARITY NOT ALL ITEMS SHOWN IN TRUE POSITION.

LAKE WHATCOM WATER & SEWER DISTRICT

DIVISION 22 RESERVOIR PRE-DESIGN

FIGURE 4-1

RESERVOIR ELEVATION

Gray & Osborne, Inc.
CONSULTING ENGINEERS
SEATTLE–YAKIMA

The proposed reservoir has a height to diameter ratio of approximately 1:1.4. Therefore, based on preliminary calculations it appears that regularly spaced anchors will be required at the base of the reservoir to resist seismic overturning forces.

These anchors are typically threaded rods spaced at three to four feet on center which are attached to a chair welded to the shell. The bottom end of the anchors would be embedded into a concrete foundation. The foundation typically would consist of a stem wall with a continuous spread footing along the bottom. The depth and width of the spread footing would need to be sized to adequately resist uplift loads due to the design-level seismic overturning forces.

FREEBOARD

Freeboard must be included in reservoir design to allow for sloshing waves during a seismic event, otherwise the reservoir roof could be damaged. The required freeboard is a function of reservoir dimensions and seismic site class. The proposed reservoir will be designed with approximately 4 feet of freeboard based on seismic calculations.

FLEXIBLE PIPE CONNECTIONS

Reservoirs may experience shell uplifts of approximately 2 inches during seismic events. In anticipation of such uplifts, the inlet/outlet piping will be equipped with "Flex-tend" assemblies utilizing ball joints and expansion sleeves to accommodate any possible uplift or horizontal movements of the pipe at the bottom connection to the reservoir.

VALVE OPERATION

The outlet vault may contain a butterfly valve with an actuator that will close the valve when triggered by a seismic sensor on-site. This would prevent the reservoir from draining if there is a water main break within the system as the result of a seismic event.

SECURITY

Security measures for the proposed reservoir will be increased compared to the existing reservoir. It is now standard practice to install intrusion alarms at reservoirs to prevent public access. Intrusion alarms will be installed on the reservoir hatch, vent, ladder/stairway access gate, and all vaults.

CHAPTER 5

PROPOSED IMPROVEMENTS

GENERAL

The existing Division 22 Reservoir site is shown in Figure 5-1. The existing facilities include the existing reservoir and access driveway, as well as a remote telemetry unit (RTU). The site will be improved with a new reservoir, access improvements, drainage improvements, and new electrical equipment as shown in Figure 5-2. This chapter identifies the proposed improvements to be made as part of the project.

RESERVOIR

A new 630,000-gallon welded steel reservoir will be installed to the north of the existing reservoir. The reservoir features are discussed in Chapter 4.

SITE IMPROVEMENTS

VEHICLE ACCESS

Truck access will be available from an extension of the existing site entrance and loop. There will be at least 15 feet of clearance around the entire reservoir for vehicle and man lift access.

WATER MAINS

A connection to the existing 12-inch water main near the entrance to the site will be made to extend a new 12-inch water main to the proposed reservoir. Valves will be added to allow either reservoir to be taken offline for maintenance while keeping the other reservoir online.

STORMWATER SYSTEM

The stormwater system in the area of the reservoir site is an open channel system of roadside drainage ditches. Discharges to the stormwater system will be limited to surface water drainage of the proposed impervious areas.

SANITARY SEWER

The drain and overflow for the existing reservoir are connected to the sanitary sewer system without an air gap. A flap valve on each discharge is the only cross connection control provided for these connections. Beside the potential for contamination, the sanitary sewer system downstream of this connection is served by a lift station with insufficient capacity for large flow events from the reservoir connections, such as an uncontrolled overflow. In order to address these issues, the sanitary sewer connection will be upgraded and rerouted to a different sanitary sewer basin. The overflow lines for the existing and proposed reservoirs will be routed to a new dedicated manhole that will be connected to the sewer system on the adjacent street to the north, which is down a steep slope approximately 50 feet below the reservoir. The new sewer connection will drain by gravity and will therefore not be limited by downstream pumping capacity. The separation from nearby sanitary sewer flows and the significant elevation difference will also minimize the potential for contamination. The potential overflow improvements are discussed in more detail in the Overflow Analyses provided in Appendix B.

LANDSCAPING

Existing vegetation will be retained to the extent possible. Priority will be given to the retention of mature trees.

ELECTRICAL AND CONTROL IMPROVEMENTS

Electrical service is already available at the existing reservoir for the existing telemetry system. Electrical conduit and facilities for the reservoir will be upgraded. SCADA equipment will be mounted on a rack above grade within the fenced area. This will replace the existing panel.

The electrical and control improvements will include the following new facilities:

- Control Panel and Programmable Logic Controller (PLC)

INSTRUMENTATION

Instrumentation that will be provided will include the following:

- Intrusion Switches
- Overflow Flood Switch
- Flow Meter
- Reservoir Outlet Pressure Transducer

The PLC will monitor the following:

- Existing Reservoir Level
- Proposed Reservoir Level
- Power Status

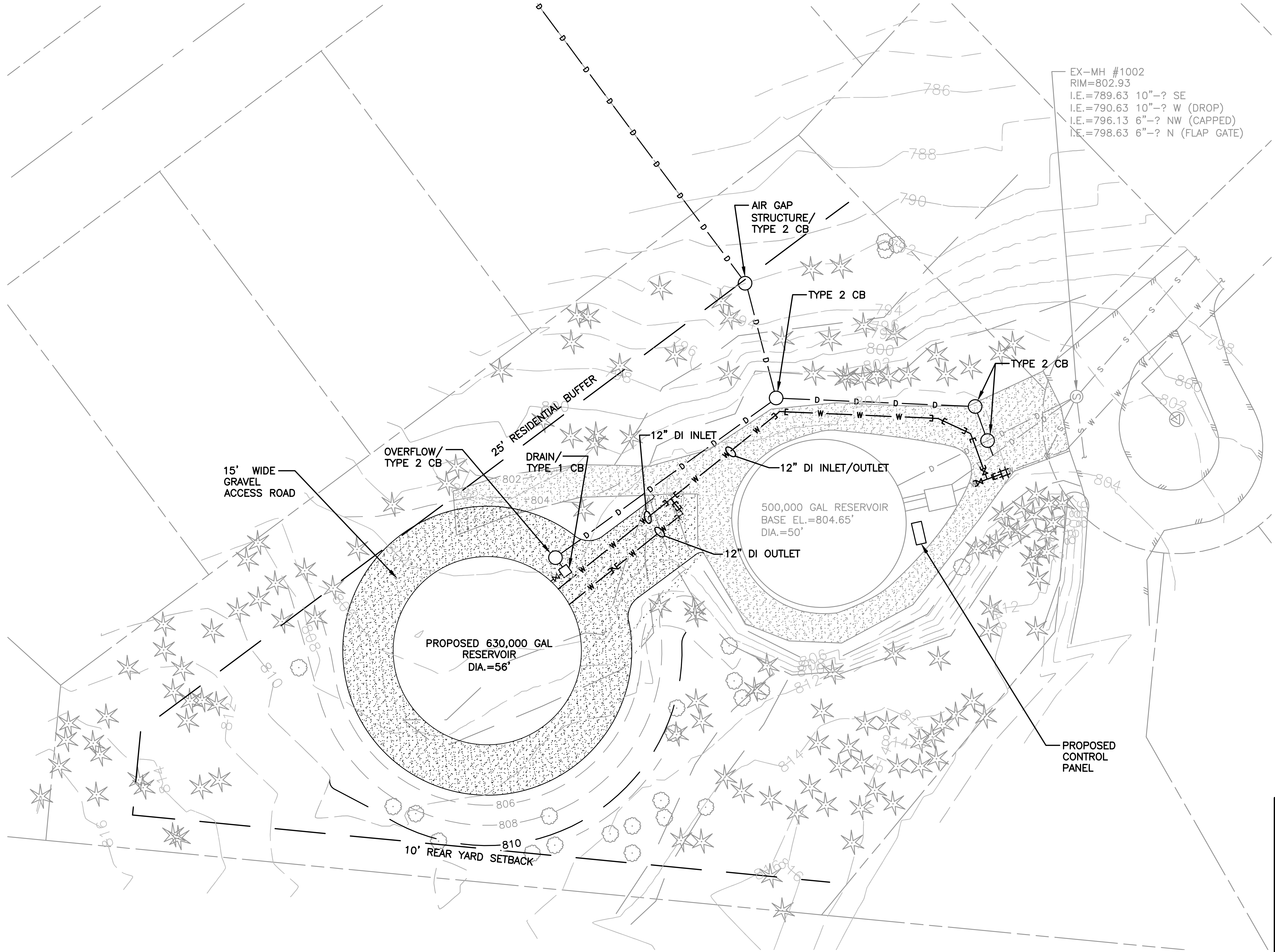
LAKE WHATCOM WATER & SEWER DISTRICT

DIVISION 22 RESERVOIR PRE-DESIGN

FIGURE 5-1

EXISTING SITE PLAN

Gray & Osborne, Inc.
CONSULTING ENGINEERS
SEATTLE–YAKIMA



LAKE WHATCOM WATER & SEWER DISTRICT

DIVISION 22 RESERVOIR PRE-DESIGN

FIGURE 5-2

PROPOSED SITE PLAN

Gray & Osborne, Inc.
CONSULTING ENGINEERS
SEATTLE–YAKIMA

The PLC will relay the following alarms:

- Intrusion
- Overflow
- Communication Failure
- Power Fail (Control and 120V)
- PLC Fail
- VFD Fail
- Existing Reservoir Low Level
- Existing Reservoir High Level
- Proposed Reservoir Low Level
- Proposed Reservoir High Level

TELEMETRY

The new facilities will be integrated into the District's existing Supervisory Control and Data Acquisition (SCADA) system for full monitoring and alarming at the District's main office.

PERMITS

Because the project is a non-residential use in a residential zone, it is allowed by conditional use. Therefore the following processes/permits are expected to be required:

- Zoning Preapplication Meeting. The following items must be turned in at, or prior to the meeting:

    - Application (includes site plan, parcel & owner information, etc.)
    - Preliminary Stormwater Proposal
    - Traffic & Concurrency Information

- The project will be required to complete a SEPA Checklist/determination. The District will act as the SEPA official for this project, most likely with a DNS issued.

- Conditional Use Application. Subsequent to the preapplication meeting, the following items must be turned in to continue the conditional use process:

    - Conditional Use Application - Master
    - Land Disturbance Permit Application
    - Zoning/Land Use consistency approval
    - Tree canopy maps
    - Notification

- Public Hearing (in addition to the public hearings required by the County, the District will meet with the Sudden Valley HOA to discuss the project and make provisions to gain approval from the HOA.)

Upon approval of the Conditional Use, the District can then get approval for the land disturbance, design & bid the project, attend a pre-construction meeting and begin construction. . The project will be restricted to land clearing and grading activities during June 1 through September 30 per Whatcom County Code 20.51.410 – Seasonal clearing activity limitations.

Acquisition of the building permit for the tank will be the responsibility of the Contractor hired by the District.

SETBACKS

While the setback requirements will be determined as part of the Conditional Use Permit requirements, the preliminary design assumes that the setbacks will be similar to those required for commercial developments. Per Whatcom County Code 20.62.550-Buffer area, the minimum side and rear yard setbacks for commercial developments adjacent to residential areas are 25 feet. The buffer area would apply near the north property line, which borders residential lots. Per the setbacks table contained in Whatcom County Code 20.80.210-Minimum setbacks, the minimum rear yard setback for a commercial property is 10 feet. The minimum rear yard setback would apply to the western property line, which borders the Stimpson Family Nature Reserve.

STORMWATER REQUIREMENTS

The site is located within the Lake Whatcom Watershed in Whatcom County, and the county's NPDES Phase II permit area. The Lake Whatcom watershed is a sensitive body of water that supplies drinking water to the Lake Whatcom Water & Sewer District and the City of Bellingham. Due to the sensitive nature of the lake, the county has implemented several restrictions regarding development projects within the watershed. The site is subject to several regulations relating to stormwater runoff:

- Zoning Code – WCC 20.32 Residential Rural District (RR) (1996)
- Zoning Code - WCC 20.51 Lake Whatcom Watershed Overlay District (2013)
- Zoning Code - WCC 20.80 Supplementary Requirements (2010)
- Whatcom County Development Standards, Chapter 2, Stormwater (1999, revised 2002)
- Whatcom County Development Standards, Chapter 2, Stormwater, Section 221 Stormwater Special District Standards (2002)
- Stormwater Management Manual for Western Washington – Ecology (2012)

- Whatcom County Phase II NPDES Permit (issued 2013)

It appears that Title 20.51, adopted in 2013, supersedes and/or modifies all of the others. The intent of Title 20.51 is to "…manage and treat stormwater runoff and establish more stringent standards on clearing activities and reduce phosphorus loading into Lake Whatcom,..." The most pertinent regulations are summarized below:

- <u>WCC 20.51 – Lake Whatcom Watershed Overlay District</u>. Passed in 2013, this code section modifies Title 20, WC development Standards Chapter 2 and Section 221. Therefore this section takes precedence over all other state and local regulations.

  - 20.51.410 – Seasonal clearing activity limitations.
  - 20.51.420 – Permanent stormwater management systems. In addition to recording a Declaration of Covenant per the county's requirements to ensure the continued maintenance and operation of the stormwater system of the site, all projects shall:

    - Not exceed the natural runoff phosphorus loading profile; and
    - Incorporate presumptive BMPs and/or demonstrative BMPs to the new impervious areas and new disturbed areas.

      - Presumptive BMPs include:

        - Full infiltration and full downspout infiltration (per Ecology Manual BMP T5.10A).
        - Full dispersion (per Ecology Manual BMP T5.30).

