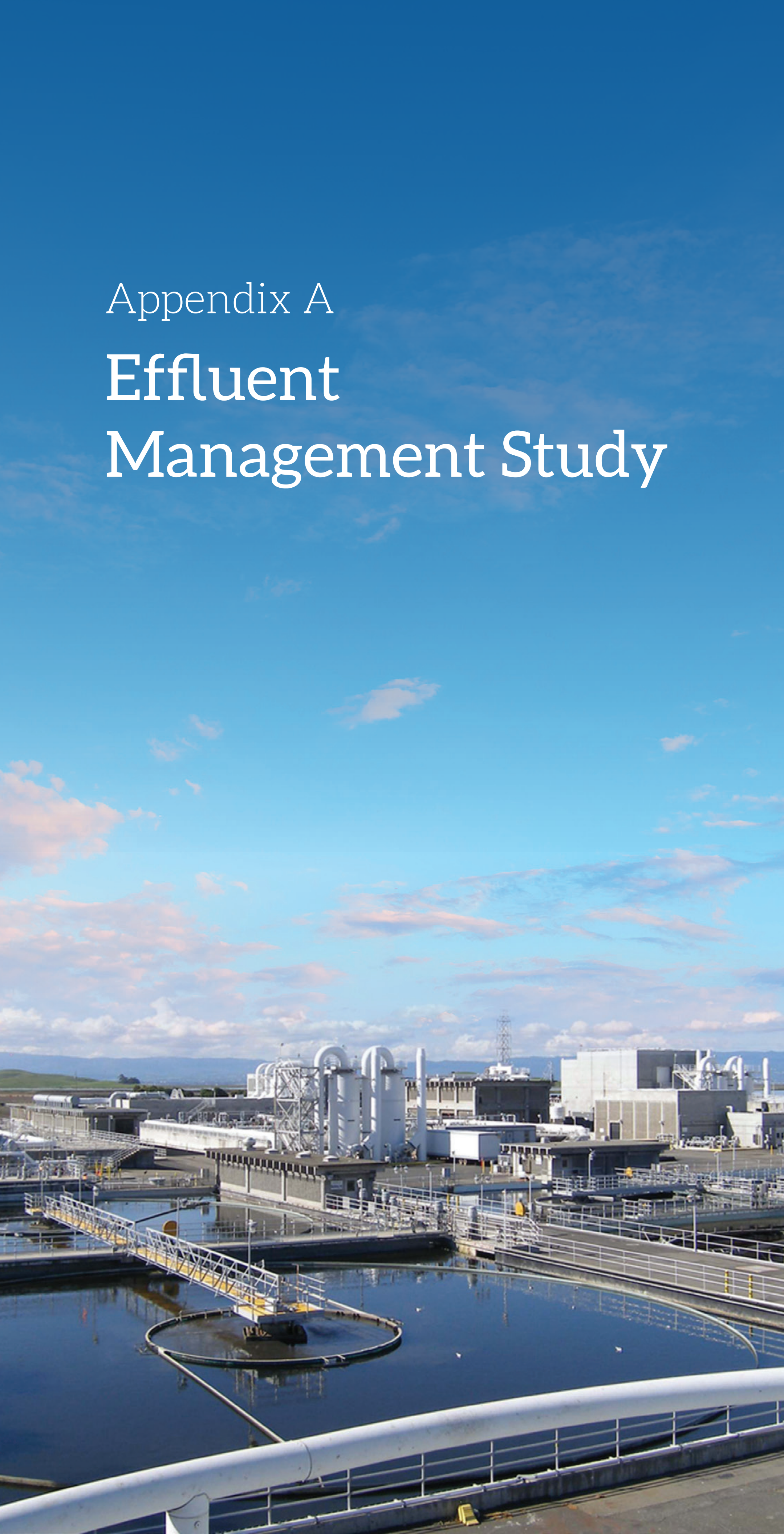
Appendix A

Effluent Management Study



A

Union Sanitary District's
Enhanced Treatment and Site Upgrade Program

EFFLUENT MANAGEMENT STUDY

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COMMITMENT & INTEGRITY DRIVE RESULTS

Union Sanitary District
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EXECUTIVE SUMMARY

The purpose of the Effluent Management Study (Study) is to develop and evaluate alternatives to manage effluent from the Union Sanitary District (District or USD) Alvarado Wastewater Treatment Plant (WWTP). More specifically, this study is focused on alternatives to address discharge of peak wet weather effluent flow from the WWTP.

USD's primary method of effluent discharge is through the East Bay Dischargers Association (EBDA) joint conveyance and outfall system. However, USD's ability to send effluent to the EBDA system is dependent on several factors, including discharge flow from the City of Hayward and the ability to discharge to the Hayward Marsh (via the EBDA system). USD can also discharge to Old Alameda Creek during unusually high flow events, however, discharge to Old Alameda Creek is not currently permitted on a routine basis. USD has an immediate effluent discharge issue given the current state of the Hayward Marsh and the limited capability of Old Alameda Creek to take additional peak flow discharge.

USD has investigated a broad variety of influent management, storage, and effluent management options to address the imminent phasing out of the Hayward Marsh as a reliable wet weather discharge location. Unless or until the East Bay Regional Parks District embraces its management role of the Marsh as an effluent reuse and disposal facility, along with providing habitat benefits, the best solution for long term effluent management at this stage appears to be upgrading effluent water quality at the WWTP, including early adoption of nutrient removal improvements, either for sidestream treatment as a partial nutrient removal, and/or full flow nutrient removal upgrade as a complete solution. Early adoption will address nutrient removal in the final effluent, which is anticipated to be needed after 2024 at some yet to be determined level. Early investment and adoption carry some level of risk to USD given that final standards are not determined. Given that these proposed improvements will benefit effluent water quality in the near term and can be repurposed to provide long term secondary treatment capacity, this risk can be mitigated. Implementing a nutrient process with a longer solids retention time will also provide benefits to operation and performance at the WWTP.

It is therefore recommended that early adoption of side-stream or full flow nutrient removal at the Alvarado WWTP and increased seasonal discharge to Old Alameda Creek be further developed as the primary effluent management project. This alternative will require extensive collaboration with the Regional Water Quality Control Board because an expansion of the use of the Old Alameda Creek outfall as a shallow water discharge requires site-specific permit conditions which have not yet been developed.

1. INTRODUCTION

The purpose of this Effluent Management Study is to present and evaluate peak wet weather effluent management strategies for Union Sanitary District (District or USD). The East Bay Dischargers Authority (EBDA) outfall provides the primary effluent disposal capability for USD, with Hayward Marsh (Marsh) and Old Alameda Creek providing essential wet weather capacity as well. The Hayward Marsh continues to function effectively as a discharge component for USD, but for the Marsh to continue to be effective for USD's effluent reuse and disposal purposes, it needs to be able to be permitted by the Regional Board, and needs to be maintained and supported (including dredging and levees reconstructed) by the East Bay Regional Parks District. East Bay Regional Parks has communicated to USD that is not in a position to invest the substantial capital (at least $20 Million) and O&M resources in the Marsh in order for USD to rely upon the Marsh for reliable disposal capacity.

Due to the potential loss of the Hayward Marsh as an effluent disposal option and the need to handle Peak Wet Weather Flows (PWWF), in excess of USD's effluent disposal capacity in EBDA, additional effluent management options need to be developed. This Study documents the effluent management options and identifies their viability at a preliminary level to help form a comprehensive and long-lasting solution. The Effluent Management Study is being developed in parallel with the interrelated Enhanced Treatment & Site Upgrade Program, which is intended to provide the long-term vision for the Alvarado Wastewater Treatment Plant (WWTP), including effluent water quality improvements.

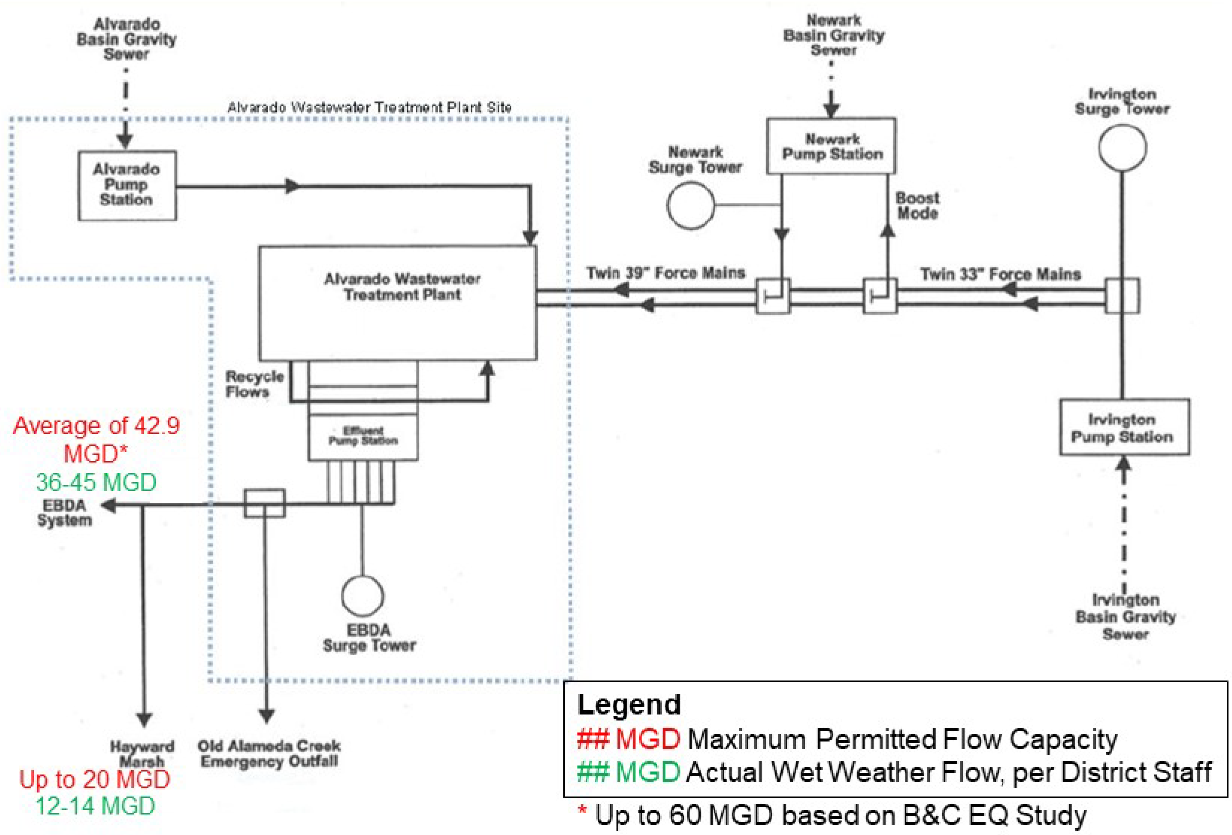
1.1 Existing Effluent Disposal System

The Union Sanitary District is evaluating strategies for disposing of treated wastewater from the WWTP. The WWTP currently provides secondary treatment of wastewater collected from Union City, Newark, and Fremont. Currently, USD is permitted to discharge secondary effluent at three discharge points:

- East Bay Dischargers Authority (EBDA) system
- Hayward Marsh
- Old Alameda Creek, during storm events only

Currently, USD is permitted to discharge up to 33 million gallons per day (MGD) average dry weather flow (ADWF) and 42.9 MGD peak daily flow of its wastewater to the EBDA outfall per its joint powers agreement (JPA) with EBDA (Order No. R2-2017-0016, NPDES No. CA 0037869). The Alvarado Effluent Pump Station (AEPS) is used to pump USD's treated effluent into the EBDA system. **Figure 1-1** shows a process flow schematic of the WWTP and the permitted flow capacities associated with its different discharge points.

Figure 1-1: Process Flow Schematic & Currently Permitted Discharge Points



Source: USD's Old Alameda Creek (Wet Weather Outfall) Permit. ORDER No. R2-2015-0045, NPDES No. CA0038733.

On average, approximately 3 MGD of effluent from USD is discharged from the EBDA pipeline to the Hayward Marsh. During peak weather events when total wastewater flow discharged by EBDA member agencies is beyond the capacity of the current system, up to 20 MGD of wastewater from USD's WWTP can be directed to Hayward Marsh. After the secondary-treated effluent flows through the freshwater treatment marsh, the reclaimed wastewater flows to San Francisco Bay.

