Pump Station Alternatives Feasibility Study

Avila Beach First Street Drainage

County of San Luis Obispo

December 09, 2024

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1. Introduction

1.1 Background

Avila Beach is a small, unincorporated community in San Luis Obispo County that is situated on San Luis Obispo Bay, as shown in Figure 1. The community is prone to flooding, particularly at the intersection of 1st St. and San Francisco St. and within the public parking lot north of this intersection, which is a low-lying area (existing ground elevation ±6 ft NAVD88). The existing drainage system for this area consists of swales along the parking lot and a gravity storm drain system that discharges to the San Luis Obispo (SLO) Creek. The existing storm drain outlet includes a Tideflex duckbill check valve to prevent backflow from the creek into existing drainage system. When water surface elevations within the creek are high, the duckbill check valve prevents or limits discharge to the creek, leading to flooding at this intersection.

Figure 1: Vicinity Map

There are several factors that may contribute to flooding in this area, including:

- San Luis Obispo Creek is tidally influenced in this area, affecting water surface elevations at the existing gravity system's outfall. The mean higher-high water (MHHW) tidal elevation is approximately 5.25 ft, which is higher than the invert elevations of the swale along the parking lot. Sea level rise may increase these tidal elevations and resulting flooding within the community.

- San Luis Obispo Creek has a large watershed of approximately 83 square miles, resulting in high water surface elevations during storm events. According to the FEMA Flood Insurance Study (FIS), the 10-year and 100-year water surface elevations within the creek at the existing outfall are approximately 9 ft and 15 ft, respectively (FEMA, 2017). These water surface elevations are well above the ground elevation at the intersection of 1st St. and San Francisco St. (elevation ±6 ft) and prevent the existing drainage system from discharging to the creek. Note that the creek is mapped in Zone AE on the FEMA Flood Insurance Rate Map (FIRM), which is an area within the 1% annual chance floodplain where base flood elevations have been established based on a detailed study (see Attachment 4).

- There is a large sand bar at the mouth of San Luis Obispo Creek, which periodically disconnects the creek from the ocean, increasing water surface elevations within the creek.

- High groundwater in the area limits infiltration of ponded stormwater and may contribute to flooding within the area. Please refer to Attachment 3 for a groundwater profile developed by England Geosystem, Inc. (2004), which displays this high groundwater.

In 2015, Cannon developed and analyzed conceptual design alternatives to reduce flooding within the public parking lot and at the intersection of 1st St. and San Francisco St. These alternatives are presented in their report entitled *Avila Beach First Street Drainage Conceptual Design Report* (Concept Report). In this report, Cannon presents four alternatives for flood improvements in the area:

1. Connect the gravity system to the existing sanitary sewer at 1st St.
2. Construct a pump station with a force main to discharge flow through a new outfall to the creek.
3. Install a standpipe above the MHHW tidal elevation on the existing outfall to allow water to discharge into the creek when the existing duckbill check valve is closed.
4. Regrade the existing parking lot and 1st St. to raise the ground elevations.

The Concept Report recommended Alternative #2, with a pump station in the parking lot, as the preferred alternative for addressing flooding issues within the area.

In 2023, GHD completed a peer review of the Concept Report and provided recommendations for a project to move forward with to reduce flooding within the public parking lot and at the intersection of 1st St. and San Francisco St. GHD's report: *Peer Review of Conceptual Design Report: Avila Beach First Street Drainage*, dated November 2023, discusses the peer review and recommended project. This report recommends a stormwater pump station with a gravity bypass that discharges to the creek through the existing outfall. Conceptual drawings of the recommended alternative were developed as part of this project and are attached to this 2023 report.

Since the completion of the GHD's peer review study, the County of San Luis Obispo (County) has identified eight alternatives for addressing the flooding at the intersection of 1st St. and San Francisco St., which are discussed in this report. Four of the identified alternative feature proposed pump stations and force mains, which are the focus of this study. The County has retained GHD to analyze these alternatives, develop concept drawings, and identify factors affecting the feasibility of the alternatives, primarily with respect to the constructability and hydraulic performance.

1.2 Project Goals

The County has identified the following goals for this project, which are discussed in GHD's *Peer Review of Conceptual Design Report* (GHD, 2023):

- Environmental and Permitting Goals: Consideration of a solution that will be feasible to construct and permit from the appropriate regulatory agencies and that minimizes soil, beach, and creek disturbance.

- Operations and Maintenance (O&M) Goals: Consideration of a solution that will minimize operations and maintenance demands and trash impacts to the system.

- Integrated Water Management (IWM) Goals: Consideration of a solution that accounts for potential beneficial use of the water and maximum water quality improvement prior to outflow.

These project goals were taken into consideration when analyzing, designing, and discussing the conceptual alternatives. Note, however, that evaluation of environmental permitting is not included in this study; permitting requirements are to be determined by the County.

1.3 Design Criteria

Several of the considered drainage improvements include pump stations. The pumps were designed to follow the drainage pump criteria outlined in Section 5.1.10 of the County of San Luis Obispo 2022 Public Improvement Standards and summarized below:

- **Gravity Outfall during Summer:** The drainage system should be designed to provide gravity outfall during the summer months and periods of low water stages, where possible. Otherwise, a pump with smaller capacity may be used to discharge low flow. Drainage alternatives presented in this study were designed to allow for gravity outfall during low flow and low water stages at the existing outfall in San Luis Obispo Creek. Note that the gravity outfall is dependent on ocean tides and creek water levels and may not always be available during the summer months.

- **Standby Equipment:** The pump station should be equipped with standby equipment and pumps, with pumps having alternating operation characteristics.

- **Floodgates:** The outfall should be equipped with floodgates when specified by the County.

- **Design Storm:** The design storm shall be specified by the County.

- **Pump Design:** Pump shall be designed so that gravity flow does not flow through the wet well. Each pump installation, including machinery, electrical system, piping system, housing installation, etc., shall receive approval from the County.

Section 5.1.10 of the County's Public Improvement Standards does not provide specific guidance on the design storm, maximum water surface elevations, or the tailwater to use in the pump station analysis. These criteria were identified for the project based on discussions with the County, and are listed below:

- **Design Storm:** 25-year, 24-hour storm event. This design storm has a rainfall depth of 4.19 inches (see Section 2.2).

- **Flood Inundation Elevation:** The maximum water surface elevation within the parking lot is 6.62 ft (NAVD88 ft), which is 6 inches below the lowest finished floor elevation of the buildings adjacent to the parking lot. This flood elevation minimizes damage to existing structures during the design storm, while allowing some ponding and attenuation within the parking lot. Please refer to Attachment 1 for a topographic survey that includes finished floor elevations of the three buildings immediately south of the parking lot, which range from 7.12 ft to 8.50 ft.

- **Tailwater:** The design tailwater elevation at the storm drain system outfall is 15 ft (NAVD88). This is a conservative condition, given that the 100-year FEMA Base Flood Elevation within the creek is

approximately 14 ft at the existing outfall location and the 100-year FEMA Base Flood Elevation in the ocean at the proposed outfalls is 15 ft. In addition, if the water surface in the creek or ocean rises above approximately elevation 15 ft, Avila Beach Dr. or Front St., respectively, would be overtopped, leading to flooding issues that could not be resolved by the proposed pump station.

Note that the design tailwater elevation of 15 ft is significantly higher than the maximum water surface elevation within the parking lot of 6.62 ft, so a pump station will be required to meet the design criteria outlined above.

1.4   Purpose of this report

The purpose of this report is to discuss the conceptual design of the alternatives identified by the County to reduce flooding at the public parking lot and the intersection of 1st St. and San Francisco St. This report also documents the hydrologic analysis of this watershed as well as preliminary hydraulic analyses of the considered alternatives. The pros and cons of the alternatives are also described, considering constructability, preliminary hydraulics, utility conflicts, and operations and maintenance.

1.5   Scope and limitations

GHD's scope for this project included in the following:

- Complete a hydrologic analysis of the 1st St. storm drain system that drains the parking lot to determine preliminary pump and force main sizes.

- Develop conceptual plan and profile drawings of Alternatives 3 to 6 identified by the County.

This report has been prepared by GHD for the County of San Luis Obispo (County) and may only be used and relied on by the County of San Luis Obispo for the purpose agreed between GHD and the County of San Luis Obispo as set out in section 1.4 of this report.

GHD otherwise disclaims responsibility to any person other than the County of San Luis Obispo arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in throughout this report. GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by the County of San Luis Obispo and others who provided information to GHD (including Government authorities), which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

Accessibility of documents

If this report is required to be accessible in any other format, this can be provided by GHD upon request and at an additional cost if necessary.

2.   Hydrologic Analysis

A hydrologic analysis of the area tributary to the existing storm drain outfall to SLO Creek was completed using the USACE HEC-HMS software (version 4.12 Beta 2). This hydrologic model was developed to perform preliminary pump station sizing, which is discussed in Section 3. This preliminary pump station sizing, and the resulting peak flow rates, were then used to analyze the hydraulics of the proposed system, as discussed in Section 6. Additionally, the results of this hydrologic analysis can be used for future refinement and analysis of the selected alternative.

The HEC-HMS model calculated inflow hydrograph to the parking lot along 1st St. during the 10-year, 25-year, 50-year, and 100-year storm events, using the Soil Conservation Service (SCS) Curve Number method and the SCS Unit Hydrograph. The key components and inputs to the HEC-HMS model are described below.

The HEC-HMS model was also used to simulate storage within the parking lot along 1st St. and size the pumps for the proposed alternatives. Discussion of pump station sizing and parking lot storage is included in Section 3 and Appendix D.

2.1   Watershed

The watershed tributary to the parking lot was delineated using a combination of survey data received from MBS Land Surveys (MBS) and 2018 Federal Emergency Management Agency (FEMA) LiDAR data downloaded from the National Oceanic and Atmospheric Administration (NOAA) Data Access Viewer. The MBS topographic survey data is included in Attachment 1 and includes topographic contours within the parking lot and along 1st St. to the existing storm drain outlet to SLO Creek. Record Drawings of the existing drainage facilities as well as the Avila Beach Community Services District (ABCSD) Storm Drain Atlas were also used to inform the watershed delineation. Please see Attachment 2 for the ABCSD Storm Drain Atlas.

The 26.7-acre watershed is shown in the Drainage Map in Appendix A alongside the 2018 LiDAR topographic contours. The delineated watershed is larger than the watershed determined by Cannon and presented in their Conceptual Design Report (Cannon, 2015). The Cannon watershed did not include the area near the intersection between Avila Beach Dr. and San Miguel St., however, the ABCSD Storm Drain Atlas shows inlets along San Miguel St. that are connected to storm drains that convey flow to the existing swale along the parking lot. The Cannon study also assumed the northeast half of Avila Beach Dr. drains toward the golf course. The current study included this area of Avila Beach Drive in the Parking lot watershed, since the curb along the northeast edge of the road would likely prevent flow from draining to the golf course and would direct it to a storm drain inlet that connects to the gravity system that drains the parking lot.

2.2   Rainfall

Precipitation was applied within the HEC-HMS model using the SCS Type I rainfall pattern, which is a 24-hour rainfall distribution curve. The SCS Type I rainfall pattern is the rainfall distribution recommended for San Luis Obispo County by US Department of Agriculture (USDA) National Resources Conservation Service (NRCS, 2009). NOAA Atlas 14 rainfall depths were input into the HEC-HMS model for the 10-year, 25-year, 50-year, and 100-year 24-hour storm events (NOAA, 2014). These rainfall depths are included in Table 1. Please refer to Appendix B for NOAA Atlas 14 point precipitation frequency estimates, which were obtained at the intersection of San Francisco St. and 1st St.

Table 1: 24-hour rainfall depths

	10-year	25-year	50-year	100-year
24-hour Rainfall (in)	3.50	4.19	4.71	5.22

2.3   Curve Number

The SCS Curve Number (CN) method was used by HEC-HMS to determine losses and excess rainfall within the watershed. A weighted average composite curve number was calculated based on land use and soil type within the watershed. The curve number for each land use was obtained from the NRCS TR-55 Table 2-2a (NRCS, 1986). The watershed was divided into the following land use categories based on aerial imagery, as shown in Figure A2 in Appendix A:

1. Low density developed, taken as residential with 1/8-acre lots.
2. High density developed, taken as urban districts: commercial and business.
3. Impervious, taken as impervious areas: paved parking lots, roofs, driveways, etc.
4. Open space, taken as open space: fair condition.

Soil type data for the watershed was obtained from the NRCS Web Soil Survey, which is included in Appendix B. The soil survey data indicates that soils in the east area of the watershed are classified as hydrologic soil group C. The hydrologic soil group for a large area of the watershed is not defined and was assumed to be type D for a conservative analysis.

The results of the calculations indicate a composite CN of 94 for the watershed. Please refer to Appendix C for curve number calculations. Note that this composite curve number considers the percentage of impervious area and therefore the percent impervious was set to 0 in HEC-HMS.

2.4   SCS Unit Hydrograph

The SCS Unit Hydrograph model was used by HEC-HMS to transform the excess rainfall to runoff. The standard dimensionless SCS unit hydrograph was selected, which was derived by the SCS from gaged rainfall and runoff data for small watersheds throughout the US and is described in Chapter 16 of Part 630 of the National Engineering Handbook (NRCS, 2007). The time of rise, or lag time, is a required input of the SCS Unit Hydrograph model and is taken as 0.6 times the time of concentration (NRCS, 2008).

The time of concentration for the watershed, the time required for runoff to travel from the most distant point in the watershed to the outlet, was calculated based on County of San Luis Obispo Department of Public Works Standard Drawings H-2. The standard drawing relates the time of concentration to the length of the longest flow path and elevation difference across this flow path. The longest flow path is shown on the Drainage Map in Appendix A and was determined based on the topographic contours and as-built drawings of the drainage facilities. The upstream elevation of the flow path was obtained from the 2018 LiDAR data, while the downstream elevation was obtained from the MBS Survey Data. The resulting time of concentration was approximately 10 minutes, which corresponds with a 6-minute lag time. Please refer to Appendix C for time of concentration calculations.

2.5   Hydrologic Results

The results of the hydrologic model, including peak flow rate, 24-hour runoff volume, and hydrographs are included in Table 2 and Figure 2 for the analyzed storm events.

Table 2: Summary of hydrologic results

	10-year	25-year	50-year	100-year
Peak Flow (cfs)	57	70	79	89
Runoff Volume (AF)	6.4	7.9	9.0	10.1
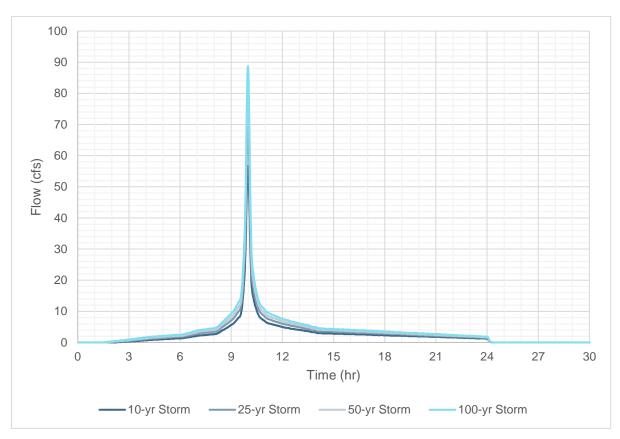
Figure 2: Avila Beach 1ˢᵗ St. Drainage Hydrographs

3. Preliminary Pump Station Design

The various alternatives considered have different locations for the pump station, however, it is anticipated that they will all function similarly and have similar hydraulic capacities. It is assumed that all the pump stations will rely on the storage capacity in the existing vegetated swale adjacent to the parking lot and the parking lot itself for storage of stormwater during large storm events. These areas will provide detention of the stormwater and allow the pump station capacity to be lower than the peak inflow to the parking lot. Based on this operation, GHD developed a preliminary design for the pump station as part of this study. The analysis performed for the design was based on the pump station and outfall location in Alternative 3 and will need to be updated once a preferred alternative is selected, but the required discharge and pump size is not expected to change significantly with the other alternatives.

The preliminary design of the pump station was completed using the HEC-HMS model discussed in Section 2. A reservoir was added to the HEC-HMS model to simulate the storage within the swale and the parking lot, with a stage-discharge curve developed using the 2024 MBS topographic survey (see Attachment 1). This stage-area curve is included in Appendix D and includes a small area to account for storage within the wet well. The initial water surface elevation within the wet well was assumed to be 4 ft, which represents a maximum ponded depth of approximately 1 ft within the swale. An initial water surface elevation of 5.3 ft was also evaluated, which the maximum water surface elevation within the swale without overtopping the parking lot. This starting water surface elevation provides a conservative estimate initial ponding due to groundwater. Note that inflow from groundwater was not modeled during the storm event, as it is expected to be negligible relative to the flow rates from runoff.

Pumps were connected to the reservoir in the HEC-HMS model and pump curves were obtained from the pump manufacturer (based on the preliminary pump selection discussed below) and input to the model. Additionally, systems curves were developed, which account for head losses through the force main (including friction and minor

losses), and these curves were used to estimate the equipment loss for the pumps in the HEC-HMS model. 12 ft was selected as the represented equipment loss, which is approximately the average through the operating range of the pump station. Note that HEC-HMS applies a constant equipment loss throughout the simulation, and thus does not consider variable flow rates and velocities throughout the storm event. In reality, the equipment loss is dynamic as it is a function of the pump discharge, which varies based on the water surface elevations in the wet well and at the force main outfall. It is recommended that the analysis be refined to consider time-varying head loss once a preferred alternative is selected.

Preliminary pump and set point selection were completed to meet the design criteria discussed in Section 1.3, including a maximum water surface elevation of 6.62 ft within the swale and parking lot (reservoir) during the 25-year storm event and a tailwater elevation of 15 ft. The performance of the pumps was also analyzed using a free outfall condition, to determine the maximum expected pump flow rates for each storm event and bracket the expected operating range for the pump station.

Output from the system curve development and HEC-HMS model is included in Appendix D. Based on the analysis, a pump station discharge of approximately 35 cfs is required to meet the criteria presented in Section 1.3. This flow results in a force main diameter of 30 inches to keep the flow velocity below 8 ft/s, which is a typical design guideline for force mains. This flow rate of 35 cfs was used to design and analyze the pump station intake structures and storm drains, discussed in Section 6. The results of the HEC-HMS model indicated no difference in peak results between the 4.0 ft and 5.3 ft starting water surface elevations within the swale.

The pump station has been designed with three equal-sized pumps, two duty pumps (lead and lag) for handling the design flow and a standby pump. The pumps preliminarily selected are Xylem (Flygt) N-series model NP 3301 submersible pumps that are intended for wastewater and stormwater applications. These pumps are 60 hp, 3-phase pumps with a 60 Hz frequency. The benefit of having two duty pumps is that smaller pumps can be used and typically only one pump will be in operation, resulting in lower energy demands and longer run times, which reduces wear and tear on the pumps. The lead pump (Pump #1) will turn on when the water surface elevation in the wet well reaches 5.50 ft. If the inflow exceeds the capacity of the lead pump and the water level in the wet well reaches elevation 6.0 ft, the lag pump (Pump #2) will turn on and the two pumps will operate in parallel. The standby pump (Pump #3) serves as a backup in case either of the other two pumps fail, and the pumps will have alternating operation characteristics, meeting the County's design criteria described in Section 1.3. A maximum of 15 starts per hour is acceptable for Flygt pumps and the HEC-HMS model output included in Appendix D shows the proposed configuration provides for much fewer starts per hour than that.

The pump station will need an electrical power supply that can power the pumps. In addition, a backup generator will be required to supply power to the pumps in the event of an electrical power outage, which often occur during storm events. Lastly, a motor control center (MCC) will be required for the pump controls. Both the backup generator and MCC will need to be elevated to protect them from damage from flooding, at least 1 ft above the maximum 100-year water surface elevation.

4. Drainage Alternatives

4.1 Alternatives Overview

The County has identified eight drainage alternatives for the proposed project, four[1] of which were analyzed and refined by GHD. Plan and profile drawings were developed for each these studied alternatives are included in Appendix E. The following section presents all of the alternatives and provides discussion of advantages and

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[1] Note that two of the studied alternatives include two possible pump station locations, so Appendix E includes drawings for six design configurations.

disadvantages of each alternative. A summary of the pros and cons of all of the alternatives is presented at the end of this section (see Section 4.10 and Table 3), along with factors considered while developing these pros and cons.

The first two County alternatives involve no construction, with Alternative 1 consisting of no change, and Alternative 2 involving implementation of an annual maintenance program for the existing drainage system. These alternatives are briefly described below but were not refined or analyzed, since the focus of this study was a new pump station. Alternative 8, which involves connecting the existing storm drain to an existing sewer lift station was also not analyzed, as the wastewater treatment plant does not have capacity for the additional inflow, based on information received from the County. Alternative 7, which involves construction of a detention pond within the Avila Beach Golf Resort was also not studied at this time per the direction of the County, as it will require significant coordination with the property owners and discussions with the property owners have not yet been initiated.

The focus of the current study was on Alternatives 3 through 6, which include proposed pump stations that pump runoff from the swale and parking lot to SLO Creek or the ocean. Note that Alternative 3 was presented in GHD's peer review study (GHD, 2023), while Alternatives 4-6 are new to the current report. As discussed in Section 3, preliminary pump station sizing was performed to maintain water surface elevations 6-inches below the lowest finished floor elevation of the buildings adjacent to the parking lot during the 25-year storm event, to meet the project's design criteria (see Section 1.3). This design criterion was also applied when designing and analyzing the proposed intake structures, proposed weirs, and/or gravity storm drains to that convey flow to the proposed pump stations (see Section 6).

4.2   Alternative 1: No Change

The first alternative considered by the County involves no changes or alterations to the existing gravity system. This alternative was not evaluated as part of this study, but a brief discussion is included for comparison. While this alternative requires no funding or construction, there are several limitations of this alternative. The existing gravity system features an outlet to San Luis Obispo Creek with a duckbill check valve and is thus dependent on ocean tides and water levels within the creek. Therefore, the system does not function well when SLO Creek water levels are high, and even small storm events may lead to significant flooding at the intersection of 1st St. and San Francisco St. Additionally, this alternative does not address the existing gravity system being heavily silted at the intersection of Avila Beach Dr. and 1st St. (per the County) or the large amount of debris observed by GHD at the inlet to the system in the vegetated swale. Lastly, this alternative also does not meet the County's IWM goal for beneficial use of the stormwater.

4.3   Alternative 2: Annual Maintenance Program

The second alternative involves implementation of maintenance program for the existing gravity system. This alternative was not evaluated as part of this study, but a brief discussion is included for comparison. This alternative requires minimal funding; however, the benefits of this alternative are limited as well. The annual maintenance program could be implemented to reduce silt problems within the gravity drainage system and debris buildup at the inlet, improving the condition and capacity of the drainage system. This alternative, however, would not address the flooding issues that occur when there are high water levels within SLO Creek. This alternative also does not meet the County's IWM goal for beneficial use of the stormwater.

4.4   Alternative 3: Pumps with Parallel Force Main

The third alternative consists of a pump station with a force main parallel to the existing 36" gravity pipe that connects to the existing outfall to SLO Creek. This alternative was the recommended design in GHD's report: *Peer Review of Conceptual Design Report*, dated November 2023. The configuration, shown in Figures E1 and E2 in Appendix E, includes a pump station within the parking lot, with an intake from the existing swale. The design of the intake structure is discussed in Section 7.1, and includes a 22.5-ft long weir at an elevation of 5.0 ft. This weir crest elevation is just under 2 ft above the invert of the swale, and it is estimated that existing swale could convey approximately 13 cfs in low tailwater conditions without overtopping the weir. A discharge structure is proposed near the existing outfall

to the SLO Creek. Both the proposed 30" force main and the existing 36" gravity pipe would outlet to this discharge structure, and then outlet to the creek via the existing 36" gravity outlet. A flap gate would be installed within the discharge structure on the 36" gravity pipe, preventing flow from the force main or the creek from entering the existing storm drain during high tailwater conditions. The existing duckbill check valve on the 36" outfall would be removed. This proposed configuration allows for the gravity drainage system to function when the water levels in SLO Creek are low, or if the pump fails, meeting the County's gravity outfall during summer criterion (see Section 1.3) and providing redundancy in the design.

An advantage of this configuration is that it could maintain 25-year water surface elevations below 6.62 ft within the parking lot, meeting the project's design criteria (see Section 1.3). The drainage system could function even when water surface elevations within SLO Creek are high. Another advantage of this alternative is that a new outfall is not required. Additionally, the proposed force main has fewer grade breaks than some of the other alternatives, improving constructability. It should be noted that the locations and elevations of existing utilities need to be verified, which may affect the force main profile.

Disadvantages of Alternative 3 that it eliminates some parking spaces within the parking lot, and it requires relocating the existing 36" storm drain within the SLO County underground drainage easement. Another disadvantage is that the existing inlet on 1st St. would need to be removed or relocated to allow for construction of the 30" force main. It would be difficult to relocate this inlet northwest with the 36" storm drain, since there is an existing driveway at that location, so the inlet would likely need to be removed. Stormwater tributary to the existing inlet would instead flow southeast along 1st St. to the sidewalk underdrain at the intersection of San Francisco St. This alternative would also require relocation of existing cable TV and telecommunication conduits within 1st St. Another consideration of Alternative 3 is that the pump station is situated within Port San Luis Harbor District (PSLHD) property, and thus would require coordination with PSLHD. This alternative also does not meet the County's IWM goal for beneficial use of the stormwater.

4.5   Alternative 4: Pumps on Existing System

The fourth alternative is shown in Figures E3 and E4 in Appendix E and consists of a new pump station near the existing outfall to SLO Creek. The pump station would be offline from the existing gravity system; there is a gravity bypass so that flow only enters the pump station's wet well when it exceeds an elevation of 4.25 ft. Flow from the pump station would be discharged to a discharge structure that is connected to the existing outfall to SLO Creek as well as the existing drainage system upstream. A flap gate to the existing 36" storm drain would prevent flow from entering the gravity system during high tailwater, and the existing duckbill check valve at the outfall would be removed. This configuration was designed so that the gravity system could still function during smaller storm events and when the water levels within SLO Creek are low, meeting the County's gravity outfall during summer criterion (see Section 1.3) and providing redundancy.

The proposed configuration includes replacing a 22.5-ft-long segment of the existing 36" storm drain with a 3-ft wide rectangular concrete channel. The concrete channel includes a side weir at an elevation of 4.25 ft that is adjacent to the pump station wet well. During low tailwater conditions, the channel would convey flow to a discharge structure, and then to the outfall within SLO Creek. During high tailwater conditions, the flap gate on the 36" storm drain that discharges to the discharge structure would close to prevent backwater within the existing storm drain and the 3-ft wide channel during high tailwater. When this flap gate is closed, the side weir within the 3-ft wide channel would be overtopped, conveying flow to the wet well. The pumps would then turn on, discharging flow into the discharge structure, through an existing segment of the 36" storm drain, and out the existing outfall. The channel and side weir were designed to maintain a 25-year water surface elevation below 6.62 ft within the 1st St. parking lot, meeting the project's design criteria (see Section 1.3). Please refer to Section 7.1 for the hydraulic analysis of Alternative 4.