      - Demonstrative BMPs must meet Ecology Minimum Requirements #3-#9, while also conforming to at least one of the following:

        - Phosphorus reduction to less than 0.1875 lb of P/acre/year;
        - No increase in monthly runoff volume; or
        - No runoff (disperse all of it).

- <u>Stormwater Management Manual for Western Washington AND the County's NPDES Phase II Permit.</u> Review of the requirements of the NPDES Phase II Permit (Appendix 1) and the Ecology Manual indicate that the project is required to apply all the minimum requirements (1-9) to the new and replaced impervious surfaces for the project.

- Minimum Requirement #4 - Preservation of Natural Drainage Systems and Outfalls. If the 100-year peak discharge from the site is less than 0.3 cfs under existing conditions and will remain under 0.3 cfs for the proposed conditions, runoff may be dispersed onsite, without needing to construct a tight-line conveyance system. The existing peak runoff is 0.161 cfs and the peak runoff for the completed project is 0.234 cfs, therefore a tight-line is not necessary.

- Minimum Requirement #5 – On-Site Stormwater Management. Requires that projects utilize BMPs to the greatest extent feasible to reduce the amount of runoff from the site. Projects that are exempt from MR #7, Flow Control, do not have to achieve the LID standard, but are required to implement soil amendments and dispersion to the extent feasible. Review of MR #7 indicates the project is exempt from flow control since the project adds less than 10,000 square feet of new or replaced impervious surface.

- Minimum Requirement # 6 – Runoff Treatment. Requires that projects creating more than 5,000 square feet (SF) of new and replaced Pollution Generating Impervious Surfaces (PGIS) construct treatment facilities. This project is exempt from this requirement since it only adds 3,976 SF of PGIS, but also removes 696 SF for a net increase of 3,280 SF of PGIS.

To summarize, under the regulations of the Ecology manual and the county's NPDES Phase II permit, the project is exempt from constructing flow control facilities (but would be required to implement flow control BMPs) and is exempt from runoff treatment. However, WCC 20.51 takes precedence over all other local regulations and can be more stringent that state regulations and therefore, the project must incorporate presumptive BMPs and/or demonstrative BMPs to be applied to the new impervious and new disturbed areas. It is believed that flow dispersion can be implemented on the site to the greatest feasible extent to control the runoff and provide adequate treatment of stormwater.

A phone call to Whatcom County Public Works Engineering Services confirmed that the project will be subject to the Ecology manual requirements, as it relates to, and as stated in WCC 20.51.420. A Preliminary Stormwater Proposal will be required to be submitted to the County at the time of Conditional Use Permit application. The County may also require a Stormwater Design Report prior to issuance of any construction permits.

INVESTMENT GRADE ENERGY AUDIT

Because the financing for the project includes a Drinking Water State Revolving Fund (DWSRF), the project must meet Investment Grade Efficiency Audit (IGEA) requirements. The IGEA requirements can be met in the following ways:

1. Documentation that you have met the IGEA requirements in the past.
2. A third party design review of your project.
3. Demonstrating there are no "obtainable" energy savings.
4. Complete a preliminary energy audit and/or an Investment Grade Efficiency Audit (IGEA) on your existing system.

This project does not include the installation or replacement of any motors, pumps, blowers, electrical, or heating/air conditioning equipment. Since the hydraulic gradeline of the existing system is set by the elevation of the existing reservoir, the installation of the proposed reservoir will not impact the power required to pump water from the Water Treatment Plant to the reservoirs. Therefore, there are no "obtainable" energy savings for this project.

OPERATIONS

During construction, the existing reservoir will be kept on-line. The existing reservoir is served by a single inlet/outlet line connected to the Sudden Valley Zone 6 distribution system. The inlet and outlet for the existing reservoir branch off from the inlet/outlet line just outside the tank. The inlet line discharges near the top of the existing tank. The outlet line draws from the bottom of the tank and contains a check valve. In order to accommodate the existing reservoir inlet/outlet configuration and ensure adequate turnover in both reservoirs, the existing and new reservoirs will be operated in parallel.

CONSTRUCTION SCHEDULE

A preliminary construction schedule is as follows:

- June 2015 – Complete Clearing and Grading Design/Advertise
- July 2015 – Award Clearing and Grading Construction Contract
- September 2014 – Complete Clearing and Grading Activities
- January 2016 – Complete Reservoir Design/Advertise
- February 2016 – Award Reservoir Construction Contract
- September 2016 – Complete Reservoir Construction

CONSTRUCTION COST ESTIMATE

Table 5-1 provides the estimated construction costs for the project.

TABLE 5-1

Project Construction Cost Estimate

No.	Item	Quantity		Unit Price	Amount
1.	Minor Changes	1	CALC	$25,000.00	$25,000.00
2.	Mobilization and Demobilization	1	LS	$95,000.00	$95,000.00
3.	Clearing and Grubbing	1	LS	$10,000.00	$10,000.00
4.	Temporary Erosion Control	1	LS	$5,000.00	$5,000.00
5.	Locate Existing Utilities	1	LS	$2,000.00	$2,000.00
6.	Trench Excavation Safety System	1	LS	$3,000.00	$3,000.00
7.	Site Earthwork	1	LS	$50,000.00	$50,000.00
8.	Unsuitable Excavation	200	CY	$40.00	$8,000.00
9.	Site Piping	1	LS	$68,000.00	$68,000.00
10.	Gravel Borrow	250	TN	$20.00	$5,000.00
11.	Crushed Surfacing Base Course	540	TN	$25.00	$13,500.00
12.	Surface Restoration	1	LS	$2,000.00	$2,000.00
13.	Welded Steel Reservoir	1	LS	$490,000.00	$490,000.00
14.	Electrical, Telemetry, and Instrumentation	1	LS	$100,000.00	$100,000.00

Subtotal $861,500.00
Contingency (15%) $87,000.00
Sales Tax at 8.5% $73,300.00
Total Construction Cost: $1,021,800.00

APPENDIX A

GEOTECHNICAL REPORT

GEOTECHNICAL REPORT
PROPOSED LWWSD Div. 22 RESERVOIR
Whatcom County, Washington

PROJECT NO. 14-250
December 2014

Prepared for:

Gray & Osborne, Inc.
CONSULTING ENGINEERS

PanGEO
INCORPORATED

Geotechnical & Earthquake
Engineering Consultants

Geotechnical & Earthquake
Engineering Consultants

December 18, 2014
PanGEO Project No. 14-250

Mr. Josef Dalaeli, P.E.
Gray & Osborne, Inc.
701 Dexter Avenue North, Suite 200
Seattle, WA 98109

Subject: **GEOTECHNICAL REPORT**
LWWSD Division 22 Reservoir
Whatcom County (Sudden Valley), Washington
Gray & Osborne IPN #14456

Dear Mr. Dalaeli,

PanGEO completed a geotechnical study to assist the project team with the design and construction of a proposed 500,000 gallon tank for the Lake Whatcom Water and Sewer District (LWWSD). The results of our study and our recommendations are presented in the attached report.

In summary, our test pits at the project site encountered up to 5 feet of loose to dense undifferentiated glacial deposits overlying stiff to hard or medium dense to very dense completely weathered siltstone and sandstone. It is our opinion that the proposed tank may be supported on a conventional shallow foundation, provided the foundation bears on competent glacial deposits or on completely weathered bedrock.

We appreciate the opportunity to be of service. Should you have any questions, please do not hesitate to call.

Sincerely,

Siew L. Tan, P.E.
Principal Geotechnical Engineer

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Seattle, WA 98102
T. (206) 262-0370
F. (206) 262-0374

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1.0 GENERAL

PanGEO completed a geotechnical engineering study to assist the project team with the design and construction of a new 500,000 gallon tank in the Sudden Valley community of Whatcom County, Washington. Our work was performed in accordance with our proposal dated April 16, 2013. The purpose of our geotechnical study was to evaluate subsurface conditions at the site and, based on the conditions encountered, provide geotechnical engineering recommendations pertinent to the design and construction of the proposed tank. Our services included a site reconnaissance, observing test pit explorations, and developing the conclusions and recommendations presented in this report.

2.0 SITE AND PROJECT DESCRIPTION

We understand it is planned to construct a new tank at the existing Lake Whatcom Water and Sewer District Division 22 reservoir facility located at the north end of Water Tower Court in the Sudden Valley community of Whatcom County, Washington. The approximate location of the project site is shown on Figure 1, Vicinity Map. We understand the proposed 500,000 gallon welded steel tank will be constructed approximately 50 feet north of the existing reservoir approximately as shown on Figure 2. We understand the new tank will be roughly 50 feet in diameter and will be benched into a gentle east-facing slope. The base elevation of the tank is anticipated to be around 804 feet. As such, excavations to reach the foundation elevation will likely be on the order of 2 to 7 feet below existing grade. We understand retaining walls up to 5 feet high may be needed to retain cuts on the west to southwest portions of the tank excavation.

Underground utilities associated with this project will include 12-inch diameter ductile iron inlet and outlet pipes that will likely be on the order of 4 to 6 feet below grade. In addition, we understand installation of a sanitary sewer line extending downslope to the east from the existing manhole at the south end of the facility is being considered.

3.0 SUBSURFACE EXPLORATIONS

Five test pits (TP-1 through TP-5) were excavated on October 16, 2014, to explore subsurface conditions at the site. The approximate test pit locations were measured from existing structures and property corners that had been staked in the field. The approximate locations of our test pits are indicated on Figure 2. The test pits were excavated to depths between 5½ and 8½ feet below the existing ground surface using a Kubota KX121-3 mini-excavator owned and operated by the LWWSD.

A geologist from PanGEO was present during the field explorations to observe the test pit excavations, obtain representative samples, and to describe and document the soils encountered in the explorations. Summary test pit logs are presented in Appendix A which provide descriptions of the materials encountered, depths to soil contacts, and depths of seepage or caving, if present, observed in the test pit sidewalls. The relative in-situ density of cohesionless soils, or the relative consistency of fine-grained soils, was estimated from the excavating action of the excavator, probing the sidewalls with a ½-inch diameter steel rod, and the stability of the test pit sidewalls. Where soil contacts were gradual or undulating, the average depth of the contact was recorded in the log. After each test pit was logged, the excavation was backfilled with the excavated soils and the surface was tamped and re-graded smooth.

4.0 CRITICAL AREAS CONSIDERATIONS

As part of our study, we reviewed the Whatcom Critical Areas Ordinance – Geologically Hazardous Areas maps available on the Whatcom County Planning and Development Services website (http://www.co.whatcom.wa.us/pds/gis/gismaps/cao.jsp). Based on our review, the subject site is not mapped within a landslide, mine, liquefaction, or volcanic hazard area.

5.0 SUBSURFACE CONDITIONS

5.1 Site Geology and Soil

Based on review of the *Geologic Map of the Bellingham 1:100,000 Quadrangle, Washington* (Lapen, 2000), the surficial geologic unit mapped at the site consists of the Eocene-aged Padden Member of the Chuckanut Formation. Lapen describes the Padden Member as moderately to well-sorted sandstone and conglomerate with alternating mudstone and minor coal.

The subsurface conditions at each of our test pit locations was generally consistent with the mapped geology and encountered completely weathered to highly weathered siltstone and sandstone at relatively shallow depths. Detailed test pit logs are provided in Appendix A of this report. The following is a summary of the subsurface conditions encountered in the test pits:

> **Glacial Deposits:** At test pits TP-1 through TP-3 and at TP-5, 2 to 5 feet of silty sand with gravel to sandy silt with gravel that we interpret to be glacial deposits were encountered. The near surface glacial deposits were typically loose to medium dense and graded to medium dense to dense within about 1 to 2 feet below grade. The glacial deposits typically exhibited a till-like appearance and were weathered.
>
> **Completely Weathered to Highly Weathered Siltstone and Sandstone:** Underlying the glacial deposits at TP-1 through TP-3 and TP-5, and near the surface at TP-4, soils that we interpret to be residual soils of the mapped Padden Member of the Chuckanut Formation were encountered. At test pits TP-1 through TP-4, the residual soils typically consisted of medium stiff to hard sandy silt to silt. The residual soil at TP-5 consisted of medium dense to very dense silty sand to poorly graded sand with silt. At each test pit location an increase in relative density/consistency with depth was noted and the residual soils were encountered to the maximum depth explored at each test pit location.

5.2 GROUNDWATER

Groundwater was not encountered at the time of test pit excavation. However, zones of iron oxide were typically observed near the contact between the glacial deposits and the underlying residual soil, which is likely indicative of surface water percolating through the upper weathered soil and perching on the lower-permeability soil. In addition, manganese oxide staining was observed in the fractured silt zones encountered at TP-3 and TP-4.

Groundwater elevations and seepage rates are likely to vary depending on the season, local subsurface conditions, and other factors. Groundwater levels and seepage rates are normally highest during the winter and early spring.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 SEISMIC CONSIDERATIONS

6.1.1 Site Seismicity

The subject site is located on the north flank of Lookout Mountain in Whatcom County, Washington. Review of the *Geologic Map of the Bellingham 1:100,000 Quadrangle, Washington* indicates that there are not any faults mapped within an approximately 8 mile radius of the site. Furthermore, review of the USGS Earthquake Hazards Program Quaternary fault map (http://geohazards.usgs.gov/qfaults/map.php), which contains information on faults that are believed to be sources of M>6 earthquakes during the Quaternary period (i.e. the past 1.6 million years), indicates that the nearest fault with M>6 Quaternary activity is the Devils Mountain Fault located approximately 25 miles south of the site.

6.1.2 Seismic Design Parameters

The seismic design of the new tank can be accomplished using the 2012 or later editions of the International Building Code (IBC), which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years). The seismic design of the tank should also follow the procedures contained in the American Water Works Association's (AWWA) Standard for Welded Carbon Steel Tanks for Water Storage (AWWA D100-11). The table on the following page presents the seismic design parameters in accordance with the 2012 IBC, which are consistent with the 2008 USGS seismic hazard maps.