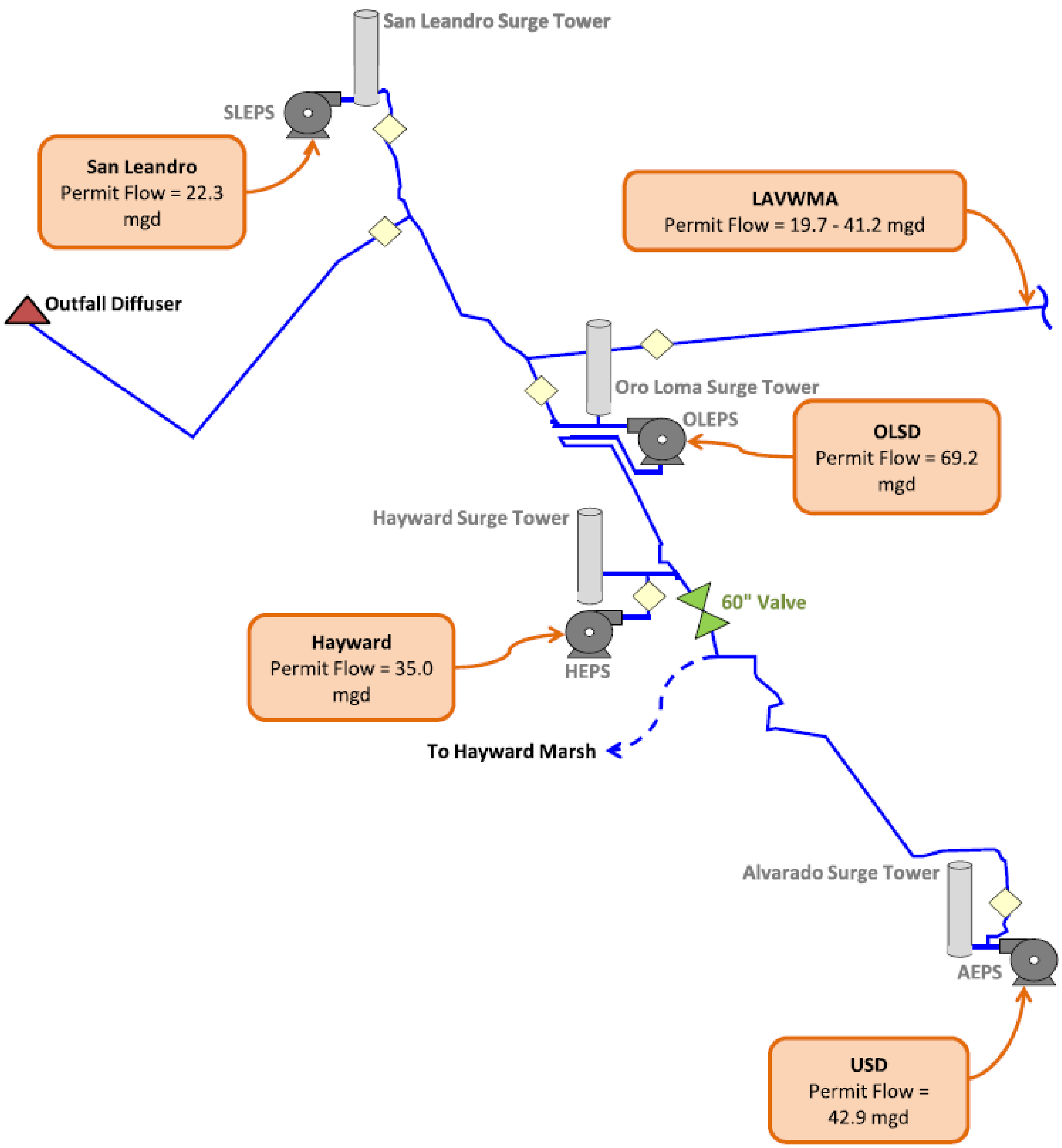
In addition to Hayward Marsh, during wet weather, USD can discharge to Old Alameda Creek. Although the previous maximum discharge flow limitation of 8.4 MG per discharge event is not retained in the current permit for Old Alameda Creek, calculations performed were based on this assumed limitation. The District has not been compelled to use this discharge point since 1998 but it typically has been exercised once per wet weather season since then.

Effluent options are required for USD in order to prepare for the elimination of Hayward Marsh as an option for wet weather discharges.

1.2 EBDA Capacity

USD's current average dry weather flow (ADWF) is 22.4 MGD and is estimated to increase to an ADWF of 33 MGD by 2058. Although there is sufficient effluent disposal capacity in the EBDA system to handle current and buildout average flows, flows through the plant increase significantly during wet weather events due to inflow and infiltration (I/I) into the collection system. A schematic of the EBDA system and permitted peak wet weather design flows is shown in **Figure 1-2**. Including flows from Livermore-Amador Valley Water Management Agency (LAVWMA), the total permitted discharge for the system is 189.1 MGD (San Francisco Bay Regional Water Quality Control Board, 2017).

Figure 1-2: EBDA System Schematic



Source: EBDA System Flow Master Plan (Carollo 2011)

Actual EBDA hydraulic capacity may not match the permitted discharge flow rate. Per District staff, USD can pump more than the permitted capacity amount, however, there are hydraulic limitations that vary depending on various conditions in the EBDA system. The District's hydraulic capacity was previously evaluated as part of the *Wastewater Equalization Storage Facilities Pre-Design* (Brown & Caldwell 1999) and more recently as part of the *Flow Equalization Report Update* (Brown & Caldwell, 2013), *Draft EBDA System Flow Master Plan* (Carollo 2011) and the *Draft EBDA Hydraulic Model Recalibration and Capacity Analysis* (Carollo 2017). The Final Version of the Hydraulic Model Recalibration and Capacity Analysis is not anticipated to have any significant new or different findings than the Draft.

The amount of available discharge capacity through the AEPS into the EBDA system is dynamic and dependent on the following primary factors:

1. Flow from Hayward Effluent Pump Station (HEPS), which is combined with the AEPS flow to the Oro Loma Effluent Pump Station (OLEPS). Increasing flows from HEPS decreases AEPS capacity.

2. Flow to the Hayward Marsh, which is diverted from the EBDA system upstream of the HEPS. Increasing flow to Hayward Marsh increases AEPS capacity.

3. The operating wet well level at OLEPS, which receives combined flow from AEPS, HEPS, and Oro Loma Sanitary District (OLSD). Increasing OLEPS wet well water levels decreases AEPS capacity.

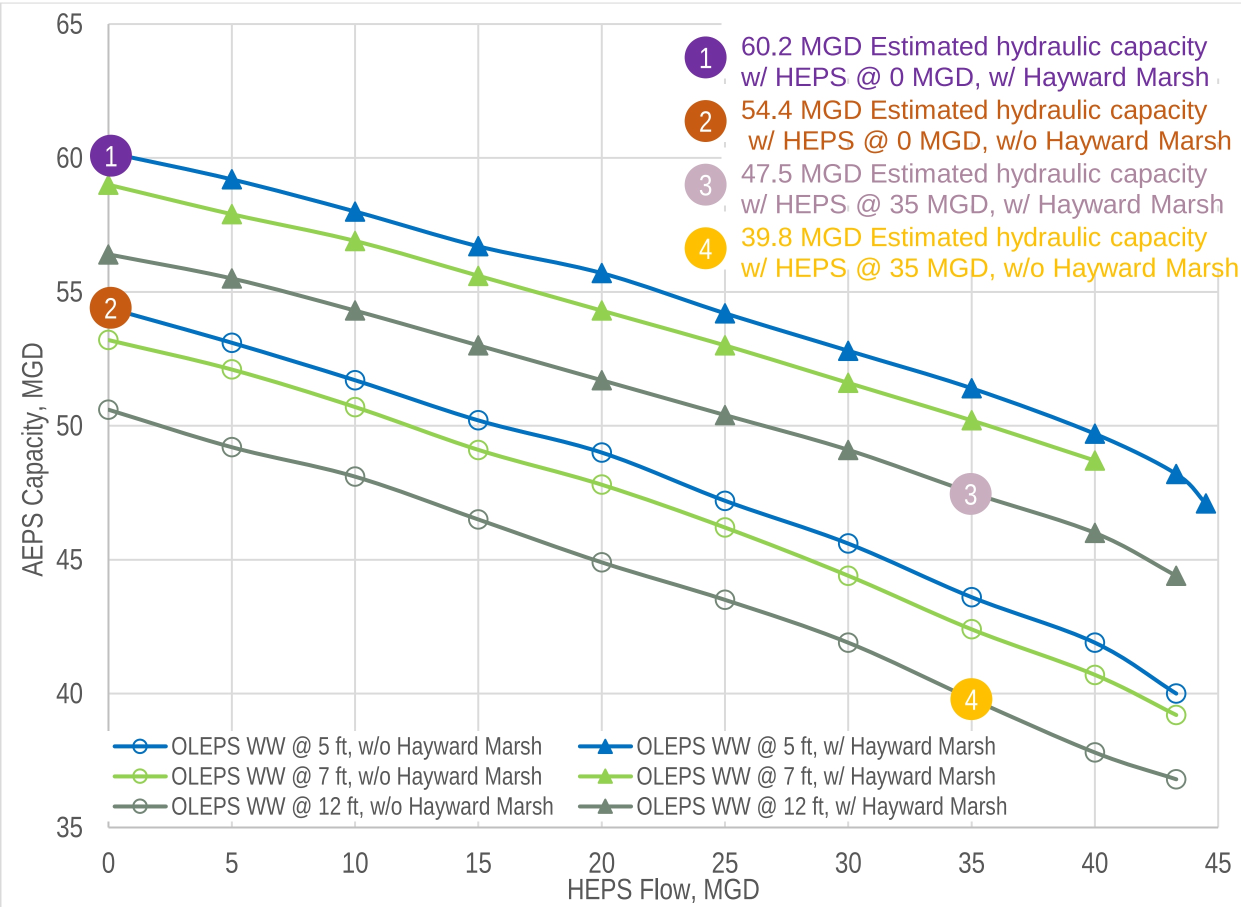
USD's discharge capacities under various conditions with and without flow to Hayward Marsh based on the 2017 study are shown in **Table 1-1**.

Table 1-1: Maximum Discharge from AEPS (USD) to EBDA

Flow from HEPS, MGD	OLEPS Wet Well Elevation, FT	Maximum Discharge to EBDA w/o Hayward Marsh, MGD	Maximum Discharge to EBDA w/ Hayward Marsh, MGD
0	5	54.4	60.2
	7	53.2	59
	12	50.6	56.4
20	5	49	55.7
	7	47.8	54.3
	12	44.9	51.7
35	5	43.6	51.4
	7	42.4	50.2
	12	39.8	47.5

As illustrated in **Table 1-1**, flow from HEPS has the largest impact on AEPS capacity, followed by flow to the Hayward Marsh and OLEPS Wet Well Elevation. Based on the *Draft EBDA Hydraulic Model Recalibration and Capacity Analysis* and illustrated in **Figure 1-3,** the capacity of AEPS without discharge to the Hayward Marsh varies from 36.8 MGD to 54.4 MGD, depending on HEPS discharge and OLEPS wet well level. With discharge to the Hayward Marsh, the AEPS capacity ranges from 44.0 MGD to 60.2 MGD.