Alternative 4 includes several benefits, as discussed in Table 3. One of the primary benefits of Alternative 4 is that it requires minimal construction relative to the other proposed alternatives (not including Alternative 1 and 2), since the alternative makes use of the existing gravity drainage system. Additionally, while the pumps will ensure adequate drainage during the 25-year event in high tailwater conditions, the existing gravity drainage system will be used when the water levels in SLO Creek are low. Analysis of the gravity drainage system indicates that the system can convey

approximately 29 cfs without overtopping the channel's side weir, which is higher than the other considered alternatives. This configuration provides redundancy and lowers the use and associated wear and maintenance of the pump station.

A disadvantage of Alternative 4 is that the pump station is located within an existing road, which would require regrading. Due to space limitations, the pump station would need to be constructed and designed to handle traffic loads, so that it could be constructed beneath the access road. Another disadvantage of Alternative 4 is that the pump station is located within the Avila Beach Golf Resort property and would require coordination with the property owner to construct the pump station. This alternative could be refined to relocate the pump station to the south side of the existing 36" storm drain to try to keep the pump station within the County property. Given the size of the proposed pump station and the County property, however, it may be difficult to fit the pump station entirely within County property. The wet well for Alternative 4 will also likely be deeper than several of the alternatives, since the intake weir (el. 4.25 ft) is lower than the configurations with the intake structure along the existing parking lot swale. Additionally, the ground elevation at the proposed pump station location is higher than the parking lot, resulting in a deeper pump station. This alternative also does not meet the County's IWM goal for beneficial use of the stormwater.

4.6   Alternative 5: Front St. Outlet via San Juan St.

Alternative 5 involves installing a force main in San Juan St. that conveys flow from a new pump station to an existing beach outfall, as shown in Figures E5 through E8 in Appendix E. This alternative can effectively maintain water surface elevations within the parking lot to below 6.62 ft within the 25-year storm event, meeting the project design criteria (see Section 1.3). Like Alternative 4, the pump station would be separate from the existing gravity drainage system, which would remain and provide redundancy.

Alternative 5 consists of two possible pump station locations. The first pump station location, referred to as Alternative 5A, is within the parking lot, with an intake from the existing swale (see Figures E5 and E6 in Appendix E). The intake from the existing swale features a 22.5-ft-long weir with a crest elevation of 5.0 ft (see Section 7.1). It is estimated that the existing drainage system could convey approximately 13 cfs (in low tailwater conditions) without starting to fill the wet well of the pump station (see Section 7.1), meeting the County's gravity outfall during summer criterion (see Section 1.3). The existing gravity system also provides redundancy in case of pump failure.

The second considered pump station location, Alternative 5B, is north of the intersection between 1st St. and San Juan St. (see Figures E7 and E8 in Appendix E). This alternative consists of a proposed gravity storm drain that diverts flow from the manhole northwest of San Juan St. to the pump station. The invert elevations and the slope of the proposed gravity storm drain were designed to maintain 25-year water surface elevations within the parking lot of below 6.62 ft, as discussed in Section 7.5.

The pros and cons for Alternatives 5A and 5B are summarized in Table 3 in Section 4.10. A benefit of this Alternative 5 (both A and B) is that it utilizes an existing outfall, which is currently maintained by County Parks. Note, however, that the outfall would require modifications to handle the additional flow from the proposed drainage system. In the current outfall configuration, the existing 24" RCP outlets to a concrete structure with a riprap apron and a side outfall gate. This configuration forces change in flow direction, dissipating some of the energy. There is not sufficient room between the existing 24" RCP and the outfall gate to install the proposed 30" force main outlet, so the existing grate would need to be relocated. Additional energy dissipation measures would also be required, so that the discharge from the force main does not lead to erosion problems on the beach. The outfall structure may not be tall enough to fit the outlet from the 30" force main. The outfall structure may also need to be extended to the north to avoid conflicts between the proposed force main and the existing 24" storm drain. A more detailed study would be needed to ensure that the outfall structure weirs have adequate capacity to convey the additional flow from the force main without causing erosion problems on the beach. Enlarging the weirs and reconstructing the existing ramp may be required.

Additional challenges associated with Alternative 5 are that it requires relocation of the existing saltwater line along San Juan St. that is presumed to supply the nearby aquarium, as well as significant construction along San Juan St., necessitating stakeholder involvement. This alternative also does not meet the County's IWM goal for beneficial use of the stormwater.

A benefit of Alternative 5A is that its force main profile has fewer grade breaks than some of the other alternatives, improving constructability. Note that utility locations will need to be verified prior to final design and construction, which may affect the force main profile. Disadvantages of this pump station location include that it eliminates some parking spaces within the parking lot, and it requires relocating the existing 36" storm drain within the SLO County underground drainage easement. Another disadvantage is that the existing inlet on 1st St. would need to be removed or relocated to allow for construction of the 30" force main. It would be difficult to relocate this inlet northwest with the 36" storm drain, since there is an existing driveway at that location. Stormwater tributary to the existing inlet would instead flow southeast long 1st St. to the sidewalk underdrain at the intersection of San Francisco St. Note that like Alternative 3, the pump station is located within PSLHD property.

Advantages of Alternative 5B are that it requires a shorter force main than Alternative 5A, it does not require relocation of the existing 36" storm drain, and construction would be limited to County roads, easements and County Parks. A disadvantage of Alternative 5B, however, is that the existing 36" drainage system can convey just under 4 cfs without diverting flow to the wet well. Therefore, this alternative will require more frequent pumping and maintenance than Alternative 5A. Also, the pump station could face constructability complications due to the limited space available and the very deep wet well required. The pump station wet well would need to be very deep to maintain a free outfall on the proposed 36" storm drain, which is exacerbated by the fact that the site features high ground elevations compared to road elevations at the intersection of San Juan St. and 1st St. The proposed pump station leaves little room between the edge of the pump station and the property boundary. Therefore, it may not be feasible to excavate for the pump station, while ensuring stability of the adjacent ground and building during construction. The proposed site also includes two palm trees which would need to be removed to facilitate construction of the pump station. Another major disadvantage of Alternative 5B is several utility conflicts as the force main crosses 1st St., including a gas conduit, cable TV conduit, 6" water line, and electrical conduits (see Figure E8 in Appendix E). While the size and locations (horizontal and vertical) of these utilities need to be verified, it would be difficult to avoid utility conflicts, due to the number of utilities at this location. Even if it is possible to avoid utility conflicts, the force main profile would include several grade breaks to avoid conflicts, which impacts constructability.

4.7   Alternative 6: Front St. Outlet via San Francisco St

Alternative 6 involves installing a force main in San Francisco St. that conveys flow from a new pump station to a new beach outfall, as shown in Figures E9 through E12 in Appendix E. This alternative includes construction of a pump station within the parking lot, which discharges to a 30" force main along San Francisco St. Similar to Alternative 5, two locations were considered for the pump station. The first considered pump station location (Alternative 6A) is adjacent to the existing swale (see Figures E9 and E10 in Appendix E). An intake structure with a 22.5-ft long weir at elevation 5.0 ft would be constructed along the swale. This configuration would allow the existing drainage system to convey approximately 13 cfs during low tailwater conditions without overtopping the weir (see Section 7.1) , meeting the County's gravity outfall during summer criterion (see Section 1.3). Alternative 5A also involves construction of a 30" force main within the parking lot and along San Francisco St.

The second pump station location (Alternative 6B) is also within the parking lot, northeast of San Francisco St. This configuration includes an intake structure adjacent to the existing swale, which is connected to a proposed 36" storm drain that conveys flow from the swale to the proposed pump station. This alternative is shown in Figures E11 and E12 in Appendix E. As discussed in Section 7.6, the intake structure and the storm drain were designed to have adequate capacity to convey 35 cfs while maintaining water surface elevations within the parking lot below 6.62 ft. This configuration would allow the existing drainage system to convey approximately 9 cfs without starting to fill the wet well, which lower than Alternative 6A, but the alternative is still expected to meet the County's gravity outfall during summer criterion (see Section 1.3). Additionally, the wet well for Alternative 6B would be lower than Alternative 6A, since the intake weir is at a lower elevation and the configuration includes a deep gravity storm drain that conveys flow from the intake structure to the proposed wet well.

An alternate intake location was considered for Alternative 6, off the existing swale adjacent to the parking lot entrance on 1st St. This location was determined to be unfeasible, because it is not directly connected to the area of the parking lot that experiences ponding during the 25-year storm event. When the pump turns on, stormwater would need to flow

from the existing swale along the northwest edge of the parking lot, through the existing 24-inch storm drain (at an adverse slope), and into the intake structure. The existing 24-inch storm drain is expected to limit the capacity of the proposed drainage system, which could prevent the alternative from meeting the design criteria in the 25-year storm event.

The pros and cons of Alternative 6 are summarized in Table 3 at the end of this Section. Like Alternatives 3-5, this alternative can meet the 25-year water surface elevation criterion within the parking lot even during high tailwater (see Section 1.3). Additionally, the existing 36" drainage system that outfalls to SLO Creek would remain and provide redundancy. A disadvantage of this alternative is that it requires a new outfall structure, which is associated with additional costs and maintenance. This outfall structure may also lead to erosion at the beach outlet; further analysis and design would be required to investigate this erosion potential. Beach outfalls are also subject to wave- and tide-driven sand build up and driftwood debris, which have the potential for blockage and increased maintenance. Like Alternatives 3 and 5A, the pump station would be located within PSLHD property, which would require coordination prior to construction. Additionally, the pump station would eliminate some parking spaces within the parking lot. Another disadvantage of this alternative is that the force main profile would likely require several grade breaks to avoid utility conflicts. The profile of the force main may be refined once the utility locations are verified. This alternative also does not meet the County's IWM goal for beneficial use of the stormwater.

4.8   Alternative 7: Creek Outlet via Avila Beach Golf Resort

Alternative 7 involves installing a pump station with a force main that conveys stormwater from the parking lot along 1st St. to a proposed retention basin within the Avila Golf Resort. The proposed basin would include an overflow outlet to San Luis Obispo Creek. This alternative has the potential to meet the County's IWM goal for beneficial use of captured stormwater, since the pond could be used for recreation, as a feature of the golf course. While this system could improve the flooding within the parking lot, it would involve coordination with the Avila Beach Golf Resort, since the proposed basin is located on their property. Analysis of this alternative is not included in the present study, as GHD was instructed by the County to wait until coordination with the property owner is initiated, however, discussion is included for comparison.

4.9   Alternative 8: Wastewater Treatment Plant

Alternative 8 involves connecting the existing gravity drainage system to the existing sanitary sewer system. The sanitary sewer system would then convey stormwater to the Avila Beach Community Services District (ABCSD) wastewater treatment plant. The benefit of this alternative is that there would be limited construction, since no new pump system would need to be constructed. Additionally, the existing gravity system could be used as redundancy. This alternative could also meet the County's IWM goal by allowing for stormwater reuse. However, the sanitary sewer conveyance system and wastewater treatment facility do not have the capacity for the additional stormwater. Therefore, this alternative was not analyzed as part of the current study, but discussion is included for clarity.

4.10   Summary of Pros and Cons

All of the identified drainage alternative are summarized in Table 3, along with their pros and cons. Note that while permitting and funding requirements are briefly mentioned in Table 3, it was not included as part of this study, as permitting requirements will be determined by the County and cost estimates will be completed in future stages of design.

The pros and cons of each alternative were developed considering:

- Project design criteria, which is outlined in Section 1.3 and includes:
  - Hydraulic performance of the system during high tailwater conditions (analyzed in Section 6). Specifically, these hydraulic analyses were used to determine the alternative's ability to meet the project-specific design storm, tailwater, and flood inundation elevation design criteria.

- o Hydraulic capacity of the gravity system (without filling the wet well) during low tailwater conditions, which was evaluated in Section 6. These analyses were completed to determine each alternative's ability to meet the County's gravity outfall during the summer criterion, as well as to estimate relative use (and wear) of the pump stations.

- Constructability complications, which could affect project timelines and costs. These complications were identified when developing the plan and profile drawings of each alternative and include high groundwater, utility conflicts, grade breaks in the force main profiles, traffic over pump stations, deep wet wells, and space limitations.

- The project's goals, which are outlined in Section 1.2 and discussed in further detail in Section 5.

- Property limits and stakeholder involvement, identified through discussions with and information received from the County, which could affect the project timeline if additional stakeholder involvement, including meetings, are required as well as the feasibility of the alternative. Additionally, consideration of the property limits could affect property acquisition requirements, which could impact project costs, feasibility, and timelines.

- Extents of construction and use of existing facilities.

- Additional information received from the County that affect the feasibility of each alternative. For example, the capacity of the wastewater treatment plan for increased flows (relevant to Alternative 8).

A constructability complication that applies to all studied alternatives is the high groundwater table at the project site. A previous study by England Geosystem, Inc. entitled *Supplemental Investigation Report: Avila Beach Drive and the Rossi Property* (England Geosystem Inc., 2004), includes a profile of groundwater levels along Avila Beach Dr., between Front St. and San Juan St. This profile is included in Attachment 3 and displays groundwater depths of approximately 12 to 13 ft below the ground surface along Avila Beach Dr. The 2018 LiDAR data shows ground elevations of approximately 15.5 ft to 16 ft (NAVD88) along this stretch of Avila Beach Dr., suggesting groundwater elevations of up to approximately 4 ft. All considered pump station alternatives feature wet wells that extend below elevation 4 ft. More detailed groundwater measurements are recommended prior to final design and construction of the selected alternative; however, it is highly likely that the groundwater table is higher than the invert elevations of the proposed pump stations, as well as sections of the proposed force mains and storm drains. The pump station will thus need to be flood-proofed to prevent groundwater intrusion and it, along with the force main, will need to be designed to resist buoyant forces from the high groundwater. Additionally, the high groundwater can lead to construction complications for the proposed storm drains, force mains, and pump stations, requiring dewatering and disposal of pumped groundwater.

Table 3: Summary of drainage alternatives pros and cons

	Alternative 1	Alternative 2	Alternative 3[5]	Alternative 4	Alternative 5A
Design Concept	No change	Annual maintenance program	Pumps with parallel force main	Pumps on existing system	Front St. outlet via San Juan St.- pump station in parking lot
Description	No changes to the existing gravity storm drain system	Implement a maintenance program for the existing gravity system, funded by taxes.	Install a pump station in the parking with an intake from the existing swale and a force main parallel to the existing drainage system along 1st St.	Install a pump station near the existing 36" storm drain outlet to SLO Creek.	Install a pump station within parking lot, with an intake from the existing swale and a force main along San Juan St. to the existing beach outfall.
Analyzed in this study?	No	No	Yes	Yes	Yes
Pros	No construction or funding required Minimizes environmental impacts No permitting required	Minimal funding required No new construction Improves capacity of existing storm drain system Addresses silt problem Minimal environmental impacts	Existing gravity system can be used when tailwater is low or if pump fails. System can convey up to ±13 cfs during low tailwater without starting to fill wet well.[4] Drainage system will function during high tailwater Meets design criteria (see Section 1.3) Few grade breaks required in force main[2] Uses existing outfall, likely reducing environmental impacts Sediment/siltation is not expected to be an issue with the proposed force main and pump station intake from swale	Gravity system can be used when tailwater is low or if pump fails. System can convey up to ±29 cfs during low tailwater without starting to fill wet well, which reduces pumping and O&M requirements.[4] Drainage system will function during high tailwater Meets design criteria (see Section 1.3) when clear of sediment and debris Minimal construction compared with other alternatives (except 1 & 2) Cooperation with ABCSD and Port San Luis Harbor District (PSLHD)	Existing gravity system can be used when tailwater is low or if pump fails. System can convey up to ±13 cfs during low tailwater without starting to fill wet well.[4] Drainage system will function during high tailwater Meets design criteria (see Section 1.3) Few grade breaks required in force main[2] Does not require construction of a new outfall Outfall is maintained by County Parks Sediment/siltation is not expected to be an issue with the proposed force main and pump station intake from swale
Cons	Does not address flooding problems. Even minor storm events lead to flooding along 1st St Does not meet design criteria (see Section 1.3) Does not address silting problems at Avila Breach Dr/1st St. intersection manhole Drainage system does not function when water levels within SLO Creek are high High groundwater does not allow infiltration within ponded areas Operations and maintenance issues affect condition of existing system Does not meet IWM goal for beneficial use of the stormwater	Does not address flooding problems. Even minor storm events lead to flooding along 1st St Does not meet design criteria (see Section 1.3) Drainage system does not function when water levels within SLO Creek are high High groundwater does not allow infiltration within ponded areas Does not meet IWM goal for beneficial use of the stormwater	Requires funding Requires state, local, and/or federal permits[1] Creek outfall may be more difficult from a permitting perspective[1] Requires additional operations and maintenance Stakeholder involvement (Avila Beach Golf Resort, ABCSD, PSLHD, businesses/residents along 1st St., County Parks) Requires construction along Avila Beach Dr, 1st St, and parking lot Existing 36" SD would need to be relocated Removal of existing inlet on 1st St Existing CaTV conduit and phone conduit would need to be relocated Eliminates parking spaces High groundwater, leading to complications during construction Does not meet IWM goal	Requires funding Requires state, local, and/or federal permits[1] Creek outfall may be more difficult from a permitting perspective.[1] Requires additional operations and maintenance Stakeholder involvement (Avila Beach Golf Resort, ABCSD, PSLHD, businesses/residents along 1st St) Requires re-grading of existing access road Traffic over the pump station High groundwater, leading to complications during construction Deeper wet well than some of the other alternatives (excluding 5B and 6B), which increases construction difficulty[3] Does not meet IWM goal Relies on gravity pipes upstream of the pump station, which are prone to siltation. May not meet design criteria if sediment builds up within these pipes.	Requires funding Requires state, local, and/or federal permits[1] Requires additional operations and maintenance Stakeholder involvement (ABCSD, PSLHD, businesses/residents along 1st St. and San Juan St., County Parks) Requires construction along 1st St. and San Juan St Existing 36" storm drain would need to be relocated Removal of existing inlet on 1st St Existing 2" saltwater line would need to be relocated Eliminates parking spaces Beach outfall would need to be modified Potential for erosion at beach outlet High groundwater, leading to complications during construction Does not meet IWM goal

Notes:
1. *Permitting requirements and feasibility will be completed by others and are not evaluated in this study.*
2. *Confirmation of existing utility locations is required to verify this statement.*
3. *A more detailed hydraulic study is required to determine required wet well invert elevation.*
4. *Meets County's gravity outfall during summer design criterion.*
5. *Alternative presented in GHD's Peer Review Report (GHD, 2023).*

	Alternative 5B	Alternative 6A	Alternative 6B	Alternative 7	Alternative 8
Design Concept	Front St. outlet via San Juan St.- pump station SE of Avila Beach Dr & 1st St	Front St. outlet via San Francisco St.- pump station in east corner of parking lot	Front St. outlet via San Francisco St.- pump station north of 1st St. & San Francisco St	Creek outlet via Avila Beach Golf Resort	Gravity Connection to Sanitary Sewer
Description	Install a pump station southeast of the intersection between Avila Beach Dr & 1st St. with a force along San Juan St. to the existing beach outfall.	Install a pump station within the parking lot with an intake from the existing swale and a force main through the parking lot and along San Francisco St. to a new beach outfall.	Install a pump station near the parking lot entrance off 1st St. connected to the existing swale with a gravity storm drain and a force main through the parking lot and along San Francisco St. to a new beach outfall.	Install a pump station with a force main that conveys flow from the parking lot to a retention basin on Avila Golf Resort property. The basin will include an overflow outlet to SLO Creek.	Connect gravity storm drain system to the sewer lift system directly east of 1st St/San Francisco St., directing stormwater to ABCSD Wastewater Treatment Plant.
Analyzed in this study?	Yes	Yes	Yes	No	No
Pros	Existing gravity system can be used for low flows when tailwater is low or if pump fails.[4] Drainage system will function during high tailwater Meets design criteria (see Section 1.3) when clear of sediment and debris Does not require construction of a new outfall Outfall is maintained by County Parks Construction limited to easements/County Roads/County Parks	Existing gravity system can be used when tailwater is low or if pump fails. System can convey up to ±13 cfs during low tailwater without starting to fill wet well.[4] Drainage system will function during high tailwater Meets design criteria (see Section 1.3) Sediment/siltation is not expected to be an issue with the proposed force main and pump station intake from swale	Existing gravity system can be used for low flows when tailwater is low or if pump fails Drainage system will function during high tailwater. System can convey up to ±9 cfs during low tailwater without starting to fill wet well.[4] Meets design criteria (see Section 1.3)	Existing gravity system can be used when tailwater is low or if pump fails Drainage system will function during high tailwater Potential to meet design criteria (see Section 1.3) Attenuation within golf course retention basin Potential for beneficial use of water collected in the retention basin (IWM Goal)	No new pump station required Minimal construction compared with other alternatives Existing gravity system can be used as a redundancy Potential for stormwater reclamation (IWM Goal) Potential to meet design criteria (see Section 1.3)
Cons	Requires funding Requires state, local and/or federal permits[1] Requires additional operations and maintenance Stakeholder involvement (ABCSD, PSLHD, businesses/residents along 1st St. and San Juan St., County Parks) Requires removal of palm trees Existing 2" SW line would need to be relocated Several utility conflicts as the force main crosses 1st St. Beach outfall would need to be modified Potential for erosion at beach outlet Constructability complications: limited space, deep wet well[3], and high groundwater Wet well starts filling at ±4 cfs, leading to increased O&M Does not meet IWM goal Relies on gravity pipes upstream of the pump station, which are prone to siltation. May not meet design criteria if sediment builds up within these pipes.	Requires funding Requires state, local and/or federal permits[1] Requires additional operations and maintenance Stakeholder involvement (ABCSD, PSLHD, businesses/residents along 1st St. and San Francisco St) Construction of a new outfall structure (increases costs & time) Potential for erosion at beach outlet Eliminates parking spaces High groundwater, leading to complications during construction Does not meet IWM goal	Requires funding Requires state, local and/or federal permits[1] Requires additional operations and maintenance Stakeholder involvement (ABCSD, PSLHD, businesses/residents along 1st St. and San Francisco St) Construction of a new outfall structure (increases costs & time) Potential for erosion at beach outlet Eliminates parking spaces Drainage system has lower capacity without filling wet well (9 cfs) compared with several of the other alternatives. High groundwater, leading to complications during construction Deeper wet well than Alternative 6A[3] Does not meet IWM goal Gravity pipe upstream of pump station may experience siltation	Requires funding Requires state, local and/or federal permits[1] Construction of a new basin, which can be expensive Requires significant coordination with stakeholders/property owners Requires additional operations and maintenance High groundwater, leading to complications during construction	Requires funding Requires state, local and/or federal permits[1] Requires additional operations and maintenance Stakeholder involvement WWTP does not have adequate capacity for increased flows

Notes:
1. Permitting requirements and feasibility will be completed by others and are not evaluated in this study.
2. Confirmation of existing utility locations is required to verify this statement.
3. A more detailed hydraulic study is required to determine required wet well invert elevation.
4. Meets County's gravity outfall during summer design criterion.

5. Evaluation of Alternatives with Respect to Project Goals

In addition to analyzing the feasibility of the studied alternatives and determining pros and cons, GHD analyzed each alternative's ability to meet the project goals, which are described in Section 1.2. A summary of each alternative with respect to the County goals is included in Table 4. Note that environmental permitting requirements are not evaluated as part of this study, so any statements regarding environmental impacts and permitting will need to be verified.

All of the proposed alternatives would include trash racks to help meet the County's O&M goal of minimizing trash impacts to the system. Trash capture and/or stormwater quality requirements may arise during the permitting process, which may affect the design. Permitting requirements, however, are not evaluated as part of this study. Additionally, all of the proposed pump station alternatives include a backup generator to reduce required staff response during power outage.

On the other hand, none of the studied pump station alternatives meet the County's integrated water management goal of providing a solution that accounts for beneficial use of the water. As discussed in GHD's 2023 report, underground storage or low impact development (LID) facilities could be incorporated into the selected alternative to store and infiltrate runoff, reducing stormwater discharge from the site and improving water quality. The high groundwater at the project site, however, may compromise the performance of these LID facilities.