Table 1 – Summary Seismic Design Parameters

Site Class	Spectral Acceleration at 0.2 sec. (g)	Spectral Acceleration at 1.0 sec. (g)	Site Coefficients		Design Spectral Response Parameters		Control Periods (sec.)		Design PGA (S_{DS}/2.5)
	S_S	S_1	F_a	F_v	S_{DS}	S_{D1}	T_O	T_S	
C	0.943	0.368	1.02	1.43	0.64	0.35	0.11	0.55	0.26

6.1.3 Liquefaction

Seismically induced liquefaction typically occurs in loose, saturated, sandy and silty materials. In our opinion, liquefaction is not a design consideration for this site because of the completely to highly weathered bedrock encountered at relatively shallow depths in our test pits.

6.2 TANK FOUNDATION DESIGN

Based on the subsurface conditions encountered in our test pit explorations at the site, it is our opinion that a conventional shallow foundation, consisting of a mat slab or a ring footing, is an appropriate foundation type to support the proposed 500,000 gallon tank, provided that the foundation bears upon at least 1 foot of Crushed Surfacing Base Course (CSBC, WSDOT 9.03.9(3)) placed upon either undisturbed dense glacial deposits or on very stiff to hard completely weathered siltstone. Based on our understanding of the current design, we anticipate competent soils will be encountered in the footing excavation.

6.2.1 Subgrade Preparation

We recommend excavating the foundation at least 1 foot below the bottom of footing and backfilling with CSBC compacted to the project requirements for structural fill. We also recommend that a geotextile fabric be placed at the bottom of the excavation before placing the CSBC. The geotextile fabric may be selected based on Table 3, Section 9-33.2(1) of the 2014 WSDOT Standard Specifications.

The bottom of the foundation excavation should be observed and verified by PanGEO to confirm that the exposed subgrade is consistent with the anticipated conditions and adequate to support the proposed reservoir. All foundation subgrade should be carefully prepared and in firm condition. If soft/loose subgrade soil is encountered, it should be overexcavated to expose competent native soil and replaced with CSBC or lean mix concrete. If overexcavation is warranted, we do not anticipate overexcavation depth would exceed 2 feet.

6.2.2 Allowable Bearing Pressure

For a foundation subgrade prepared as discussed above, we recommend that an allowable soil bearing pressure of 4,000 pounds per square foot (psf) be used for sizing the foundation. For allowable stress design, the recommended allowable bearing pressure may be increased by 1/3 for transient conditions such as wind and seismic loading. A modulus of subgrade reaction of 200 pci may be utilized for design of a mat slab. The reservoir foundation should be placed at a minimum depth of 18 inches below the final exterior grade.

Total and differential settlements are anticipated to be within tolerable limits for foundations designed and constructed as discussed above. Footing settlement under static loading conditions is estimated to be less than approximately ½-inch, and differential settlement across the reservoir should be less than about ¼-inch.

6.2.3 Lateral Resistance

Lateral forces from wind or seismic loading may be resisted by a combination of passive earth pressures acting against the embedded portions of the foundation, and by friction acting on the base of the foundation. Passive resistance values may be determined using an equivalent fluid weight of 350 pounds per cubic foot (pcf). This value includes a factor safety of at least 1.5 assuming that properly compacted structural fill will be placed adjacent to the sides of the footings. A friction coefficient of 0.40 may be used to determine the frictional resistance at the base of the footings, provided the footings are poured on CSBC as recommended. This coefficient includes a factor safety of approximately 1.5.

6.3 Retaining Walls

We understand retaining walls up to about 5 feet high may be constructed on the west to southwest portions of the tank to retain excavations on the west and southwest portions of the tanks. Given the limited height of the retaining wall, several wall options may be considered. The selection of wall type depends on several factors, including cost, performance, aesthetics, and constructability. For this project, it is our opinion that gravity walls such as a pre-cast concrete block walls are appropriate. Although a conventional cast-in-place concrete wall is also considered appropriate, a gravity wall is likely the more economical wall option.

6.3.1 Gravity Walls

The principal advantage of a gravity wall is the ease and speed of construction, and the relatively low construction cost. If a gravity wall will be used for this project, we recommend a concrete block wall be utilized.

Concrete blocks should have a minimum dimension of 2½ feet by 2½ feet by 5 feet such as Ultrablocks (www.ultrablocks.com) and be made of new concrete. Blocks made of returned concrete, or having dimensions of 2 feet by 2 feet by 6 feet (i.e. ecology blocks) should not be used. Concrete blocks can be made with various finishes or textures to provide the desired aesthetics. Typical block layouts for Ultrablock walls up to 3-blocks high are shown on Figure 3.

Minimum Width – For Ultrablock walls up to 3-blocks high constructed in front of stable cuts, the wall should have a minimum width of 2½ feet.

Minimum Embedment - Walls should have a minimum of one foot of embedment. All walls should be founded on competent native soils or properly compacted fill. If needed, a 6-inch layer of granular structural fill such as crushed rock may be placed as a leveling course before placing the base course of blocks.

Foundation Preparation – Competent soils are anticipated to be encountered at the wall subgrade elevation. If unstable soils are encountered at the foundation subgrade elevation, it should be removed to competent soil and the excavation should be backfilled with adequately compacted CSBC. As a minimum, we recommend at least 4 inches of CSBC be placed as levelling course below the bottom blocks.

Surcharge -. For the typical wall section shown on Figure 3, we assume that no surcharge will be present behind the block wall.

6.3.2 Cast-In-Place Concrete Walls

Concrete retaining walls may be designed for an earth pressure based upon an equivalent fluid weight of 35 pcf. The recommended lateral pressures assume that adequate wall drainage provisions will be incorporated into the design and construction of the walls, and that properly compacted free-draining structural fill will be used for wall backfill. On-site soils should not be used as wall backfill because of its poor drainage characteristics.

Wall footings should be supported on relatively undisturbed native soils, or compacted structural fill placed on native soils. As such, an allowable bearing pressure of 2,000 psf may be used to size the footing. Lateral resistance may be computed using an allowable friction coefficient of 0.35 at the base of footings, and an allowable passive resistance of 350 pcf against the embedded portion of the foundation element.

Lateral pressures from surface surcharges located within a distance equal to the exposed wall height should be estimated using a lateral pressure coefficient of 0.3 (i.e. the ratio of lateral pressure to vertical pressure). Where applicable, a lateral uniform pressure of 80 psf should be used to account for traffic surcharge.

6.4 NEW UTILITIES

6.4.1 Trench Excavation

We anticipate that utility excavations will generally be less than 8 feet deep and will encounter material that can be excavated with conventional excavation equipment. If site excavations extend deeper that the depths explored at our test pit locations, less weathered, stronger bedrock (i.e. siltstone and sandstone) that may require specialized excavating equipment could be encountered. All excavations in excess of 4 feet in depth should be sloped in accordance with Washington Administrative Code (WAC) 296-155, or be shored. It is the contractor's responsibility to maintain safe working conditions, including temporary excavation stability and dewatering.

6.4.2 Pipe Support and Bedding

Based on our field explorations, we anticipate medium stiff to very stiff sandy silt or medium dense to dense silty sand suitable to support utility pipes will be encountered in utility trench excavations. Utility installation should be conducted in accordance with the 2014 WSDOT Standard Specifications or other applicable specifications for placement and compaction of pipe bedding and backfill. In general, pipe bedding should be placed in loose lifts not exceeding 6 inches in thickness, and compacted to a firm and unyielding condition. Bedding materials and thicknesses provided should be suitable for the utility system and materials installed, and in accordance with any applicable manufacturers' recommendations. Pipe bedding materials should be placed on relatively undisturbed native soil. Soft soils, if present, should be removed from the bottom of the trench and replaced with pipe bedding material.

6.4.3 Trench Backfill

The onsite soils are not considered suitable for use as trench backfill due to an excessive fines content. Trench backfill should consist of imported granular material meeting the requirements for Gravel Borrow as specified in Section 9-03.14(1) of the 2014 WSDOT *Standard Specifications*, CSBC, or an approved equivalent. The trench backfill should be placed in 8- to 12-inch, loose lifts and compacted using mechanical equipment to at least 95 percent maximum dry density, per ASTM D1557 (Modified Proctor). Heavy compaction equipment should not be permitted to operate directly over utilities until a minimum of 2 feet of backfill has been placed.

6.4.4 Thrust Blocks

Where needed, we recommend that thrust blocks be sized using an allowable passive pressure calculated using an equivalent fluid unit weight of 350 pcf, assuming the thrust blocks will be constructed against undisturbed native soils or against properly compacted structural fill.

7.0 EARTHWORK CONSIDERATIONS

7.1 Site Preparation

Site preparation for the proposed project includes striping and clearing of any remaining surface vegetation and rootballs and excavating to the design subgrade. All stripped materials should be disposed off-site or be "wasted" on site in non-structural landscaping areas.

7.2 Temporary Excavations and Permanent Slopes

We anticipate that utility excavations will generally be less than 8 feet deep. Based on our understanding of the subsurface conditions at the sites, we anticipate that the excavations will largely encounter medium dense to dense silty sand with gravel and medium stiff to very stiff sandy silt. All temporary excavations should be performed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring. For planning purposes, the temporary excavations may be sloped as steep as 1H:1V, but should be re-evaluated in the field during construction based on actual observed soil conditions. During wet weather, the cut slopes may need to be flattened to reduce potential erosion.

Permanent cut slopes should be graded no steeper than 2H:1V and should be track-walked then promptly planted with an appropriate species of vegetation. Alternatively, permanent slopes may be armored with quarry spalls (WSDOT 9-13.6) for erosion protection.

7.3 Material Reuse

The onsite soils generally have an estimated fines content in excess of 50 percent. Due to the high fines content of the soils expected to be encountered at the site, it is our opinion that the on-site soils should not be used as a structural fill. The on-site soils may only be used as general fill in non-structural areas.

7.4 Structural Fill and Compaction

Reservoir Foundation Backfill – Within the footprint of the proposed reservoir, we recommend that the structural fill consist of Crushed Surfacing Base Course as specified in section 9-03.9(3) of the 2014 WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (WSDOT, 2012), or an approved similar material.

Areas Outside of Reservoir Footprint – If structural fill is needed outside of the reservoir footprint, such as for access roads, or to raise grades below associated structures, we recommend importing structural fill. Imported structural fill, if needed, should consist of clean, free-draining granular soils that are relatively free from organic matter or other deleterious materials. Such materials should be less than 4 inches in maximum dimension, with less than 7 percent fines (portion passing the U. S. Standard No. 200 sieve), as specified for Gravel Borrow in Section 9-03.14(1) of the 2014 WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction*. The fine-grained portion of structural fill soils should be non-plastic. A fines content greater than 7 percent may be acceptable if the earthwork is performed during relatively dry weather and the contractor's methods are conducive to proper compaction of the soil. The use of material with a fines content greater than 7 percent should be approved by the project engineer prior to use.

All structural fill should be moisture conditioned to within about 3 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and compacted to at least 95 percent maximum density, determined using ASTM D 1557 (Modified Proctor). The procedure to achieve proper density of a compacted fill depends on the size and type of the compacting equipment, the number of passes, thickness of the layer being compacted, and certain soil properties. In areas where the size of the excavation restricts the use of heavy equipment, smaller equipment can be used, but the soil must be placed in thin enough layers to achieve the required relative compaction.

Generally, loosely compacted soils are a result of poor construction technique or improper moisture content. Soils with high fines contents are particularly susceptible to becoming too wet, and coarse-grained materials easily become too dry, for proper compaction. Silty or clayey soils with a moisture content too high for adequate compaction should be dried as necessary, or moisture conditioned by mixing with drier materials, or other methods.

7.5 Wet Weather Construction

General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below. The following procedures are best management practices recommended for use in wet weather construction:

- Earthwork should be performed in small areas to minimize subgrade exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.

- During wet weather, the allowable fines content of the structural fill should be reduced to no more than 5 percent by weight based on the portion passing ¾-inch sieve. The fines should be non-plastic.

- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water.

- Bales of straw and/or geotextile silt fences should be strategically located to control erosion and the movement of soil.

- Excavation slopes and soils stockpiled on site should be covered with plastic sheets.

7.6 Surface Drainage and Erosion Considerations

Surface runoff can be controlled during construction by careful grading practices. Typically, this includes the construction of shallow, upgrade perimeter ditches or low earthen berms in conjunction with silt fences to collect runoff and prevent water from entering excavations or to prevent runoff from the construction area from leaving the immediate work site. Temporary erosion control may require the use of geotextile silt fences or hay bales on the downhill side of the project to prevent water from leaving the site and potential storm water detention to trap sand and silt before the water is discharged to a suitable outlet. All collected water should be directed under control to a positive and permanent discharge system.

Permanent control of surface water should be incorporated in the final grading design. Adequate surface gradients and drainage systems should be incorporated into the design such that surface runoff is collected and directed away from the tank and to a suitable outlet. Potential problems associated with erosion may also be reduced by establishing vegetation within disturbed areas immediately following grading operations.

8.0 UNCERTAINTY AND LIMITATIONS

We have prepared this report for use by Gray & Osborne, the Lake Whatcom Water and Sewer District, and other project team members. Recommendations contained in this report are based on a site reconnaissance, a subsurface exploration program, review of pertinent geologic publications, and our understanding of the project. The study was performed using a mutually agreed-upon scope of work.

Variations in soil conditions may exist between the locations of the explorations and the actual conditions underlying the site. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at the site that are different from those described in this report, we should be notified immediately to review the applicability of our recommendations. Additionally, we should also be notified to review the applicability of our recommendations if there are any changes in the project scope.