Figure 1-3: Estimated AEPS Discharge Limitations



Source: Draft EBDA Hydraulic Model Recalibration and Capacity Analysis (Carollo 2017)

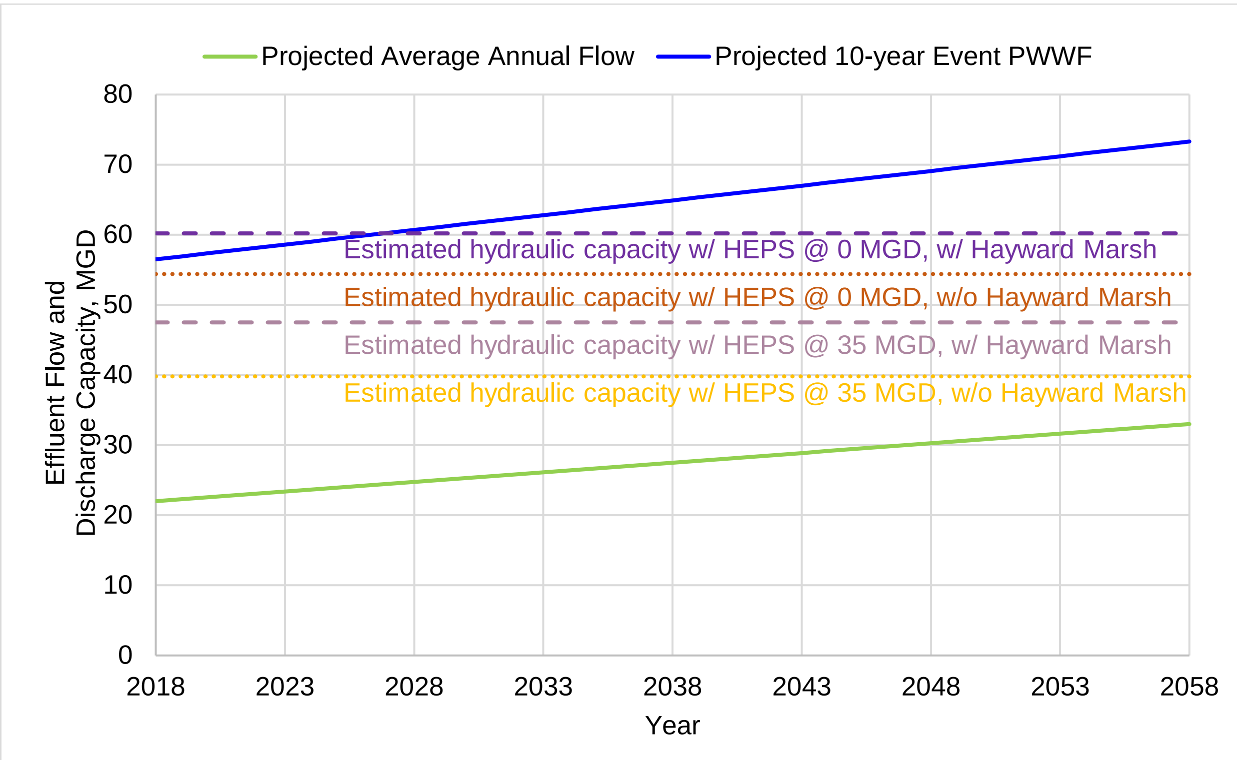
Although the allocated capacity for HEPS is 35 MGD, during peak wet weather events, some flow from the City of Hayward would be diverted to the City of Hayward storage ponds. Actual peak flow from HEPS may be less than 35 MGD due to the use of storage. As part of the Draft EBDA Hydraulic Model Recalibration and Capacity Analysis, EBDA requested that HEPS be evaluated using flows of 20 MGD and 15 MGD.

EBDA has developed as a standard operating procedure, which is used to manage capacity in the EBDA system during wet weather events. Based on the standard operating procedure, if the OLEPS wet well level continues to rise at ~ 110 MGD of flow through OLEPS, EBDA will direct the City of Hayward to begin diverting flow to the City of Hayward storage ponds. If the wet well level at OLEPS continues to rise, and USD cannot dispose of all of its flow through EBDA and the Hayward Marsh, USD may then need to discharge to Old Alameda Creek (that direction is not given by EBDA). A copy of the 2017-2018 EBDA Standard Operating Procedure is included as Appendix A.

1.3 Excess Effluent Storage (or Discharge) Requirements

A summary of USD's effluent discharge (current and future for both average and peak wet weather) and estimated EBDA hydraulic capacity is presented in **Figure 1-4.**

Figure 1-4: USD's Estimated Effluent Flow and Discharge Capacity



Notes: Maximum capacities shown are based on an OLEPS wet well level of 5 feet; minimum capacities are based on an OLEPS wet well level of 12 feet. Additional flow can be discharged to Old Alameda Creek (nominally up to 8.4 MG) during peak wet weather events. There is no longer a volumetric limit in the NPDES permit for Old Alameda Creek, but the frequency of use allowed in the Permit is tied to an excess flow event of approximately 8.4 MG.

Excess flows above USD's discharge capacity through the EBDA system will need to be managed (i.e. stored or discharged to an alternative location). USD's equalization storage (EQ) requirements (i.e. managed volume) for a 10-year storm event were also previously evaluated in the *Flow Equalization Report Update*. Due to water conservation measures, the current dry weather influent wastewater flow is similar to influent flow rates presented in the *Flow Equalization Report Update*, which was prepared in 2013. Although the current peak flow of 56.9 MGD would typically exceed the available capacity that can be accommodated in the EBDA pipeline, Brown and Caldwell concluded that the excess flow could currently be discharged to

Old Alameda Creek without the need for additional storage. The estimated current storage requirements from the *Flow Equalization Report* are presented in **Table 1-2** below.

Table 1-2: Effluent Storage Analysis at Current Peak Flows (56.9 MGD)

Assumed Available EBDA System Capacity, MGD	Hayward Marsh Flow, MGD	Excess Effluent Volume, MG	Old Alameda Creek Discharge Limit, MG	Effluent Storage Required, MG
42.9	0	2.3	8.4	0
51	0	0.4	8.4	0
60	20	0	8.4	0

Source: Flow Equalization Update Report (Brown and Caldwell, 2013)

However, flows are anticipated to increase at a rate of 1% per year and within two to three years the Hayward Marsh may not be available to USD for effluent discharge. For the full buildout condition of USD's service area (2058), the 10-year storm event would result in a projected flow of 73.3 MGD. Under this condition (with the worst case EBDA capacity of 42.9 MGD), 53.6 MG of equalization storage would be required with 8.4 MG of discharge capacity through Old Alameda Creek and no discharge to Hayward Marsh. However, with better than worst-case hydraulic conditions, the District could pump 51 MGD to the EBDA system, which would significantly reduce the amount of effluent storage required to 2.3 MG (with 8.4 MG discharged to Old Alameda Creek). Approximately, 60 MGD is the estimated maximum flow that USD can discharge into the EBDA system (including 20 MGD to Hayward Marsh) when downstream conditions are optimal. In the last case, no effluent storage is needed (with 2.2 MG discharged to Old Alameda Creek). **Figure 1-3** above shows the estimated AEPS discharge limitations; **Table 1-3** below summarizes the findings of the effluent storage analysis at buildout flows.

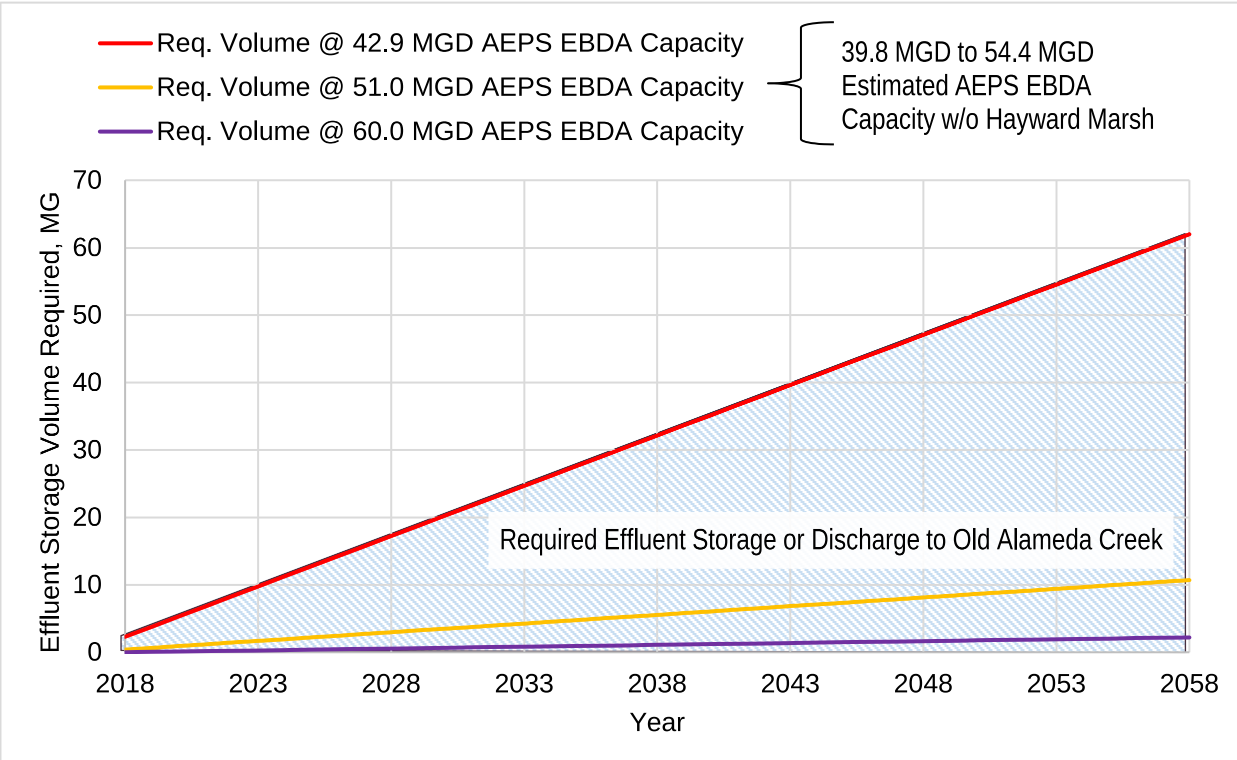
Table 1-3: Effluent Storage Analysis at Future Peak Flows (73.3 MGD)

Assumed Available EBDA System Capacity, MGD	Hayward Marsh Flow, MGD	Excess Effluent Volume, MG	Old Alameda Creek Discharge Limit, MG	Effluent Storage Required, MG
42.9	0	62.0	8.4	53.6
51	0	10.7	8.4	2.3
60	20	2.2	8.4	0

Source: Flow Equalization Update Report (Brown and Caldwell 2013)

Therefore, USD is identifying and evaluating alternatives for management and/or disposal of peak wet weather flows. A timeline showing the effluent storage requirement under the three EBDA system capacities, from current flows to build out, is presented in **Figure 1-5**.

Figure 1-5: Estimated Effluent Storage Requirements



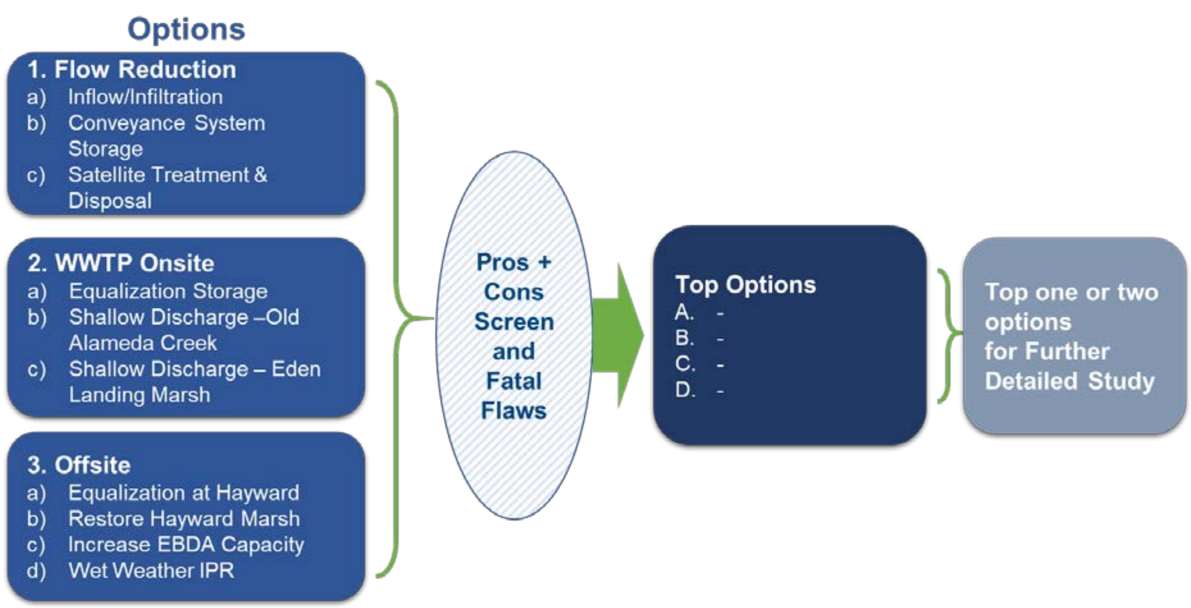
Note: Required volumes do not account for potential discharge to Old Alameda Creek (nominally up to 8.4 MG) during peak wet weather events.

2. EFFLUENT MANAGEMENT OPTIONS

A range of effluent management and discharge options were identified and evaluated in this first phase of the Effluent Management Study. The feasibility of the effluent management options were evaluated based on their viability and the extent to which they provide a solution to the amount of effluent storage required. Parameters considered for option viability included but were not limited to the following: permit compliance, operational complexity, capital cost, and life cycle cost. Based on the initial screening process, some options were eliminated from further consideration, leaving a narrower set of four top options. In Phase II, the costs, benefits and implementation plan for the preferred effluent management option(s) will be identified.