	Environmental and Permitting Goal[1]	Operations and Maintenance Goal	Integrated Water Management
Alternative 1: No Change	No environmental impacts No permitting required	No additional O&M required Trash rack to minimize trash impacts	Does not meet this goal
Alternative 2: Annual Maintenance Program	No environmental impacts No permitting anticipated	No additional O&M required Trash rack to minimize trash impacts	Does not meet this goal
Alternative 3: Pumps with Parallel Force Main	Utilizes existing outfall, which is expected to reduce environmental impacts Use of existing outfall may reduce permitting requirements[1] Potential impacts to existing swale (likely a jurisdictional wetland) for relocation of existing inlet and new wet well intake	Existing drainage system can be used as a gravity bypass system (with low tailwater and flows <±13 cfs), which reduces pump station O&M Backup generator to reduce required staff response during power outage Separate intake structure from existing gravity system, which may reduce trash/debris load from existing trash rack Eliminates duckbill check valve at SLO Creek and replaces it with a flap gate in the discharge structure, which is expected to be easier to inspect and maintain Trash rack to minimize trash impacts Minimal siltation is expected within the force main	Does not meet this goal[3]
Alternative 4: Pumps on Existing System	Utilizes existing outfall, which is expected to reduce environmental impacts Use of existing outfall may reduce permitting requirements No construction within existing swale (likely a jurisdictional wetland), which is expected to reduce environmental impacts and permitting requirements	Existing drainage system can be used as a gravity bypass system, which reduces pump station O&M. Existing drainage system can convey up to 29 cfs during low tailwater without filling wet well, which is higher than the other alternatives. Backup generator to reduce required staff response during power outage Eliminates duckbill check valve at SLO Creek and replaces it with a flap gate in the discharge structure, which is expected to be easier to inspect and maintain Shorter force main than other alternatives, which may reduce O&M Trash rack to minimize trash impacts Relies on gravity pipe upstream of pump station, which could be prone to sediment issues	Does not meet this goal[3]
Alternative 5A: Front St. Outlet via San Juan St. – pump station in parking lot	Utilizes existing outfall, which is expected to reduce environmental impacts Discharges to beach, which may reduce environmental impacts and permitting requirements relative to the Creek outfall Potential impacts to existing swale (likely a jurisdictional wetland) for relocation of existing inlet and new wet well intake	Existing drainage system can be used as a gravity bypass system (with low tailwater and flows <±13 cfs), which reduces pump station O&M Backup generator to reduce required staff response during power outage Existing beach outfall is currently maintained by County Parks Trash rack to minimize trash impacts Minimal siltation is expected within the force main	Does not meet this goal[3]
Alternative 5B: Front St. Outlet via San Juan St. – pump station SE of Avila Beach Dr & 1st St	Utilizes existing outfall, which is expected to reduce environmental impacts Discharges to beach, which may reduce environmental impacts and permitting requirements relative to the Creek outfall No construction within existing swale (likely a jurisdictional wetland), which is expected to reduce environmental impacts and permitting requirements	Existing drainage system can be used as a gravity bypass system, which reduces pump station O&M. Note, however, the system can only convey <±4 cfs without filling wet well, which is lower than the other proposed alternatives Backup generator to reduce required staff response during power outage Existing beach outfall is currently maintained by County Parks Trash rack to minimize trash impacts Relies on gravity pipe upstream of pump station, which could be prone to sediment issues	Does not meet this goal[3]
Alternative 6A: outlet via San Francisco St.- pump station in east corner of parking lot	Discharges to beach, which may reduce environmental impacts and permitting requirements relative to the Creek outfall Potential impacts to existing swale (likely a jurisdictional wetland) for new wet well intake	Existing drainage system can be used as a gravity bypass system (with low tailwater and flows <±13 cfs), which reduces pump station O&M Backup generator to reduce required staff response during power outage Trash rack to minimize trash impacts Minimal siltation is expected within the force main	Does not meet this goal[3]
Alternative 6B: Front St. outlet via San Francisco St.- pump station north of 1st St. & San Francisco St	Discharges to beach, which may reduce environmental impacts and permitting requirements relative to the Creek outfall Potential impacts to existing swale (likely a jurisdictional wetland) for new wet well intake	Existing drainage system can be used as a gravity bypass system (with low tailwater and flows <±13 cfs), which reduces pump station O&M Backup generator to reduce required staff response during power outage Trash rack to minimize trash impacts Gravity pipe upstream of pump station may experience siltation	Does not meet this goal[3]
Alternative 7: Creek outlet via Avila Beach Golf Resort[2]	Detention basin may reduce outflow and thus environmental impacts to the Creek Potential for naturalized treatment of stormwater in proposed basin, reducing environmental impacts	Existing drainage system can be used as a gravity bypass system, which reduces pump station O&M. Backup generator to reduce required staff response during power outage Trash rack to minimize trash impacts	Potential for beneficial use of captured stormwater, such as by providing recreational use (the proposed basin can be feature of the golf course)
Alternative 8: Gravity Connection to Sanitary Sewer[2]	No new outfall or construction within existing swale (likely a jurisdictional wetland), which is expected to reduce environmental impacts and permitting requirements	Existing drainage system can be used as a gravity bypass system, which reduces O&M No new pump station, reducing O&M Trash rack to minimize trash impacts	Potential for stormwater reuse

Notes:

1. *Environmental permitting is not included in this study. Any statements regarding environmental impacts and permitting have not been verified.*
2. *Alternative was not analyzed as a part of this study. Statements regarding these alternatives need to be verified.*
3. *Upstream LID facilities could be incorporated into the design to help meet this goal; however, high groundwater may compromise performance of these systems.*

6.   Qualitative Cost Ranking

A qualitative cost ranking has been applied to the analyzed alternatives in this study. This ranking is very preliminary and approximate, as cost estimates would be required to accurately rank the alternatives. The ranking was completed considering the approximate length of proposed pipe, as well as additional cost considerations such as relocation of existing utilities, and construction or modification of outfall structures. The ranking does not consider depth of the proposed pipes, which will impact the cost. The ranking also assumes a similar pump station cost for each alternative. Finally, the cost ranking does not include operations and maintenance costs associated with each alternative.

Table 5: Qualitative cost ranking of analyzed drainage alternatives

Alternative	Approximate Length of Proposed Pipe[1]	Additional Cost Considerations	Cost Ranking[2]
Alternative 3: Pumps with Parallel Force Main	370 ft	Relocation of up to 230 ft of existing phone conduit and cable TV conduit Relocation approximately 90 ft of existing 36" storm drain	Medium
Alternative 4: Pumps on Existing System	50 ft[3]		Lowest
Alternative 5A: Front St. Outlet via San Juan St. – pump station in parking lot	730 ft	Relocation of 90 ft of existing 36" SD and existing trash rack Modification of existing beach outfall Relocation of up to 270 ft of 2" saltwater line	High
Alternative 5B: Front St. Outlet via San Juan St. – pump station SE of Avila Beach Dr & 1st St	610 ft	Modification of existing beach outfall Relocation of up to 270 ft of 2" saltwater line May require shoring to construct the deep pump station in the limited space May require relocation of water, gas, electrical and/or TV conduits	Highest
Alternative 6A: outlet via San Francisco St.- pump station in east corner of parking lot	660 ft	New beach outfall	Medium
Alternative 6B: Front St. outlet via San Francisco St.- pump station north of 1st St. & San Francisco St	710 ft	New beach outfall	Medium

Notes:
1. *Approximate horizontal length, not considering depth/slope of pipe.*
2. *Cost ranking is very approximate; cost estimates are required to accurately rank these alternatives.*
3. *Includes length of proposed channel.*

7.   Hydraulic Analysis

7.1   Objectives

Concept-level hydraulic analyses were completed to preliminarily design and analyze the proposed drainage systems. The objectives of the hydraulic analyses are as follows:

1. Provide preliminary design of intake structures, weirs, channels, and gravity storm drains that convey flow to the proposed pump station. These structures were designed so that they would not limit to the flow to the pump station (i.e. the structures are able to convey the pump station's 25-year peak flow rate) while maintaining water surface elevation below 6.62 ft within the parking lot, meeting the project's design criteria (see Section 1.3).

2. Analyze the existing 36" gravity system upstream of the pump station in Alternatives 4 and 5B, to determine if it can convey the pump station's peak 25-year flow rate (±35 cfs) while maintaining a water surface elevation in the parking lot below 6.62 ft. This analysis was completed to determine the feasibility of Alternative 4 and 5B and their ability to meet the project's design criteria outlined in Section 1.3.

3. Evaluate the capacity of the existing gravity system during low tailwater conditions in each alternative prior to flow entering the pump station's wet well. This analysis was completed to evaluate whether the proposed designs meet the County's gravity outfall during the summer criterion (see Section 1.3).

These hydraulic analyses were completed using a StormCAD models, weir calculations, and normal depth calculations and are described in the following section.

7.2   Assumptions and Limitations

The preliminary hydraulic analyses were completed to evaluate the feasibility of each alternative and are based on several assumptions. These assumptions and limitations are discussed throughout the following section, with some of the general assumptions described below:

- The hydraulic analyses are steady state.
  - The analyses and preliminary designs of the structures upstream of the pump station (objectives 1 and 2 above) estimate the hydraulic behavior of the proposed drainage system at the peak pumping flow rate of 35 cfs (see Section 3), which assumes the pump station and attenuation within the parking lot behave similarly in all the alternatives and can be approximated by the pump station and reservoir modeled in HEC-HMS (see Section 3). Additionally, these analyses assume that the hydraulic behavior at the peak flow governs, and thus can be used to evaluate the alternatives' ability to meet the project design criteria.
  - The evaluation of the capacity of the existing gravity system estimates the maximum steady-state flow rate through the system. The actual capacity of the existing gravity system may differ in dynamic conditions, since the steady-state models do not account for any attenuation throughout the system.
- The tailwater conditions in the analysis for the proposed drainage system (objectives 1 and 2 above) was assumed to be 15 ft, consistent with the project's design criteria (see Section 1.3).
- A free outfall was used to evaluate the capacity of the existing gravity system during low tailwater conditions (objective 3 above). Given the low invert elevation at the existing outfall (1.79 ft), a free outfall may not be realistic, however, the analysis provides an estimate of the maximum capacity of the system in ideal conditions. Analysis of historic water surface elevations within SLO Creek could be used to determine more realistic boundary conditions for future hydraulic modeling.
- The analyses of the existing and proposed drainage systems assume that the structures are maintained and that there is no debris or silt reducing the capacity of the systems. While sediment is not expected to be an

issue within the proposed force mains, it has been observed within the existing system and would reduce the conveyance capacity of gravity pipes. Thus, the capacity calculations for gravity bypass systems indicate the maximum capacity in ideal conditions; capacity would be less than these values if sediment accumulates within the pipes and they are not appropriately maintained. Additionally, the performance of proposed Alternatives 4, 5B, and 6, which rely on gravity systems upstream of the pump station, may be prone to sedimentation within the pipes. These potential impacts are discussed in their respective sections of the report.

It is recommended that more detailed hydraulic analyses be completed prior to developing the 30% design of the selected alternative. A dynamic model is recommended to more accurately simulate the behavior of the pump, the ponding within the parking lot, and hydraulics within the proposed and existing storm drain systems. This dynamic model should be used to refine the design of the selected alternative, including the pump station selection, set points, intake structures, gravity storm drains, and force mains. Additionally, should an alternative be selected that relies on gravity drainage, sediment should be considered in the dynamic model.

7.3   Alternative 3, 5A, and 6A

7.3.1   Proposed Intake Weirs

The intake weirs for Alternatives 3, 5A, and 6A were sized and analyzed to ensure that the weirs have sufficient capacity to convey flow from the swale to the pump station (objective 1 above). The capacity of the 22.5-ft-long weir openings was analyzed considering water surface elevations of 5.5 ft and 6.0 ft within the swale, which represent the elevations at which the first and second pumps turn on, respectively. The capacity of the weirs was calculated using the weir equation (in FlowMaster), with a weir coefficient of 2.65 to account for the change in direction of flow due to orientation of the weir being perpendicular to the flow direction in the swale. This represents a conservative analysis; during high tailwater, flow will be ponded in the swale with minimal flow velocities. The weir flow calculations assumed that the tailwater does not restrict the flow over the weir. The weirs were designed with a crest elevation of 5.0 ft, resulting in capacities of 21 cfs and 60 cfs at water surface elevations of 5.5 ft and 6.0 ft, respectively. The pump station capacity with one and two pumps turned on is approximately 20 cfs and 35 cfs, respectively (see Section 3). Therefore, the designed intake weirs are expected to have adequate capacity to convey flow to the pump stations. Please refer to Appendix F for FlowMaster output.

7.3.2   Existing Gravity System Capacity (Low Tailwater)

The capacity of the existing gravity drainage system prior to overtopping the weir in low tailwater conditions was approximated using a StormCAD model, to determine the alternative's ability to meet the County's gravity outfall during summer criterion (objective 3 above). The StormCAD model was used to simulate the water surface profile from the outlet of the existing 36" storm drain to the inlet within the swale. This method assumes that water is ponded within the swale and that water surface elevations are controlled by the inlet of the 36" storm drain. Given that the swale invert elevation at the location of the pump station inlet is lower than the invert elevation of the 36" storm drain, it is likely that the storm drain will control the water surface elevations within the swale. However, a more detailed, unsteady analysis of the channel and the storm drain would provide more accurate estimates of the system's capacity without overtopping the weir.

The StormCAD model consisted of the following elements:

- Headwall (H-1): The 36" storm drain inlet was modeled as a headwall. The inlet was defined as square edge concrete inlet with headwall, which is represented with an entrance loss coefficient of 0.5.

- Storm Drain (SD-1): The segment of the 36" storm drain that conveys flow from the swale to the junction with the inlet east of the intersection of 1st St. and San Juan St. was modeled as a 36" circular conduit with a Manning's roughness of 0.014. This Manning's roughness coefficient is within the range of the values presented by Chow (1959) for concrete closed conduits flowing partly full. The pipe geometry, including invert elevations and lengths, were obtained from 2024 survey data from MBS (see Attachment 1).

- Junction (J-1): The junction connecting the two 36" storm drain segments was modeled with a loss coefficient of 0.6, which was selected to represent the bend losses and junction losses associated with the manhole. This loss coefficient was selected using StormCAD guidance for junctions with 45-degree bends.

- SD- 2: The segment of the 36" storm drain that connects the junction on 1st St. to the manhole was modeled as a 36" circular conduit with a Manning's roughness of 0.014. The pipe geometry, including invert elevations and lengths, were obtained from the MBS survey data (see Attachment 1).

- Manhole (MH-1): The manhole northwest of the intersection between 1st St. and San Juan St. was modeled with a loss coefficient of 0.5, following StormCAD guidance for a junction with no bend in the trunkline.

- SD-3: The segment of the 36" storm drain that conveys flow from the manhole to existing outlet to SLO Creek was modeled as a 36" circular conduit with a Manning's roughness of 0.014. Like pipe segments 1 and 2, the length and invert elevations were obtained from the MBS Survey data (see Attachment 1).

- Outfall (O-1): The downstream boundary condition of the StormCAD model was set to a free outfall to simulate low tailwater conditions within SLO Creek. This boundary condition represents ideal conditions for the system and was used to estimate the maximum capacity within the existing drainage system (prior to flow entering the proposed wet well).

The inflow to the StormCAD model was adjusted until the hydraulic grade upstream of the headwall was less than 5.0 ft. The results indicate that approximately 13 cfs can be conveyed through the existing storm drain system in low tailwater conditions without overtopping the weir into the wet well. Please refer to Appendix G for StormCAD results.

7.4   Alternative 4

7.4.1   Proposed Drainage System

The existing 36" storm drain was analyzed to ensure that it has adequate capacity to convey flow to the proposed pump station while maintaining a water surface elevation within the 1st St. parking lot below an elevation of 6.62 ft during the 25-year storm event (objective 2 above). A hydraulic analysis of the proposed concept was completed using StormCAD to determine the hydraulic grade line (HGL) along the 36" storm drain that conveys flow from the 1st St. parking lot to SLO Creek. A peak flow rate of 35 cfs was selected for the analysis, which is the peak flow rate from the pump station analysis described in Section 3. This analysis was also used to design the proposed side weir elevation that conveys flow to the pump station's wet well (objective 1 above).

The StormCAD model included the following elements:

- H-1, SD-1, J-1, and SD-2: These elements were modeled as described in Section 7.1.

- SD-3: The segment of the 36" storm drain that conveys flow from the manhole to the proposed 3-ft wide concrete channel was modeled as a 36" circular conduit with a Manning's roughness of 0.014. Like pipe segments 1 and 2, the length and upstream invert elevation was obtained from the MBS Survey data (see Attachment 1). The downstream invert elevation was interpolated.

- Outfall (O-1): The downstream boundary condition of the StormCAD model was set to a constant water surface elevation, representing the water surface elevation in the proposed 3-ft channel required to convey 35 cfs to the wet well. This elevation, 4.95 ft, was set so that the ponded water surface elevation within the parking lot remained below 6.62 ft.

The results of the analysis yielded a water surface elevation of 6.59 ft upstream of the pipe's inlet, indicating that the existing system has adequate capacity to convey 35 cfs to the pump station without exceeding the target water surface elevation within the parking lot when the system is clear of debris or sediment. Please refer to Appendix G for StormCAD output.

The water surface elevation in the proposed channel of 4.95 ft was used to calculate the elevation of the 22.5-ft-long side weir using the standard weir equation (in FlowMaster). A weir coefficient of 2.65 was selected to represent the

side weir, which on the lower end of the range of weir coefficients provided in StormCAD guidance. This weir coefficient was selected for a conservative analysis and to consider the change in direction of flow due to the side weir. The resulting elevation of the side weir is 4.25 ft. Please refer to Appendix F for FlowMaster output.

A sensitivity analysis was completed in which the manning's roughness of the 36" storm drains in the StormCAD model was increased to 0.025 to simulate increased roughness due to sediment within the pipes. The results of this analysis yielded a water surface elevation within the parking lot that exceeds the design elevation of 6.62 ft, even with a free outfall downstream boundary condition. These results indicate that Alternative 4 is sensitive to sediment conditions and would require regular maintenance to prevent buildup of sediment within the pipes.

7.4.2   Existing Drainage System (Low Tailwater)

A second StormCAD model was created to determine the performance of the system during low tailwater to determine the alternative's ability to meet the County's gravity outfall during summer criterion (objective 3 above). The StormCAD model includes the components defined above. The model was extended to also include the following:

- Proposed Channel (CHAN): The 22.5-ft long and 3-ft wide proposed rectangular concrete channel was modeled as a rectangular conduit with a Manning's roughness of 0.014. The height of the conduit was set sufficiently large to prevent pressurized flow within the conduit. The slope of the channel was set to the same slope as pipe segment 2.

- Discharge Structure (MH-3): The discharge structure was modeled as a manhole with a loss coefficient of 0.5, following StormCAD guidance for a manhole with no bend at the junction.

- SD-4: The existing segment of the pipe that connects the discharge structure to the existing outfall was modeled as a 36" circular conduit with a Manning's roughness of 0.014 and the same slope as pipe segment 2.

- Free outfall (O-1): The downstream model boundary was set to a free outfall, assuming low tailwater conditions within San Luis Obispo Creek. This boundary condition represents ideal conditions for the system and was used to estimate the maximum capacity within the existing drainage system (prior to flow entering the proposed wet well).

The model inflow was adjusted until the water surface elevation within the channel was below the weir elevation (4.25 ft), to determine the gravity system's capacity during low tailwater. The results indicate that the system can convey approximately 29 cfs without overtopping the weir. StormCAD output for this model is also included in Appendix G.

7.5   Alternative 5B

7.5.1   Proposed Drainage System

The gravity drainage system upstream of the pump station in Alternative 5B was also analyzed in StormCAD, to ensure the system has adequate capacity to maintain water surface elevations within the parking lot below 6.62 ft (objective 2 above). The model was also used to design the proposed gravity pipe that diverts flow from the existing 36" pipe to the proposed pump station (objective 1 above). The StormCAD model extends from the existing inlet in the parking lot swale to the proposed pump station's wet well and was modeled with a constant inflow of 35 cfs. The model includes the following components:

- Pipe Inlet, Pipe Segment 1, Junction, Pipe Segment 2: These elements were modeled using the same assumptions and geometry as Alternative 4.

- Diversion Manhole: The manhole northwest of the intersection between San Juan St. and 1st St. was modeled as a manhole with a head loss coefficient of 0.8, which was selected to represent the bend losses and junction losses associated with the manhole. This loss coefficient was selected using StormCAD guidance for junctions with 90-degree bends.

- Proposed Pipe: The proposed pipe was modeled as a 36" circular conduit with a roughness of 0.014. The upstream invert elevation (3.06 ft) and slope (1%) of the pipe were designed to ensure the ponding within the parking lot remains below elevation 6.62 ft.

- Model Outfall: The downstream boundary condition is located at the pump station's wet well. The boundary condition for the proposed pipe was set to a free outfall condition, which assumes that the pump station maintains low water surface elevations within the wet well.

The results of the analysis yielded a water surface elevation of 6.60 ft within the parking lot, meeting the project's design criteria (see Section 1.3) when free of sediment or debris. Please refer to Appendix G for StormCAD output. Note that ensuring a free outfall on the proposed storm drain would require a deep wet well. A dynamic hydraulic model of the drainage system and pump station would be required to determine invert elevations and pump operation points to maintain this free outfall.

A sensitivity analysis was completed by changing the roughness of the gravity pipes to 0.025 to simulate increased roughness due sediment within the pipes. The results of the analysis indicated that the upstream gravity system does not have adequate capacity to convey 35 cfs while meeting the target water surface elevation within the parking lot, even if the proposed pipe is lowered. Thus, this alternative would only meet the design criteria in ideal conditions, where the gravity pipes are free of sediment. As siltation is reportedly an issue with the existing storm drains, this alternative may not be feasible.

7.5.2   Existing Gravity System (Low Tailwater)

Like Alternative 4, the existing gravity system was modeled under low tailwater conditions to determine the capacity of the existing system without diverting to the wet well (objective 3 above). The Alternative 5B model, described above, was modified by removing the proposed pipe, and instead modelling the existing 36" pipe to the existing outfall (assumed to be a free outfall). The head loss coefficient of the diversion manhole was reduced to 0.5, following StormCAD guidance for a junction with no bend in the trunkline. The inflow to the model was adjusted until the hydraulic grade line within the diversion manhole was below the proposed pipe invert elevation (3.06 ft). The results indicate that the system can convey just below 4 cfs before stormwater is diverted to the wet well. StormCAD results for this model are also included in Appendix G.

7.6   Alternative 6B

7.6.1   Proposed Drainage System

The proposed intake structure and gravity system for Alternative 6B were designed and analyzed to ensure that they would not limit flow to the pump station in the 25-year storm event (objective 1 above). The proposed gravity storm drain was designed to ensure the full flow capacity exceeded 35 cfs, which is the peak design flow of the pump station (see Section 3). Calculations were complete using Manning's equation (in FlowMaster) to determine the full flow capacity of the proposed 42" storm drain with a 0.2%. The full flow capacity of the 42" storm drain is approximately 42 cfs, which exceeds the design flow of 35 cfs. Therefore, the 42" storm drains should have adequate capacity to convey flow from the existing swale to the parking lot, to maintain water surface elevations within the parking lot of below 6.62 ft. A dynamic model is recommended to confirm this finding and refine the size of the proposed gravity storm drain. Additionally, the effects of siltation on this storm drain should be considered, which may result in a larger pipe. Please refer to Appendix F for FlowMaster output.

The proposed intake structure for Alternative 6B includes two weir openings. The first weir opening is located along the side of the existing swale and is 12 ft long, with a crest elevation of 4.7 ft. The capacity of this weir was calculated (in FlowMaster) using a conservative weir coefficient of 2.65 to represent the side weir. The capacity of the intake weir was calculated with water surface elevations of 5.5 ft and 6.0 ft, which are the elevations at which the first and second pumps turn on, respectively. The weir flow rate with water surface elevation of 5.5 ft is 22.8 cfs, which exceeds the first pump's capacity of 20 cfs. The weir flow rate with a water surface elevation of 6.0 ft is 47.1 cfs, which exceeds the

pump station's capacity of 35 cfs with two pumps running. Therefore, the weir openings are not expected to limit the flow to the pump station. A second 4-ft-long weir with a crest elevation of 5.0 ft is proposed along the east wall of the intake structure, to provide redundancy. Please refer to Appendix F for FlowMaster weir calculations.

7.6.2   Existing Gravity System (Low Tailwater)

The capacity of the channel before the stormwater starts overflowing the weirs (objective 3 above) was calculated by modifying the inflow to the StormCAD model described in Section 7.1 so that the water surface elevation upstream of the 36" inlet did not exceed 4.7 ft. Results of this model indicate that the existing drainage system can convey approximately 9 cfs without starting to fill the wet well. Model output is included in Appendix G.

8.   Conclusion

GHD has analyzed and developed concept designs for four pump station alternatives to address flooding within the parking lot and at the intersection of 1st St. and San Francisco St., in Avila Beach, California. Preliminary designs of pump stations, intake structures, gravity storm drains, and force mains were developed and analyzed to maintain 25-year water surface elevations within the parking lot 6 inches below the lowest finished floor elevation of the adjacent buildings considering high tailwater conditions at the outfall. The technical feasibility of each alternative is presented, including hydraulic performance, constructability, and utility conflicts. This study revealed that some alternatives will face significant constructability challenges. For example, Alternative 5B faces several constructability complications, such as a very deep wet well, high groundwater, and limited space to construct the proposed pump station, which may affect stability of the adjacent residential structures. Additionally, sediment issues may hinder feasibility of alternatives that rely on existing gravity pipes upstream of the pump station, including Alternatives 4 and 5B. The preferred alternative, however, depends on additional factors not studied as a part of this project, including environmental impacts, permitting requirements, property acquisition, stakeholder involvement, and costs. Prior to selecting a preferred alternative for 30% design, it is recommended that these factors be evaluated.

The conceptual designs developed as part of this study are based on preliminary hydraulic analyses described in this report, which are based on several assumptions. It is recommended that detailed dynamic models of the drainage system be developed as part of the 30% design effort, which can be used to more accurately analyze the performance of the existing and proposed drainage systems, determine the set points of the proposed pump station, and refine the design of intake structures, pump stations, force mains, and gravity storm drains. Additionally, should an alternative with a beach outfall be selected, outfall erosion should be analyzed to provide a design that minimizes impacts to the beach. The tailwater condition for the beach outfall should also be reevaluated.

It is recommended that several data gaps be addressed while refining the design of the selected alternative. Geotechnical data and recommendations, including groundwater data, should be obtained to inform the proposed design and construction, including any dewatering requirements. Several of the utility sizes and locations were assumed while developing the concept drawings, due to limited available utility data. These locations and sizes should be verified to refine the design, including the force main location and profile. Finally, the location and capacity of the electrical service of the proposed pump station should be identified as the design is refined.