The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design. Additionally, the scope of our work specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances. We are not mold consultants nor are our recommendations to be interpreted as being preventative of mold development. A mold specialist should be consulted for all mold-related issues.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or other factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 24 months from its issuance. PanGEO should be notified if the project is delayed by more than 24 months from the date of this report so that we may review the applicability of our conclusions considering the time lapse.

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk. Any party other than the client who wishes to use this report shall notify PanGEO of such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use this report.

Within the limitation of scope, schedule and budget, PanGEO engages in the practice of geotechnical engineering and endeavors to perform its services in accordance with generally accepted professional principles and practices at the time the Report or its contents were prepared. No warranty, express or implied, is made.

We appreciate the opportunity to be of service to you on this project. Please feel free to contact our office with any questions you have regarding our study, this report, or any geotechnical engineering related project issues.

Sincerely,

PanGEO, Inc.

Steven T. Swenson, L.G.
Project Geologist

Siew L. Tan, P.E.
Principal Geotechnical Engineer

9.0 REFERENCES

American Water Works Association (AWWA), 2011, *AWWA Standard for Welded Carbon Steel Tanks for Water Storage*, ANSI/AWWA D100-11.

International Building Code (IBC), 2012, International Code Council.

Lapen, Thomas J., 2000, Geologic Map of the Bellingham 1:100,000 Quadrangle, Washington: Washington Division of Geology and Earth Resources Open File Report 2000-5, 36 p., 2 plates, scale 1:100,000

WSDOT, 2014, *Standard Specifications for Road, Bridges, and Municipal Construction*.

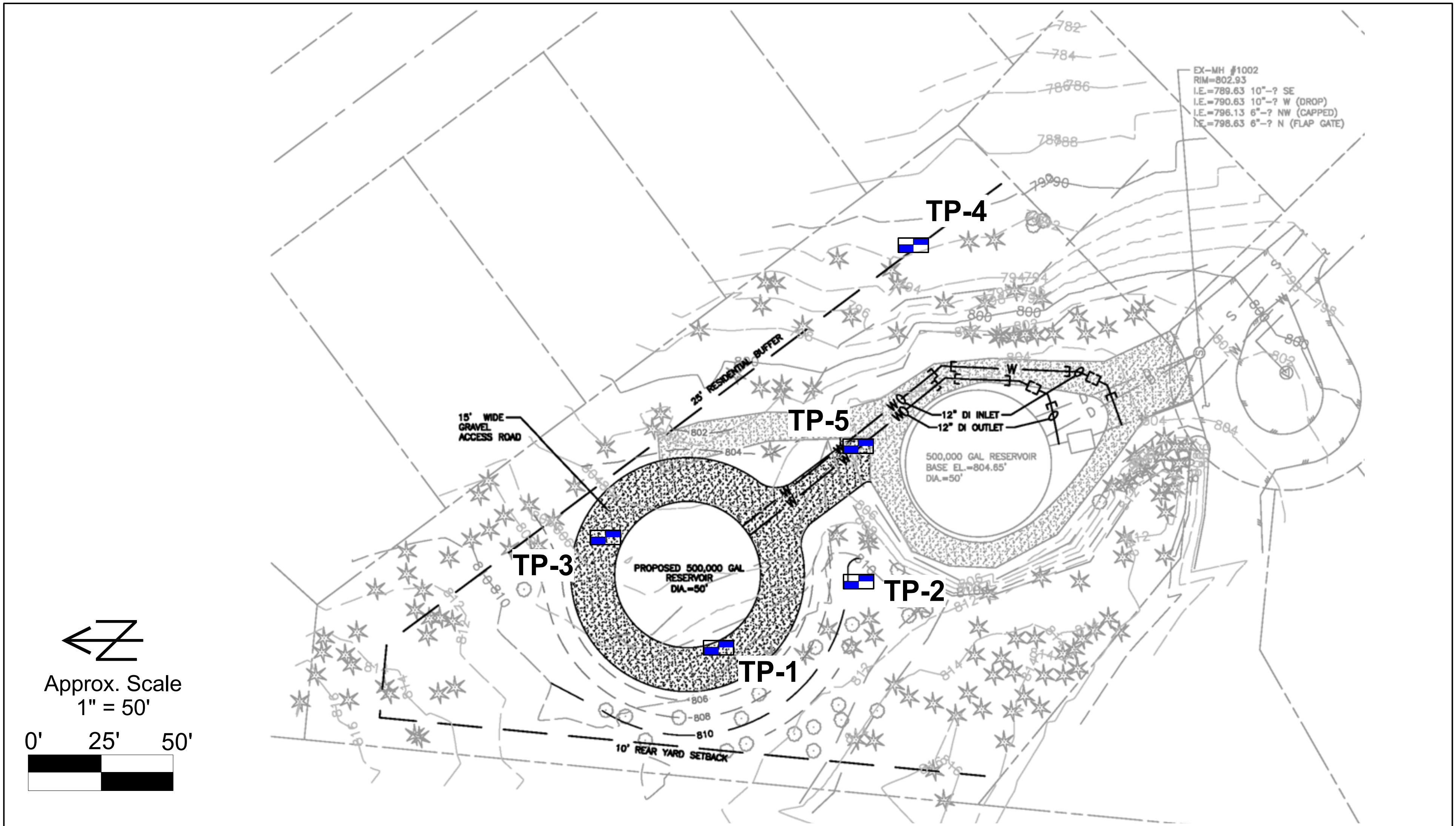
N

No Scale

Image Source: Google Maps

PanGEO INCORPORATED	**Proposed LWWSD Division 22 Reservoir Whatcom County, WA**	**VICINITY MAP**	
		Project No. **14-250**	Figure No. **1**

14-250 Fig 1 Vicinity.grf 11/24/14 (12:00) STS



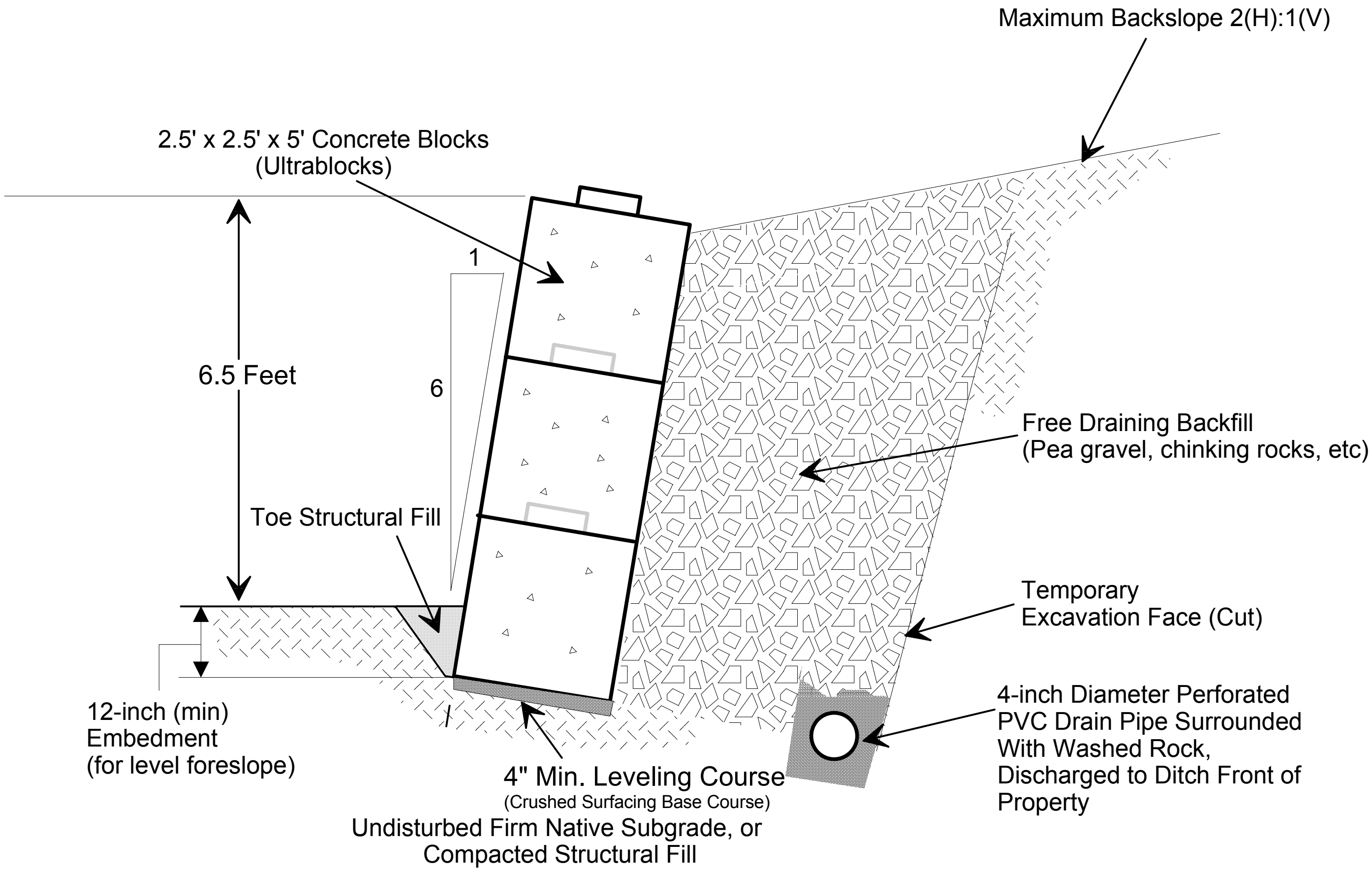
Legend:

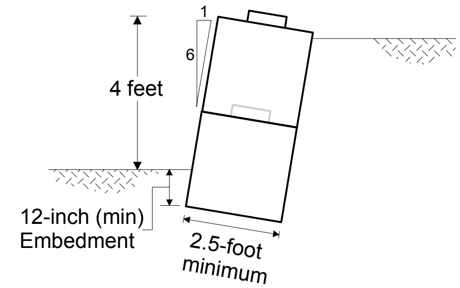
Approx. Test Pit Location

Note: Base map modified a pre-design proposed site plan provided by Gray & Osborne.
Elevation contours are approximate and based on the NAVD88 Datum.

PanGEO INCORPORATED	**Proposed LWWSD Division 22 Reservoir Whatcom County, WA**	**SITE AND EXPLORATION PLAN**
		Project No. **14-250** / Figure No. **2**

14-250 Fig 3 Ultrablock.grf 12/18/14 (16:22) STS





2-Block Configuration

Typical Modular Block Wall Schematic
(Not To Scale)

PanGEO INCORPORATED	Proposed LWWSD Division 22 Reservoir Whatcom County, WA	ULTRABLOCK GRAVITY WALL DETAIL (Retaining Cut)	
		Project No. **14-250**	Figure No. **3**

APPENDIX A

TEST PIT LOGS

RELATIVE DENSITY / CONSISTENCY

SAND / GRAVEL			SILT / CLAY		
Density	SPT N-values	Approx. Relative Density (%)	Consistency	SPT N-values	Approx. Undrained Shear Strength (psf)
Very Loose	<4	<15	Very Soft	<2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Med. Dense	10 to 30	35 - 65	Med. Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	>50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	>30	>4000

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		GROUP DESCRIPTIONS	
Gravel 50% or more of the coarse fraction retained on the #4 sieve. Use dual symbols (eg. GP-GM) for 5% to 12% fines.	GRAVEL (<5% fines)	GW	Well-graded GRAVEL
		GP	Poorly-graded GRAVEL
	GRAVEL (>12% fines)	GM	Silty GRAVEL
		GC	Clayey GRAVEL
Sand 50% or more of the coarse fraction passing the #4 sieve. Use dual symbols (eg. SP-SM) for 5% to 12% fines.	SAND (<5% fines)	SW	Well-graded SAND
		SP	Poorly-graded SAND
	SAND (>12% fines)	SM	Silty SAND
		SC	Clayey SAND
Silt and Clay 50%or more passing #200 sieve	Liquid Limit < 50	ML	SILT
		CL	Lean CLAY
		OL	Organic SILT or CLAY
	Liquid Limit > 50	MH	Elastic SILT
		CH	Fat CLAY
		OH	Organic SILT or CLAY
Highly Organic Soils		PT	PEAT

Notes:
1. Soil exploration logs contain material descriptions based on visual observation and field tests using a system modified from the Uniform Soil Classification System (USCS). Where necessary laboratory tests have been conducted (as noted in the "Other Tests" column), unit descriptions may include a classification. Please refer to the discussions in the report text for a more complete description of the subsurface conditions.

2. The graphic symbols given above are not inclusive of all symbols that may appear on the borehole logs. Other symbols may be used where field observations indicated mixed soil constituents or dual constituent materials.

DESCRIPTIONS OF SOIL STRUCTURES

Layered:	Units of material distinguished by color and/or composition from material units above and below	**Fissured:**	Breaks along defined planes
Laminated:	Layers of soil typically 0.05 to 1mm thick, max. 1 cm	**Slickensided:**	Fracture planes that are polished or glossy
Lens:	Layer of soil that pinches out laterally	**Blocky:**	Angular soil lumps that resist breakdown
Interlayered:	Alternating layers of differing soil material	**Disrupted:**	Soil that is broken and mixed
Pocket:	Erratic, discontinuous deposit of limited extent	**Scattered:**	Less than one per foot
Homogeneous:	Soil with uniform color and composition throughout	**Numerous:**	More than one per foot
		BCN:	Angle between bedding plane and a plane normal to core axis

COMPONENT DEFINITIONS

COMPONENT	SIZE / SIEVE RANGE	COMPONENT	SIZE / SIEVE RANGE
Boulder:	> 12 inches	Sand	
Cobbles:	3 to 12 inches	Coarse Sand:	#4 to #10 sieve (4.5 to 2.0 mm)
Gravel		Medium Sand:	#10 to #40 sieve (2.0 to 0.42 mm)
Coarse Gravel:	3 to 3/4 inches	Fine Sand:	#40 to #200 sieve (0.42 to 0.074 mm)
Fine Gravel:	3/4 inches to #4 sieve	Silt	0.074 to 0.002 mm
		Clay	<0.002 mm

TEST SYMBOLS

for In Situ and Laboratory Tests listed in "Other Tests" column.