This chapter describes each of the identified options that will be subjected to the initial feasibility comparison. **Figure 2-1** below shows the initial set of options to be considered and the anticipated approach for the Effluent Management Study. Management options are classified into three categories: 1) Flow Reductions, which are focused on reducing flows coming into the WWTP, 2) On-site at the WWTP, and 3) Off-site, which are focused on managing effluent downstream of the AEPS.

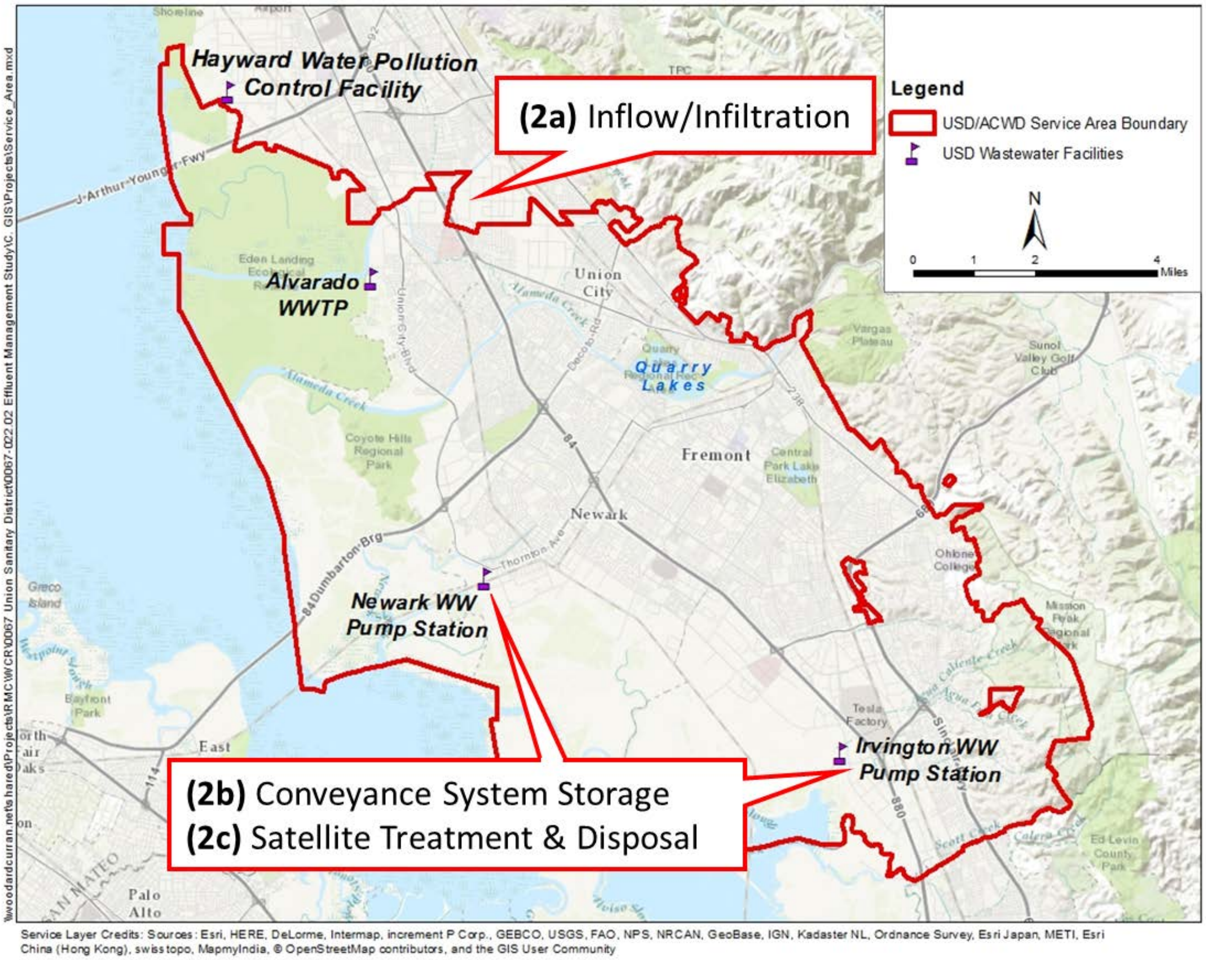
Figure 2-1: Effluent Management Study Approach



2.1 Influent Flow Reduction Options

This section describes three options that would address effluent management via influent flow reduction. This strategy reduces effluent by reducing flows in the collection system or delaying peaks from reaching the plant. **Figure 2-2** shows the location of these options relative to USD's pump station facilities, as well as the service area for USD/ACWD.

Figure 2-2: Locational Map of Influent Flow Reduction Options



2.1.1 Inflow/Infiltration (I/I)

Stormwater I/I into the wastewater infrastructure system during wet weather increases the volume of wastewater conveyed through the collection system to the treatment plant. This option would involve the implementation of strategies that would reduce the level of I/I into the wastewater system. However, District staff has confirmed that current I/I in the collection system is minimal (the USD peaking factor of just over 2 is contrasted with systems throughout the East Bay and greater Bay Area of between 4 and 5) and that additional I/I reduction available would need to be addressed through upper laterals. Given the limited flow reduction possible with this option and the costs and time that would be associated with trying to reduce these minimal flows, it is ranked as a minor solution with low viability.

2.1.2 Conveyance System Storage

This option involves expanding the use of available storage within the existing conveyance system for peak flow attenuation. There is an existing wet weather equalization tank at the Irvington pump station, with a capacity of 1.8 MG. According to the Flow Equalization Update Project, this basin could be increased to 3.6 MG. However, per the *2013 Brown and Caldwell Flow Equalization Update Report*, the Irvington force main's capacity is currently reduced whenever influent flow is diverted into the Irvington pump station storage. The hydraulic impacts of additional storage at Irvington Pump Station would need to be confirmed. The Newark Pump Station site could allow for another 2 MG of similar influent storage. The old treatment plant at the Newark site on USD's property would be demolished to create space for influent storage. The District currently has plans to both expand the Irvington basin as well as to construct a new basin at Newark Pump Station in coming years.

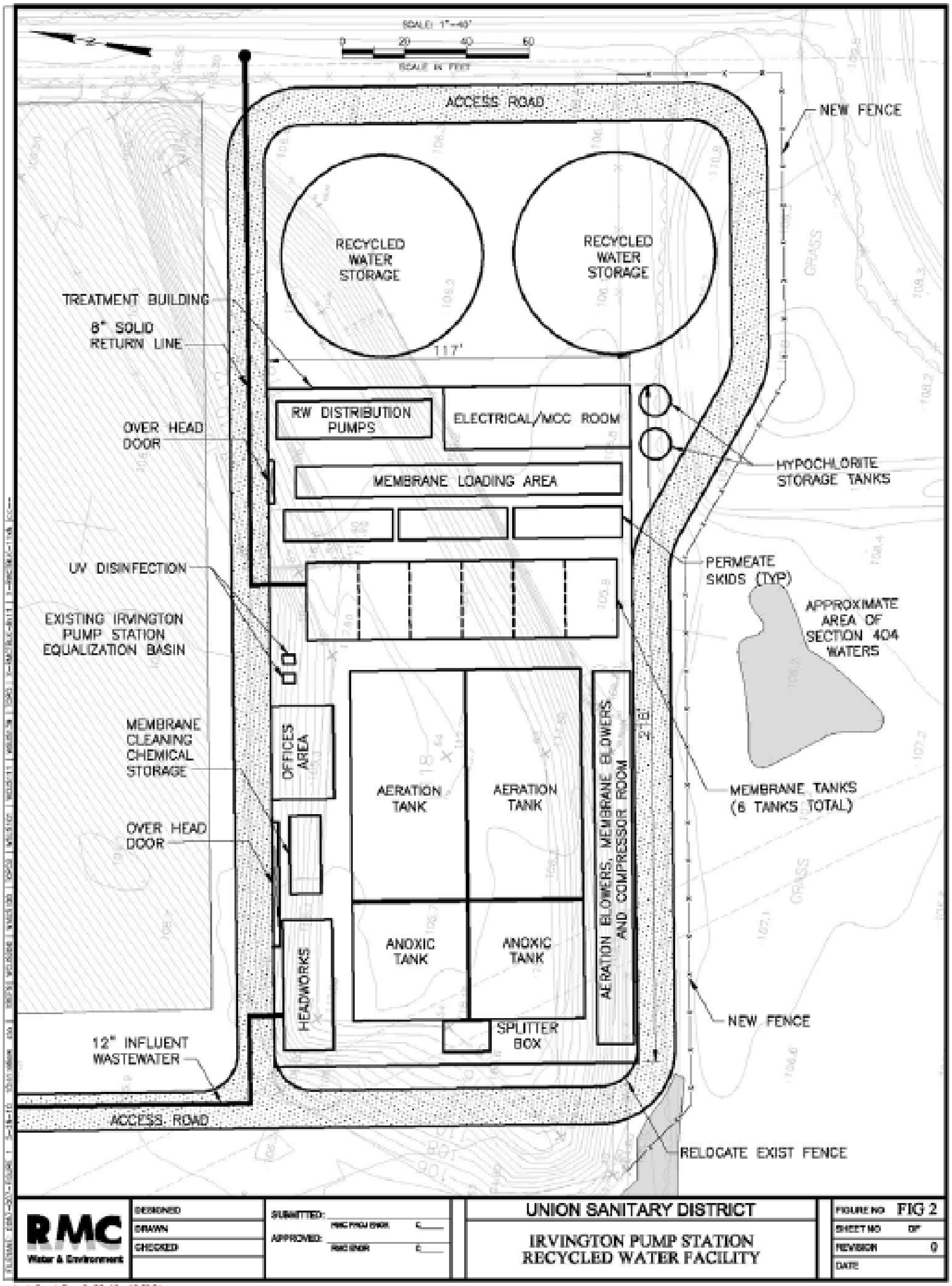
In order to further vet this option, USD would need to identify the efforts and costs needed to avoid impact to the force main when diverting influent flow into the Irvington PS, and/or to create a new influent storage basin at the Newark pump station. The identified influent storage available in the conveyance system is limited compared to the buildout storage needed (33 MG for secondary effluent storage up to 2038, and potentially beyond) so it would only provide a <u>minor</u> solution. Previous evaluation has determined that conveyance system storage is possible and is of <u>moderate</u> viability due to USD ownership of the facilities.

2.1.3 Satellite Treatment and Disposal

This option would involve treatment of USD wastewater at either Irvington or Newark Pump Stations and reuse and/or disposal of the treated effluent in that basin. This option was evaluated in the 2010 Recycled Water Feasibility Study for USD and ACWD. A satellite facility at Newark Pump Station was not recommended because (1) the project cost on a per unit of water basis was much higher compared to other available alternatives, and (2) the customer base was highly dependent on future users. A satellite facility at Irvington Pump Station serving the south end of the study area, however, was considered a preferred project at the time and is shown below in **Figure 2-3** below. The proposed facility is estimated to reduce flows downstream of the pump station by 1.7 MGD.

The 2010 Feasibility Study identified three major risks associated with the satellite treatment project: (1) sensitivity to nature and timing of future water demands, (2) uncertainties associated with obtaining and maintaining a new NPDES permit, and (3) uncertainty in influent ammonia levels that impact sizing satellite facilities. Additionally, further work on this satellite project has not been pursued due to lack of customers and demands on the south side of USD's service area. As such, this option is a <u>partial</u> solution with the limited demands and is ranked with <u>low</u> viability with the cost of the option likely not being justified by the demands.