9.   References

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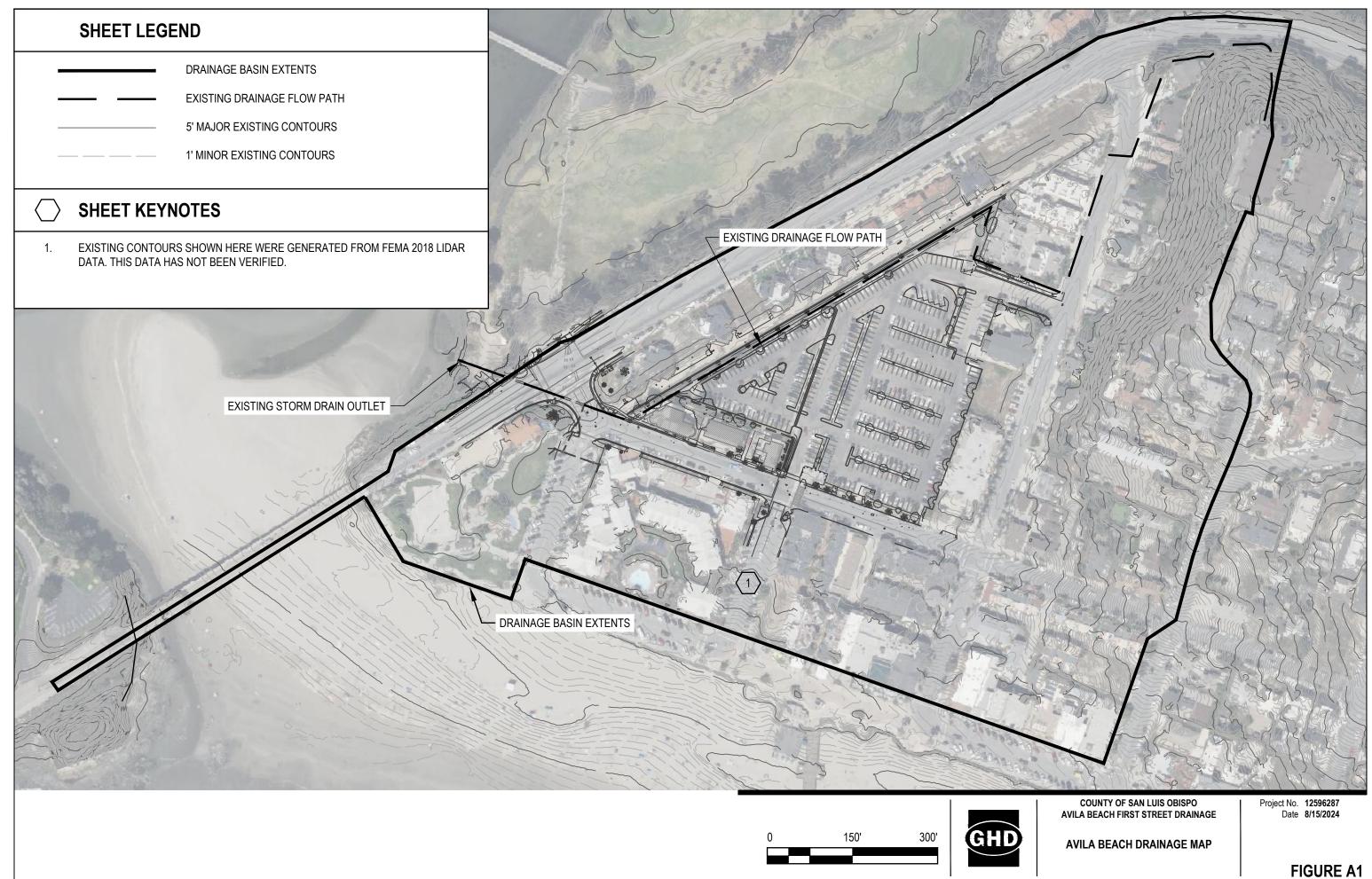
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Appendices
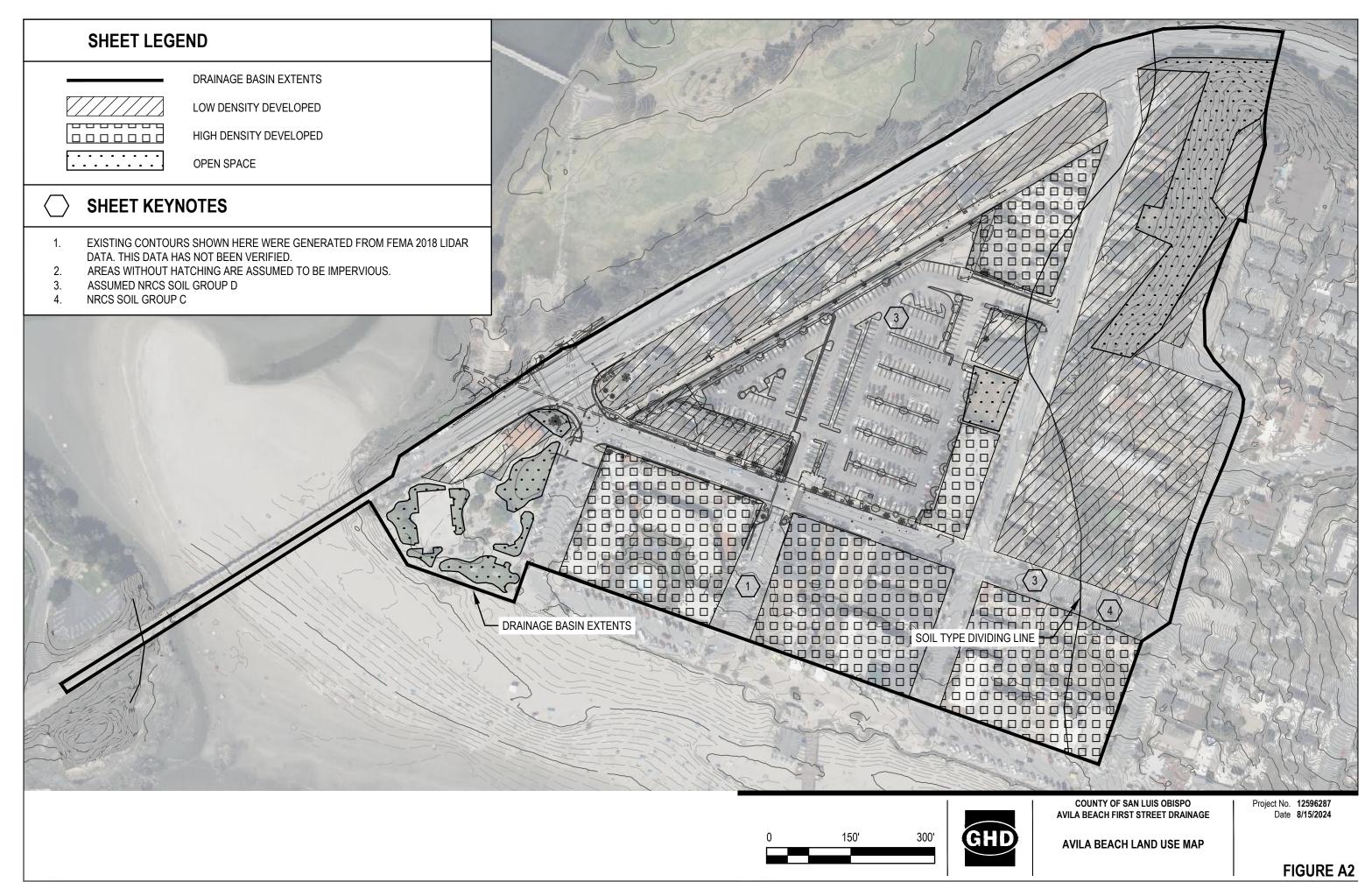
Appendix A

Drainage Maps

SHEET LEGEND

— DRAINAGE BASIN EXTENTS

– – – EXISTING DRAINAGE FLOW PATH

— 5' MAJOR EXISTING CONTOURS

– – – 1' MINOR EXISTING CONTOURS

SHEET KEYNOTES

1. EXISTING CONTOURS SHOWN HERE WERE GENERATED FROM FEMA 2018 LIDAR DATA. THIS DATA HAS NOT BEEN VERIFIED.

EXISTING DRAINAGE FLOW PATH

EXISTING STORM DRAIN OUTLET

DRAINAGE BASIN EXTENTS

0   150'   300'

Filename: \\ghdnet\ghd\US\San Luis Obispo\Projects\561\12596287\Digital_Design\ACAD\Figures\12596287-FIG-Drainage Map.dwg
Plot Date: 15 August 2024 9:30 AM

Data Source

DRAINAGE BASIN EXTENTS

SOIL TYPE DIVIDING LINE

0   150'   300'

Appendix B

Hydrologic Analysis Inputs

B-1 NOAA Atlas 14 Point Precipitation Frequency Estimates

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.121 (0.104-0.140)	0.154 (0.133-0.180)	0.197 (0.170-0.231)	0.232 (0.198-0.274)	0.278 (0.229-0.342)	0.313 (0.251-0.395)	0.348 (0.271-0.452)	0.384 (0.289-0.515)	0.432 (0.310-0.608)	0.468 (0.323-0.686)
10-min	0.173 (0.150-0.201)	0.221 (0.191-0.258)	0.283 (0.244-0.331)	0.332 (0.284-0.393)	0.399 (0.328-0.490)	0.449 (0.360-0.566)	0.499 (0.389-0.648)	0.551 (0.415-0.738)	0.619 (0.444-0.871)	0.671 (0.463-0.983)
15-min	0.209 (0.181-0.243)	0.267 (0.231-0.312)	0.342 (0.295-0.400)	0.402 (0.344-0.475)	0.482 (0.396-0.593)	0.543 (0.435-0.685)	0.604 (0.470-0.784)	0.666 (0.502-0.893)	0.749 (0.538-1.05)	0.812 (0.560-1.19)
30-min	0.295 (0.255-0.343)	0.377 (0.326-0.439)	0.482 (0.416-0.564)	0.567 (0.484-0.670)	0.680 (0.559-0.836)	0.766 (0.614-0.965)	0.852 (0.663-1.10)	0.939 (0.708-1.26)	1.06 (0.758-1.48)	1.14 (0.790-1.68)
60-min	0.420 (0.364-0.490)	0.537 (0.465-0.627)	0.688 (0.593-0.805)	0.808 (0.691-0.956)	0.970 (0.797-1.19)	1.09 (0.875-1.38)	1.21 (0.946-1.58)	1.34 (1.01-1.80)	1.50 (1.08-2.12)	1.63 (1.13-2.39)
2-hr	0.648 (0.562-0.755)	0.810 (0.701-0.945)	1.01 (0.874-1.18)	1.17 (1.00-1.38)	1.37 (1.13-1.69)	1.52 (1.22-1.92)	1.67 (1.30-2.16)	1.81 (1.36-2.43)	2.00 (1.44-2.81)	2.14 (1.47-3.13)
3-hr	0.827 (0.717-0.964)	1.03 (0.891-1.20)	1.28 (1.10-1.49)	1.47 (1.25-1.73)	1.71 (1.40-2.10)	1.88 (1.51-2.37)	2.05 (1.60-2.66)	2.22 (1.67-2.97)	2.43 (1.74-3.42)	2.58 (1.78-3.78)
6-hr	1.16 (1.01-1.36)	1.45 (1.25-1.69)	1.79 (1.54-2.09)	2.04 (1.74-2.41)	2.36 (1.94-2.91)	2.59 (2.08-3.27)	2.81 (2.19-3.64)	3.02 (2.27-4.04)	3.28 (2.36-4.62)	3.48 (2.40-5.09)
12-hr	1.48 (1.28-1.73)	1.86 (1.61-2.17)	2.32 (2.00-2.72)	2.68 (2.29-3.16)	3.13 (2.57-3.85)	3.46 (2.77-4.36)	3.77 (2.94-4.89)	4.08 (3.08-5.47)	4.48 (3.22-6.30)	4.77 (3.29-6.98)
24-hr	1.85 (1.69-2.06)	2.34 (2.15-2.61)	2.99 (2.73-3.34)	3.50 (3.17-3.94)	4.19 (3.68-4.87)	4.71 (4.05-5.58)	5.22 (4.40-6.34)	5.75 (4.71-7.16)	6.46 (5.09-8.36)	6.99 (5.33-9.35)
2-day	2.25 (2.06-2.51)	2.90 (2.65-3.23)	3.74 (3.42-4.18)	4.44 (4.02-5.00)	5.38 (4.72-6.25)	6.11 (5.26-7.24)	6.85 (5.76-8.30)	7.62 (6.24-9.48)	8.66 (6.83-11.2)	9.48 (7.23-12.7)
3-day	2.58 (2.36-2.87)	3.34 (3.06-3.72)	4.35 (3.97-4.86)	5.18 (4.69-5.83)	6.33 (5.56-7.35)	7.22 (6.22-8.56)	8.15 (6.86-9.88)	9.11 (7.47-11.3)	10.4 (8.23-13.5)	11.5 (8.76-15.4)
4-day	2.80 (2.56-3.12)	3.64 (3.33-4.06)	4.76 (4.35-5.32)	5.70 (5.16-6.42)	6.99 (6.14-8.13)	8.01 (6.90-9.50)	9.07 (7.63-11.0)	10.2 (8.34-12.7)	11.7 (9.23-15.2)	12.9 (9.87-17.3)
7-day	3.30 (3.02-3.67)	4.31 (3.94-4.80)	5.67 (5.18-6.34)	6.82 (6.18-7.68)	8.43 (7.40-9.80)	9.71 (8.36-11.5)	11.1 (9.30-13.4)	12.5 (10.2-15.5)	14.5 (11.4-18.7)	16.1 (12.3-21.5)
10-day	3.72 (3.40-4.14)	4.87 (4.46-5.43)	6.44 (5.87-7.19)	7.76 (7.03-8.74)	9.62 (8.45-11.2)	11.1 (9.57-13.2)	12.7 (10.7-15.4)	14.4 (11.8-17.9)	16.7 (13.2-21.6)	18.6 (14.2-24.9)
20-day	4.76 (4.36-5.30)	6.29 (5.76-7.02)	8.38 (7.65-9.37)	10.1 (9.19-11.4)	12.6 (11.1-14.7)	14.6 (12.6-17.4)	16.8 (14.1-20.3)	19.0 (15.6-23.7)	22.2 (17.5-28.7)	24.8 (18.9-33.1)
30-day	5.82 (5.34-6.48)	7.71 (7.06-8.60)	10.3 (9.38-11.5)	12.5 (11.3-14.0)	15.5 (13.6-18.0)	18.0 (15.5-21.3)	20.6 (17.3-24.9)	23.3 (19.1-29.0)	27.2 (21.4-35.2)	30.3 (23.1-40.6)
45-day	7.02 (6.44-7.82)	9.29 (8.50-10.4)	12.4 (11.3-13.8)	14.9 (13.5-16.8)	18.6 (16.3-21.6)	21.5 (18.5-25.4)	24.5 (20.6-29.7)	27.7 (22.7-34.5)	32.2 (25.4-41.7)	35.9 (27.4-48.0)
60-day	8.28 (7.58-9.22)	10.9 (9.97-12.1)	14.4 (13.2-16.1)	17.4 (15.7-19.6)	21.5 (18.9-25.0)	24.8 (21.3-29.4)	28.2 (23.7-34.2)	31.8 (26.1-39.6)	36.9 (29.1-47.7)	40.9 (31.2-54.7)
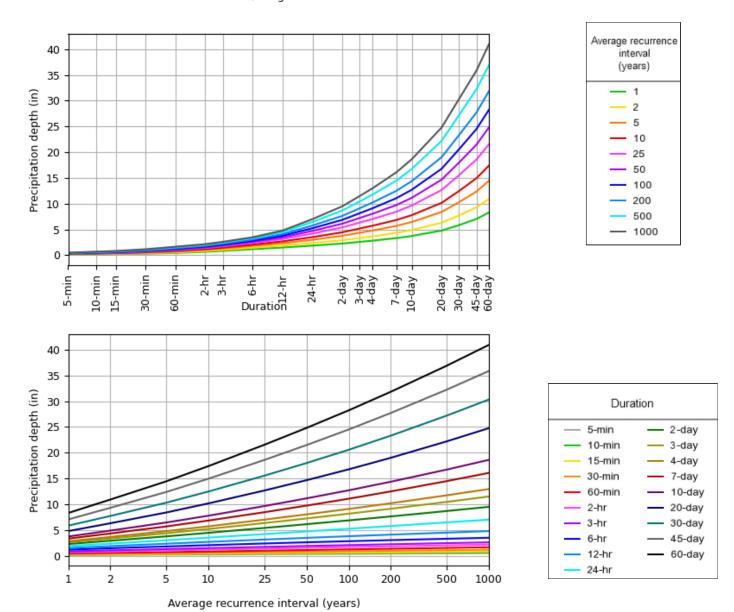
[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Back to Top

PF graphical

PDS-based depth-duration-frequency (DDF) curves
Latitude: 35.1799°, Longitude: -120.7346°
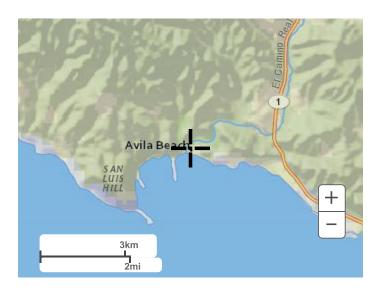
Created (GMT): Wed Jun 14 20:37:53 2023
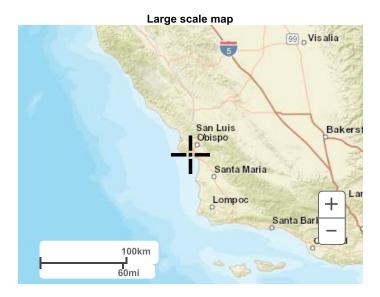
Maps & aerials
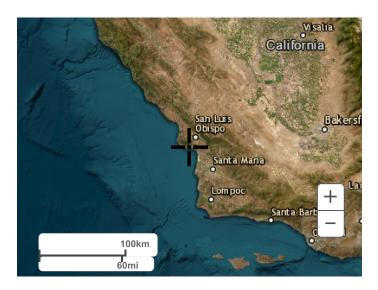
Small scale terrain

Large scale terrain

Large scale map

Large scale aerial
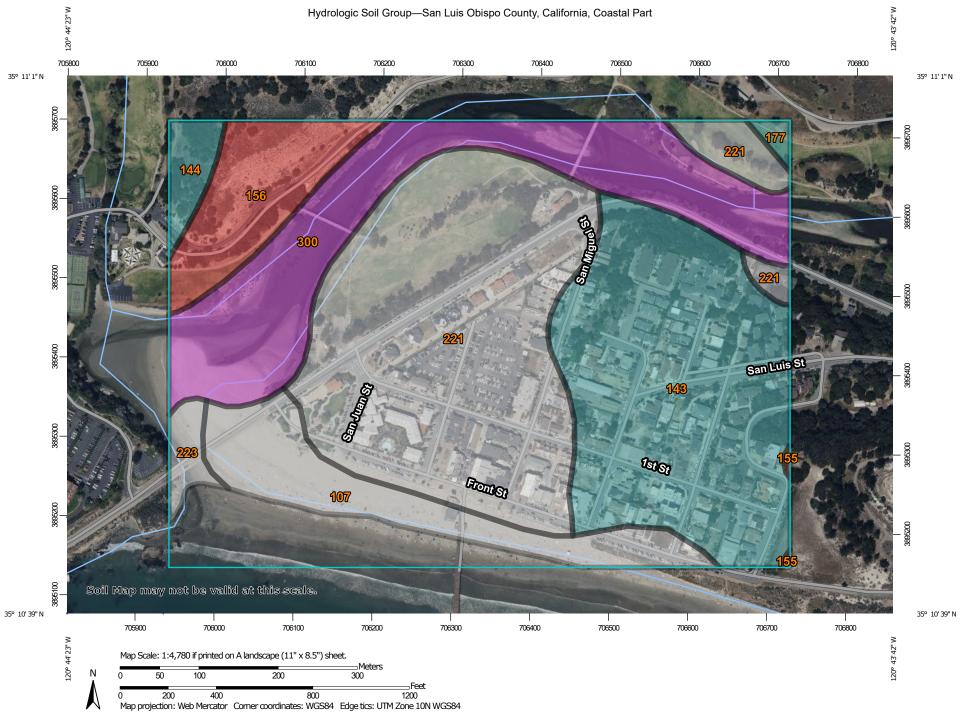
[Back to Top](#)

B-2   NRCS Hydrologic Soil Group Map

Map Scale: 1:4,780 if printed on A landscape (11" x 8.5") sheet.

N

Meters				
0	50	100	200	300

Feet			
0	200	400	800	1200

Map projection: Web Mercator   Corner coordinates: WGS84   Edge tics: UTM Zone 10N WGS84

USDA **Natural Resources**
Conservation Service

Web Soil Survey
National Cooperative Soil Survey

4/25/2024
Page 1 of 4

MAP LEGEND

MAP INFORMATION

Area of Interest (AOI)
☐ Area of Interest (AOI)

Soils

Soil Rating Polygons
▦ A

▦ A/D

▦ B

▦ B/D

▦ C

▦ C/D

▦ D

▦ Not rated or not available

Soil Rating Lines
〰 A

〰 A/D

〰 B

〰 B/D

〰 C

〰 C/D

〰 D

〰 Not rated or not available

Soil Rating Points
▪ A

▪ A/D

▪ B

▪ B/D

▪ C

▪ C/D

▪ D

▪ Not rated or not available

Water Features
〰 Streams and Canals

Transportation
┿┿┿ Rails

〰 Interstate Highways

〰 US Routes

〰 Major Roads

〰 Local Roads

Background
▦ Aerial Photography

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: San Luis Obispo County, California, Coastal Part
Survey Area Data: Version 16, Sep 11, 2023

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Mar 12, 2022—Apr 12, 2022

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
107	Beaches		9.3	8.4%
143	Gazos-Lodo clay loams, 15 to 30 percent slopes	C	27.2	24.7%
144	Gazos-Lodo clay loams, 30 to 50 percent slopes	C	1.9	1.7%
155	Lopez very shaly clay loam, 9 to 30 percent slopes	D	0.1	0.1%
156	Lopez very shaly clay loam, 30 to 75 percent slopes	D	7.3	6.6%
177	Nacimiento silty clay loam, 15 to 30 percent slopes	C	0.5	0.4%
221	Xererts-Xerolls-Urban land complex, 0 to 15 percent slopes		35.9	32.6%
223	Xerorthents, escarpment		1.4	1.3%
300	Corducci and Typic Xerofluvents, 0 to 5 percent slopes, occasionally flooded, MLRA 14	A	19.3	17.6%
Totals for Area of Interest			**110.1**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

Appendix C

Hydrologic Calculations

C-1　Curve Number Calculations

Composite Curve Number Calculations

Project: Avila Beach Drainage
Updated: 5/31/2024

Curve Numbers

Land Cover	Hydrologic Soil Group (HSG)	Curve Number, *CN*	
Low Density Developed	C	90	(TR-55, Table 2-2a: Residential 1/8 acre)
	D	92	(TR-55, Table 2-2a: Residential 1/8 acre)
High Density Developed	C	94	(TR-55, Table 2-2a: Urban districts; commercial and business)
	D	95	(TR-55, Table 2-2a: Urban districts; commercial and business)
Impervious	C	98	(TR-55, Table 2-2a: Impervious areas; paved parking lots, roofs, driveways etc)
	D	98	(TR-55, Table 2-2a: Impervious areas; paved parking lots, roofs, driveways etc)
Open Space	C	79	(TR-55, Table 2-2a: Open space; fair condition)
	D	84	(TR-55, Table 2-2a: Open space; fair condition)

Calculation Description

Composite curve numbers are calculated by using the Area Weighted Average method as follows:

Total Area, *A* (ft^2)
1,163,340

$$Composite\ CN = \frac{\Sigma(CN_1 A_1 + CN_2 A_2 + ... + CN_n A_n)}{\Sigma(A_1 + A_2 + ... + A_n)}$$

Area by HSG and Land Cover (ft^2)							
HSG C				HSG D			
Low-density	High-density	Impervious	Open Space	Low-density	High-density	Impervious	Open Space
158,444.59	26,366.05	51645.42768	54507.925	142,639.25	235,731.80	464808.849	29195.787

Area by HSG and Land Cover (ac)							
HSG C				HSG D			
Low-density	High-density	Impervious	Open Space	Low-density	High-density	Impervious	Open Space
3.64	0.61	1.19	1.25	3.27	5.41	10.67	0.67

Composite Curve Numbers

Sub-basin	Total Area, *A* (ac)	Total Area, *A* (sq. mi.)	Percent Developed	Composite Curve Number, *CN*
Avila	26.71	0.042	92.8%	94.23

References

1. NRCS. 1986. TR-55: Urban Hydrology for Small Watersheds.

\\ghdnet\ghd\US\San Luis Obispo\Projects\561\12596287\Tech\H&H\Hydrology\12596287-CAL-Hydrology Calcs-SCS Hydrograph Method.xlsx

GHD | County of San Luis Obispo | 12596287 | Feasibility Study 1/1

C-2 Time of Concentration Calculations

Time of Concentration Calculation
Project: Avila Beach Drainage
Updated: 6/6/2024

Calculation Description

Time of concentration calculated based on SLO County Department of Public Works Standard Drawing H-2. See: \\ghdnet\ghd\US\San Luis Obispo\Projects\561\12596287\Tech\H&H\Hydrology\1256287-ToC.pdf

EQUATIONS FOR ESTIMATED
"TIME OF CONCENTRATION"

$$Tc = \left(\frac{11.9L^3}{H}\right)^{0.385}$$

LEGEND:
Tc = TIME OF CONCENTRATION
      IN HOURS.
L = LENGTH OF CHANNEL IN
      MILES.
H = DIFFERENCE IN ELEVATION
      BETWEEN MOST REMOTE
      POINT AND THE POINT OF
      CONCENTRATION IN FEET.

Length of Flowpath, L (ft)	2120.1	*see drainage map
Length of Flowpath, L (mi)	0.4	
Upstream Elevation (ft)	86.0	*based on LiDAR data
Downstream Elevation (ft)	1.8	*based on MBS topo map, invert elevation of outlet
H (ft)	84.2	
Tc (hrs)	0.2	
Tc (min)	9.8	
Lag time (min)	5.9	*Taken as 0.6 times Tc
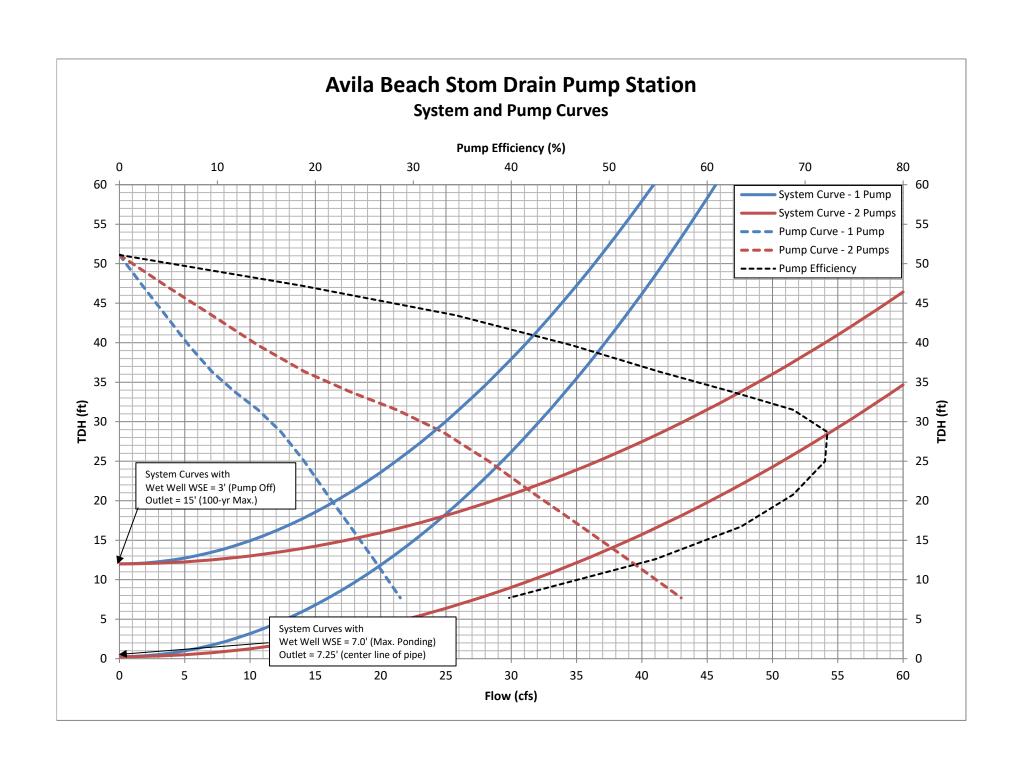
\\ghdnet\ghd\US\San Luis Obispo\Projects\561\12596287\Tech\H&H\Hydrology\12596287-CAL-Hydrology Calcs-SCS Hydrograph Method.xlsx

GHD | County of San Luis Obispo | 12596287 | Feasibility Study   1/1

Appendix D
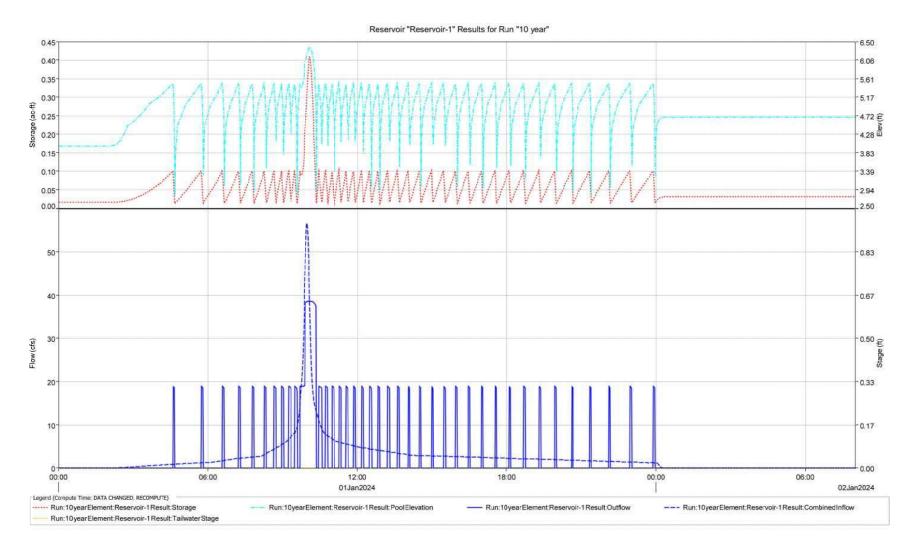
Preliminary Pump Station Design Calculations

Avila Beach Stom Drain Pump Station
System and Pump Curves

Pump Efficiency (%)

Legend:
- System Curve - 1 Pump
- System Curve - 2 Pumps
- Pump Curve - 1 Pump
- Pump Curve - 2 Pumps
- Pump Efficiency

System Curves with
Wet Well WSE = 3' (Pump Off)
Outlet = 15' (100-yr Max.)