ATT	Atterberg Limit Test
Comp	Compaction Tests
Con	Consolidation
DD	Dry Density
DS	Direct Shear
%F	Fines Content
GS	Grain Size
Perm	Permeability
PP	Pocket Penetrometer
R	R-value
SG	Specific Gravity
TV	Torvane
TXC	Triaxial Compression
UCC	Unconfined Compression

SYMBOLS

Sample/In Situ test types and intervals

- 2-inch OD Split Spoon, SPT (140-lb. hammer, 30" drop)
- 3.25-inch OD Spilt Spoon (300-lb hammer, 30" drop)
- Non-standard penetration test (see boring log for details)
- Thin wall (Shelby) tube
- Grab
- Rock core
- Vane Shear

MONITORING WELL

- Groundwater Level at time of drilling (ATD)
- Static Groundwater Level
- Cement / Concrete Seal
- Bentonite grout / seal
- Silica sand backfill
- Slotted tip
- Slough
- Bottom of Boring

MOISTURE CONTENT

Dry	Dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

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Terms and Symbols for Boring and Test Pit Logs

Figure A-1

TEST PIT LOGS

Test Pit No. 1 Location: See Figure 2 Approximate ground surface elevation: 810 feet	
Depth (ft)	Material Description
0 – 5	Loose to medium dense, dark brown to brown, sandy SILT with gravel, moist. Weathered. **(Glacial Deposits)** -Charcoal fragments near surface, abundant roots to 2’ -Becomes medium dense to dense around 2’, till-like -Gravels subround, trace cobbles -Iron oxide staining starting around 3’
5 – 8½	Stiff to very stiff, light brown to tan, fine sandy SILT, moist. **(Completely Weathered Siltstone)** -Iron oxide staining near top of soil unit -Increase in relative density with depth
	Test Pit terminated approximately 8½ feet below ground surface. No groundwater observed at the time of excavation.

Figure A-2

Test Pit No. 2

Location: See Figure 2

Approximate ground surface elevation: 810 feet

Depth (ft)	Material Description
0 – 3	Medium dense to dense, brown to brownish-gray, silty SAND with gravel to sandy SILT with gravel, moist. Weathered, till-like. **(Glacial Deposits)** -Charcoal fragments near surface, abundant roots to 2' -Gravels subround, trace cobbles -Iron oxide staining starting around 2'
3 – 7	Stiff to very stiff, light brown to tan, fine sandy SILT, moist. **(Completely to Highly Weathered Siltstone)** -Iron oxide staining near top of soil unit -Becomes gray around 6' -Increase in relative density with depth, practical excavation refusal at 7'
	Test Pit terminated approximately 7 feet below ground surface due to practical excavation refusal. No groundwater observed at the time of excavation.

Figure A-3

Test Pit No. 3 Location: See Figure 2 Approximate ground surface elevation: 805 feet	
Depth (ft)	Material Description
0 – 2	Medium dense to dense, brown to brownish-gray, silty SAND with gravel to sandy SILT with gravel, moist. Weathered. **(Glacial Deposits)** -Charcoal fragments near surface, abundant roots to 1½' -Gravels subround, trace cobbles
2 – 7	Very stiff to hard, gray, fine sandy SILT, moist. **(Completely to Highly Weathered Siltstone)** -Fractured, magnesium oxide staining along fracture planes -Increase in relative density with depth, practical excavation refusal at 7'
	Test Pit terminated approximately 7 feet below ground surface due to practical excavation refusal. No groundwater observed at the time of excavation.
Figure A-4	

Test Pit No. 4

Location: See Figure 2

Approximate ground surface elevation: 792 feet

Depth (ft)	Material Description
0 – 6½	Medium stiff to hard, orangish-brown, SILT with sand and gravel, moist. **(Completely Weathered Siltstone)** -Gravels comprised of less weathered pieces of siltstone -Numerous roots to 2' -Around 6 feet becomes gray and fractured, magnesium oxide staining along fracture planes -Increase in relative density with depth
	Test Pit terminated approximately 6½ feet below ground surface. No groundwater observed at the time of excavation.

Figure A-5

Test Pit No. 5 Location: See Figure 2 Approximate ground surface elevation: 806 feet	
Depth (ft)	Material Description
0 – 3½	Loose to medium dense, brown to brownish-gray, silty SAND with gravel, moist. Weathered, till-like. **(Glacial Deposits)** -Gravels subround, trace cobbles
3½ – 5½	Medium dense to very dense, brown, silty SAND to poorly graded SAND with silt, moist. **(Completely to Highly Weathered Sandstone)** -Increase in relative density with depth
	Test Pit terminated approximately 5½ feet below ground surface. No groundwater observed at the time of excavation.

Completed test pit.

Weathered sandstone from around 5 feet.

Figure A-6

Date Test Pits Excavated: October 16, 2014 using a Kubota KX121-3 mini-excavator owned and operated by the Lake Whatcom Water and Sewer District.
Test Pits Logged by: STS

APPENDIX B

DIVISION 22 RESERVOIR OVERFLOW ANALYSES

DIVISION 22 RESERVOIR OVERFLOW ANALYSES

LWWSD Project #C1401 – Division 22 Reservoir

Prepared by

Kristin Hemenway, PE and Bill Hunter, PE

March 18, 2015

EXISTING AND PROPOSED FACILITIES

The existing Division 22 reservoir has a nominal capacity of 500,000 gallons. The base elevation is approximately 805 feet (NAVD 88 datum), with an overflow elevation of 840 feet. The Division 22 Reservoir is fed from the Sudden Valley Water Treatment Plant via the Division 22 Transmission Pump Station, which contains two pumps with a capacity of 700 GPM at 608 TDH.

The second Division 22 Reservoir will be built adjacent to the existing Division 22 Reservoir, and is proposed to have a capacity of 630,000 gallons. A means for handling overflow of the existing and new reservoirs will be addressed as part of the design and bid package for the new reservoir.

ANALYSIS OBJECTIVES

The objective of this analysis is to analyze the current system hydraulics for handling overflow via the existing overflow route to the Strawberry Canyon Pump Station and to address and evaluate options if surcharge conditions exist. This analysis will be used as a foundation to further develop overflow features that may be required to handle the reservoir overflow.

EXECUTIVE SUMMARY

From the standpoint of cost and simplicity it appears that a combination of the alternatives is the District's preferred option. The preferred combination is to install an overflow pipe to Kinglet Court (Scenario 2) and raise the manhole rim elevation of MH 22-36 (Scenario 3). The additional pipe work would have minimal, if any, impact on the watershed with minimal impact to the project schedule.

Prioritized Overflow Solution Alternatives	
Scenario 2 + Scenario 3	Install 8" overflow pipe to Kinglet Court and raise MH 22-36 to allow surcharge back-up into the MH.
Scenario 2 + Scenario 4	Install 8" overflow pipe to Kinglet Court and install a flow splitting structure at the site of the new Division 22 reservoir.
Scenario 2 + Scenario 3	Install 8" overflow pipe to Kinglet Court and upsize the sewer pipe segment 22-087 to a 12" DI pipe.

Scenario Summary of Hydraulic Modeling	
Scenario 1 – Existing Conditions	Existing overflow conditions @ 700 GPM surcharges the system and exceed the pump-out capacity at Strawberry Canyon Pump Station.
Scenario 2 – Direct 700 GPM to Lake Whatcom Boulevard Interceptor	Install 8" overflow pipe from Division 22 reservoir to Kinglet Court. Hydraulic analysis shows that overflow conditions surcharge the system.
Scenario 3a – Direct 700 GPM to Lake Whatcom Boulevard Interceptor and Upsize Sewer Pipe Segment S22-087	Install 8" overflow pipe from Division 22 reservoir to Kinglet Court. Hydraulic analysis shows that replacing 277 LF of existing 8" VCP (segment 22-087) with a 12" DI pipe will handle the flow. The pipe replacement is along the high bank of a seasonal creek.
Scenario 3b – Direct 700 GPM to Lake Whatcom Boulevard Interceptor and Raise Manhole S22-36	Install 8" overflow pipe from Division 22 reservoir to Kinglet Court. Hydraulic analysis shows that raising MH S22-36 an additional 3' will handle the surcharge conditions in the system.
Scenario 4 – Install a Flow Splitting Structure	Install 8" overflow pipe from Division 22 reservoir to Kinglet Court along with a flow splitting structure at the Division 22 Reservoir site with a 3-way flow split. Splitting the flows will allow partial routing through the Strawberry Canyon Pump Station, Lake Whatcom Boulevard Interceptor and also via a level spreader.

The following scenarios were considered:

<u>Scenario 1 – Use Existing System Overflow</u>

Overflow at the existing Division 22 reservoir is routed through a 10-inch cast-iron overflow pipe that transitions to an 8" vitrified clay pipe and connects to the Division 22 sanitary sewer system at MH 22-89. There is a flapper valve and 50.4' air gap as cross connection control between the two systems. This air gap exceeds the current DOH minimum requirement of 34'.

The overflow path for the existing Division 22 Reservoir is shown on the attachment labelled "Scenario 1". InfoSewer was used to model the HGL along the overflow route from the Division 22 reservoir to the Strawberry Canyon Pump Station (STCPS). The overflow route begins at MH 22-89 and outputs into the STCPS wetwell. The STCPS was upgraded in 2007 and retrofitted with two submersible, non-clog wastewater pumps, Flygt Model NP3102.090 (465 Impeller). These pumps were selected to comply with the project design requirement to deliver 130 GPM at 28.75', with a shut-off head at 35 feet. These pumps are designed to operate as a lead and lag pumping system, alternating wet well pump-outs with each cycle. As this station was designed to operate with single pumps, little information is available on actual pump performance when both pumps operate in parallel. Performance curves for the Flygt pumps are included on the attachment labelled "Scenario 1".

The HGL profile from Division 22 reservoir to the Strawberry Canyon Pump Station is shown on the attachment "Scenario 1: Existing System Overflow". As shown on the profile, surcharge conditions are present from MH S6-6 to MH S6-3. The system is limited by low-slope pipe (pipe slopes ranging from s=0.001 to s=0.005) immediately upstream of the Strawberry Canyon Pump Station. Consequently, it is observed that the current condition does not adequately accommodate the overflow risk at the reservoir.

The system hydraulics were then evaluated to determine the peak flow that the existing gravity sewer system can handle without surcharging. The as-configured sanitary sewer system piping configuration can handle a load up to 430 GPM. However, the pump station capacity is limited by the Flygt pumps pumping in parallel, with an upper limit estimated conservatively at approximately 150 GPM.

In summary, the current overflow configuration is limited by the capacity of the Strawberry Canyon Sewer Pump Station and is not able to handle an overflow event @ 700 GPM. The design of the new Division 22 reservoir needs to include an overflow design that meets the design overflow requirement of 700 GPM.

Scenario 2 – Direct 700 GPM to the Lake Whatcom Boulevard Interceptor

As an option for reservoir overflow for the new reservoir construction, we considered the option to install a new manhole and approximately 225 LF of 8-inch diameter overflow pipe from the new Division 22 Reservoir to the sanitary sewer system along Kinglet Court, connecting to the existing system at MH S22-22. The additional pipe would be installed in the existing 6-foot side lot line utility easement from the reservoir to the District system at Kinglet Court. This scenario is shown on the attachment labelled "Scenario 2".

Upon analysis of this scenario, we found that the system surcharges at existing MH S22-36 adjacent to Doe Court and very near the point of tie-in with the Lake Whatcom Boulevard Interceptor. The surcharge occurs where a low slope pipe (s=0.004) is adjacent to a seasonal creek.

Surcharge conditions can be eliminated at this location with either of the following options:

a. Upsize a gravity sewer pipe segment to eliminate surcharge (Scenario 3),
b. Raise a manhole to allow surcharge into the manhole instead of ground overflow (Scenario 3), or
c. Install a flow splitting structure at the Division 22 Reservoir (Scenario 4).

Scenario 3 – 8" Overflow Pipe to Kinglet Ct. and Upsize Pipe S22-087 or Raise MH S22-36

The hydraulic analysis was evaluated with the option to install the 8" overflow pipe to Kinglet Court and also upsize an existing low slope 8" VCP segment to accommodate the additional volume (Pipe ID S22-087) from MH S22-35 to S22-90, as shown on the attachment labelled Scenario 3. This option would require a temporary sewer bypass, removal of existing pipe and installation of approximately 277 LF of upsized pipe. When running the hydraulic analysis, it was determined that while a 10" ductile iron pipe would eliminate the surcharge, a 12" ductile iron pipe segment is recommended. The increase of this pipe segment to a 12" ductile iron pipe (n=0.010) will eliminate the system surcharge due to overflow with both the increased diameter and improved coefficient of friction with a new material. The additional volume will allow for a factor of safety for the baseline flows that were not considered as part of the analysis.

This pipe section is along the upper bank of a seasonal creek. While not directly in the creek bed, due to the constraints of working within the Lake Whatcom Watershed (associated permitting and construction can be problematic), it is assumed that upsizing this section of pipe adjacent to a season creek will not be the preferred alternate. This option would not require an additional easement.