Figure 2-3: Satellite Treatment Facility at Irvington Pump Station

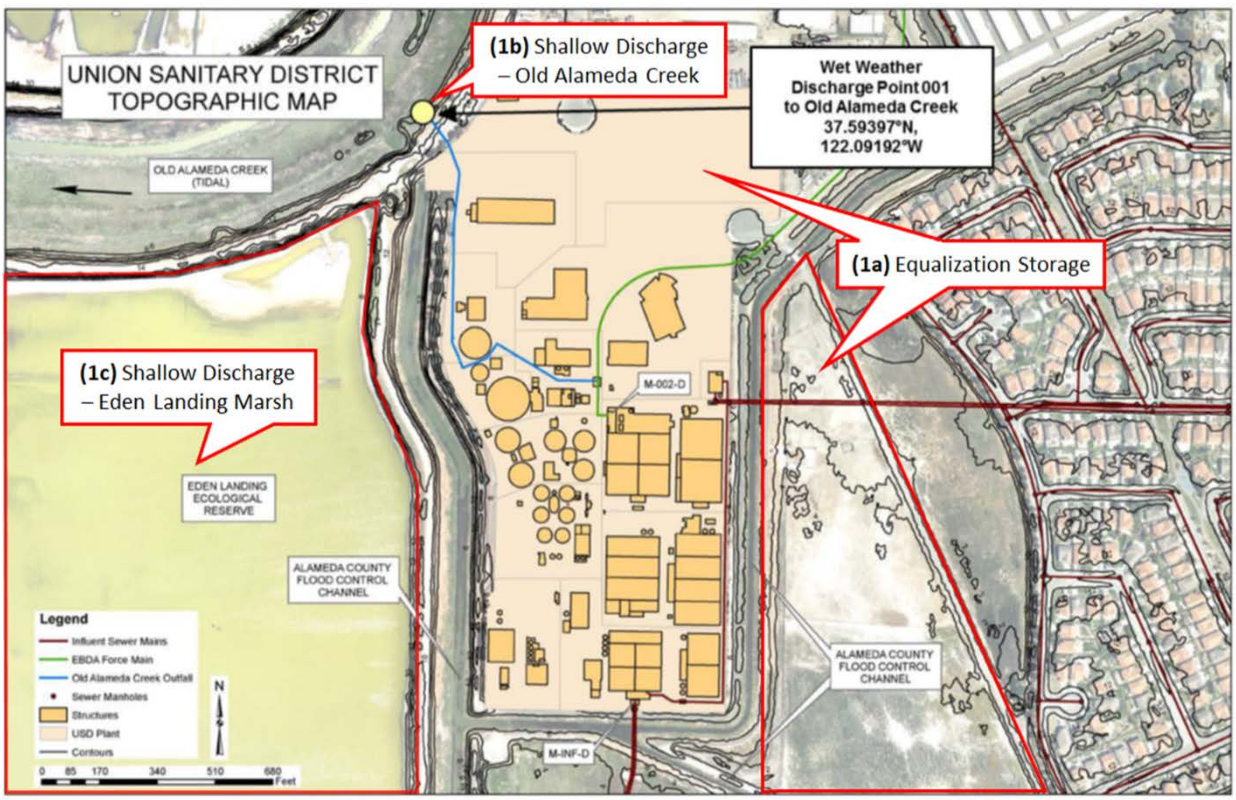


Source: ACWD and USD Recycled Water Feasibility Study Update (RMC 2010)

2.2 Wastewater Treatment Plant On-Site Options

This section describes three effluent management options that would be located onsite at the WWTP site. **Figure 2-4** below shows the location of these options relative to USD's WWTP facilities.

Figure 2-4: Locational Map of Wastewater Treatment Plant On-Site Options



Source: Base map from USD's Old Alameda Creek (Wet Weather Outfall) Permit. ORDER No. R2-2015-0045, NPDES No. CA0038733.

2.2.1 Equalization Storage (Secondary Effluent or Stormwater)

Under this option, some secondary effluent flow would be temporarily diverted into EQ storage to reduce the required discharge flow to EBDA during wet weather events. USD could create an EQ basin east of the WWTP to potentially provide a full solution, and/or create a smaller EQ basin on the north side of the WWTP for a partial solution.

2.2.1.1 New EQ Basin East of WWTP

For this equalization storage option, USD would utilize the triangular 17-acre parcel adjacent to the WWTP, which is owned by the Alameda County Flood Control District (ACFCD), to construct an equalization basin for temporary storage of secondary treated effluent. The east EQ area is highlighted in red in **Figure 2-4** above. The equalization basin could be built in several phases to match the required storage volume needed to match peak wet weather flows at that time.

Secondary effluent would be pumped to the EQ basin during peak flow events and metered back into the EBDA system after peak flows subside. WRA, Inc. (WRA) previously conducted a preliminary wetland delineation of the site in April 2016. As shown in **Figure 2-5** below, the study had classified the majority of the site as jurisdictional wetlands. As such, construction of an EQ basin on this site would require a Standard Permit from the U.S. Army Corps of Engineers (USACE), per Section 404 of the Clean Water Act. To justify the discharge of dredged or fill material into the waters of the United States, which includes jurisdictional wetlands as defined by the Environmental Protection Agency (EPA), a 404(b)(1) alternatives analysis would need to be conducted demonstrating that there are no practicable alternatives that would be less environmentally damaging. WRA will continue to advise on this option for this Effluent Management Study.

To obtain a standard 404 permit the following information would need to be developed and submitted to the USACE:

- 404 Permit application (Department of the Army ENG Form 4345), including drawings depicting the proposed plans for the project
- Biological Assessment to facilitate USACE Section 7 consultation with U.S. Fish and Wildlife Service
- Cultural Resources Assessment to facilitate USACE Section 106 consultation with State Historic Preservation Officer.
- 401 Water Quality Certification from the Regional Water Quality Control Board
- 404(b)(1) Alternatives Analysis, documenting that alternatives that would avoid wetland fill have been considered and that none of those alternatives is practicable.
- Compensatory Mitigation Plan, demonstrating how any permanent wetlands that would be permanently removed would be replaced. Mitigation could include on-site mitigation (e.g. retaining some wetland vegetation within the existing site and managing the basin to maintain wetland function), or off-site mitigation through the restoration of the treatment portions of the Hayward Marsh to a natural wetland area or through the purchase of mitigation credits.

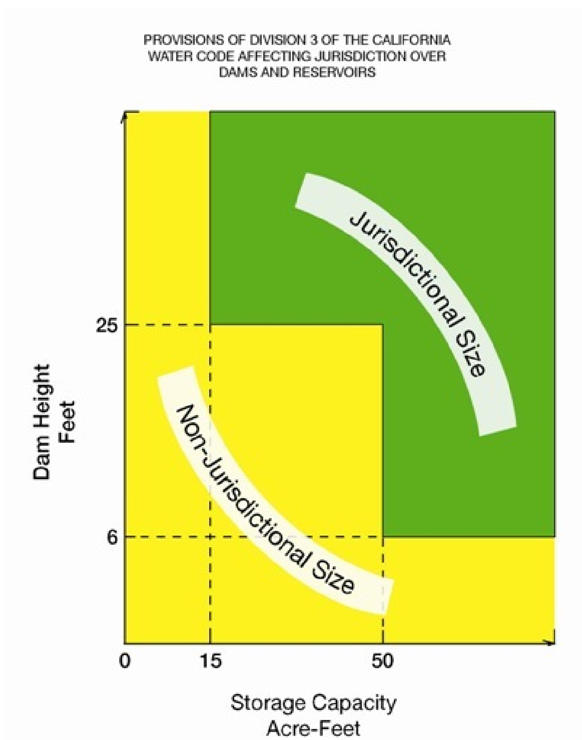
A Standard 404 Permit requires that the USACE issue a public notice and respond to any public comments regarding the project. Time to obtain a Standard Permit is usually about a year after submittal of the application, though it may be possible to streamline the process if the project can be designed to provide wetlands value within the existing site; and the process can take longer if Section 7 or Section 106 consultation is delayed or if there is public opposition.

Figure 2-5: Wetland Delineation of Potential Equalization Basin Site

Source: Delineation of Waters of the U.S. Report, Alvarado Equalization Storage Basin Project (WRA, 2016)

The equalization storage facilities should be specifically sited and designed to minimize impacts on designated wetlands. For example, the land area of the EQ basin site could decrease with consideration of a deeper equalization storage basin. Note, also, that at a certain storage capacity and at a certain depth, the sidewalls of the equalization basin may be considered a dam and subjected to additional regulations from the California Department of Water Resources' Division of Safety of Dams, as shown in **Figure 2-6** below. Based on these restrictions, the maximum storage capacity of an EQ basin that could be constructed in a 17-acre area would be 102 acre-feet without being classified as jurisdictional size (maximum dam height of 6 feet). This storage capacity is equivalent to approximately 33 MG, less than the 53.6 MG of storage anticipated to be needed during buildout 10-year storm events. Although, it achieves adequate storage to about 2038 based on current EBDA capacity.

Figure 2-6: Division of Safety of Dams Jurisdictional Classification



Source: California Department of Water Resources, Division of Safety of Dams. Available at: http://www.water.ca.gov/damsafety/jurischart/index.cfm

Furthermore, the water table level and soil conditions at the site could also affect the height of the levee and should be taken into careful consideration in the design of the equalization basin. Alameda County Water District (ACWD) has expressed concerns with volatile organic compounds in the stored effluent, whereby constituents in the effluent may contaminate the groundwater and volatilize into homes.

As described above, the project would require a Compensatory Mitigation Plan committing to mitigation projects to compensate for wetlands lost to the EQ basin footprint. Note that mitigation requirements associated with creating an impervious basin would be much greater than the mitigation requirements for creating a seasonal wetland.

The USACE may be more receptive to a project that retains wetland functions and values within the site. It was therefore recommended by Woodard & Curran and our subconsultant, WRA, that if this option were to be evaluated further, USD should present the project at one of the monthly Interagency Meetings held by USACE. USD agreed and authorized the project team to hold this introductory meeting. The results of the meeting are as follows:

- The USACE staff at the meeting were receptive to the EQ basin approaches presented and identified that the mitigation for lost habitat would be anticipated to be at least 1-acre mitigation per 1 acre of jurisdictional wetland affected, and potentially higher mitigation ratios if equally valuable habitat were not available thru a mitigation bank. Mitigation bank acreage is roughly estimated at $1 Million/acre, so the mitigation costs for a portion or all of the 17-acre site would be very high.

  The Regional Board staff member present was not amenable to the EQ basin approach presented in any form due to the perspective that all possible alternatives would have to be proven infeasible in order to affect any jurisdictional wetland acreage. It appears that the three sections at the Bay Area Regional Water Quality Control Board (Permitting, Basin Planning and Enforcement) each have different views on the value of the Hayward Marsh continuing to be permitted for secondary effluent shallow water discharge. Until those varying views are reconciled, gaining a clear regulatory message from the Regional Board on how the treatment portions of the Hayward Marsh might be used for mitigation acreage would be highly problematic. For this reason, the USD project team left the interagency meeting recognizing that the permitting of this alternative is highly uncertain at this time.

Based upon the input summarized above, this option is considered as a partial to full solution, with a potential capacity to store to 33 MG of effluent. The permitting process is expected to take several years based on initial opposition to the project received from potential permitting agencies. Due to permitting and mitigation obstacles, the viability of this option is ranked as low.

2.2.1.2 New EQ Basin on the North Side of the WWTP Site (Site Drainage)

RMC/Woodard &Curran conducted a Treatment Plant Drainage Study in 2011 which highlighted capacity constraints with the Plant's Site Waste Pump Station (SWPS). During high rainfall events, peak onsite stormwater flows stress the capacity of the SWPS and increase the risk of exceeding USD's contractual peak capacity with the EBDA system. The study identified and evaluated alternatives for storing stormwater runoff and reducing peak flows at the Plant. Peak stormwater modeling results from the Study for different return periods are summarized in **Table 2-1**. Under existing conditions, site stormwater drains offsite, while at future build-out conditions, site stormwater would drain to the Plant.