System Curves with
Wet Well WSE = 7.0' (Max. Ponding)
Outlet = 7.25' (center line of pipe)

TDH (ft)

Flow (cfs)

Parking Lot and Swale Stage-Area Curve

Contour (ft)	Area (sf)	Source
-3.0	100	Small area used to
4.0	100	represent the wet well
4.5	1,025	
5.0	3,592	
5.5	6,019	
6.0	12,234	MBS Topo
6.5	38,747	
7.0	114,262	
7.5	183,901	

Pump Station Hydrologic Model Output
10-year Storm with Free Discharge Condition at Outfall

Reservoir "Reservoir-1" Results for Run "10 year"

Legend (Compute Time: DATA CHANGED, RECOMPUTE)
- Run:10year Element:Reservoir-1 Result:Storage
- Run:10year Element:Reservoir-1 Result:Pool Elevation
- Run:10year Element:Reservoir-1 Result:Outflow
- Run:10year Element:Reservoir-1 Result:Combined Inflow
- Run:10year Element:Reservoir-1 Result:Tailwater Stage

Computed Results
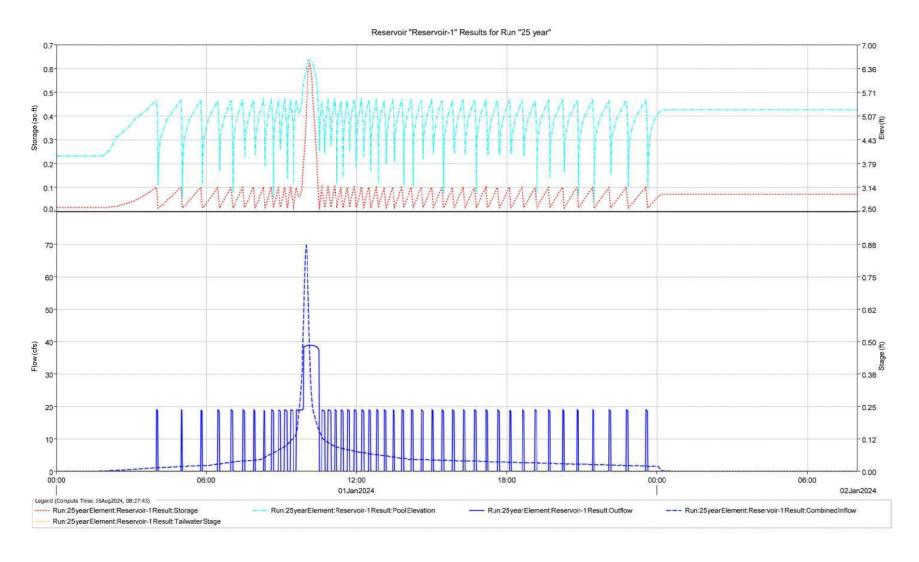
Peak Inflow:	56.7 (CFS)	Date/Time of Peak Inflow:	01Jan2024, 09:59
Peak Discharge:	38.7 (CFS)	Date/Time of Peak Discharge:	01Jan2024, 10:05
Inflow Volume:	2.86 (IN)	Peak Storage:	0.4 (ACRE-FT)
Discharge Volume:	2.71 (IN)	Peak Elevation:	6.4 (FT)
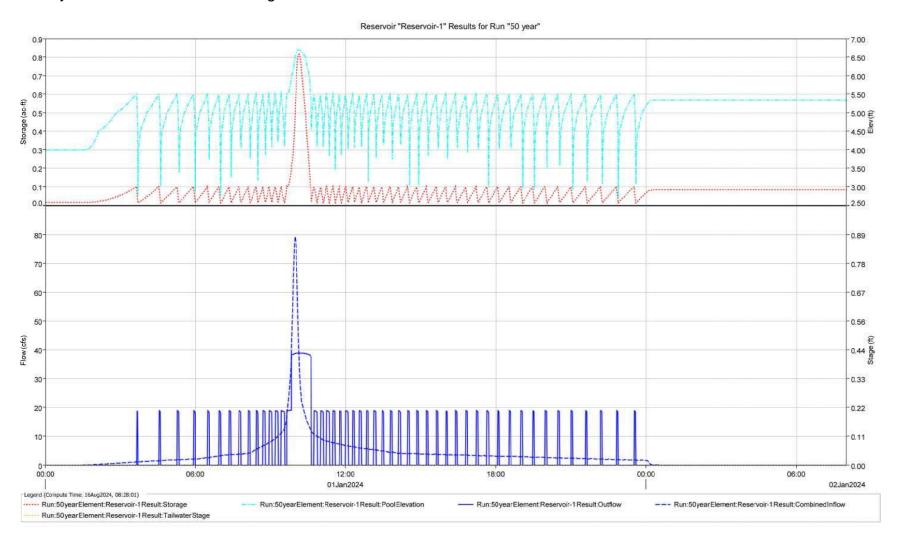
Pump Station Hydrologic Model Output
25-year Storm with Free Discharge Condition at Outfall

Reservoir "Reservoir-1" Results for Run "25 year"

Legend (Compute Time: 16Aug2024, 08:27:43)
- Run:25yearElement:Reservoir-1Result:Storage
- Run:25yearElement:Reservoir-1Result:PoolElevation
- Run:25yearElement:Reservoir-1Result:Outflow
- Run:25yearElement:Reservoir-1Result:CombinedInflow
- Run:25yearElement:Reservoir-1Result:TailwaterStage

Computed Results

Peak Inflow:	69.6 (CFS)	Date/Time of Peak Inflow:	01Jan2024, 09:59
Peak Discharge:	38.9 (CFS)	Date/Time of Peak Discharge:	01Jan2024, 10:07
Inflow Volume:	3.53 (IN)	Peak Storage:	0.6 (ACRE-FT)
Discharge Volume:	3.33 (IN)	Peak Elevation:	6.6 (FT)
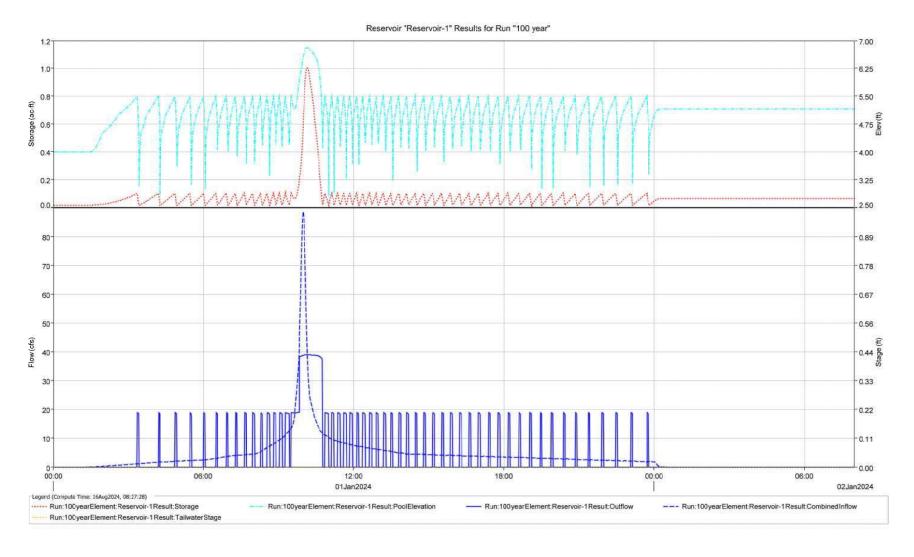
Pump Station Hydrologic Model Output
50-year Storm with Free Discharge Condition at Outfall

Reservoir "Reservoir-1" Results for Run "50 year"

Legend (Compute Time: 16Aug2024, 08:28:01)
- Run:50year Element:Reservoir-1 Result:Storage
- Run:50year Element:Reservoir-1 Result:Pool Elevation
- Run:50year Element:Reservoir-1 Result:Outflow
- Run:50year Element:Reservoir-1 Result:Combined Inflow
- Run:50year Element:Reservoir-1 Result:Tailwater Stage

Computed Results

Peak Inflow:	79.3 (CFS)	Date/Time of Peak Inflow:	01Jan2024, 09:59
Peak Discharge:	39.0 (CFS)	Date/Time of Peak Discharge:	01Jan2024, 10:08
Inflow Volume:	4.04 (IN)	Peak Storage:	0.8 (ACRE-FT)
Discharge Volume:	3.84 (IN)	Peak Elevation:	6.7 (FT)
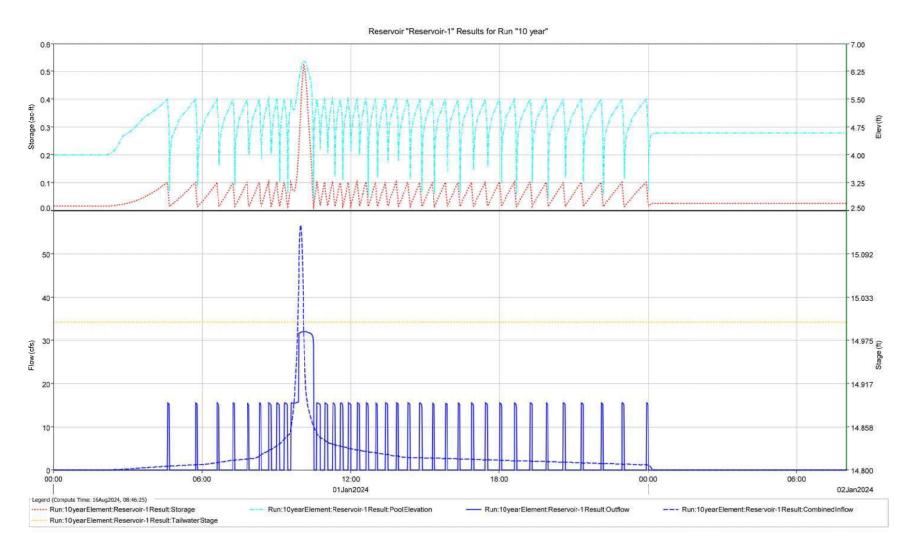
Pump Station Hydrologic Model Output
100-year Storm with Free Discharge Condition at Outfall

Reservoir "Reservoir-1" Results for Run "100 year"

Computed Results

Peak Inflow:	88.7 (CFS)	Date/Time of Peak Inflow:	01Jan2024, 09:59
Peak Discharge:	39.0 (CFS)	Date/Time of Peak Discharge:	01Jan2024, 10:09
Inflow Volume:	4.55 (IN)	Peak Storage:	1.0 (ACRE-FT)
Discharge Volume:	4.37 (IN)	Peak Elevation:	6.8 (FT)

Pump Station Hydrologic Model Output
10-year Storm with Tailwater Elevation of 15.0' at Outfall

Reservoir "Reservoir-1" Results for Run "10 year"

Legend (Compute Time: 16Aug2024, 08:46:25)
- Run:10yearElement:Reservoir-1 Result:Storage
- Run:10yearElement:Reservoir-1 Result:Pool Elevation
- Run:10yearElement:Reservoir-1 Result:Outflow
- Run:10yearElement:Reservoir-1 Result:Combined Inflow
- Run:10yearElement:Reservoir-1 Result:Tailwater Stage

Computed Results
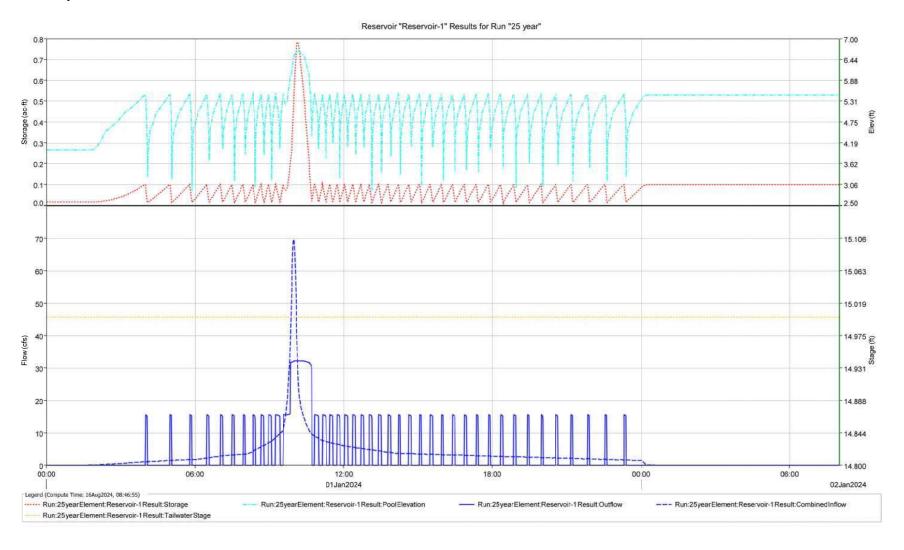
Peak Inflow:	56.7 (CFS)	Date/Time of Peak Inflow:	01Jan2024, 09:59
Peak Discharge:	32.1 (CFS)	Date/Time of Peak Discharge:	01Jan2024, 10:07
Inflow Volume:	2.86 (IN)	Peak Storage:	0.5 (ACRE-FT)
Discharge Volume:	2.69 (IN)	Peak Elevation:	6.5 (FT)
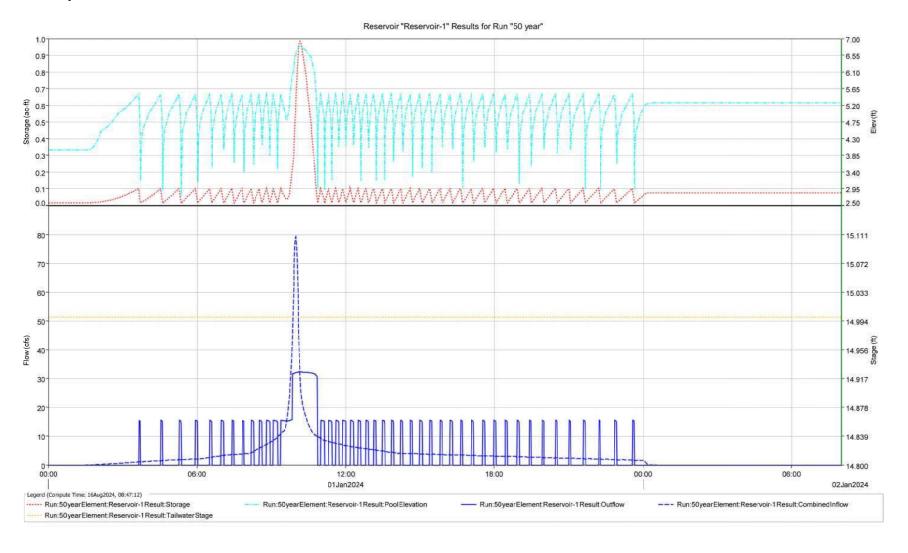
Pump Station Hydrologic Model Output
25-year Storm with Tailwater Elevation of 15.0' at Outfall

Reservoir "Reservoir-1" Results for Run "25 year"

Legend (Compute Time: 16Aug2024, 08:46:55)
····· Run:25yearElement:Reservoir-1Result:Storage   — · — Run:25yearElement:Reservoir-1Result:PoolElevation   —— Run:25yearElement:Reservoir-1Result:Outflow   – – Run:25yearElement:Reservoir-1Result:CombinedInflow
····· Run:25yearElement:Reservoir-1Result:TailwaterStage

Computed Results
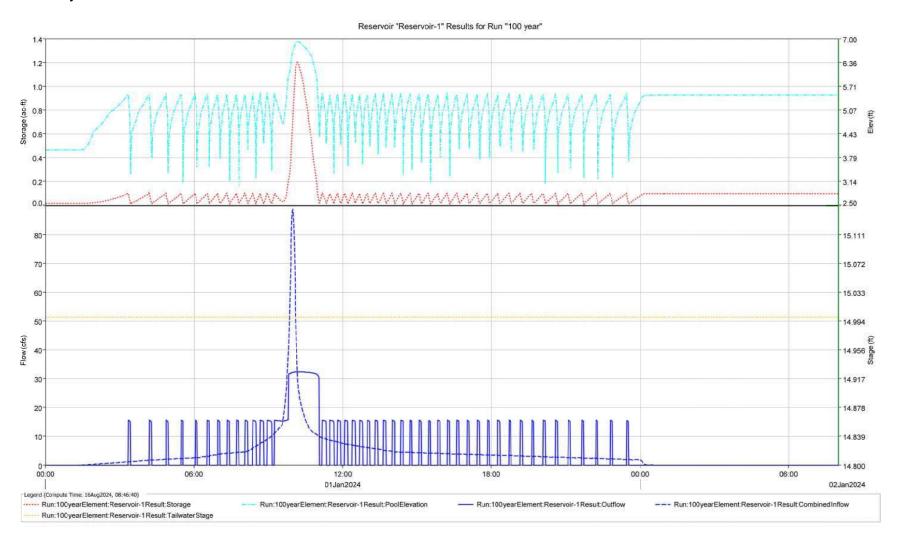
Peak Inflow:	69.6 (CFS)	Date/Time of Peak Inflow:	01Jan2024, 09:59
Peak Discharge:	32.2 (CFS)	Date/Time of Peak Discharge:	01Jan2024, 10:08
Inflow Volume:	3.53 (IN)	Peak Storage:	0.8 (ACRE-FT)
Discharge Volume:	3.40 (IN)	Peak Elevation:	6.7 (FT)

Pump Station Hydrologic Model Output
50-year Storm with Tailwater Elevation of 15.0' at Outfall

Reservoir "Reservoir-1" Results for Run "50 year"

Legend (Compute Time: 16Aug2024, 08:47:12)
- Run:50year Element:Reservoir-1 Result:Storage
- Run:50year Element:Reservoir-1 Result:Pool Elevation
- Run:50year Element:Reservoir-1 Result:Outflow
- Run:50year Element:Reservoir-1 Result:Combined Inflow
- Run:50year Element:Reservoir-1 Result:Tailwater Stage

Computed Results

Peak Inflow:	79.3 (CFS)	Date/Time of Peak Inflow:	01Jan2024, 09:59
Peak Discharge:	32.3 (CFS)	Date/Time of Peak Discharge:	01Jan2024, 10:09
Inflow Volume:	4.04 (IN)	Peak Storage:	1.0 (ACRE-FT)
Discharge Volume:	3.85 (IN)	Peak Elevation:	6.8 (FT)

Pump Station Hydrologic Model Output
100-year Storm with Tailwater Elevation of 15.0' at Outfall

Reservoir "Reservoir-1" Results for Run "100 year"

Legend (Compute Time: 16Aug2024, 08:46:40)
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- Run:100yearElement:Reservoir-1Result:PoolElevation
- Run:100yearElement:Reservoir-1Result:Outflow
- Run:100yearElement:Reservoir-1Result:CombinedInflow
- Run:100yearElement:Reservoir-1Result:TailwaterStage

Computed Results

Peak Inflow:	88.7 (CFS)	Date/Time of Peak Inflow:	01Jan2024, 09:59
Peak Discharge:	32.5 (CFS)	Date/Time of Peak Discharge:	01Jan2024, 10:10
Inflow Volume:	4.55 (IN)	Peak Storage:	1.2 (ACRE-FT)
Discharge Volume:	4.39 (IN)	Peak Elevation:	6.9 (FT)

Pump Station Hydrologic Model Output
25-year Storm with Free Discharge Condition at Outfall
5.5 ft Initial Water Surface Elevation

Reservoir "Reservoir-1" Results for Run "25 year"

Legend (Compute Time: **10/23/2024 11:26 am**
······ Run:25yearElement:Reservoir-1Result:Storage
─·─·─ Run:25yearElement:Reservoir-1Result:PoolElevation
───── Run:25yearElement:Reservoir-1Result:Outflow
─ ─ ─ Run:25yearElement:Reservoir-1Result:CombinedInflow
······ Run:25yearElement:Reservoir-1Result:TailwaterStage

Computed Results

Peak Inflow:	69.6 (CFS)	Date/Time of Peak Inflow:	01Jan2024, 09:59
Peak Discharge:	38.9 (CFS)	Date/Time of Peak Discharge:	01Jan2024, 10:07
Inflow Volume:	3.53 (IN)	Peak Storage:	0.6 (ACRE-FT)
Discharge Volume:	3.34 (IN)	Peak Elevation:	6.6 (FT)
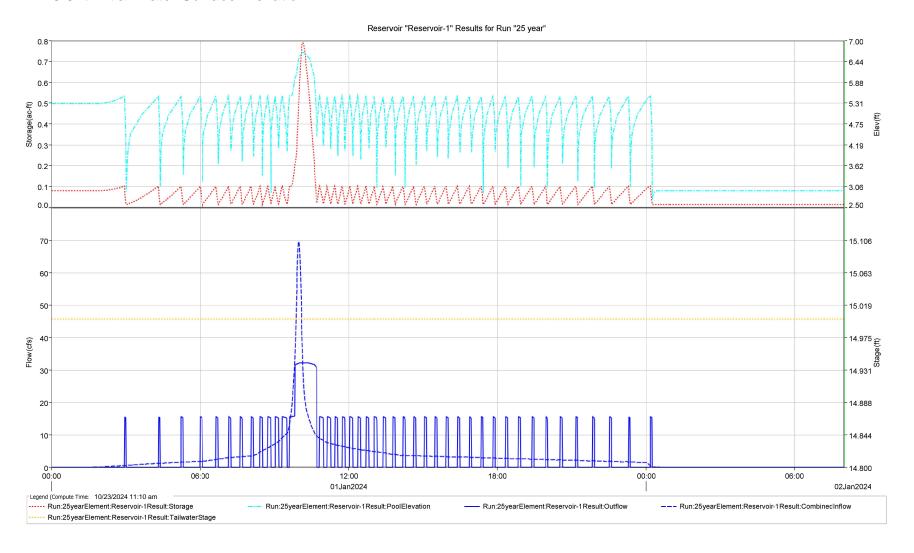
Pump Station Hydrologic Model Output
25-year Storm with Tailwater Elevation of 15.0' at Outfall
5.5 ft Initial Water Surface Elevation

Reservoir "Reservoir-1" Results for Run "25 year"

Legend (Compute Time: 10/23/2024 11:10 am)
- Run:25yearElement:Reservoir-1Result:Storage
- Run:25yearElement:Reservoir-1Result:PoolElevation
- Run:25yearElement:Reservoir-1Result:Outflow
- Run:25yearElement:Reservoir-1Result:CombinedInflow
- Run:25yearElement:Reservoir-1Result:TailwaterStage

Computed Results

Peak Inflow:	69.6 (CFS)	Date/Time of Peak Inflow:	01Jan2024, 09:59
Peak Discharge:	32.2 (CFS)	Date/Time of Peak Discharge:	01Jan2024, 10:08
Inflow Volume:	3.53 (IN)	Peak Storage:	0.8 (ACRE-FT)
Discharge Volume:	3.42 (IN)	Peak Elevation:	6.7 (FT)

NP 3301 LT 3~ 816

Patented self cleaning semi-open channel impeller, ideal for pumping in waste water applications. Modular based design with high adaptation grade.

FLYGT
a xylem brand

Technical specification

Curves according to: Water, pure Water, pure [100%], 39.2 °F, 62.42 lb/ft³, 1.6891E-5 ft²/s

79.7%

816 394mm

Curve: ISO 9906

Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances.
Please consult your local Flygt representative for performance guarantees.

Configuration

Motor number	**Installation type**
N3301.185 35-29-8AA-W 60hp	P - Semi permanent, Wet
Impeller diameter	**Discharge diameter**
394 mm	14 inch

Pump information

Material

Impeller diameter
394 mm

Impeller
Hard-Iron ™

Discharge diameter
14 inch

Inlet diameter
350 mm

Maximum operating speed
885 rpm

Number of blades
2

Max. fluid temperature
40 °C

Project	Avila Beach SD Pump Station	**Created by**	Seth Stevens
Block	0	**Created on**	8/16/2024 **Last update** 8/16/2024

Program version
74.0 - 7/1/2024 (Build 136)

Data version
8/9/2024 14:06 A8P8

User group(s)
Xylem:USA - EXT

NP 3301 LT 3~ 816

Technical specification

FLYGT
a xylem brand

Motor - General

Motor number N3301.185 35-29-8AA-W 60hp	**Phases** 3~	**Rated speed** 885 rpm	**Rated power** 60 hp
ATEX approved No	**Number of poles** 8	**Rated current** 99 A	**Stator variant** 6
Frequency 60 Hz	**Rated voltage** 380 V	**Insulation class** H	**Type of Duty** S1
Version code 185			

Motor - Technical

Power factor - 1/1 Load 0.76	**Motor efficiency - 1/1 Load** 90.5 %	**Total moment of inertia** 34.9 lb ft²	**Starts per hour max.** 30
Power factor - 3/4 Load 0.71	**Motor efficiency - 3/4 Load** 91.0 %	**Starting current, direct starting** 555 A	
Power factor - 1/2 Load 0.59	**Motor efficiency - 1/2 Load** 90.5 %	**Starting current, star-delta** 185 A	

Program version
74.0 - 7/1/2024 (Build 136)

Data version
8/9/2024 14:06 A8P8

User group(s)
Xylem:USA - EXT

NP 3301 LT 3~ 816

Performance curve

FLYGT
a **xylem** brand

Duty point

Flow	Head
458 l/s	20.2 ft

Curves according to: Water, pure Water, pure [100%],39.2 °F,62.42 lb/ft³,1.6891E-5 ft²/s

[ft] Head

79.7%

20.2 ft

816 394mm

[%] Overall Efficiency
Efficiency

74.7 %

68.1 %

816 394mm
816 394mm

[hp] Power input P1
Shaft power P2

54.4 hp

49.7 hp

816 394mm (P1)
816 394mm (P2)

[ft] NPSHR-values

816 394mm

20.7 ft

457.98 l/s

[l/s]

Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances.
Please consult your local Flygt representative for performance guarantees.