Another alternate considered, in lieu of replacing the pipe section, is to raise the rim elevation of MH22-36 by 3-feet or use a gasketed, bolt-down lid. This manhole is located along the high-bank of the seasonal creek. Further review of upstream service lateral tie-ins and basement elevations would need to be reviewed if this scenario is exercised.

If either of the above alternatives is selected, baseline usage flows will need to be added to the hydraulic model to verify system capacity. Additionally, the hydraulic analysis will need to be extended along the Lake Whatcom Boulevard Interceptor route, to verify capacity through the system from the tie-in point at the interceptor all the way through Cable Street Pump Station.

Scenario 4 – Install a Flow Splitting Structure:

This scenario evaluated the option of installing a flow splitting structure at the Division 22 Reservoir site to split the flow to the STCPS, the LWBI and also through a level spreader. With this structure in place, flows could be split to both basins to better accommodate the limited pump capacity at STCPS and avoid surcharging or replacing pipes along the low-slope pipe section to the Boulevard. Additionally, a level spreader could be constructed to handle part of the overflow. The 8" overflow pipe described in Scenario 2 would need to be installed in order to split flows to the LWBI.

It was determined that the system could handle the following flow split:

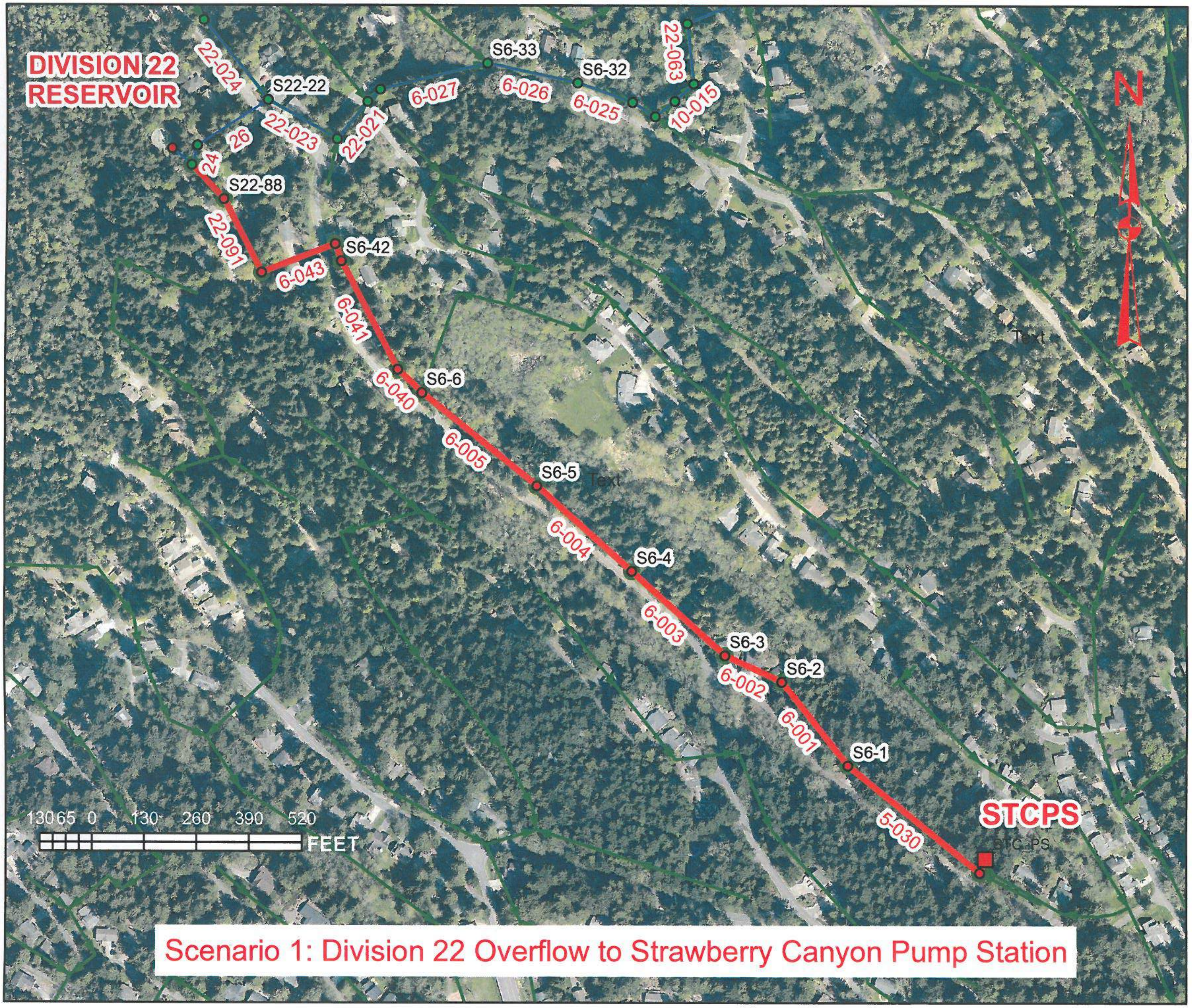
- STCPS: 15% Flow, equivalent to 105 GPM,
- LWBI: 85% Flow, equivalent to 595 GPM, and optionally
- Level Spreader (split volume to be determined with additional research) to relieve a small amount of system flow to alleviate burden through STCPS and LWBI.

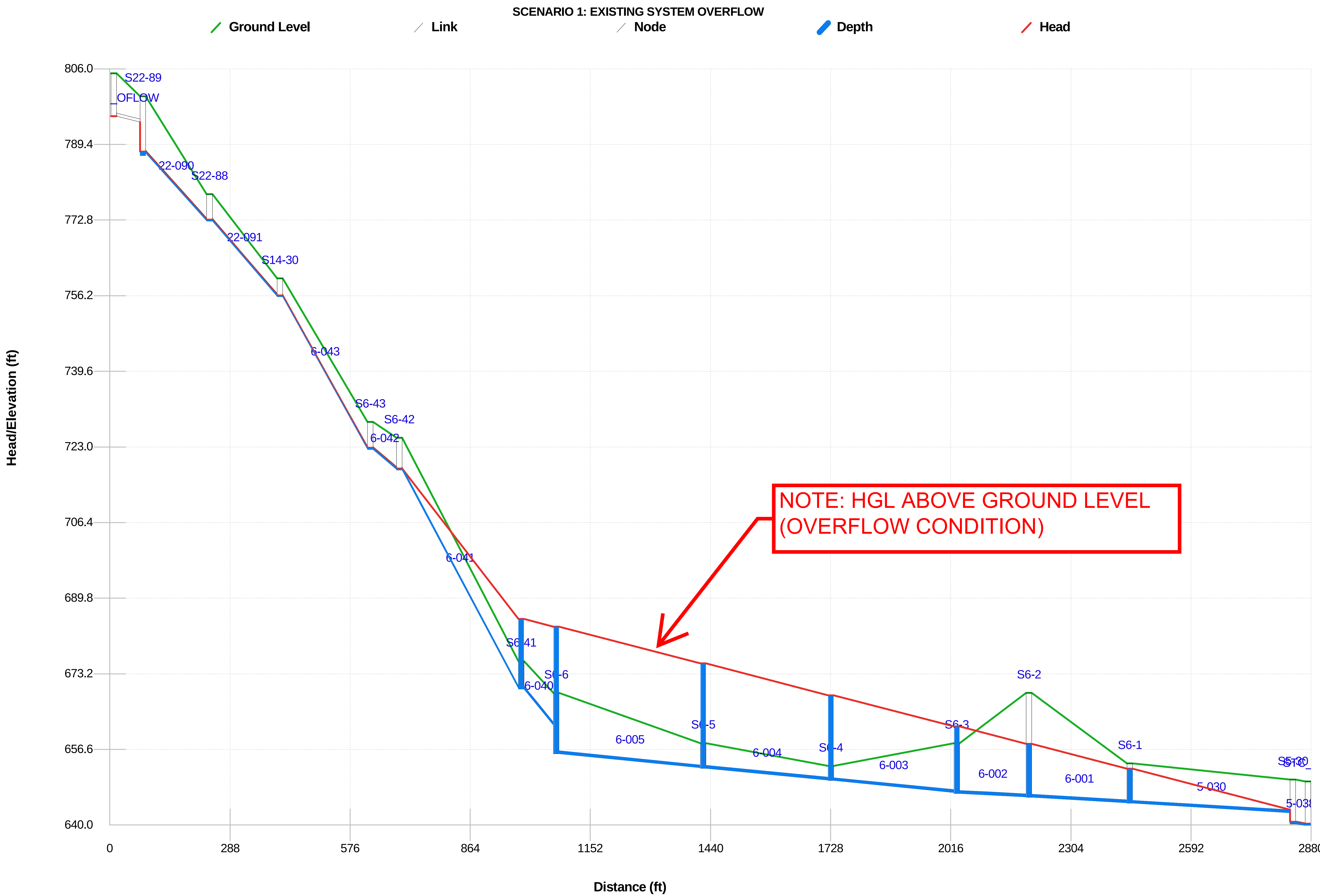
The hydraulic model run under this scenario, shows that all pipes perform without surcharge at manholes. However, if this alternate is selected, baseline usage flows will need to be added to the model to verify system capacity. Additionally, the hydraulic analysis will need to be extended along the Lake Whatcom Boulevard Interceptor route, to verify capacity through the system from the tie-in point all the way through Cable Street Pump Station.

Hydraulic Modeling Assumptions:

InfoSewer was used to model the HGL along the overflow routes. The following parameters were used within InfoSewer and for consideration of the overflow options:

- Overflow load added at MH 22-89 @ 700 GPM (representing the pump fill rate at which the Division 22 reservoir fills).
- The Manning's n value for the existing system was selected at n=0.015. This coefficient of roughness was selected based on research data available for aged vitrified clay pipe, assumed to be in "fair" condition. The existing sewer system was installed in 1972. We feel that using a "fair" condition value is appropriate. This is based on visual inspection as well as understanding the 40+ year age of the system.
- Baseline sewer collection system flows were not analyzed and therefore not represented in the hydraulic analysis. To move forward with any Scenario it is recommended that a more detailed analysis be performed to include baseline flows and extension of the model downstream.
- When considering the pump-out of overflow at the wetwell, Scenario 1, it is unknown how the two Flygt pumps at STCPS operate in parallel.
- Installation costs of the alternatives were broadly considered, but detailed installation estimates were not performed.


DIVISION 22
RESERVOIR
N
22-024
S22-22
22-023
26
24
S22-88
22-091
6-043
S6-42
6-041
22-021
6-027
S6-33
6-026
S6-32
6-025
22-063
10-015
S6-6
6-040
6-005
S6-5
Text
6-004
S6-4
6-003
S6-3
6-002
S6-2
6-001
S6-1
5-030
STCPS
STC_PS
130 65 0 130 260 390 520
FEET
Scenario 1: Division 22 Overflow to Strawberry Canyon Pump Station



SCENARIO 1: EXISTING SYSTEM OVERFLOW
Ground Level
Link
Node
Depth
Head
Head/Elevation (ft)
Distance (ft)
806.0
789.4
772.8
756.2
739.6
723.0
706.4
689.8
673.2
656.6
640.0
0
288
576
864
1152
1440
1728
2016
2304
2592
2880
S22-89
OFLOW
22-090
S22-88
22-091
S14-30
6-043
S6-43
S6-42
6-042
6-041
S6-41
S6-6
6-040
6-005
S6-5
6-004
S6-4
6-003
S6-3
6-002
S6-2
6-001
S6-1
5-030
NOTE: HGL ABOVE GROUND LEVEL
(OVERFLOW CONDITION)

STCPS PUMPS

SYSTEM DESIGNED @ 130 GPM

FLYGT	PERFORMANCE CURVE	PRODUCT NP3102.181	TYPE MT
DATE 2007-06-11	PROJECT STRAWBERRY P.S.	CURVE NO 63-465-00-3703	ISSUE 5

	1/1-LOAD	3/4-LOAD	1/2-LOAD
POWER FACTOR	0.81	0.75	0.63
EFFICIENCY	85.0 %	85.0 %	83.5 %
MOTOR DATA	—	—	—

RATED POWER	5	hp
STARTING CURRENT	42	A
RATED CURRENT	6.8	A
RATED SPEED	1745	rpm
TOT.MOM.OF INERTIA	0.026	kgm2
NO. OF BLADES	2	