Table 2-1: Plant Peak Stormwater Flows and Volumes

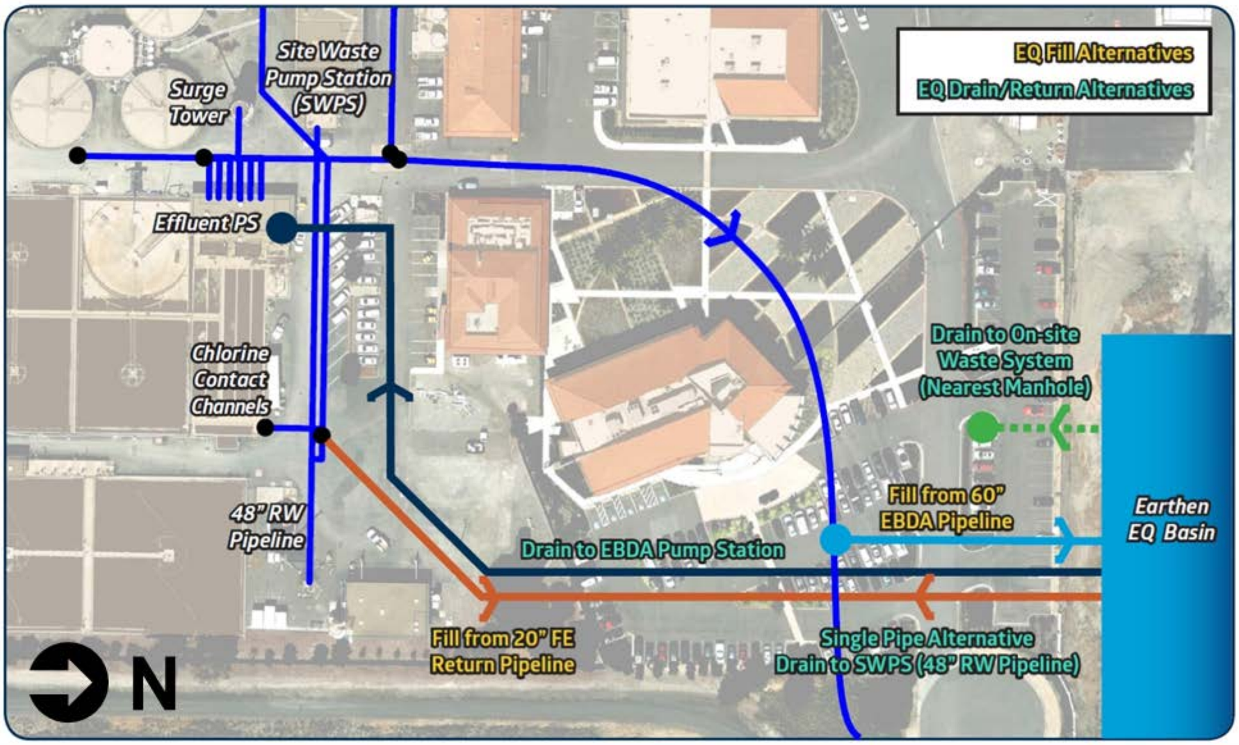
	Existing		Build-Out	
Return Period	Peak Flow (mgd)	Volume (MG)	Peak Flow (mgd)	Volume (MG)
5-Year	8.9	1.6	10.5	1.8
10-Year	10.6	1.9	12.5	2.2
15-Year	11.5	2.0	13.6	2.4
25-Year	12.7	2.2	15.0	2.6

Notes: *Flows do not include wastewater process flows to the SWPS.*
Source: USD Treatment Plant Drainage Study (RMC 2011)

Within this study, one of the preferred alternatives involved a new aboveground storage pond located on site north of the parking lot at the WWTP. However, this location may change in coordination with the Site Use Study currently being conducted by RMC/Woodard & Curran. The pond footprint was based on a storage capacity equal to the 25-year stormwater runoff volume of 2.6 MG with some amount of freeboard. The sizing of this pond in the study was based on offsetting the impact of the Plant's stormwater runoff on the discharge to the EBDA pipeline. As a result, the pond was sized to capture the entire storm runoff volume. However, additional effluent storage beyond the stormwater volume may be required in the future to maintain effluent flows within the EBDA system capacity.

The EQ basin proposed in the drainage study could be filled or drained either through direct connections to the EBDA pipeline or to existing USD-owned facilities. The former may be a less viable approach as it would require EBDA review and approval. **Figure 2-7** shows some of the EQ filling and draining alternatives. This figure shows two drainage alternatives which would involve pumping, at either EBDA's pump station or USD's SWPS.

Figure 2-7: Fill and Drain Alternatives for Onsite Storage Pond

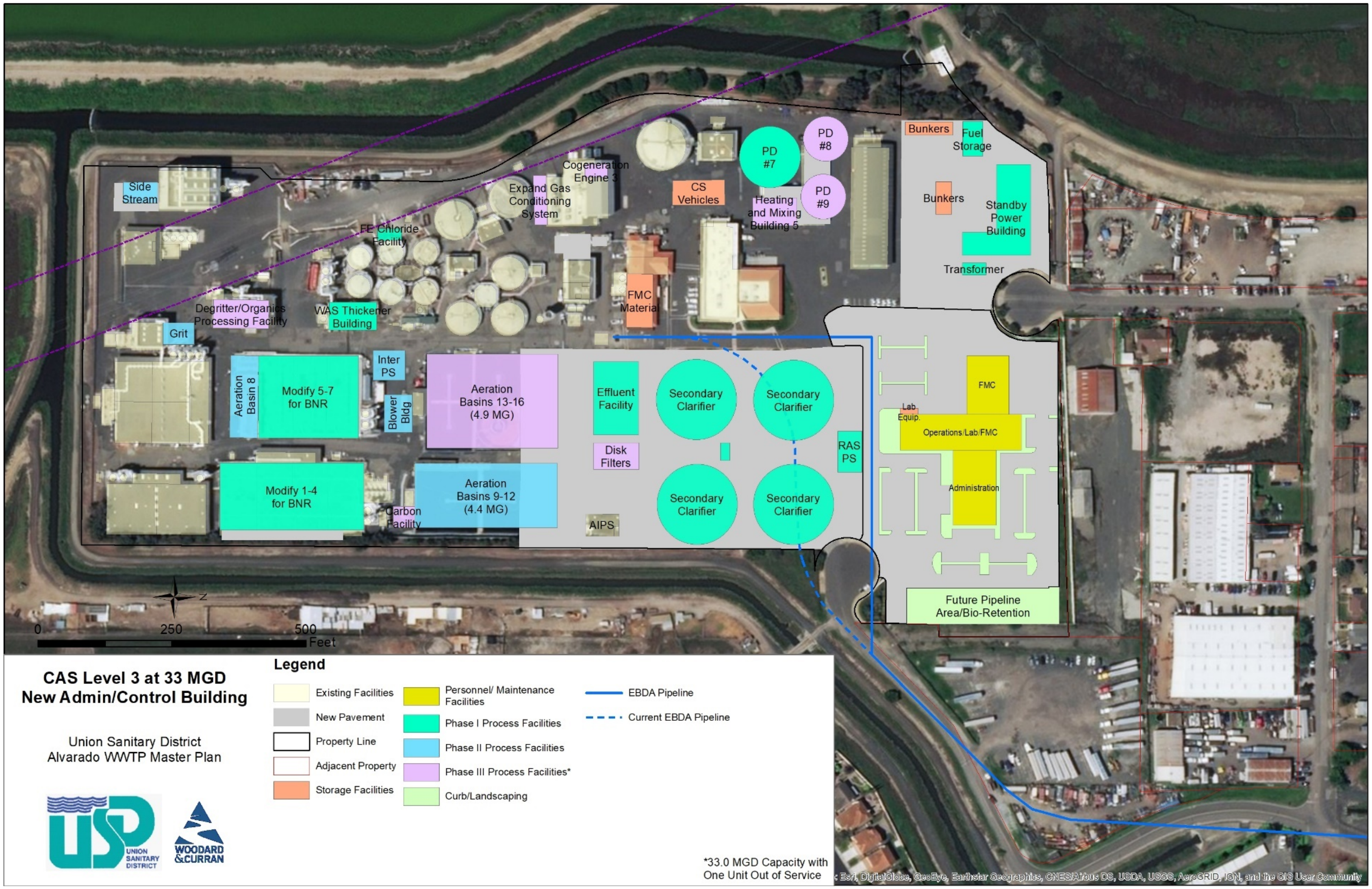


Source: Proposal for the Union Sanitary District Alvarado Equalization Storage Basin Project (RMC 2015b)

Although there is currently some open USD property to the north of existing facilities, these lands may be used for a future new plant and there is no parcel large enough to accommodate effluent equalization that is currently on the market; any such parcel would appear to require condemnation, and that would only be justified if other viable alternatives had been exhausted. **Figure 2-8** below shows the potential layout for Phase III of the Enhanced Treatment & Site Upgrade Program (Woodard & Curran 2019).

Given that this potential equalization storage option would only provide a portion of the potentially required EQ volume, it is a partial solution. Although the onsite location uses land already belonging to the District, this open area currently is reserved for higher priority facilities and upgrades, therefore the viability of this option is low. The timing for implementation of this onsite equalization basin option, however, would be shorter compared to an EQ basin outside the existing fence line, as this onsite option is both smaller and located on land already belonging to USD.

Figure 2-8: USD Plant Layout at Builout (2058)



Source: Enhanced Treatment & Site Upgrade Program (Woodard & Curran 2019)

2.2.2 Shallow Water Discharge: Old Alameda Creek

Under this option, additional effluent capacity could be obtained by increasing the permitted capacity of the Old Alameda Creek (OAC) discharge location. Old Alameda Creek currently serves as an emergency outfall during peak wet weather flow conditions, but no maximum discharge rate is specified in the permit. The previous permit order dictated a maximum discharge volume limitation of 8.4 MG per discharge event, which was the expected flow from a storm with a 20-year return frequency (i.e., a 20-year storm). According to the permit, this number was determined from the USD's 1994 District Wide Master Plan and 1999 Wastewater Equalization Storage Facilities Pre-Design. The current order replaces the discharge flow limitation with a standard prohibition against the bypass of treatment systems. For more long-term use, its discharge capacity could be increased. Some increase in treatment level at the WWTP would likely be required by the Regional Water Quality Control Board (RWQCB) to allow this increase because it is a shallow water discharge and could have more impacts on beneficial uses with increased frequency of use and increased volume of discharge. Consequently, it is anticipated that future nutrient removal WWTP improvements would be needed for the portion of flow discharge to Old Alameda Creek.

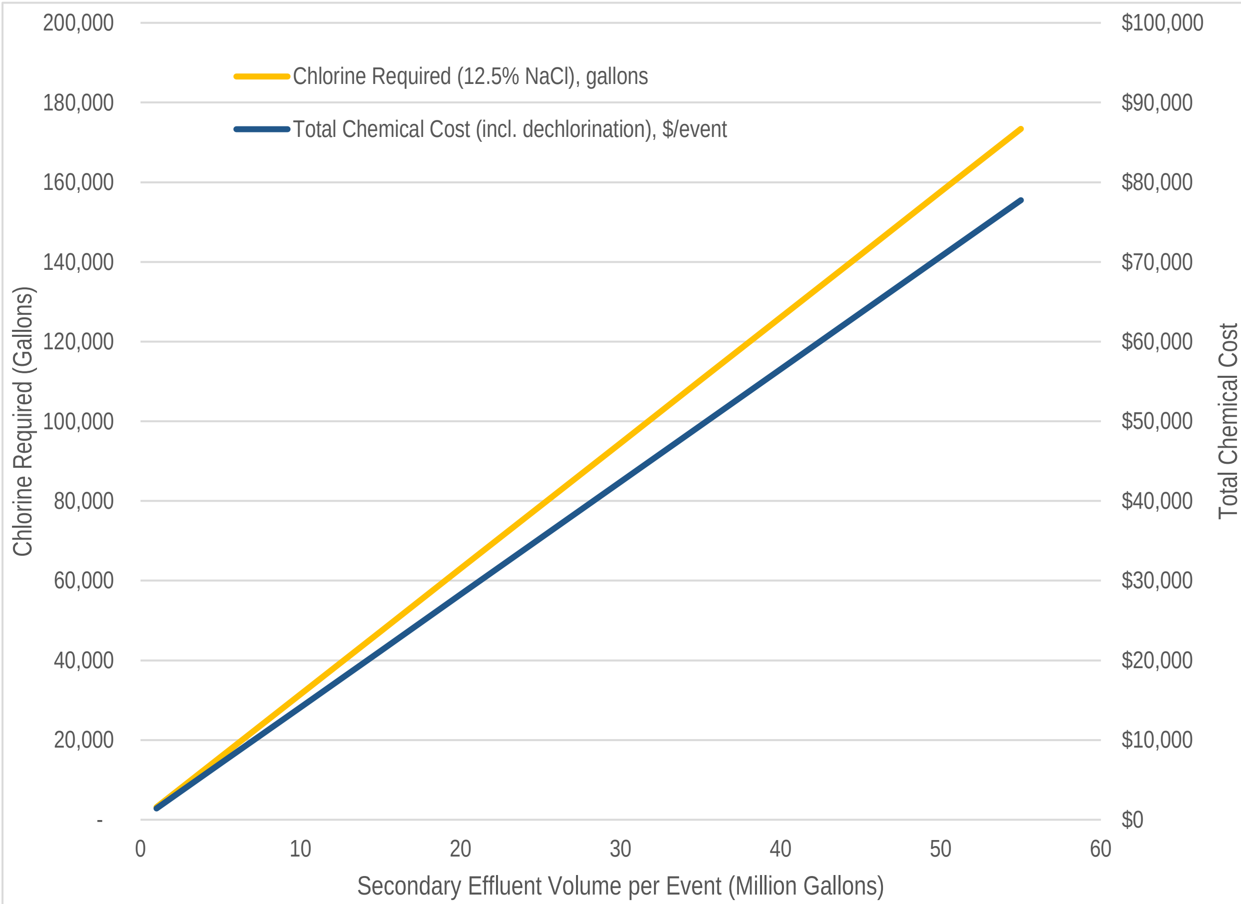
2.2.2.1 Ammonia Removal via Breakpoint Chlorination

For short duration discharges, ammonia toxicity may be the primary consideration for the RWQCB in terms of expanded use of the Old Alameda Creek outfall. A potential approach to reduce ammonia concentrations in the effluent prior to discharge would be to implement breakpoint chlorination. The use of breakpoint chlorination involves the use of chlorine to convert ammonia to nitrogen gas. The use of breakpoint chlorination is presented in Principles and Practice of *Phosphorus and Nitrogen Removal from Municipal Wastewater (The Soap and Detergent Association, September 1989)*. The overall reaction for breakpoint chlorination is expressed in the following equation.

$$NH_4^+ + 1.5\,HOCl \rightarrow 0.5\,N_2 + 1.5H_2O + 2.5H^+ + 1.5Cl^-$$

Although the stoichiometric ratio of chlorine to ammonia is 1.5 to 1, in practice, the required chlorine dose is 10 parts chlorine per part of ammonia (The Soap and Detergent Association 1989). Assuming a secondary effluent concentration of 45 mg/L NH_4-N, the required volume of chlorine for breakpoint chlorination is presented in **Figure 2-9**. Total chemical cost of chlorine (12.5% NaCl solution @ \$0.447/gal) including the cost of sodium bisulfite for dechlorination (25% NaSO3 @ \$0.97 gallon) are also presented in **Figure 2-9**.