Curve: ISO 9906

Avila Beach SD Pump Station		Seth Stevens		
0		**Created on**	8/16/2024 **Last update**	8/16/2024

Program version
74.0 - 7/1/2024 (Build 136)

Data version
8/9/2024 14:06 A8P8

User group(s)
Xylem:USA - EXT

NP 3301 LT 3~ 816

Duty Analysis

FLYGT
a **xylem** brand

Curves according to: Water, pure [100%] ; 39.2°F; 62.42lb/ft³; 1.6891E-5ft²/s

Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances.
Please consult your local Flygt representative for performance guarantees.

Operating characteristics

Pumps / Systems	Flow	Head	Shaft power	Flow	Head	Shaft power	Hydr.eff.	Spec. Energy	NPSHre
	l/s	ft	hp	l/s	ft	hp		kWh/US MG	ft
2 / 1	458	20.2	49.7	916	20.2	99.3	74.7 %	93.2	20.7
1 / 1	526	14.7	47.4	526	14.7	47.4	65.4 %	77.3	22.7

Project		Created by	Seth Stevens		
Block	Avila Beach SD Pump Station	Created on	8/16/2024	Last update	8/16/2024

Program version
74.0 - 7/1/2024 (Build 136)

Data version
8/9/2024 14:06 A8P8

User group(s)
Xylem:USA - EXT

NP 3301 LT 3~ 816

VFD Curve

Curves according to: Water, pure , 39.2 °F , 62.42 lb/ft³ , 1.6891E-5 ft²/s

FLYGT
a xylem brand

Head [ft]

816 394mm [Pump 1]   816 394mm [Pump 1+2]

79.7%

55 Hz 50 Hz 45 Hz 40 Hz

Overall Efficiency [%]
Efficiency

816 394mm [Pump 1]   816 394mm [Pump 1+2]

Power input P1 [hp]
Shaft power P2

816 394mm [Pump 1+2] (P1)
816 394mm [Pump 1+2] (P2)
816 394mm [Pump 1] (P1)
816 394mm [Pump 1] (P2)

NPSHR-values [ft]

816 394mm [Pump 1]   816 394mm [Pump 1+2]

[l/s]

Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances.
Please consult your local Flygt representative for performance guarantees.

Curve: ISO 9906

Project	Avila Beach SD Pump Station	Created by	Seth Stevens		
Block	0	Created on	8/16/2024	Last update	8/16/2024

NP 3301 LT 3~ 816

VFD Analysis

FLYGT
a xylem brand

Curves according to: Water, pure [100%] ; 39.2°F; 62.42lb/ft³; 1.6891E-5ft²/s

Specific energy [kWh/US MG]

93.3 kWh/US MG

64.9 kWh/US MG

20.2 ft

79.7%

816 394mm [Pump 1]

816 394mm [Pump 1+2]

55 Hz   50 Hz   45 Hz   40 Hz

463.03 l/s
41.4 Hz

915.95 l/s

Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances.
Please consult your local Flygt representative for performance guarantees.

Operating Characteristics

Pumps / Systems	Frequency	Flow	Head	Shaft power	Flow	Head	Shaft power	Hydr.eff.	Specific energy	NPSHre
		l/s	ft	hp	l/s	ft	hp		kWh/US MG	ft
2 / 1	60 Hz	458	20.2	49.7	916	20.2	99.3	74.7 %	93.2	20.7
2 / 1	55 Hz	402	18.3	38.6	804	18.3	77.2	76.5 %	82.6	17.8
2 / 1	50 Hz	344	16.6	29.3	689	16.6	58.5	78.3 %	73.6	15.1
2 / 1	45 Hz	283	15.1	21.5	565	15.1	43	79.6 %	67	12.7

Project	Avila Beach SD Pump Station	**Created by**	Seth Stevens	
Block	0	**Created on**	8/16/2024	**Last update** 8/16/2024

NP 3301 LT 3~ 816

VFD Analysis

FLYGT
a **xylem** brand

Curves according to: Water, pure [100%] ; 39.2°F; 62.42lb/ft³; 1.6891E-5ft²/s

Specific energy
[kWh/US MG]

93.3 kWh/US MG

64.9 kWh/US MG

20.2 ft

79.7%

816 394mm [Pump 1]   816 394mm [Pump 1+2]

55 Hz   50 Hz   45 Hz   40 Hz

463.03 l/s
41.4 Hz

915.95 l/s

Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances.
Please consult your local Flygt representative for performance guarantees.

Operating Characteristics
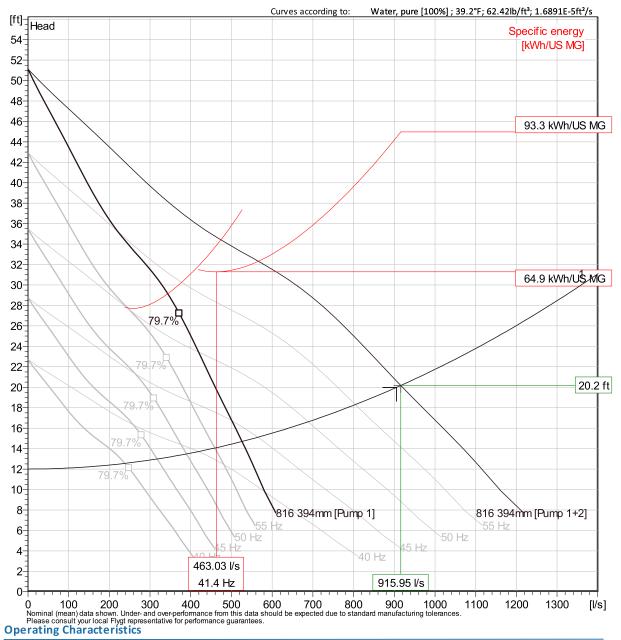
Pumps / Systems	Frequency	Flow	Head	Shaft power	Flow	Head	Shaft power	Hydr.eff.	Specific energy	NPSHre
		l/s	ft	hp	l/s	ft	hp		kWh/US MG	ft
2 / 1	40 Hz	209	13.7	15	419	13.7	30	76.8 %	65.4	10.7
1 / 1	60 Hz	526	14.7	47.4	526	14.7	47.4	65.4 %	77.3	22.7
1 / 1	55 Hz	459	14.1	37.2	459	14.1	37.2	69.5 %	69.8	18.9
1 / 1	50 Hz	389	13.5	28.6	389	13.5	28.6	73.7 %	63.7	15.6

Project	Avila Beach SD Pump Station	**Created by**	Seth Stevens	
Block	0	**Created on**	8/16/2024	**Last update**	8/16/2024

Program version
74.0 - 7/1/2024 (Build 136)

Data version
8/9/2024 14:06 A8P8

User group(s)
Xylem:USA - EXT

NP 3301 LT 3~ 816

VFD Analysis

FLYGT
a **xylem** brand

Curves according to: Water, pure [100%] ; 39.2°F; 62.42lb/ft³; 1.6891E-5ft²/s

Chart labels:
- Head [ft]
- Specific energy [kWh/US MG]
- 93.3 kWh/US MG
- 64.9 kWh/US MG
- 79.7%
- 20.2 ft
- 816 394mm [Pump 1]
- 816 394mm [Pump 1+2]
- 55 Hz, 50 Hz, 45 Hz, 40 Hz
- 463.03 l/s / 41.4 Hz
- 915.95 l/s

Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances.
Please consult your local Flygt representative for performance guarantees.

Operating Characteristics

Pumps / Systems	Frequency	Flow	Head	Shaft power	Flow	Head	Shaft power	Hydr.eff.	Specific energy	NPSHre
		l/s	ft	hp	l/s	ft	hp		kWh/US MG	ft
1 / 1	45 Hz	318	13	21.3	318	13	21.3	77.7 %	59.1	12.8
1 / 1	40 Hz	238	12.6	15.1	238	12.6	15.1	79.5 %	57.7	10.6

Project	Avila Beach SD Pump Station	**Created by**	Seth Stevens	
Block	0	**Created on**	8/16/2024	**Last update** 8/16/2024

Program version
74.0 - 7/1/2024 (Build 136)

Data version
8/9/2024 14:06 A8P8

User group(s)
Xylem:USA - EXT

NP 3301 LT 3~ 816

Dimensional drawing

FLYGT
a **xylem** brand

Ref. line

$80\frac{3}{4}$

$61\frac{13}{8}$

$45°$

$10\frac{5}{16}$

$18\frac{1}{4}$

$8\frac{5}{8}°$

$15"$

$22\frac{1}{8}$

$38\frac{9}{16}$

$5\frac{3}{8}$

$33\frac{1}{4}$

3" Guide Bars

$18\frac{3}{4}$

$6\frac{7}{8}$

Min water level*

Ø14"

$59\frac{5}{8}$

$1\frac{3}{4}$

$2\frac{5}{8}$

$2\frac{7}{8}$

$\frac{7}{8}$

$1\frac{1}{8}$

$45°$

min $7\frac{1}{8}$

$1\frac{3}{4}$

$5\frac{3}{4}$

Outlet center line

16"

$2\frac{5}{8}$

$11\frac{13}{16}$

$26\frac{9}{16}$

Ø15(4x)

VIEW Z — Z

* Only applicable for intermittent duty.
  Consult the IOM for more info.

Weight (lbs)	Pump	Discharge
with cooling jacket	2625	530
without cooling jacket	2470	530

	NP	3301	LT	8pdes	Discharge outlet Ø14"	Scale	Date
					Pump outlet Ø14"	1:40	230330
	090,095,180,185,660,670				Pump inlet	Drawing number	Revision
					Suction inlet	6844700	14

Project	Avila Beach SD Pump Station	Created by	Seth Stevens	
Block	0	Created on	8/16/2024 Last update	8/16/2024

Program version
74.0 - 7/1/2024 (Build 136)

Data version
8/9/2024 14:06 A8P8

User group(s)
Xylem:USA - EXT

Appendix E

Concept Plan and Profile Drawings

(E) OUTFALL WITH CHECK VALVE (TO BE REMOVED) IE = 1.79' (36")

(E) SDMH
RIM = 11.71'
IE = 2.26' (36")
IE NE = 6.68' (18")

PARKING LOT OWNED BY PORT SAN LUIS HARBOR DISTRICT

(N) PAD FOR BACKUP GENERATOR AND CONTROL PANEL

(N) DISCHARGE STRUCTURE WITH FLAP GATE ON (E) 36" PIPE
RIM = 15.0' ±
IE = 2.5' ±

(E) 36" GRAVITY PIPE

RELOCATED 36" SD INLET WITH TRASH RACK

(N) PUMP STATION WITH INTAKE FROM SWALE

(N) 30" FORCE MAIN

RELOCATE EXISTING CaTV CONDUIT

RELOCATED EXISTING 36" SD

(E) INLET WITH TRASH RACK
IE = 3.24' (36")

(E) 24" OUTFALL
IE = 4.01'

(E) 24" STORM DRAIN

12' UNDERGROUND DRAINAGE EASEMENT TO SLO COUNTY

RELOCATE EXISTING PHONE CONDUIT

(E) 24" INLET
IE = 4.73'

(E) DI GRATE = 7.32'
36" NE IE = 2.56'
36" SE IE = 2.43'
(EXISTING GRATE TO BE REMOVED)

FIRST STREET

AVILA BEACH COMMUNITY PARK

(E) SIDEWALK UNDERDRAIN GUTTER FL = 5.76'

AVILA BEACH DRIVE

SAN JUAN STREET

SAN FRANCISCO STREET

N

NOTES:
1. ALL ELEVATIONS ARE IN NAVD88 (FT).
2. PARCEL BOUNDARIES PROVIDED BY PARCEL QUEST FOR SLO COUNTY.
3. EXISTING STORM DRAIN ELEVATIONS ARE FROM 2024 MBS SURVEY.
4. UTILITY LOCATIONS APPROXIMATED FROM SLO COUNTY RECORD DRAWINGS AND 2024 MBS SURVEY. THESE LOCATIONS ARE TO BE POTHOLED FOR AT 60% DESIGN.

0   20'   40'   80'

GHD

COUNTY OF SAN LUIS OBISPO
AVILA BEACH FIRST STREET DRAINAGE

Project No. 12596287
Date 8/15/2024

ALTERNATIVE 3 - PARALLEL PRESSURE SYSTEM CONCEPTUAL PLAN

FIGURE E1

Data Source

PROPOSED FORCE MAIN PROFILE

SCALE H: 1" = 40', V: 1" = 5'

Labels in figure:
- (N) DISCHARGE STRUCTURE
- (E) TWO 4" PVC SS FORCE MAINS (ESTIMATED LOCATION)
- (E) LOPEZ WATERLINE (ASSUMED LOCATION WITH 3FT COVER)
- EXISTING GROUND
- (E) 18" STORM DRAIN IN MANHOLE
- (E) 8" W PVC (ASSUMED LOCATION)
- (E) PUMP STATION
- (N) PUMP STATION
- (N) BACKUP GENERATOR PAD
- (E) 6" W ACP (ASSUMED LOCATION WITH 3FT COVER)
- (E) PG&E (ASSUMED SIZE / LOCATION)
- (E) 24" STORM DRAIN
- (E) TRASH RACK
- SAN LUIS OBISPO CREEK
- (N) FLAP GATE
- (E) 36" STORM DRAIN
- (E) OUTFALL WITH CHECK VALVE (TO BE REMOVED)
- (N) 30" FORCE MAIN
- (E) TV CONDUIT (ASSUMED SIZE / LOCATION)
- (E) PHONE CONDUIT (ASSUMED SIZE / LOCATION)
- Elevation (vertical axes, -15 to 25)
- STATION (horizontal axis, 0+00 to 5+75)

NOTES:
1. ALL ELEVATIONS ARE IN NAVD88 (FT).
2. EXISTING GROUND SURFACE GENERATED FROM 2024 MBS SURVEY.
3. EXISTING STORM DRAIN ELEVATIONS ARE FROM 2024 MBS SURVEY, UNLESS NOTED OTHERWISE.
4. UTILITY LOCATIONS APPROXIMATED FROM SLO COUNTY RECORD DRAWINGS AND 2024 MBS SURVEY. THESE LOCATIONS ARE TO BE POTHOLED FOR AT 60% DESIGN.

Scale bar: 0 20' 40' 80'

GHD

COUNTY OF SAN LUIS OBISPO
AVILA BEACH FIRST STREET DRAINAGE

Project No. **12596287**
Date **8/15/2024**

ALTERNATIVE 3 -
PARALLEL PRESSURE SYSTEM
CONCEPTUAL PROFILE

FIGURE E2

Data Source

(E) OUTFALL WITH CHECK VALVE (TO BE REMOVED) IE = 1.79' (36")

(N) 24" FORCE MAIN

(N) PAD FOR BACKUP GENERATOR AND CONTROL PANEL

(N) PUMP STATION

(E) SDMH RIM = 11.71' IE = 2.26' (36") IE NE = 6.68' (18")

PARKING LOT OWNED BY PORT SAN LUIS HARBOR DISTRICT

(N) DISCHARGE STRUCTURE WITH FLAP GATE ON (E) 36" PIPE RIM = 15.0' ± IE = 2.5' ±

(E) 36" GRAVITY PIPE

(N) BYPASS CHANNEL IE = 1.93'

(E) 24" OUTFALL IE = 4.01'

(E) 24" STORM DRAIN

(N) OVERFLOW WEIR IE = 4.25'

(E) INLET WITH TRASH RACK IE = 3.24' (36")

12' UNDERGROUND DRAINAGE EASEMENT TO SLO COUNTY

(E) 24" INLET IE = 4.73'

(E) DI GRATE = 7.32' 36" NE IE = 2.56' 36" SE IE = 2.43'

AVILA BEACH COMMUNITY PARK

SAN JUAN STREET

FIRST STREET

(E) SIDEWALK UNDERDRAIN GUTTER FL = 5.76'

AVILA BEACH DRIVE

N

NOTES:
1. ALL ELEVATIONS ARE IN NAVD88 (FT).
2. PARCEL BOUNDARIES PROVIDED BY PARCEL QUEST FOR SLO COUNTY.
3. EXISTING STORM DRAIN ELEVATIONS ARE FROM 2024 MBS SURVEY.
4. UTILITY LOCATIONS APPROXIMATED FROM SLO COUNTY RECORD DRAWINGS AND 2024 MBS SURVEY. THESE LOCATIONS ARE TO BE POTHOLED FOR AT 60% DESIGN.

0   20'   40'   80'

GHD

COUNTY OF SAN LUIS OBISPO
AVILA BEACH FIRST STREET DRAINAGE

Project No. **12596287**
Date **8/15/2024**

ALTERNATIVE 4 -
PUMP ON EXISTING SYSTEM
CONCEPTUAL PLAN

FIGURE E3

Data Source

ALTERED GRAVITY LINE PROFILE

- 25
- 20
- 15
- 10
- 5
- 0
- -5
- -10
- -15

Elevation

(E) TWO 4" PVC SS FORCE MAINS (ESTIMATED LOCATION)

(E) LOPEZ WATERLINE (ASSUMED LOCATION WITH 3FT COVER)

EXISTING GROUND

(E) 18" STORM DRAIN IN MANHOLE

(E) 6" W ACP (ASSUMED LOCATION WITH 3FT COVER)

(E) 8" W PVC (ASSUMED LOCATION)

(E) PG&E (ASSUMED SIZE / LOCATION)

(E) 24" STORM DRAIN

(E) TRASH RACK

SAN LUIS OBISPO CREEK

(N) 30" FORCE MAIN

(N) OVERFLOW WEIR EL = 4.25'

(N) BYPASS CHANNEL

(E) 36" STORM DRAIN

(E) OUTFALL WITH CHECK VALVE (TO BE REMOVED)

(N) FLAP GATE

(N) DISCHARGE STRUCTURE

(E) PHONE CONDUIT (ASSUMED SIZE / LOCATION)

(E) TV CONDUIT (ASSUMED SIZE / LOCATION)

STATION: 0+00, 0+25, 0+50, 0+75, 1+00, 1+25, 1+50, 1+75, 2+00, 2+25, 2+50, 2+75, 3+00, 3+25, 3+50, 3+75, 4+00, 4+25, 4+50, 4+75

① ALTERED GRAVITY LINE PROFILE
SCALE H: 1" = 40', V: 1" = 5'

PROPOSED FORCE MAIN PROFILE

(N) DISCHARGE STRUCTURE

(N) 24" FORCE MAIN

(N) OVERFLOW WEIR

(N) PUMP STATION

(N) FLAP GATE

(N) BYPASS CHANNEL

STATION: 0+00, 0+25, 0+50, 0+75, 1+00

② PROPOSED FORCE MAIN PROFILE
SCALE H: 1" = 40', V: 1" = 5'

NOTES:
1. ALL ELEVATIONS ARE IN NAVD88 (FT).
2. EXISTING GROUND SURFACE GENERATED FROM 2024 MBS SURVEY.
3. EXISTING STORM DRAIN ELEVATIONS ARE FROM 2024 MBS SURVEY, UNLESS NOTED OTHERWISE.
4. UTILITY LOCATIONS APPROXIMATED FROM SLO COUNTY RECORD DRAWINGS AND 2024 MBS SURVEY. THESE LOCATIONS ARE TO BE POTHOLED FOR AT 60% DESIGN.

Scale: 0   20'   40'   80'

GHD

FIGURE E5

(E) 36" GRAVITY PIPE

(E) 24" OUTFALL
IE = 4.01'

PARKING LOT OWNED BY PORT SAN LUIS HARBOR DISTRICT

(N) PAD FOR BACKUP GENERATOR AND CONTROL PANEL

(N) PUMP STATION WITH INTAKE FROM SWALE

RELOCATED 36" SD INLET WITH TRASH RACK

RELOCATED EXISTING 36" SD

(E) 24" STORM DRAIN

12' UNDERGROUND DRAINAGE EASEMENT TO SLO COUNTY

(E) INLET WITH TRASH RACK
IE = 3.24' (36")

AVILA BEACH DRIVE

(E) DI GRATE = 7.32'
36" NE IE = 2.56'
36" SE IE = 2.43'
(EXISTING INLET TO BE REMOVED)

FIRST STREET

AVILA BEACH COMMUNITY PARK

(N) 30" FORCE MAIN

2" SW TO BE RELOCATED

SAN JUAN STREET

SAN FRANCISCO STREET

(E) 24" DIA. DRAINAGE PIPE OUTLET INV. 8.12'
(FROM RECORD DRAWING)

(E) OUTFALL STRUCTURE (TO BE MODIFIED)

6.74' MIN

6' FROM CENTERLINE

4" SS

12" W

2" SW

(N) OUTLET TO EXISTING OUTFALL STRUCTURE

N

NOTES:
1. ALL ELEVATIONS ARE IN NAVD88 (FT).
2. PARCEL BOUNDARIES PROVIDED BY PARCEL QUEST FOR SLO COUNTY.
3. EXISTING STORM DRAIN ELEVATIONS ARE FROM 2024 MBS SURVEY, UNLESS NOTED OTHERWISE.
4. UTILITY LOCATIONS APPROXIMATED FROM SLO COUNTY RECORD DRAWINGS AND 2024 MBS SURVEY. THESE LOCATIONS ARE TO BE POTHOLED FOR AT 60% DESIGN.

0   50'   100'

GHD

COUNTY OF SAN LUIS OBISPO
AVILA BEACH FIRST STREET DRAINAGE

Project No. 12596287
Date 8/15/2024

ALTERNATIVE 5A -
FRONT STREET OUTLET VIA SAN JUAN
STREET CONCEPTUAL PLAN

FIGURE E5

Filename: \\ghdnet\ghd\US\San Luis Obispo\Projects\561\12596287\Digital_Design\ACAD\Figures\12596287-FIG-Alternative 5-Pump Location 1.dwg
Plot Date: 15 August 2024 4:12 PM

Data Source

PROPOSED FORCE MAIN PROFILE
SCALE H: 1" = 60', V: 1" = 7.5'

NOTES:
1. ALL ELEVATIONS ARE IN NAVD88 (FT).
2. EXISTING GROUND SURFACE GENERATED FROM 2024 MBS SURVEY UNLESS NOTED OTHERWISE.
3. EXISTING GROUND SURFACE OUTSIDE OF MBS SURVEY EXTENTS GENERATED FROM FEMA 2018 LIDAR DATA. THIS DATA HAS NOT BEEN VERIFIED.
4. EXISTING STORM DRAIN ELEVATIONS ARE FROM 2024 MBS SURVEY, UNLESS NOTED OTHERWISE.
5. UTILITY LOCATIONS APPROXIMATED FROM SLO COUNTY RECORD DRAWINGS AND 2024 MBS SURVEY. THESE LOCATIONS ARE TO BE POTHOLED FOR AT 60% DESIGN.

COUNTY OF SAN LUIS OBISPO
AVILA BEACH FIRST STREET DRAINAGE

Project No. **12596287**
Date **8/15/2024**

GHD

ALTERNATIVE 5A -
FRONT STREET OUTLET VIA SAN JUAN STREET CONCEPTUAL PROFILE

FIGURE E6

Filename: \\ghdnet\ghd\US\San Luis Obispo\Projects\561\12596287\Digital_Design\ACAD\Figures\12596287-FIG-Alternative 5-Pump Location 1.dwg
Plot Date: 15 August 2024 4:12 PM

Data Source

(E) 36" GRAVITY PIPE

(N) 36" GRAVITY PIPE

(N) PAD FOR BACKUP GENERATOR AND CONTROL PANEL

PARKING LOT OWNED BY PORT SAN LUIS HARBOR DISTRICT

(E) SDMH RIM = 11.71'
IE = 2.26' (E 36")
IE NE = 6.68' (E 18")
IE N = 3.06' (N 36")

(N) PUMP STATION

(E) 24" OUTFALL
IE = 4.01'

(E) 24" STORM DRAIN

AVILA BEACH DRIVE

(E) DI
GRATE = 10.72'
IE = 7.18' (18")

(E) INLET WITH TRASH RACK
IE = 3.24' (36")

12' UNDERGROUND DRAINAGE EASEMENT TO SLO COUNTY

(E) DI
GRATE = 7.32'
36" NE IE = 2.56'
36" SE IE = 2.43'

FIRST STREET

(N) 30" FORCE MAIN

AVILA BEACH COMMUNITY PARK

2" SW TO BE RELOCATED

SAN JUAN STREET

SAN FRANCISCO STREET

(E) 24" DIA. DRAINAGE PIPE OUTLET INV. 8.12' (FROM RECORD DRAWING)

(E) OUTFALL STRUCTURE (TO BE MODIFIED)

6.74' MIN

4" SS

12" W

6' FROM CENTERLINE

(N) OUTLET TO EXISTING OUTFALL STRUCTURE

2" SW

N

NOTES:
1. ALL ELEVATIONS ARE IN NAVD88 (FT).
2. PARCEL BOUNDARIES PROVIDED BY PARCEL QUEST FOR SLO COUNTY.
3. STORM DRAIN ELEVATIONS ARE FROM 2024 MBS SURVEY, UNLESS NOTED OTHERWISE.
4. UTILITY LOCATIONS APPROXIMATED FROM SLO COUNTY RECORD DRAWINGS AND 2024 MBS SURVEY. THESE LOCATIONS ARE TO BE POTHOLED FOR AT 60% DESIGN.

0   50'   100'

GHD

COUNTY OF SAN LUIS OBISPO
AVILA BEACH FIRST STREET DRAINAGE

Project No. **12596287**
Date **8/15/2024**

ALTERNATIVE 5B -
FRONT STREET OUTLET VIA SAN JUAN STREET CONCEPTUAL PLAN

FIGURE E7

Data Source

PROPOSED FORCE MAIN PROFILE
SCALE H: 1" = 60', V: 1" = 7.5'

NOTES:
1. ALL ELEVATIONS ARE IN NAVD88 (FT).
2. EXISTING GROUND SURFACE GENERATED FROM 2024 MBS SURVEY UNLESS NOTED OTHERWISE.
3. EXISTING GROUND SURFACE OUTSIDE OF MBS SURVEY EXTENTS GENERATED FROM FEMA 2018 LIDAR DATA. THIS DATA HAS NOT BEEN VERIFIED.
4. EXISTING STORM DRAIN ELEVATIONS ARE FROM 2024 MBS SURVEY, UNLESS NOTED OTHERWISE.
5. UTILITY LOCATIONS APPROXIMATED FROM SLO COUNTY RECORD DRAWINGS AND 2024 MBS SURVEY. THESE LOCATIONS ARE TO BE POTHOLED FOR AT 60% DESIGN.