COMMENTS

INLET/OUTLET -/ 4 inch

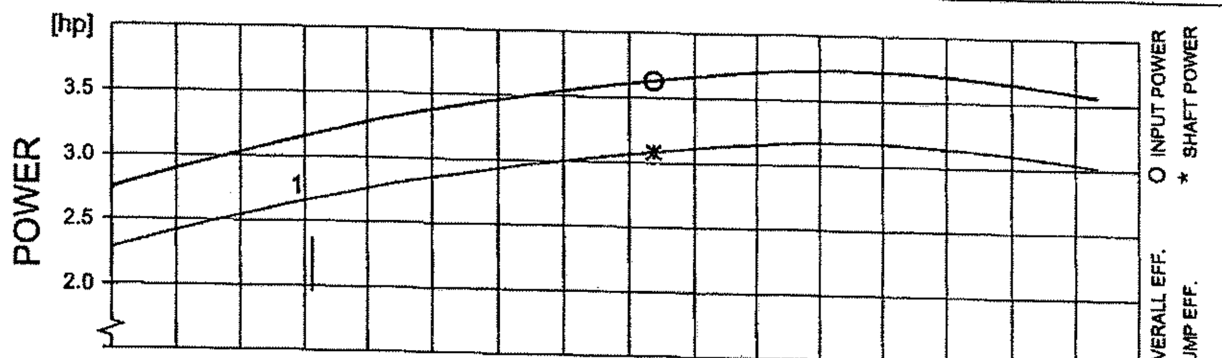
IMP. THROUGHLET —

IMPELLER DIAMETER 152 mm

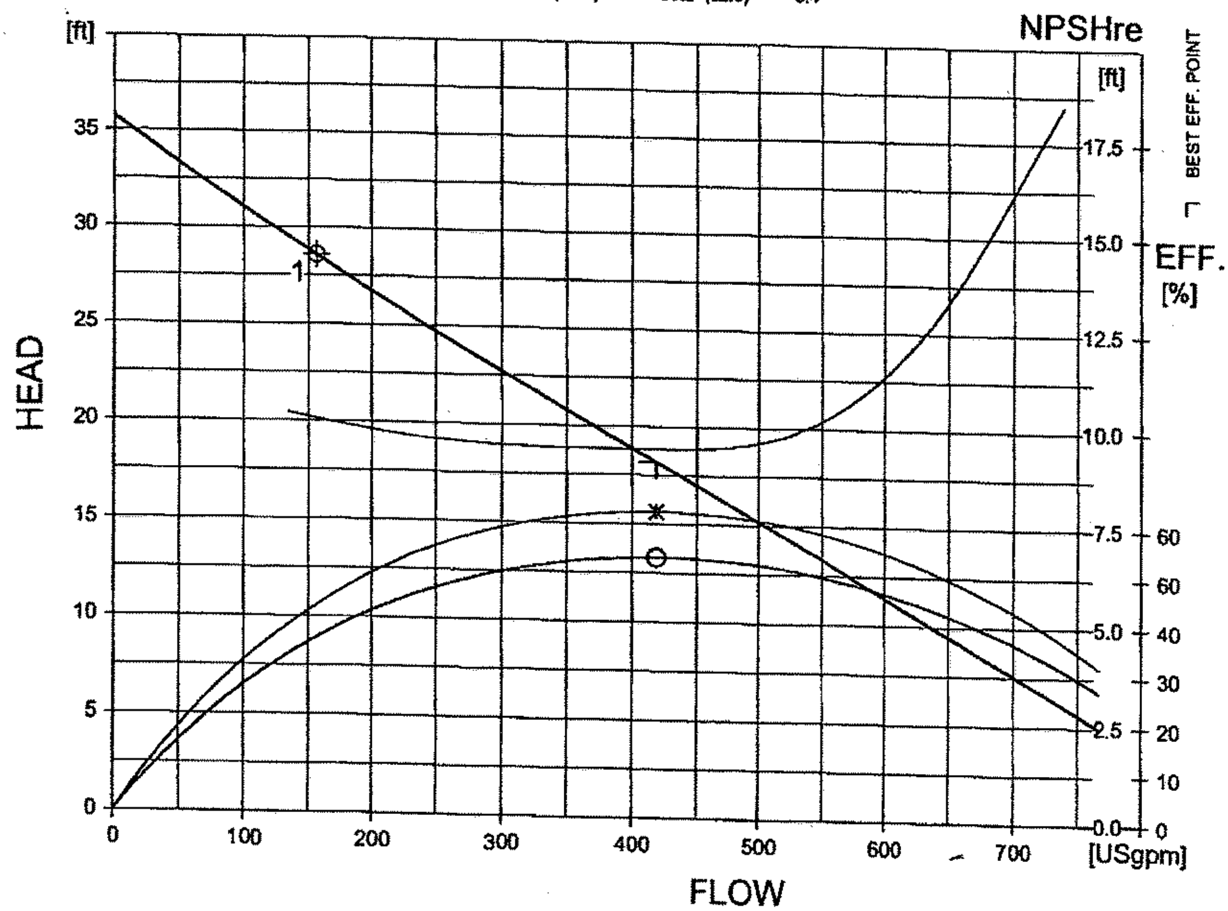
MOTOR #	STATOR	REV
18-11-4AL	01YSER	11

FREQ.	PHASES	VOLTAGE	POLES
60 Hz	3	~~460~~ 216 V	4

GEARTYPE	RATIO
—	—



DUTY-POINT	FLOW[USgpm]	HEAD[ft]	POWER [hp]	EFF. [%]	NPSHre[ft]
1	156	28.6	3.21 (2.71)	35.7 (42.3)	10.1
B.E.P.	419	18.2	3.63 (3.09)	53.2 (62.6)	9.4



FLYPS3.1.5.8 (20060531)

NPSHre = NPSH3% + min. operational margin

Performance with clear water and ambient temp 40 °C

FLYGT HI B Curve

FLYGT

STRAW #2

TEST REPORT

PRODUCT

Serial Number		Performance Curve No.	Motor Module/type	Voltage (V)
3102.090	0680188	63-465-00-3703	130	230

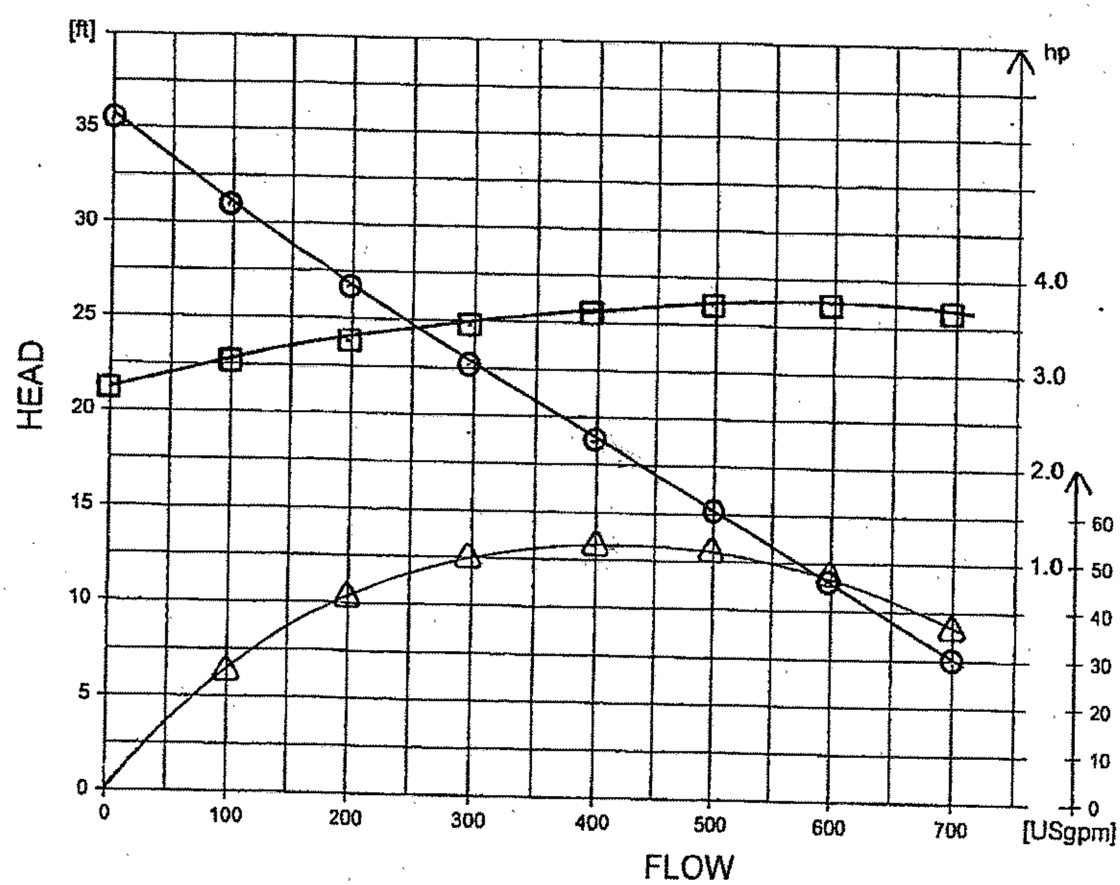
Base Module	Impeller No.	Gear Type	Gear Ratio	Imp. Diam/Blade Angle	Water temp °C
004	6066745			152	21

TEST RESULTS

Pump total head H (ft)	Volumn rate of flow Q (USGpm)	Motor input power P (HP)	Voltage U (V)	Current I (A)	Overall efficiency N (%)
34.83	0.00	2.72	231	7.7	0.00
30.85	99.30	3.00	231	8.1	26.61
26.28	197.81	3.23	230	8.5	41.11
22.40	293.23	3.44	230	8.8	49.76
18.74	393.22	3.57	230	8.9	52.75
14.93	493.62	3.67	230	9.1	51.15
10.99	593.42	3.71	230	9.3	46.59
7.41	696.10	3.62	230	9.3	36.55

Accepted after	Test facility	Test date	Time	Chief tester 2255
HI	Lindas Q1 Sweden	07 08 30	09:20	

PLOTTED TEST RESULTS Measured Point: ○ = Q/H, □ = Q/P Calculated Point: △ = Q/ETA overall



FLYGT

TEST REPORT

STRAW #1

PRODUCT

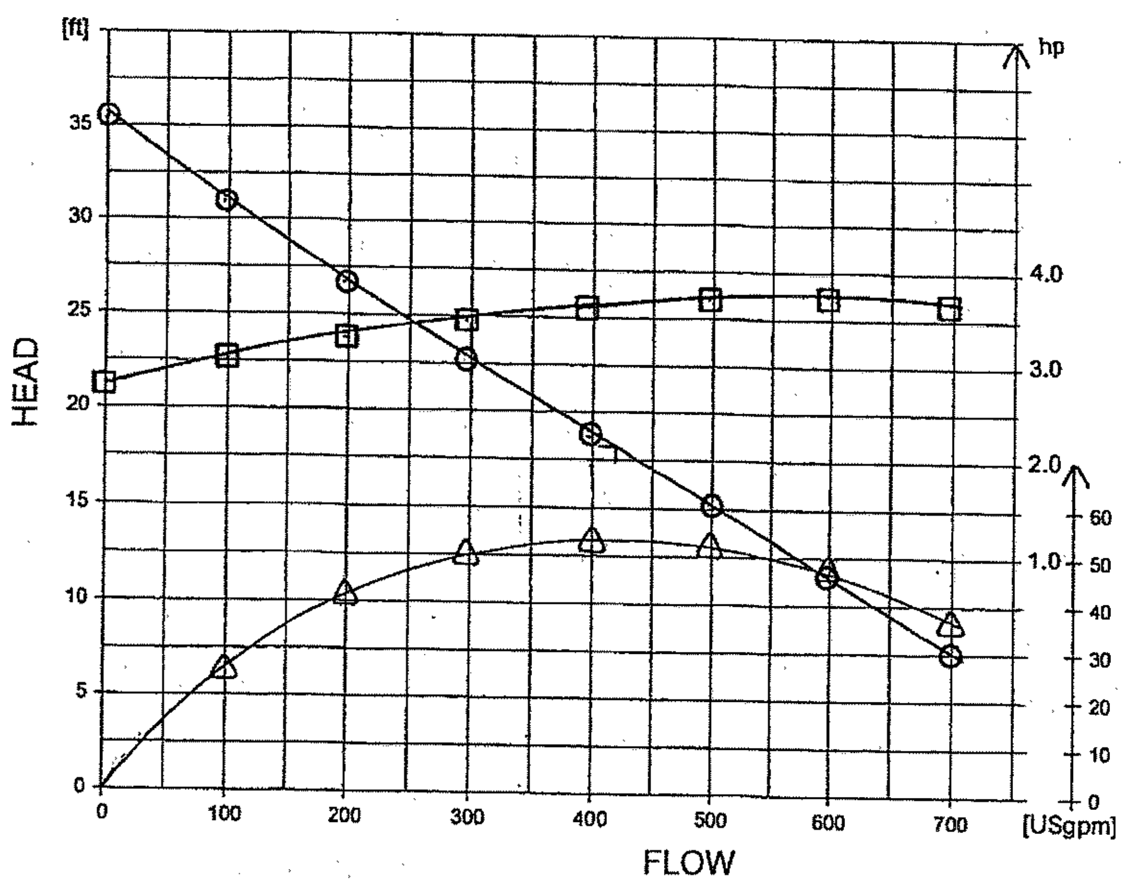
Serial Number		Performance Curve No.		Motor Module/type	Voltage (V)
3102.090 0680187		63-465-00-3703		130	230
Base Module	**Impeller No.**	**Gear Type**	**Gear Ratio**	**Imp. Diam/Blade Angle**	**Water temp °C**
004	6066745			152	21