Figure 2-9: Chlorine Volume and Chemical Cost for Breakpoint Chlorination



Although the cost of chemicals per event can be expensive, the need for breakpoint chlorination in the near-term would be relatively infrequent. However, breakpoint chlorination would require a large volume of hypochlorite to be stored on site, or onsite hypochlorite generation. Given the relatively short shelf life of sodium hypochlorite, ~2 weeks, USD may end up wasting a significant amount of hypochlorite during the wet weather season.

As an example, if USD implemented 10 MG of breakpoint chlorination capacity, approximately 30,000 gallons of additional hypochlorite storage would be required. USD currently has two 8,232-gallon hypochlorite storage tanks. Therefore, USD would need approximately three more tanks of the same size for 41,160 gallons of storage, which includes one standby tank for filling. The current estimated annual hypochlorite usage for disinfection is 780,000 gallons per year, which is equal to 2,137 gallons per day. Based on a daily use of 2,137 gallons per day, the average hypochlorite storage time with five tanks would be approximately 19.3 days, which is close to maximum desired storage time to minimize hypochlorite degradation.

Due to potential degradation, storing hypochlorite on-site in volumes in excess of 40,000 gallons is likely impractical, therefore providing breakpoint chlorination of secondary effluent volumes greater than 10 million gallons may not be a viable alternative unless hypochlorite generation is included. Either way, the generation and use of such quantities of hypochlorite and bisulfite would not be sustainable from an energy consumption and emissions standpoint.

Given the potential issues with storing large volumes of sodium hypochlorite, or undertaking the on-site generation of hypochlorite, this option is considered as a partial solution with low viability for the long term. It could be part of a short-term solution while a longer-term alternative is pursued.

2.2.2.2 Early Action Nutrient Removal

Limits on nutrients discharged from publicly owned treatment works (POTWs) to San Francisco Bay are anticipated because of growing concern for impairment of water quality in San Francisco Bay. The association of POTWs in the Bay Area, Bay Area Clean Water Agencies (BACWA), is conducting a study to analyze nutrient reduction alternatives at Bay Area POTWs. **Table 2-2** presents three different treatment levels used to bracket potential future nutrient discharge limits. These provided the basis for the evaluation of treatment options and costs in the BACWA study.

Table 2-2: BACWA Nutrient Removal Levels

Treatment Level	Study	Ammonia	Total Nitrogen	Total Phosphorus
Level 1	Optimization	--	--	--
Level 2	Upgrades	2 mg N/L	15 mg N/L	1.0 mg P/L
Level 3	Upgrades	2 mg N/L	6 mg N/L	0.3 mg P/L

Source: Potential Nutrient Reduction by Treatment Optimization and Treatment Upgrades, Scoping and Evaluation Plan (HDR & Brown and Caldwell 2014)

A draft Nutrient Reduction Study for USD's WWTP was recently made available. The report identifies Level 1 optimization strategies of adding ferric chloride upstream of the primary clarifiers to remove phosphorous, and deammonification sidestream technology for reducing nitrogen/phosphorus loads.

Based on current land availability at USD (note that the feasibility of purchasing additional land is being evaluated as part of the Enhanced Treatment & Site Upgrade Program), the only feasible nutrient removal technology found in this limited study and recommended would be membrane bioreactors for meeting Levels 2 and 3. Costs for these process improvements range up to $610 million for Level 3 wet season upgrades (HDR & Brown and Caldwell 2016). Level 2 and 3 recommendations for the entire plant flow are listed below:

- Level 2:
    - Construct chemical facilities for ferric chloride addition upstream of primary clarifiers,
    - Convert the secondary process to a membrane bioreactor process. Convert existing aeration basins and three of the existing secondary clarifiers to MLE aeration tanks.

Construct new membrane tanks. Construct fine screening to protect membranes. Construct facilities for methanol and alkalinity addition.

- Level 3
  - Same as Level 2, plus
  - Add additional ferric chloride after the aeration basins for phosphorus polishing.
  - Convert three additional existing secondary clarifiers (six total) to 4-stage BNR and configure all tanks as 4-stage BNR. Add additional methanol for denitrification.

Per a more recent evaluation, USD would be able to reduce ammonia and total inorganic nitrogen (TIN) levels to near “Level 2” nutrient benchmark through a year-round BNR process with the implementation of Secondary Treatment Process Improvements Phase I (Hazen and Sawyer, August 2019). This could potentially allow for increasing shallow water discharges during wet weather to be transitioned proportionately from the Hayward Marsh, where ammonia removal occurs within the Marsh, to Old Alameda Creek where there is no ammonia removal, but some dilution. The “early action” element of Phase I would be used by USD to provide the basis for a request to the Regional Board for more time to meet future nutrient limits than the agencies within the same sub-embayment who do not implement “early action”.

Given the potential discharge capacity available at the outfall, this option is considered as a <u>partial</u> solution for sidestream treatment only and a <u>full</u> solution for year-round, full flow BNR. Due to the regulatory requirements and a 5 to 7 year implementation schedule it is ranked as <u>moderate</u> viability for sidestream treatment and <u>high</u> viability for year-round, full flow BNR.

2.2.2.3 Continuous Discharge

If the District were to pursue a continuous discharge to Old Alameda Creek, additional regulatory requirements would apply. The requirements associated with continuous shallow water discharge were previously evaluated by RMC in the USD Regulatory Requirements for Continuous Shallow Water Discharge TM (2015c). Discharge Prohibition 1 of the Basin Plan prohibits the discharge of any wastewater that does not receive a minimum initial dilution of at least 10:1 (i.e., shallow water discharge) or discharges to dead-end sloughs. Section 4.2 of the Basin Plan provides for exceptions to this prohibition only under certain circumstances, with the following ones being potentially applicable in this case:

1. An inordinate burden would be placed on the Discharger relative to the beneficial uses protected, and an equivalent level of environmental protection can be achieved by alternate means;
2. A discharge is approved as part of a reclamation project; or
3. Net environmental benefits will be derived because of the discharge

In USD’s case, qualification for the first exception would require evidence demonstrating that continued participation in EBDA is an “inordinate burden”. Qualification for the second exception would likely require coordination with ACWD and is expected to be capital-intensive and with timing outside of USD's control. Qualification for the third exception is rarely granted but could potentially be done by creating brackish or freshwater marsh habitat.

In addition to the efforts necessary to qualify for shallow water discharge, obtaining and maintaining the NPDES permit for discharge would also require significant time and effort. Special studies which may span several years are needed to complete the application, including an Antidegradation Analysis, a Dilution Study, and a Mixing Zone Analysis. An Environmental Review would also likely be needed for modification of the District's outfall facilities. Furthermore, maintaining the NPDES permit would require the District to perform additional sampling and water quality analyses. Effluent and receiving water would likely require more frequent monitoring, and the District would likely need to install an online monitoring system for hourly monitoring of chlorine residual. New monitoring requirements are expected to be most costly for whole effluent acute and/or chronic toxicity monitoring, which carry an estimated cost of $60,000-120,000 per five-year permit cycle. The permit renewal effort is also significantly greater for shallow water dischargers compared to deep water dischargers. Such permits often require additional professional services and District staff time, not only at the time of permit reissuance but throughout the course of the permit term.

Given the potential discharge capacity available at the outfall, this option is considered as a partial to full solution. Due to the additional regulatory requirements and the unknown but costly facility upgrades required for this option, it is ranked as low viability.

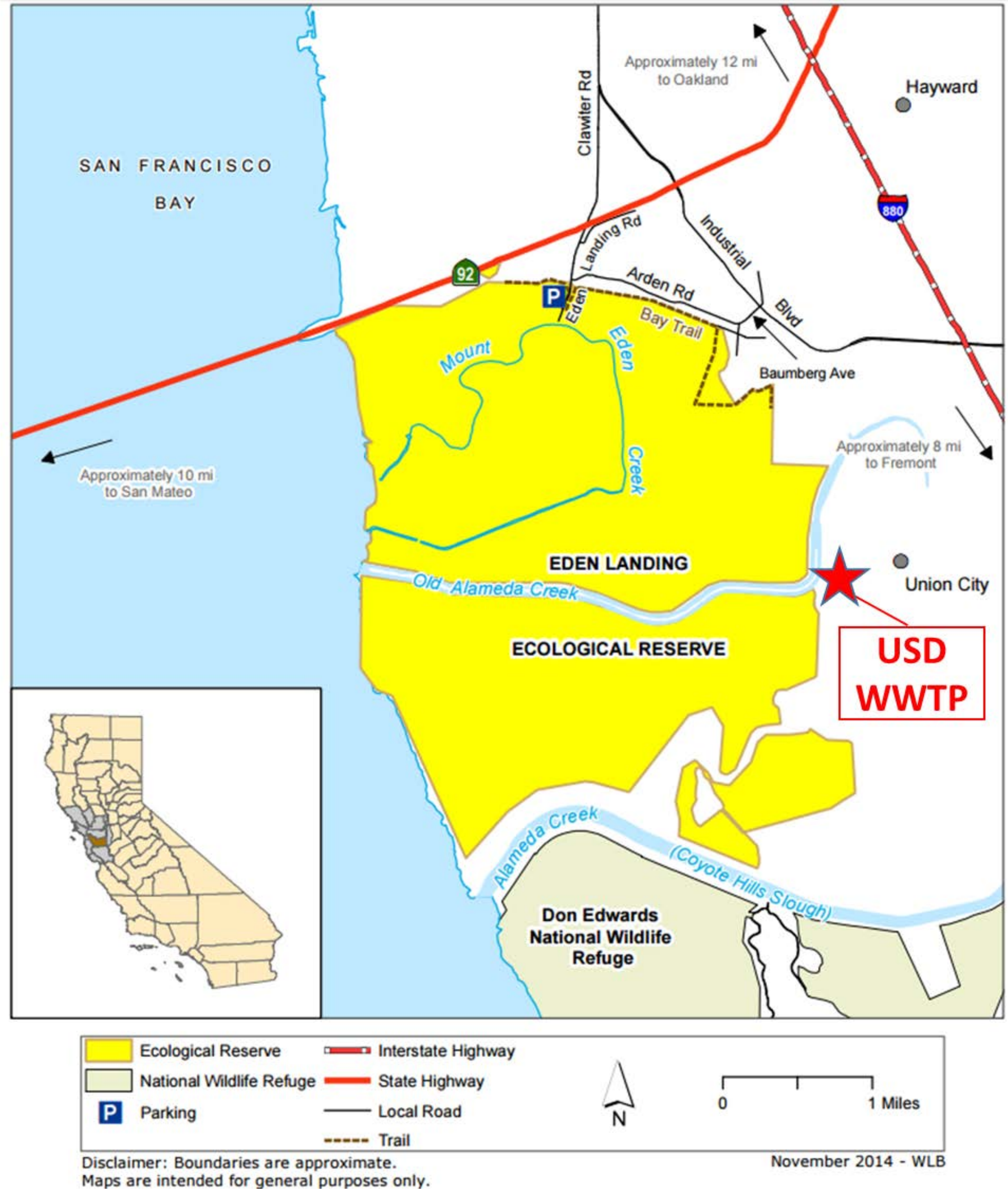
2.2.3 Shallow Water Discharge: Eden Landing Marsh

As the California State Coastal Conservancy (SCC) continues to implement salt marsh restoration in the South Bay and pursues wetland restoration in the vicinity of USD, the District may be able to create a wetland with a shallow water discharge project with the SCC. **Figure 2-10** shows a map of the Eden Landing Ecological Reserve with reference to the WWTP location. Unlike Hayward Marsh, this option is not a wet weather discharge option. Only a small amount of flow would be provided on an annual basis to SCC for vegetation and irrigation of the marsh. Oro Loma recently implemented a similar project where utilizing treated wastewater for marsh restoration will be studied. However, the results of the pilot study will not be available for several years.