COUNTY OF SAN LUIS OBISPO
AVILA BEACH FIRST STREET DRAINAGE

Project No. **12596287**
Date **8/15/2024**

ALTERNATIVE 5B -
FRONT STREET OUTLET VIA SAN JUAN
STREET CONCEPTUAL PROFILE

FIGURE E8

Filename: \\ghdnet\ghd\US\San Luis Obispo\Projects\561\12596287\Digital_Design\ACAD\Figures\12596287-FIG-Alternative 5-Pump Location 2.dwg
Plot Date: 15 August 2024 4:14 PM

Data Source

(E) 36" GRAVITY PIPE

(E) INLET WITH TRASH RACK
IE = 3.24' (36")

(E) 24" OUTFALL
IE = 4.01'

(N) PAD FOR BACKUP GENERATOR AND CONTROL PANEL

(N) PUMP STATION WITH INTAKE FROM SWALE

(E) 24" STORM DRAIN

FIRST STREET

FRONT STREET

(N) BEACH OUTFALL
APPROX IE = 13.0'

(E) SIDEWALK UNDERDRAIN
GUTTER FL = 5.76'

(E) 24" INLET
IE = 4.73'

SAN FRANCISCO STREET

(N) 30" FORCE MAIN

NOTES:
1. ALL ELEVATIONS ARE IN NAVD88 (FT).
2. PARCEL BOUNDARIES PROVIDED BY PARCEL QUEST FOR SLO COUNTY.
3. EXISTING STORM DRAIN ELEVATIONS ARE FROM 2024 MBS SURVEY.
4. UTILITY LOCATIONS APPROXIMATED FROM SLO COUNTY RECORD DRAWINGS AND 2024 MBS SURVEY. THESE LOCATIONS ARE TO BE POTHOLED FOR AT 60% DESIGN.

0   20'   40'   80'

COUNTY OF SAN LUIS OBISPO
AVILA BEACH FIRST STREET DRAINAGE

Project No. **12596287**
Date **8/15/2024**

GHD

ALTERNATIVE 6A - FRONT STREET OUTLET VIA SAN FRANCISCO STREET (NEW OUTLET) CONCEPTUAL PLAN

FIGURE E9

Filename: \\ghdnet\ghd\US\San Luis Obispo\Projects\561\12596287\Digital_Design\ACAD\Figures\12596287-FIG-Alternative 6-Pump Location 1.dwg
Plot Date: 15 August 2024 4:15 PM

Data Source

PROPOSED FORCE MAIN PROFILE - PUMP AT PIPE INTAKE
SCALE H: 1" = 50', V: 1" = 6.25'

NOTES:
1. ALL ELEVATIONS ARE IN NAVD88 (FT).
2. EXISTING GROUND SURFACE GENERATED FROM 2024 MBS SURVEY UNLESS NOTED OTHERWISE.
3. EXISTING GROUND SURFACE OUTSIDE OF MBS SURVEY EXTENTS GENERATED FROM FEMA 2018 LIDAR DATA. THIS DATA HAS NOT BEEN VERIFIED.
4. EXISTING STORM DRAIN ELEVATIONS ARE FROM 2024 MBS SURVEY, UNLESS NOTED OTHERWISE.
5. UTILITY LOCATIONS APPROXIMATED FROM SLO COUNTY RECORD DRAWINGS AND 2024 MBS SURVEY. THESE LOCATIONS ARE TO BE POTHOLED FOR AT 60% DESIGN.

COUNTY OF SAN LUIS OBISPO
AVILA BEACH FIRST STREET DRAINAGE
Project No. **12596287**
Date **8/15/2024**

ALTERNATIVE 6A - FRONT STREET
OUTLET VIA SAN FRANCISCO STREET
(NEW OUTLET) CONCEPTUAL PROFILE

FIGURE E10

Filename: \\ghdnet\ghd\US\San Luis Obispo\Projects\561\12596287\Digital_Design\ACAD\Figures\12596287-FIG-Alternative 6-Pump Location 1.dwg
Plot Date: 15 August 2024 4:15 PM

Data Source

(E) 36" STORM DRAIN

(E) INLET WITH
TRASH RACK
IE = 3.24' (36")

(E) 24" OUTFALL
IE = 4.01'

(N) INTAKE STRUCTURE WITH
RECTANGULAR WEIRS
N IE = 4.7' (12' X 3' WEIR)
W IE = 5.0' (4' X 2.7' WEIR)

(N) 42" GRAVITY PIPE

(E) 24" STORM DRAIN

FIRST STREET

FRONT STREET

(E) 24" INLET
IE = 4.73'

(N) BEACH OUTFALL
APPROX IE=13.0'

(N) 30" FORCE MAIN

SAN FRANCISCO STREET

(N) PUMP STATION

(N) PAD FOR
BACKUP
GENERATOR AND
CONTROL PANEL

0+00   1+00   2+00   3+00   4+00   5+00   6+00   7+00   7+70

NOTES:
1. ALL ELEVATIONS ARE IN NAVD88 (FT).
2. PARCEL BOUNDARIES PROVIDED BY PARCEL QUEST FOR SLO COUNTY.
3. EXISTING STORM DRAIN ELEVATIONS ARE FROM 2024 MBS SURVEY.
4. UTILITY LOCATIONS APPROXIMATED FROM SLO COUNTY RECORD DRAWINGS AND 2024 MBS SURVEY. THESE LOCATIONS ARE TO BE POTHOLED FOR AT 60% DESIGN.

0   20'   40'   80'

GHD

COUNTY OF SAN LUIS OBISPO
AVILA BEACH FIRST STREET DRAINAGE

Project No. **12596287**
Date **8/15/2024**

ALTERNATIVE 6B - FRONT STREET
OUTLET VIA SAN FRANCISCO STREET
(NEW OUTLET) CONCEPTUAL PLAN

FIGURE E11

Filename: \\ghdnet\ghd\US\San Luis Obispo\Projects\561\12596287\Digital_Design\ACAD\Figures\12596287-FIG-Alternative 6 - Pump Location 2.dwg
Plot Date: 15 August 2024 4:16 PM

Data Source

PROPOSED FORCE MAIN PROFILE - PUMP IN PARKING LOT
SCALE H: 1" = 50', V: 1" = 7.5'

NOTES:
1. ALL ELEVATIONS ARE IN NAVD88 (FT).
2. EXISTING GROUND SURFACE GENERATED FROM 2024 MBS SURVEY UNLESS NOTED OTHERWISE.
3. EXISTING GROUND SURFACE OUTSIDE OF MBS SURVEY EXTENTS GENERATED FROM FEMA 2018 LIDAR DATA. THIS DATA HAS NOT BEEN VERIFIED.
4. EXISTING STORM DRAIN ELEVATIONS ARE FROM 2024 MBS SURVEY, UNLESS NOTED OTHERWISE.
5. UTILITY LOCATIONS APPROXIMATED FROM SLO COUNTY RECORD DRAWINGS AND 2024 MBS SURVEY. THESE LOCATIONS ARE TO BE POTHOLED FOR AT 60% DESIGN.

Filename: \\ghdnet\ghd\US\San Luis Obispo\Projects\561\12596287\Digital_Design\ACAD\Figures\12596287-FIG-Alternative 6 - Pump Location 2.dwg
Plot Date: 04 December 2024 8:32 AM

Data Source

Appendix F

FlowMaster Results

F-1   Alternatives 3, 5A, and 6A Intake Weir Calculations

Worksheet for Swale Intake Weir-5.5'

Project Description

Solve For	Discharge

Input Data

Headwater Elevation	5.50 ft
Crest Elevation	5.00 ft
Tailwater Elevation	0.00 ft*
Weir Coefficient	2.65 ft^(1/2)/s
Crest Length	22.5 ft
Number Of Contractions	0

Results

Discharge	21.08 cfs
Headwater Height Above Crest	0.50 ft
Tailwater Height Above Crest	-5.00 ft*
Flow Area	11.3 ft²
Velocity	1.87 ft/s
Wetted Perimeter	23.5 ft
Top Width	22.50 ft

*assume tailwater does not restrict weir flow

12596287-CALC-Hydraulics_MH.fm8
8/13/2024

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FlowMaster
[10.03.00.03]
Page 1 of 1

Worksheet for Swale Intake Weir-6.0'

Project Description

Solve For	Discharge

Input Data

Headwater Elevation	6.00 ft
Crest Elevation	5.00 ft
Tailwater Elevation	0.00 ft*
Weir Coefficient	2.65 ft^(1/2)/s
Crest Length	22.5 ft
Number Of Contractions	0

Results

Discharge	59.63 cfs
Headwater Height Above Crest	1.00 ft
Tailwater Height Above Crest	-5.00 ft*
Flow Area	22.5 ft²
Velocity	2.65 ft/s
Wetted Perimeter	24.5 ft
Top Width	22.50 ft

*assume tailwater does not restrict weir flow

12596287-CALC-Hydraulics_MH.fm8
8/13/2024

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FlowMaster
[10.03.00.03]
Page 1 of 1

F-2 Alternative 4 Side Weir Calculations

Worksheet for Alternative 4 Weir

Project Description

Solve For	Discharge

Input Data

Headwater Elevation	4.95 ft
Crest Elevation	4.24 ft
Tailwater Elevation	0.00 ft*
Weir Coefficient	2.65 ft^(1/2)/s
Crest Length	22.5 ft
Number Of Contractions	0

Results

Discharge	35.67 cfs
Headwater Height Above Crest	0.71 ft
Tailwater Height Above Crest	-4.24 ft*
Flow Area	16.0 ft²
Velocity	2.23 ft/s
Wetted Perimeter	23.9 ft
Top Width	22.50 ft

*assume tailwater does not restrict weir flow

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[10.03.00.03]
Page 1 of 1

F-3 Alternative 6B Gravity Storm Drain and Intake Weir Calculations

Worksheet for Alternative 6B Proposed Gravity SD

Project Description

Friction Method	Manning Formula
Solve For	Full Flow Capacity

Input Data

Roughness Coefficient	0.014
Channel Slope	0.002 ft/ft
Normal Depth	42.0 in
Diameter	42.0 in
Discharge	41.78 cfs

Results

Discharge	41.78 cfs
Normal Depth	42.0 in
Flow Area	9.6 ft²
Wetted Perimeter	11.0 ft
Hydraulic Radius	10.5 in
Top Width	0.00 ft
Critical Depth	24.1 in
Percent Full	100.0 %
Critical Slope	0.005 ft/ft
Velocity	4.34 ft/s
Velocity Head	0.29 ft
Specific Energy	3.79 ft
Froude Number	(N/A)
Maximum Discharge	44.94 cfs
Discharge Full	41.78 cfs
Slope Full	0.002 ft/ft
Flow Type	Undefined

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[10.03.00.03]
Page 1 of 1

Worksheet for Alternaitve 6B Weir - 5.5'

Project Description

Solve For	Discharge

Input Data

Headwater Elevation	6.00 ft
Crest Elevation	4.70 ft
Tailwater Elevation	0.00 ft*
Weir Coefficient	2.65 ft^(1/2)/s
Crest Length	12.0 ft
Number Of Contractions	0

Results

Discharge	47.13 cfs
Headwater Height Above Crest	1.30 ft
Tailwater Height Above Crest	-4.70 ft*
Flow Area	15.6 ft²
Velocity	3.02 ft/s
Wetted Perimeter	14.6 ft
Top Width	12.00 ft

*assume tailwater does not restrict weir flow

12596287-CALC-Hydraulics_MH.fm8
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[10.03.00.03]
Page 1 of 1
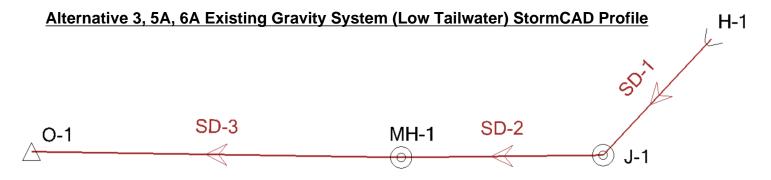
Worksheet for Alternaitve 6B Weir - 6.0'

Project Description

Solve For	Discharge

Input Data

Headwater Elevation	5.50 ft
Crest Elevation	4.70 ft
Tailwater Elevation	0.00 ft*
Weir Coefficient	2.65 ft^(1/2)/s
Crest Length	12.0 ft
Number Of Contractions	0
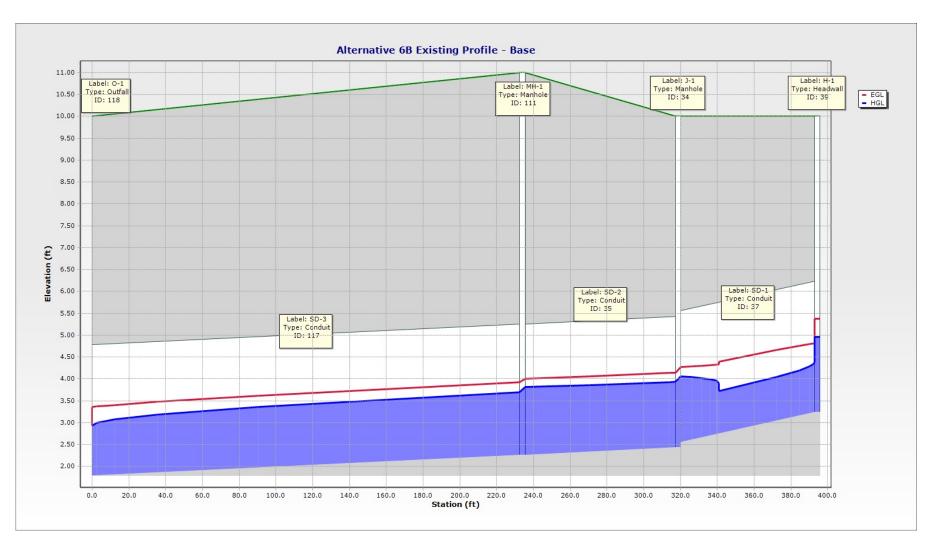
Results

Discharge	22.75 cfs
Headwater Height Above Crest	0.80 ft
Tailwater Height Above Crest	-4.70 ft*
Flow Area	9.6 ft²
Velocity	2.37 ft/s
Wetted Perimeter	13.6 ft
Top Width	12.00 ft

*assume tailwater does not restrict weir flow

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FlowMaster
[10.03.00.03]
Page 1 of 1

Appendix G

StormCAD Results

G-1   Alternatives 3, 5A, and 6A Existing Drainage System (Low Tailwater) StormCAD Results

Alternative 3, 5A, 6A Existing Gravity System (Low Tailwater) StormCAD Profile
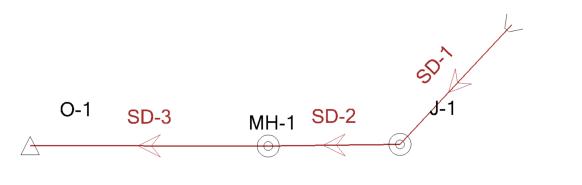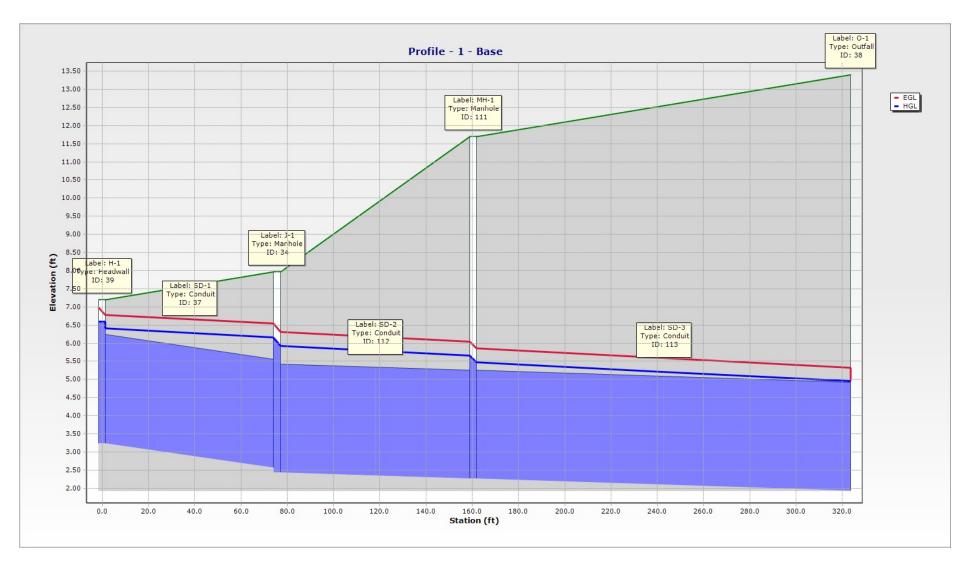
Alternative 3, 5A, and 6A Existing Drainage System (Low Tailwater) StormCAD Table

Label	Station (Calculated) (ft)	Flow (cfs)	Start Node	Stop Node	Length (Unified) (ft)	Slope (Calculated) (ft/ft)	Friction Slope (ft/ft)	Headloss (ft)
J-1	3+19							0.12
MH-1	2+34							0.12
O-1								
H-1	3+94							
SD-2		13.00	J-1	MH-1	84.8	0.002	0.002	0.13
SD-1		13.00	H-1	J-1	75.6	0.009	0.007	0.33
SD-3		13.00	MH-1	O-1	234.0	0.002	0.002	0.76

Velocity (ft/s)	Invert (Start) (ft)	Hydraulic Grade Line (In) (ft)	Invert (Stop) (ft)	Hydraulic Grade Line (Out) (ft)	Headloss Coefficient (Standard)	Diameter (in)
		4.06		3.94	0.600	
		3.81		3.69	0.500	
		4.96		4.96		
3.86	2.43	3.94	2.26	3.81		36.0
6.68	3.24	4.39	2.56	4.06		36.0
3.86	2.26	3.69	1.79	2.94		36.0

12596287-CALC-Alternative3,5A,6A_Existing_20240813.stsw
8/13/2024

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StormCAD
[10.04.00.158]
Page 1 of 1

G-2   Alternative 4 Proposed System (High Tailwater) StormCAD Results

Alternative 4 Proposed System StormCAD Profile (High Tailwater)
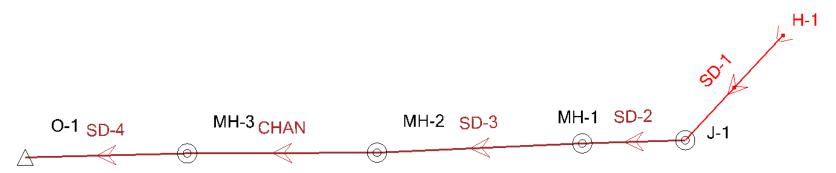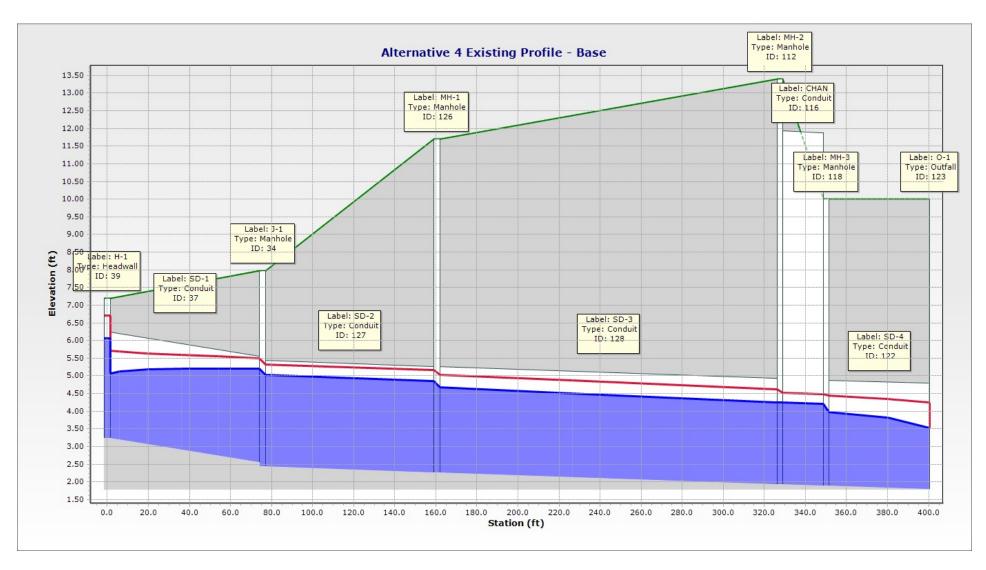
Alternative 4 Proposed Drainage System (High Tailwater) StormCAD Table

Label	Station (Calculated) (ft)	Flow (cfs)	Start Node	Stop Node	Length (Unified) (ft)	Slope (Calculated) (ft/ft)	Friction Slope (ft/ft)	Headloss (ft)
J-1	2+48							0.23
MH-1	1+63							0.19
O-1								
H-1	3+24							
SD-1		35.00	H-1	J-1	75.6	0.009	0.003	0.24
SD-2		35.00	J-1	MH-1	84.8	0.002	0.003	0.27
SD-3		35.00	MH-1	O-1	163.2	0.002	0.003	0.52

Velocity (ft/s)	Invert (Start) (ft)	Hydraulic Grade Line (In) (ft)	Invert (Stop) (ft)	Hydraulic Grade Line (Out) (ft)	Headloss Coefficient (Standard)	Diameter (in)
		6.16		5.93	0.600	
		5.66		5.47	0.500	
		6.59		6.40		
4.95	3.24	6.40	2.56	6.16		36.0
4.95	2.43	5.93	2.26	5.66		36.0
4.95	2.26	5.47	1.93	4.95		36.0

12596287-CALC-Alternative4_20240802.stsw
8/13/2024

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StormCAD
[10.04.00.158]
Page 1 of 1

G-3 Alternative 4 Existing Drainage System (Low Tailwater) StormCAD Results

Alternative 4 Existing Gravity System StormCAD Profile (Low Tailwater)
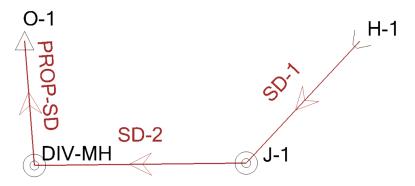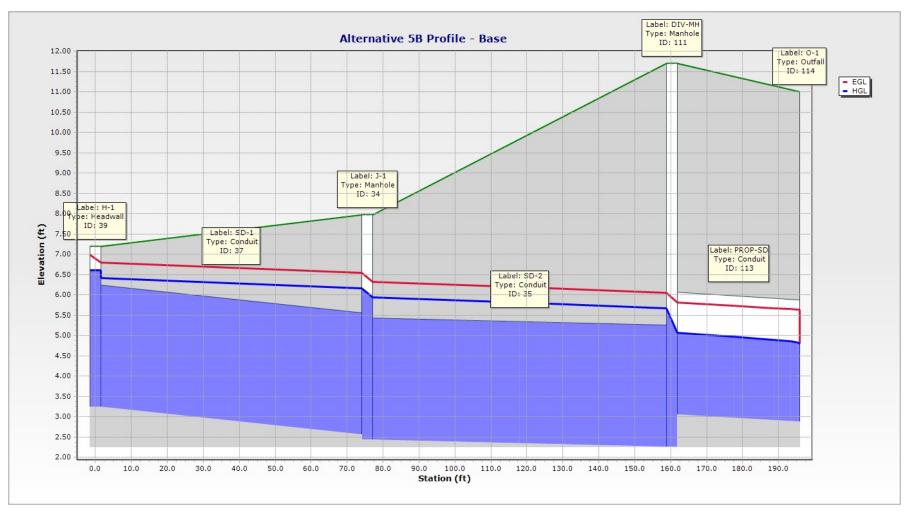
Alternative 4 Existing Drainage System (Low Tailwater) StormCAD Table

Label	Station (Calculated) (ft)	Flow (cfs)	Start Node	Stop Node	Length (Unified) (ft)	Slope (Calculated) (ft/ft)	Friction Slope (ft/ft)	Headloss (ft)
J-1	3+25							0.19
MH-2	0+73							0.00
MH-3	0+50							0.23
MH-1	2+40							0.18
O-1								
H-1	4+00							
SD-1		29.00	H-1	J-1	75.6	0.009	0.003	-0.13
CHAN		29.00	MH-2	MH-3	22.5	0.002	0.002	0.04
SD-4		29.00	MH-3	O-1	50.4	0.002	0.004	0.44
SD-2		29.00	J-1	MH-1	84.8	0.002	0.002	0.17
SD-3		29.00	MH-1	MH-2	167.2	0.002	0.002	0.42

Velocity (ft/s)	Invert (Start) (ft)	Hydraulic Grade Line (In) (ft)	Invert (Stop) (ft)	Hydraulic Grade Line (Out) (ft)	Headloss Coefficient (Standard)	Diameter (in)
		5.21		5.02	0.600	
		4.25		4.25	0.000	
		4.21		3.98	0.500	
		4.85		4.67	0.500	
		6.06		6.06		
8.28	3.24	5.08	2.56	5.21		36.0
4.58	1.93	4.25	1.88	4.21		
4.10	1.88	3.98	1.79	3.53		36.0
4.40	2.43	5.02	2.26	4.85		36.0
4.43	2.26	4.67	1.93	4.25		36.0

12596287-CALC-Alternative4_Freeoutfalltest_20240802.stsw
8/13/2024

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[10.04.00.158]
Page 1 of 1

G-4 Alternative 5B Proposed System (High Tailwater) StormCAD Results

Alternative 5B Proposed Drainage System StormCAD Profile (High Tailwater)
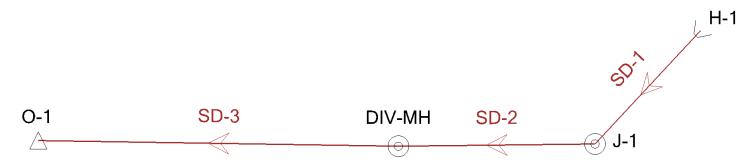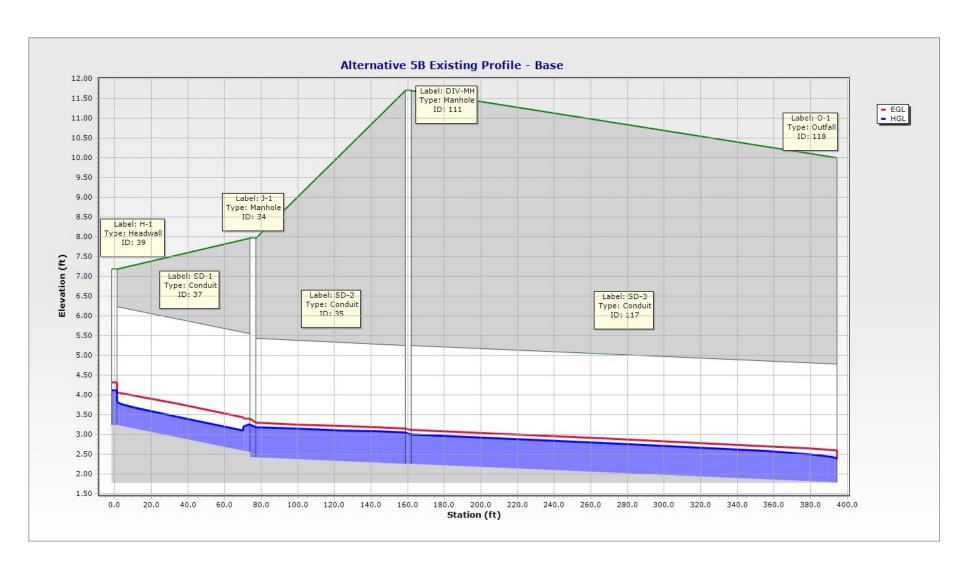
Alternative 5B Proposed Drainage System (High Tailwater) StormCAD Table