TEST RESULTS

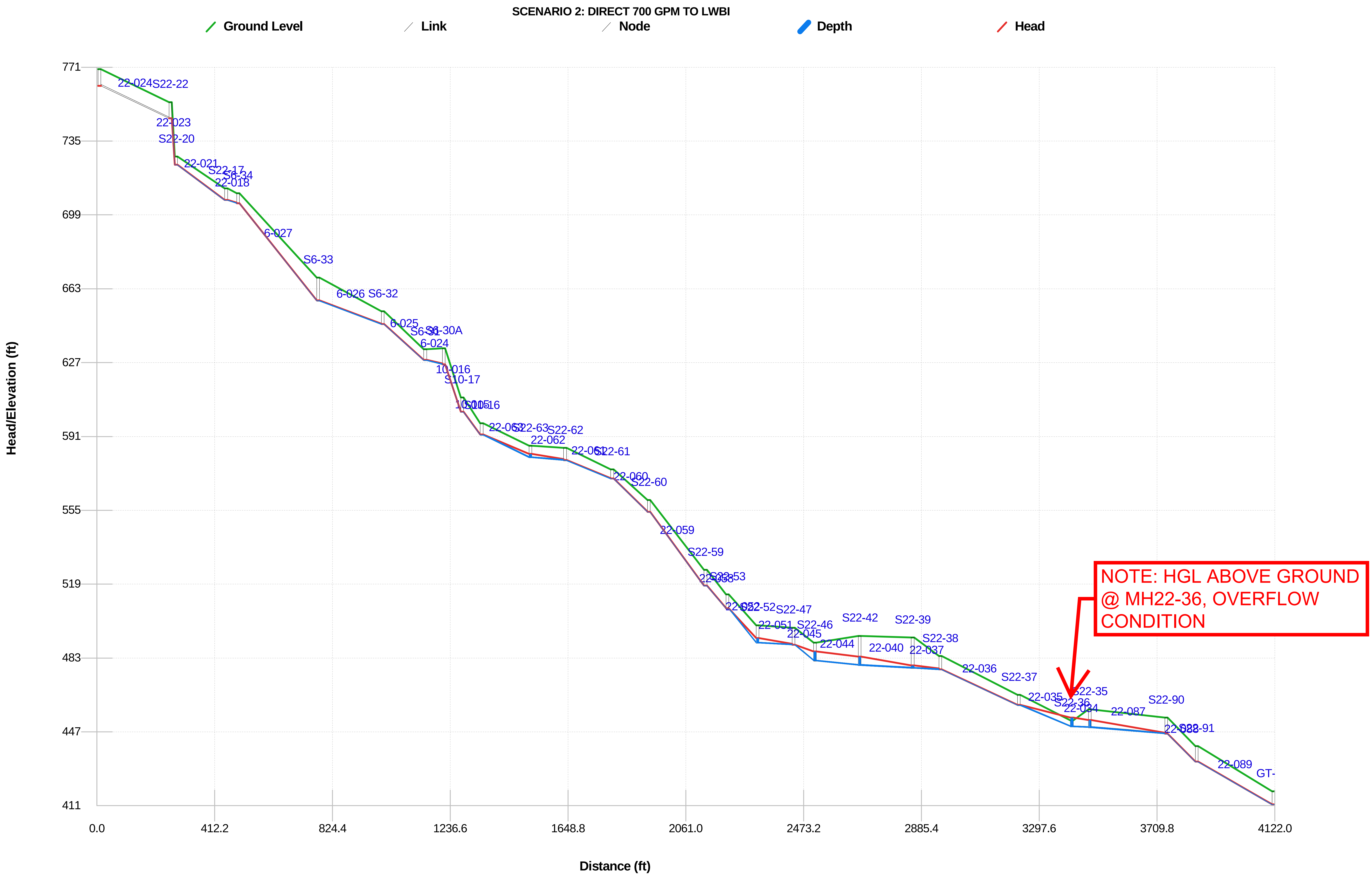
Pump total head H (ft)	Volumn rate of flow Q (USGpm)	Motor input power P (HP)	Voltage U (V)	Current I (A)	Overall efficiency N (%)
35.18	0.00	2.74	231	7.7	0.00
31.15	100.29	3.03	230	8.1	26.87
26.54	199.78	3.27	230	8.5	41.52
22.62	296.16	3.48	230	8.9	50.26
18.92	397.16	3.61	231	9.0	53.28
15.08	498.56	3.71	231	9.1	51.66
11.10	599.35	3.75	230	9.4	47.05
7.49	703.06	3.66	230	9.3	36.91

Accepted after	Test facility	Test date	Time	Chief tester 2255
HI	Lindas Q1 Sweden	07 08 30	08:30	

PLOTTED TEST RESULTS Measured Point: ○ = Q/H □ = Q/P Calculated Point: △ = Q/ETA overall

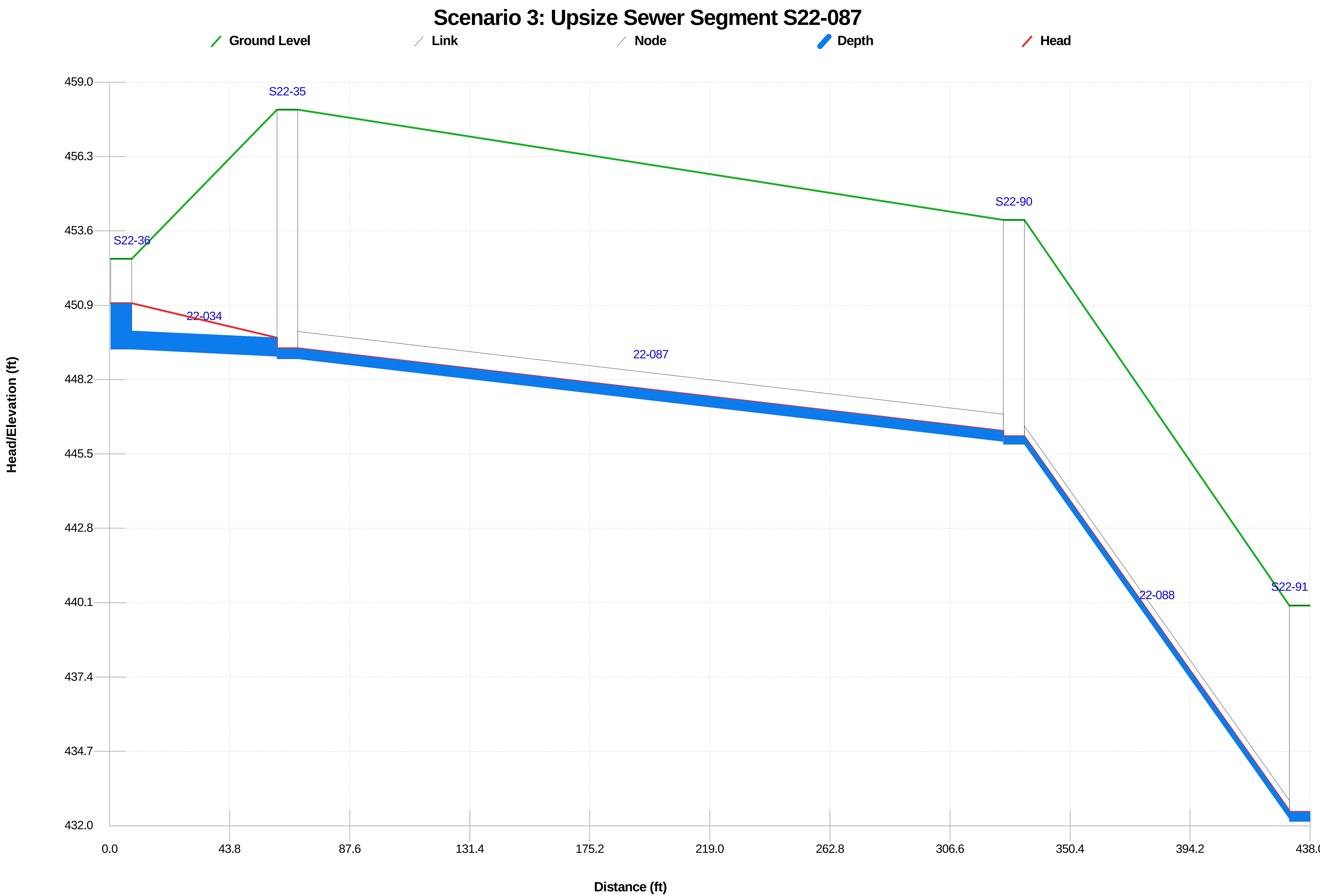


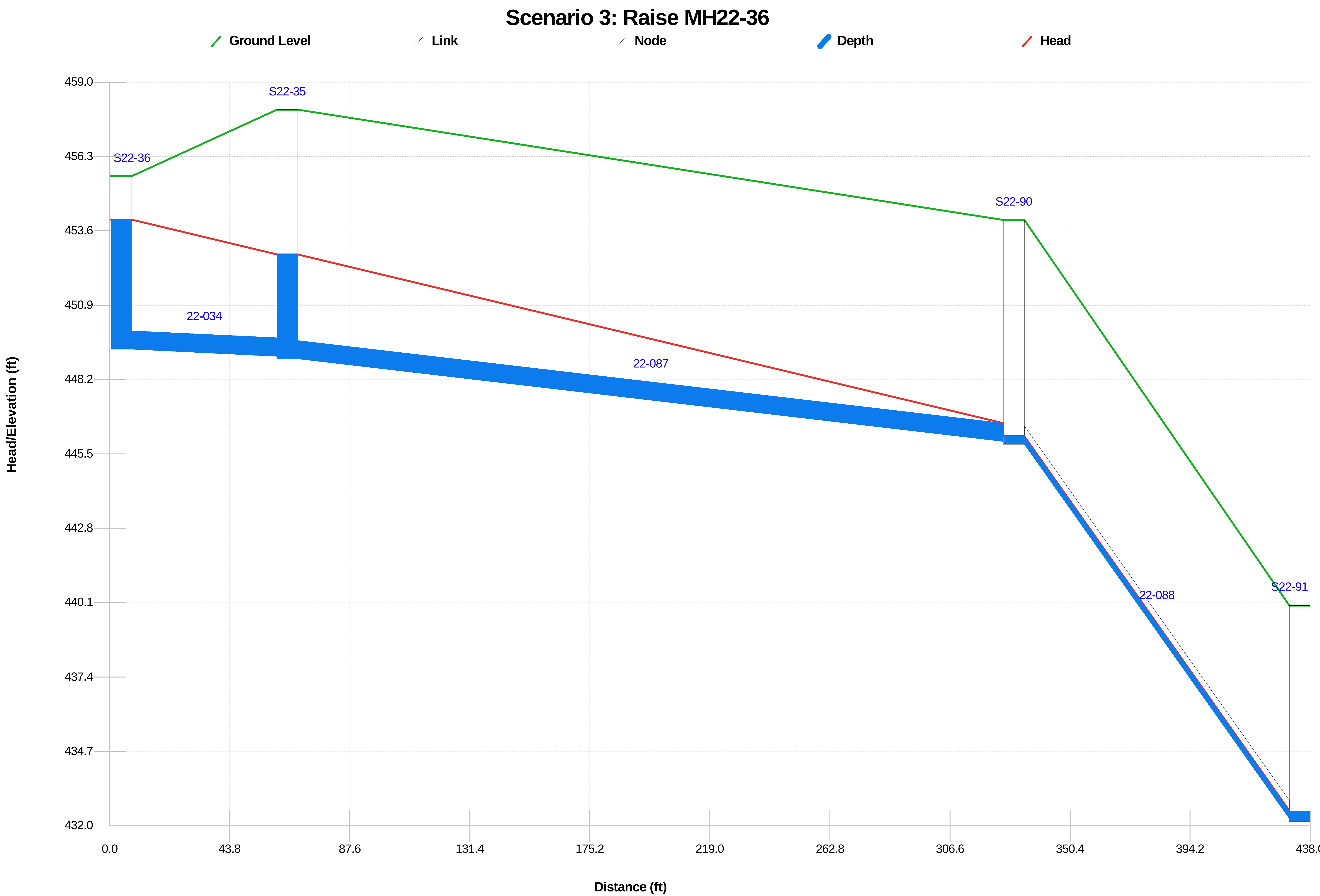
N
LWBI
GT-34
22-089
S22-91
22-087
S22-35
22-035
S22-37
22-036
S22-38
Text
22-040
S22-53
S22-47
22-051
22-059
S22-60
S22-61
22-061
S22-62
22-063
S10-16
6-024
6-026
S6-33
6-027
S22-22
S22-23
22-024
26
8" OVERFLOW PIPE
DIVISION 22
RESERVOIR
22-091
6-043
S6-42
6-041
S6-41
Text
100 50 0 100 200 300 400
FEET
Scenario 2: Direct 700 GPM to LWBI



SCENARIO 2: DIRECT 700 GPM TO LWBI
Ground Level
Link
Node
Depth
Head
Head/Elevation (ft)
771
735
699
663
627
591
555
519
483
447
411
Distance (ft)
0.0
412.2
824.4
1236.6
1648.8
2061.0
2473.2
2885.4
3297.6
3709.8
4122.0
NOTE: HGL ABOVE GROUND @ MH22-36, OVERFLOW CONDITION
22-024
S22-22
22-023
S22-20
22-021
S22-17
S6-34
22-018
6-027
S6-33
6-026
S6-32
6-025
6-024
10-016
S10-17
22-062
S22-62
S22-61
22-060
S22-60
22-059
S22-59
22-051
S22-47
S22-46
22-045
22-044
S22-42
22-040
S22-39
S22-38
22-037
22-036
S22-37
S22-35
22-087
S22-90
22-089


UPSIZE 22-087 OR
RAISE MH 22-036
N
LWBI
GT-34
22-089
S22-91
22-087
S22-35
22-035
S22-37
22-036
S22-38
22-040
S22-53
S22-47
22-051
22-059
S22-60
S22-61
22-061
S22-62
22-063
S10-16
S6-33
6-026
6-024
6-027
S22-22
S22-23
22-024
26
8" OVERFLOW PIPE
DIVISION 22
RESERVOIR
22-091
S6-42
6-043
6-041
S6-41
Text
Text
FEET
Scenario 3: Upsize Segment S22-087


Scenario 3: Upsize Sewer Segment S22-087
Ground Level
Link
Node
Depth
Head
459.0
456.3
453.6
450.9
448.2
445.5
442.8
440.1
437.4
434.7
432.0
Head/Elevation (ft)
S22-36
S22-35
S22-90
S22-91
22-034
22-087
22-088
0.0
43.8
87.6
131.4
175.2
219.0
262.8
306.6
350.4
394.2
438.0
Distance (ft)



Scenario 3: Raise MH22-36
Ground Level
Link
Node
Depth
Head
459.0
456.3
453.6
450.9
448.2
445.5
442.8
440.1
437.4
434.7
432.0
Head/Elevation (ft)
S22-35
S22-36
S22-90
S22-91
22-034
22-087
22-088
0.0
43.8
87.6
131.4
175.2
219.0
262.8
306.6
350.4
394.2
438.0
Distance (ft)

Scenario 4: Install Flow Splitting Structure