Label	Station (Calculated) (ft)	Flow (cfs)	Start Node	Stop Node	Length (Unified) (ft)	Slope (Calculated) (ft/ft)	Friction Slope (ft/ft)	Headloss (ft)	Velocity (ft/s)	Invert (Start) (ft)	Hydraulic Grade Line (In) (ft)	Invert (Stop) (ft)	Hydraulic Grade Line (Out) (ft)	Headloss Coefficient (Standard)	Diameter (in)
J-1	1+20							0.23			6.17		5.94	0.600	
DIV-MH	0+36							0.60			5.67		5.07	0.800	
O-1															
H-1	1+96										6.60		6.41		
SD-2		35.00	J-1	DIV-MH	84.8	0.002	0.003	0.27	4.95	2.43	5.94	2.26	5.67		36.0
SD-1		35.00	H-1	J-1	75.6	0.009	0.003	0.24	4.95	3.24	6.41	2.56	6.17		36.0
PROP-SD		35.00	DIV-MH	O-1	35.5	0.005	0.005	0.27	6.92	3.06	5.07	2.88	4.80		36.0

12596287-CALC-Alternative5B_20240805.stsw
8/13/2024

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[10.04.00.158]
Page 1 of 1

G-5 Alternative 5B Existing Drainage System (Low Tailwater) StormCAD Results

Alternative 5B Existing Drainage System StormCAD Profile (Low Tailwater)

Alternative 5B Existing Gravity System (Low Tailwater) StormCAD Table

Label	Station (Calculated) (ft)	Flow (cfs)	Start Node	Stop Node	Length (Unified) (ft)	Slope (Calculated) (ft/ft)	Friction Slope (ft/ft)	Headloss (ft)
J-1	3+19							0.07
DIV-MH	2+34							0.06
O-1								
H-1	3+94							
SD-2		3.75	J-1	DIV-MH	84.8	0.002	0.002	0.13
SD-1		3.75	H-1	J-1	75.6	0.009	0.009	0.59
SD-3		3.75	DIV-MH	O-1	234.0	0.002	0.002	0.61

Velocity (ft/s)	Invert (Start) (ft)	Hydraulic Grade Line (In) (ft)	Invert (Stop) (ft)	Hydraulic Grade Line (Out) (ft)	Headloss Coefficient (Standard)	Diameter (in)
		3.26		3.19	0.600	
		3.06		3.00	0.500	
		4.12		4.12		
2.74	2.43	3.19	2.26	3.06		36.0
4.66	3.24	3.84	2.56	3.26		36.0
2.74	2.26	3.00	1.79	2.39		36.0

12596287-CALC-Alternative5B_Freeoutfalltest_20240805.stsw
8/13/2024

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[10.04.00.158]
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G-6   Alternative 6B Existing Drainage System (Low Tailwater) StormCAD Results

Alternative 6B Existing Drainage System StormCAD Profile (Low Tailwater)


Alternative 6B Existing Profile - Base

Alternative 6B Existing Gravity System (Low Tailwater) StormCAD Table
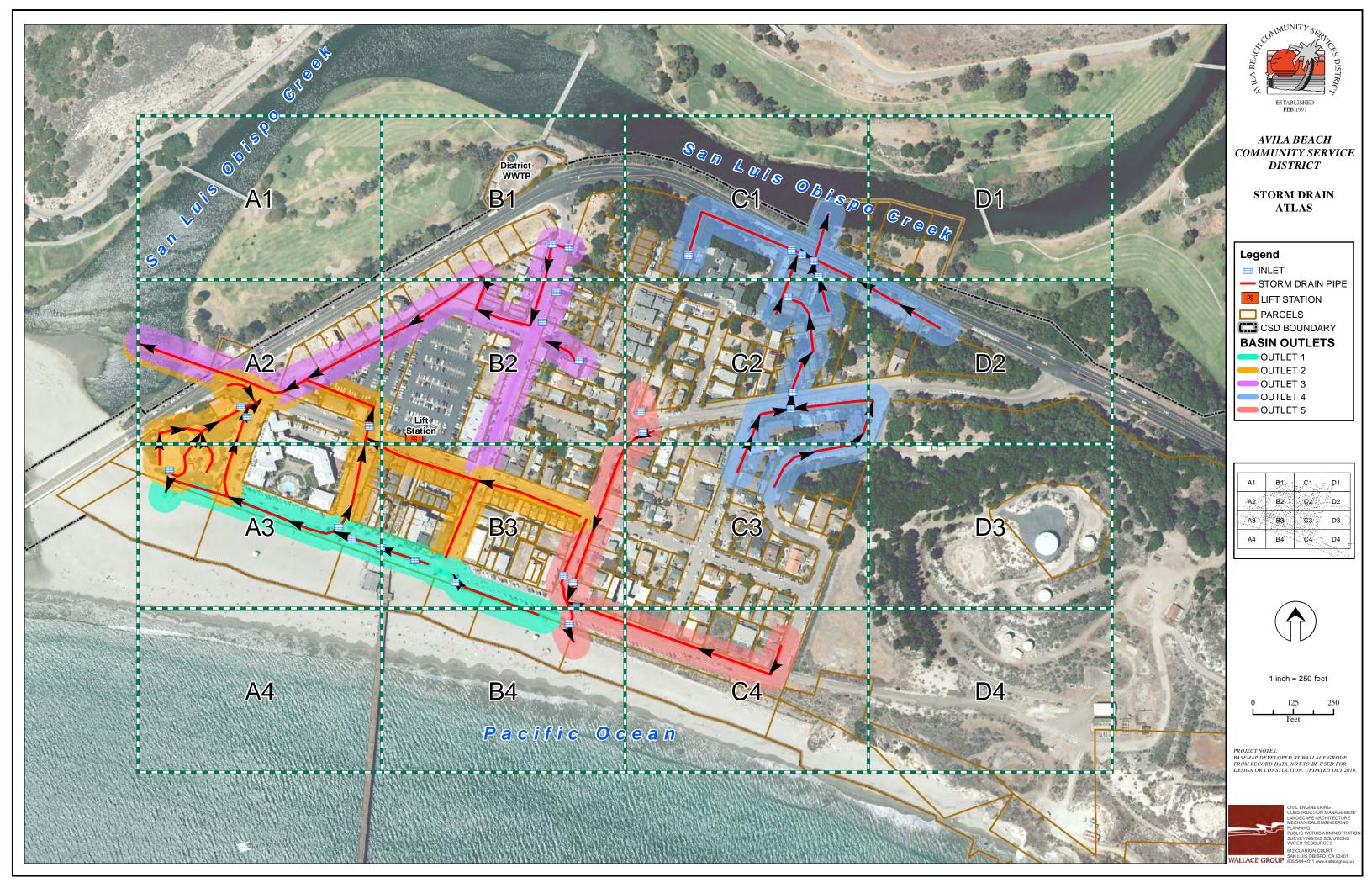
Label	Station (Calculated) (ft)	Flow (cfs)	Start Node	Stop Node	Length (Unified) (ft)	Slope (Calculated) (ft/ft)	Friction Slope (ft/ft)	Headloss (ft)
J-1	3+19							0.10
MH-1	2+34							0.10
O-1								
H-1	3+94							
SD-2		9.00	J-1	MH-1	84.8	0.002	0.002	0.12
SD-1		9.00	H-1	J-1	75.6	0.009	0.008	0.43
SD-3		9.00	MH-1	O-1	234.0	0.002	0.002	0.70

Velocity (ft/s)	Invert (Start) (ft)	Hydraulic Grade Line (In) (ft)	Invert (Stop) (ft)	Hydraulic Grade Line (Out) (ft)	Headloss Coefficient (Standard)	Diameter (in)
		3.76		3.65	0.600	
		3.53		3.44	0.500	
		4.65		4.65		
3.50	2.43	3.65	2.26	3.53		36.0
6.01	3.24	4.19	2.56	3.76		36.0
3.51	2.26	3.44	1.79	2.74		36.0

12596287-CALC-Alternative6B_Freeoutfalltest_20240813.stsw
8/13/2024

Bentley Systems, Inc. Haestad Methods Solution Center
76 Watertown Road, Suite 2D Thomaston, CT 06787 USA +1-203-755-1666

StormCAD
[10.04.00.158]
Page 1 of 1

Attachments

Attachment 1

MBS Land Surveys Topographic Survey

SYMBOL LEGEND:

Symbol	Description	Symbol	Description
—x—	FENCE LINE	▭▭	RETAINING WALL
—SS—	SEWER MAIN	PG&E	PG&E BOX
—W—	WATER MAIN	GM	GAS METER
—G—	GAS MAIN	GW	GAS METER
—ETC—	ELEC/TELEPHONE/CABLE	TEL	TELEPHONE BOX
—OHE—	OVERHEAD ELECTRIC	SIG	SIGNAL BOX
	DROP INLET AT CURB	CATV	CABLE T.V. BOX
	DROP INLET	EB	ELECTRIC BOX
	STORM DRAIN MANHOLE	T	TELEPHONE MANHOLE
	FIRE HYDRANT		STREET LIGHT
	WATER WELL	JP	JOINT POLE
	WATER VALVE	PP	POWER POLE
	WATER METER		GUY WIRE
	SEWER MANHOLE		
	SEWER CLEANOUT		

RECORD RIGHT-OF-WAY or PROPERTY LINE

ABBREVIATIONS

AC ASPHALT CONCRETE
AP ANGLE POINT
BM BENCH MARK
BLDG BUILDING
BOW BACK OF WALK
CB CATCH BASIN
CF CURB FACE
CO CLEAN OUT
COL COLUMN
COR CORNER
CONC CONCRETE
CMP CORRUGATED METAL PIPE
CMU CONCRETE MASONRY UNITS
CRN CROWN OF STREET
DI DROP INLET
EG EXISTING GRADE
EP EDGE OF PAVEMENT
FD FOUND
FL FLOW LINE
FF FINISH FLOOR
FOW FACE OF WALL
HSE HOUSE COR
GR GRASS
GM GAS METER
IP IRON PIPE

IP IRON PIPE
GB GRADE BREAK
GM GAS METER
HP HIGH POINT
LT LIGHT
MH MANHOLE
PP POWER POLE
PVC POLYVINYL PIPE
RB REBAR
RCP REINFORCED CONCRETE PIPE
R10 CANOPY RADIUS
SD STORM DRAIN
SL POINT ON SLOPE
SS SEWER
STP STEP
STP STAIRS
TOP TOP OF SLOPE
TOE TOE OF SLOPE
TW TOP OF WALL
W WATER
WL WALL
WM WATER METER
WV WATER VALVE

DI-1.5FL TOP OF GRATE -1.5' FLOW LINE

SURVEYOR'S STATEMENT:

THIS MAP REPRESENTS AN AERIAL TOPOGRAPHIC SURVEY PERFORMED BY CENTRAL COAST AERIAL MAPPING, PHOTOGRAPHY DATED DECEMBER 15, 2023, CCAM JOB NO. 23-236, WITH GROUND CONTROL SET BY MBS LAND SURVEYS.
CONTACT: ROBERT LAFICA, CCAM 805-543-4307

MICHAEL B. STANTON, PLS 5702   DATE

SURVEYOR'S NOTES:

1. EASEMENTS FOR STORM DRAINAGE HAVE BEEN PLOTTED ACCORDING TO THE TITLE REPORTS LISTED BELOW:
076-181-032 FIRST AMERICAN TITLE NO. 4001-7064081
076-181-039 FIRST AMERICAN TITLE NO. 4001-7064081
076-181-061 FIRST AMERICAN TITLE NO. 4001-7064081
076-205-001 FIRST AMERICAN TITLE NO. 4001-7064106
076-196-001 FIRST AMERICAN TITLE NO. 4001-7064106
076-211-002 FIRST AMERICAN TITLE NO. 4001-7064089
076-211-003 FIRST AMERICAN TITLE NO. 4001-7064089
076-213-012 FIRST AMERICAN TITLE NO. 4001-7064126
076-213-003 FIRST AMERICAN TITLE NO. 4001-7064153
076-213-007 FIRST AMERICAN TITLE NO. 4001-7064153
076-213-003 FIRST AMERICAN TITLE NO. 4001-7064131
076-196-006 FIRST AMERICAN TITLE NO. 4001-7064148
076-213-012 FIRST AMERICAN TITLE NO. 4001-7064134

2. ONLY THE SURFACE EVIDENCE OF UNDERGROUND UTILITIES HAVE BEEN MEASURED IN THE FIELD ON THIS SURVEY. UNDERGROUND ALIGNMENTS MAY BE SHOWN FROM AGENCY ATLAS INFORMATION, HOWEVER, I MAKE NO WARRANTY AS TO THE ACTUAL LOCATION, TYPE OR DEPTH OF THOSE UNDERGROUND UTILITIES. CALL UNDERGROUND SERVICE ALERT (USA) AT 1-800-642-2444 TO VERIFY THE ACTUAL LOCATION OF UTILITIES PRIOR TO ANY EXCAVATION.

3. THE SIGNED AND SEALED ORIGINAL DRAWING OF THIS MAP CONSTITUTES THE FINAL WORK PRODUCT. MBS LAND SURVEYS WILL NOT BE LIABLE FOR ELECTRONIC VERSIONS OF THIS MAP PROVIDED TO OTHER PARTIES.

5. THE RIGHT-OF-WAY LINES AND BOUNDARY LINES SHOWN HEREON WERE COMPILED FROM RECORD INFORMATION (I.E. RECORDED MAPS OR DEEDS) AND ARE NOT INTENDED TO REPRESENT THE TRUE OR ACTUAL BOUNDARY LINES OF THE SUBJECT PROPERTY.

GRAPHIC SCALE

(IN FEET)
1 inch = 20 ft.

Parcel / Owner labels

PAUL & ROBIN METCHIK
2975 AVILA BEACH DR.
APN 076-196-007

ROB GARCIA ET. UX.
2995 AVILA BEACH DR.
APN 076-196-006

ABR PROPERTY, LP
APN 076-181-039

MILLER ACORD LP
550 1st STREET
APN 076-213-003

GNOS FAMILY TRUST
546 1st STREET
APN 076-213-012

VICTOR MONTGOMERY
502 1st STREET
APN 076-213-007

PORT SAN LUIS HARBOR DISTRICT
APN 076-215-009

PORT SAN LUIS HARBOR DISTRICT
APN 076-215-012

AVILA LIGHTHOUSE SUITES
595 1st STREET

Street labels

AVILA GOLF RESORT
BLOCK 5
BLOCK 2
PM COAL 96-112
54 PM 73 5
COUNTY ROAD
2703 OR 515
SAN LUIS OBISPO CREEK
AVILA BEACH DRIVE
SAN JUAN ST.
FIRST STREET
SAN FRANCISCO ST.

TOPOGRAPHIC MAP

PORTIONS OF BLOCKS 4, 5 AND 9 IN THE TOWN OF AVILA BEACH AS SHOWN ON MAP FILED IN BOOK A AT PAGE 6 OF MAPS, AND LOTS 1-7 OF PARCEL MAP BOOK 54 AT PAGE 73, IN THE COUNTY OF SAN LUIS OBISPO, CALIFORNIA

AT THE REQUEST OF COUNTY OF SAN LUIS OBISPO

MICHAEL B. STANTON, PLS 5702
MBS LAND SURVEYS
3559 SOUTH HIGUERA ST.
SAN LUIS OBISPO, CA 93401
805-594-1960

April 3, 2024   JOB #23-111   SHEET 1 OF 2

DATE	REVISION
—	—

GRAPHIC SCALE

(IN FEET)
1 inch = 20 ft.

SECOND STREET

FIRST STREET

SAN MIGUEL ST.

COUNTY OF SAN LUIS OBISPO
APN 076-196-001

PARCEL 3
2500 OR 664

PCL B

660 OR 23

800 OR 450

BLOCK 5

PM COAL 96-112

54 PM 73

PM COAL 72-89

16 PM 40

PARCEL A

RESORT

COUNTY ROAD

103 OR 515

COUNTY OF SAN LUIS OBISPO
RIGHT-OF-WAY PER OAK CANY...

10' P.U.E. PER
DOC. 2005-040921

PEDESTRIAN ACCESS
EASEMENT PER
DOC. 2005-031763

5' WIDE PUBLIC ACCESS
EASEMENT PER
DOC. 2005-031763

PAUL & ROBIN METCHIK
2975 AVILA BEACH DR.
APN 076-196-007

ROB GARCIA ET. UX.
2995 AVILA BEACH DR.
APN 076-196-006

LOPEZ
METER STATION

UNABLE TO DETERMINE EDGE/LIMITS
OF CONCRETE IN CERTAIN AREAS
DUE TO HEAVY SILT AND GABION
ROCKS BELOW WATER SURFACE

GABIONS

DRAINAGE CHANNEL

20' WIDE FIRE LANE PER 78 PM 53
20' PUE PER DOC 2005-40921
20' WIDE PSMT TO A.B.C.S.D. PER DOC 2013-053975
20' WIDE PRIVATE ACCESS EASEMENT PER 54 PM 73
20' WIDE SEWER AND WATER EASEMENT PER DOC 2013-053975
20' WIDE SO. CAL. GAS EASEMENT PER DOC. 2005-03896?

PORT SAN LUIS HARBOR DISTRICT
APN 076-215-009

PORT SAN LUIS HARBOR DISTRICT
APN 076-215-012

OFFER TO DEDICATE
10' PUBLIC FOR DRAINAGE
STRUCTURES AND APPURTENANCES
PER DOC. 2001-025022

MILLER ACCORD LP
550 1st STREET
APN 076-213-003

GNOS FAMILY TRUST
546 1st STREET
APN 076-213-012
1997-044329

VICTOR MONTGOMERY
502 1st STREET
APN 076-213-007
1995-057534

20' WIDE EASEMENT TO CO. OF S.L.O.
FOR DRAINAGE AND APPURTENANCES
PER 1455 OR 248

PG&E
VAULT

1204
SET 5/8REBR CAP -MBS CTRL HV/AV BASE PT

1205
6.33

1203
6.33
SET MAG -AT HV3

1206
7.84
SET MAG -AT CHK1

1701
9.05
SET SPIKE

1214
SET 3/25AD LS5702 MW

1213
FD 2.25AD LS5139 MW

1712
FD 1.0R 0AD -5LEG

24" RCP

16" PEPPER
8" PEPPER
16" PEPPER
8" PEPPER
8" PINE

12" PALM
8" PALM
8" PALM
12" PALM

3x8" PALMS
3x8" PALMS
22" PALM

14" PINE
12" PINE

2x18" PEPPER

TOPOGRAPHIC MAP

PORTIONS OF BLOCKS 4, 5 AND 9 IN THE TOWN OF AVILA BEACH AS
SHOWN ON MAP FILED IN BOOK A AT PAGE 5 OF MAPS, AND LOTS
1-7 OF PARCEL MAP BOOK 54 AT PAGE 73, IN THE, COUNTY OF SAN
LUIS OBISPO, CALIFORNIA

AT THE REQUEST OF COUNTY OF SAN LUIS OBISPO

MICHAEL B. STANTON, PLS 5702
3559 SOUTH HIGUERA ST.
SAN LUIS OBISPO, CA 93401
805-594-1960

MBS
LAND SURVEYS

April 3, 2024   JOB #23-111   SHEET 2 OF 2

DATE   REVISION

Attachment 2

ABCSD Storm Drain Atlas

San Luis Obispo Creek

District WWTP

Lift Station
PS

Pacific Ocean

A1 B1 C1 D1
A2 B2 C2 D2
A3 B3 C3 D3
A4 B4 C4 D4

AVILA BEACH
COMMUNITY SERVICE
DISTRICT

STORM DRAIN
ATLAS

Legend
- INLET
- STORM DRAIN PIPE
- PS LIFT STATION
- PARCELS
- CSD BOUNDARY

BASIN OUTLETS
- OUTLET 1
- OUTLET 2
- OUTLET 3
- OUTLET 4
- OUTLET 5

1 inch = 250 feet

0   125   250
Feet

CIVIL ENGINEERING
CONSTRUCTION MANAGEMENT
LANDSCAPE ARCHITECTURE
MECHANICAL ENGINEERING
PLANNING
PUBLIC WORKS ADMINISTRATION
SURVEYING/GIS SOLUTIONS
WATER RESOURCES

612 CLARION COURT
SAN LUIS OBISPO, CA 93401
805 544-4011 www.wallacegroup.us

WALLACE GROUP

ESTABLISHED
FEB.1997

Attachment 3

England Geosystem Groundwater Profile

FIGURE 2

SITE PLAN

AVILA BEACH DRIVE
AND ROSSI PROPERTY
AVILA BEACH, CALIFORNIA

PREPARED FOR

UNOCAL CORPORATION
SAN LUIS OBISPO CALIFORNIA

ENGLAND
GEOSYSTEM
ENVIRONMENTAL ENGINEERING

EXPLANATION

/////	AREA OF REMEDIAL EXCAVATION
GB-1 ⊙	EXISTING SAMPLING POINT
GP-7 ◐	GEOPROBE BORING
⊔	CROSS SECTION LOCATION

1227.001\007\11/13/03

CROSS SECTION A-A'

AVILA BEACH DRIVE
AND ROSSI PROPERTY
AVILA BEACH, CALIFORNIA

FIGURE 3

PREPARED FOR

UNOCAL CORPORATION
SAN LUIS OBISPO CALIFORNIA

ENGLAND
GEOSYSTEM
ENVIRONMENTAL ENGINEERING

CROSS SECTION B-B'

FIGURE 4

CROSS SECTION B-B'

AVILA BEACH DRIVE
AND ROSSI PROPERTY
AVILA BEACH, CALIFORNIA

PREPARED FOR

UNOCAL CORPORATION
SAN LUIS OBISPO CALIFORNIA

ENGLAND
GEOSYSTEM
ENVIRONMENTAL ENGINEERING

Attachment 4

Effective FIRM and FIS Profile

National Flood Hazard Layer FIRMette

FEMA

120°44'28"W 35°11'4"N

AREA OF MINIMAL FLOOD HAZARD
Zone X

San Luis Obispo Creek

Zone AE

Zone A

Zone A

T R S0

San Luis Obispo County
060304

06079C1336H
eff. 5/16/2017

Zone AE

Zone AE

FLOODWAY
Zone AE

Zone AE

Zone AE

15.5 FEET

16 FEET

14 FEET

13.4 FEET

13 FEET

57

Zone AE
(EL 15 Feet)

58

Zone AE
(EL 16 Feet)

59

Zone AE
(EL 13 Feet)

Zone VE
(EL 15 Feet)

Zone VE
(EL 16 Feet)

Zone VE
(EL 22 Feet)

Zone VE
(EL 17 Feet)

Feet
0   250   500   1,000   1,500   2,000

1:6,000

120°43'51"W 35°10'35"N

Basemap: USGS National Map: Orthoimagery: Data refreshed October, 2020

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
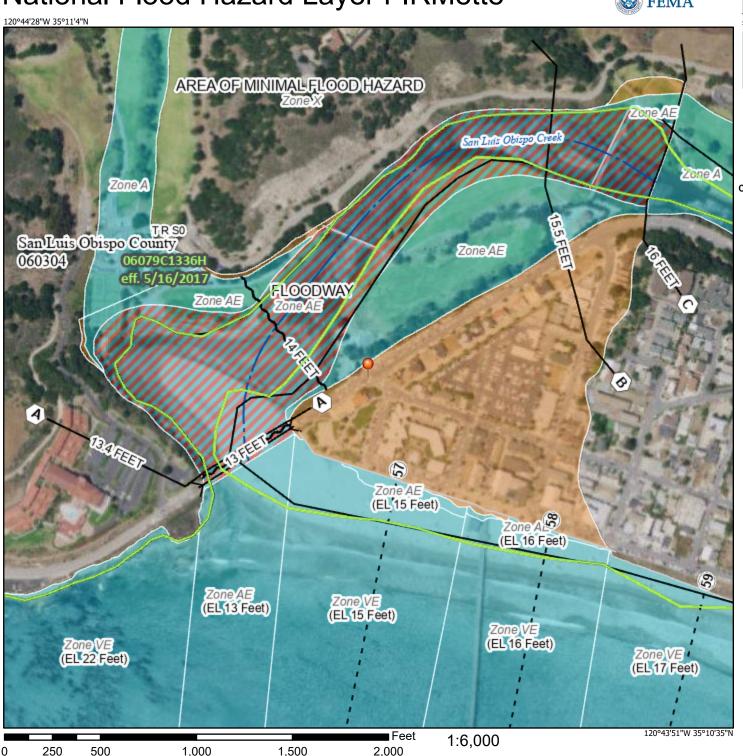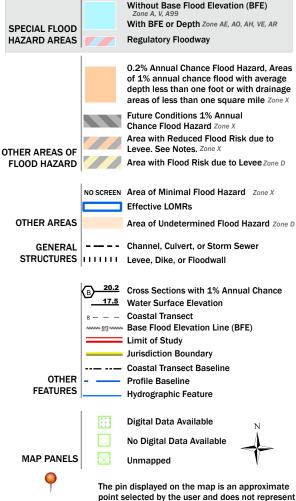- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- — — — Channel, Culvert, or Storm Sewer
- ⊩⊩⊩⊩ Levee, Dike, or Floodwall

OTHER FEATURES
- 20.2 / 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- 8 — — — Coastal Transect
- ∿∿∿ 513 ∿∿∿ Base Flood Elevation Line (BFE)
- —— Limit of Study
- —— Jurisdiction Boundary
- — · — · — Coastal Transect Baseline
- — · — Profile Baseline
- —— Hydrographic Feature

MAP PANELS
- Digital Data Available
- No Digital Data Available
- Unmapped

N

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 10/26/2022 at 1:15 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

FLOOD PROFILES

SAN LUIS OBISPO CREEK

ELEVATION IN FEET (NAVD 88)

STREAM DISTANCE IN FEET ABOVE PACIFIC OCEAN

AVILA BEACH DRIVE

FOOTBRIDGE

FOOTBRIDGE

LIMIT OF DETAILED STUDY

ELEVATIONS DOWNSTREAM OF THIS POINT CONTROLLED BY THE PACIFIC OCEAN

LEGEND

0.2% ANNUAL CHANCE FLOOD

1% ANNUAL CHANCE FLOOD

2% ANNUAL CHANCE FLOOD

10% ANNUAL CHANCE FLOOD

STREAM BED

CROSS SECTION LOCATION

FEDERAL EMERGENCY MANAGEMENT AGENCY

SAN LUIS OBISPO COUNTY, CA

AND INCORPORATED AREAS

67P