Due to the lengthy regulatory and negotiation process anticipated, implementing this solution may not be possible prior to discontinuing discharge to the Hayward Marsh. While this option would face similar regulatory hurdles as those described, particularly for the Old Alameda Creek option above, it would likely require fewer near-term facility improvements because the wetland would provide a degree of additional effluent treatment or water quality enhancement.

Given the option's implementation timeline and the limited discharge capacity of the option, it is considered a partial solution with low viability.

Figure 2-10: Map of Eden Landing Ecological Reserve

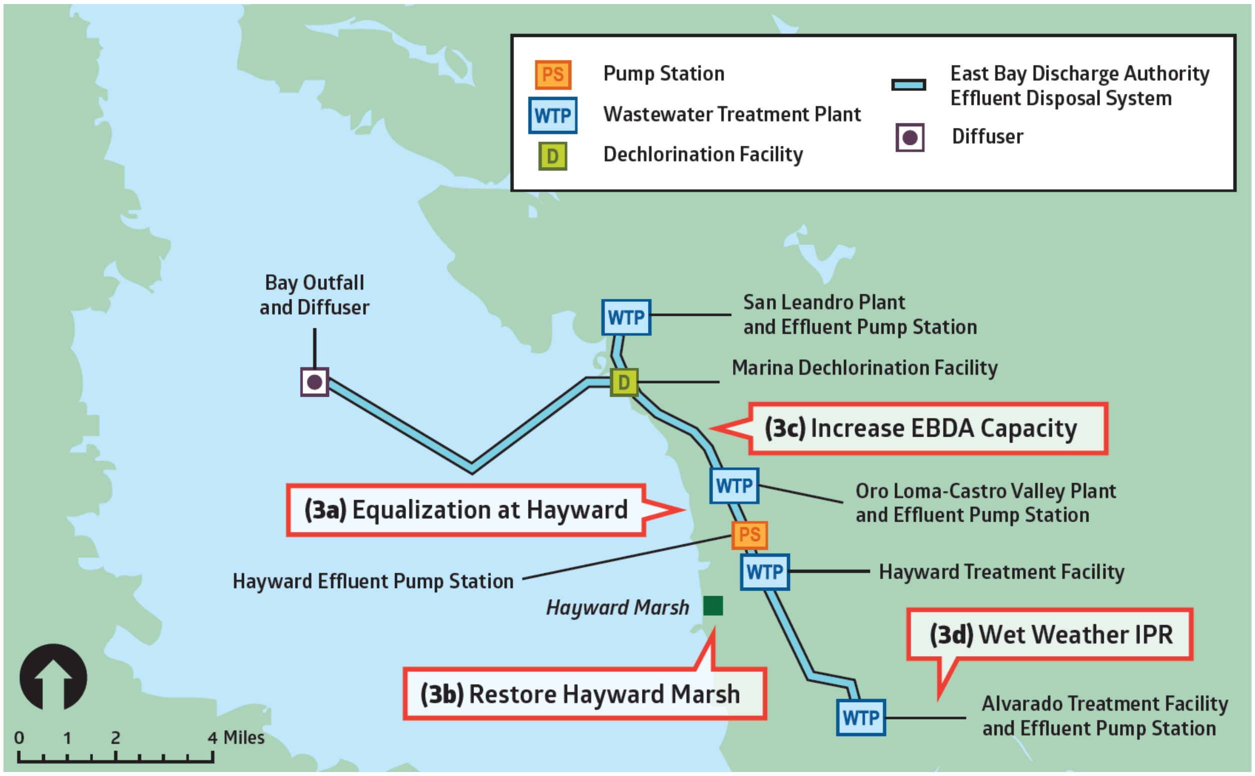


Source: California Department of Fish and Wildlife (2014)
Base map available at https://nrm.dfg.ca.gov/FileHandler.ashx?DocumentID=85029&inline

2.3 Off-Site Options

This section describes four effluent management options that would be located offsite from the WWTP site and the USD collection system. **Figure 2-11** below shows the location of these options relative to the EBDA system.

Figure 2-11: Locational Map of Off-Site Options



Source: Base map from EBDA Permit. ORDER No. R2-2017-0016, NPDES No. CA0037869.

2.3.1 Off-Site EQ Storage using Hayward Ponds

Under this option, storage could be developed at the City of Hayward oxidation ponds. The ponds' total available storage volume available is approximately 200 MG, which is more than sufficient for this option (Carollo 2011). Stored secondary effluent would be metered back into the EBDA system after a peak flow event. Use of oxidation ponds 3 and 4 was previously evaluated in the *Hayward Marsh Rehabilitation Options Study,* as shown in **Figure 2-12** below.

Figure 2-12: Hayward Ponds Site Map

Source: Hayward Marsh Rehabilitation Options Study (RMC 2015d). Base map from Google Earth.

According to the study, approximately 50 acres of existing clay-bottom ponds near the Hayward wastewater treatment plant could be converted to storage basins for USD's use. Given the amount of storage potentially available, a minimal effort would be made to regrade the existing pond bottom.

New facilities would include a 48-inch diameter equalization diversion and return pipeline to connect to the 60-inch diameter EBDA pipeline on the east side of the oxidation ponds, isolation valves, a flow metering flume, and a 10 MGD equalization return pump station (RMC 2015d). Based on previous work done during the Hayward Marsh Rehabilitation Options Study, the estimated capital cost for this option is $10.4M.

Alternatively, similar permutations of this option can also be considered. For example, flows may be pumped from USD and the City of Hayward into shared storage, with stored water repumped via a new pump station and wet well at Hayward. These options depend upon other agencies within EBDA (foremost, Hayward) to collaborate to make this project feasible and to implement it in a timely manner. Although this option has the potential to be a partial to full solution given that over 200 MG storage is potentially available at the ponds, its viability is ranked as low due to the anticipated level of effort associated with coordinating with other agencies who may not have the same level of urgency as USD.

2.3.2 Hayward Marsh Options

The current NPDES permit for Hayward Marsh (Order No. R2-2011-0058, NPDES No. CA 0038636) allows USD to discharge up to 20 MGD of its treated wastewater to the Hayward Marsh when EBDA is at capacity. The East Bay Regional Park District (EBRPD) owns and operates the Hayward Marsh. Treatment Basins 1, 2A, and 2B are three freshwater marsh basins where USD discharges its wastewater effluent during wet weather. The marsh acts as part of the treatment process, where biotic transformation, sorption, and volatilization further reduce pollutant loads. Basins 3A and 3B on the western side of the marsh are two brackish water basins considered as receiving waters along with the San Francisco Bay. Operational difficulties have led the EBPRD to explore full marsh restoration or discontinuing effluent discharges to the Marsh. If restoration were chosen, the EBRPD and USD would partner to complete the necessary improvements.

The operational difficulties include a variety of vector and avian management challenges in the freshwater basins associated with USD's discharge. Additionally, the marsh does not always reliably achieve sufficient water quality enhancement to maximize downstream beneficial uses. Rehabilitation options and their costs were previously evaluated in the Hayward Marsh Rehabilitation Options Study (RMC 2015d).

2.3.2.1 Baseline Restoration

The baseline restoration would include dredging existing channels, levee repair and maintenance, and island modification, and to return the marsh to the original design condition. This would also provide operational improvements and habitat enhancement. This reflects the bare minimum improvements to restore the entire Hayward Marsh and estimated to cost approximately $20 million. As indicated in **Table 1-3**, this option could reduce the effluent storage required from a maximum value of 62 MG (or less) to 2.2 MG. A diagram of Hayward Marsh and the baseline restoration option is shown in **Figure 2-13**. With increasing flows anticipated in the future, the Baseline Hayward Marsh restoration option as an effluent management option would be a partial solution and low viability due to the cost of restoration and dependency on partnerships with other agencies.

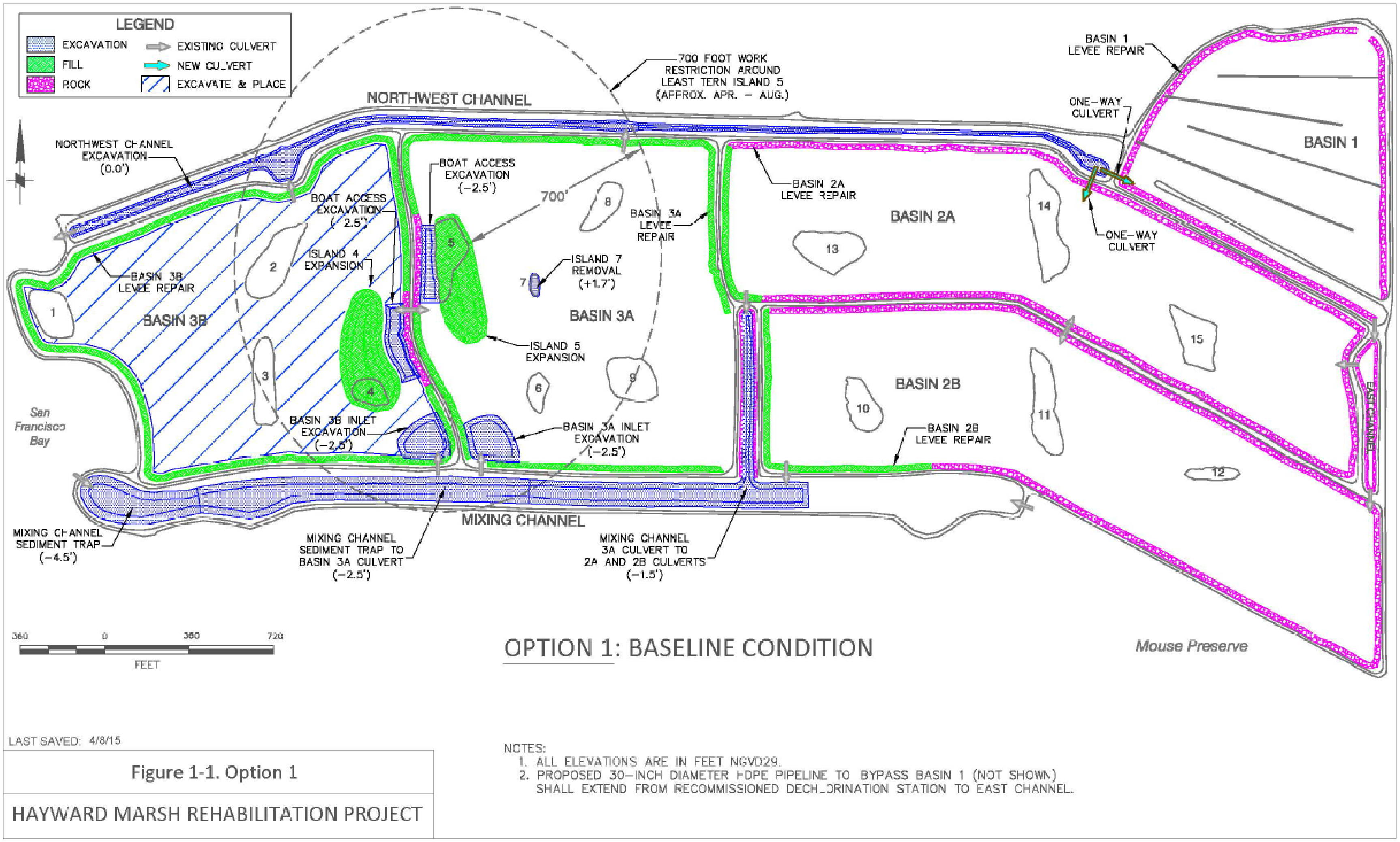
2.3.2.2 Convert Basin 1 to Equalization Storage

A less costly $15 million option was also evaluated in the study which would eliminate freshwater flow in the marsh. This option would provide muted tidal exchange for Basins 3A and 3B as well as for Basins 2A and 2B for the purpose of avian bird health. Basin 1 would be converted to a 30 MG equalization storage for USD's treated wastewater during wet weather, which provides adequate storage to about 2037 based on current EBDA capacity.

USD would construct a pumping station in Basin 1 to return wastewater to the EBDA pipeline. However, of the $15 million estimated cost, about half ($7.2 million) was related to the conversion of Basin 1 to an equalization basin. The remainder was associated with the limited restoration of the marsh for tidal exchange. This simplified option is presented in **Figure 2-14**.

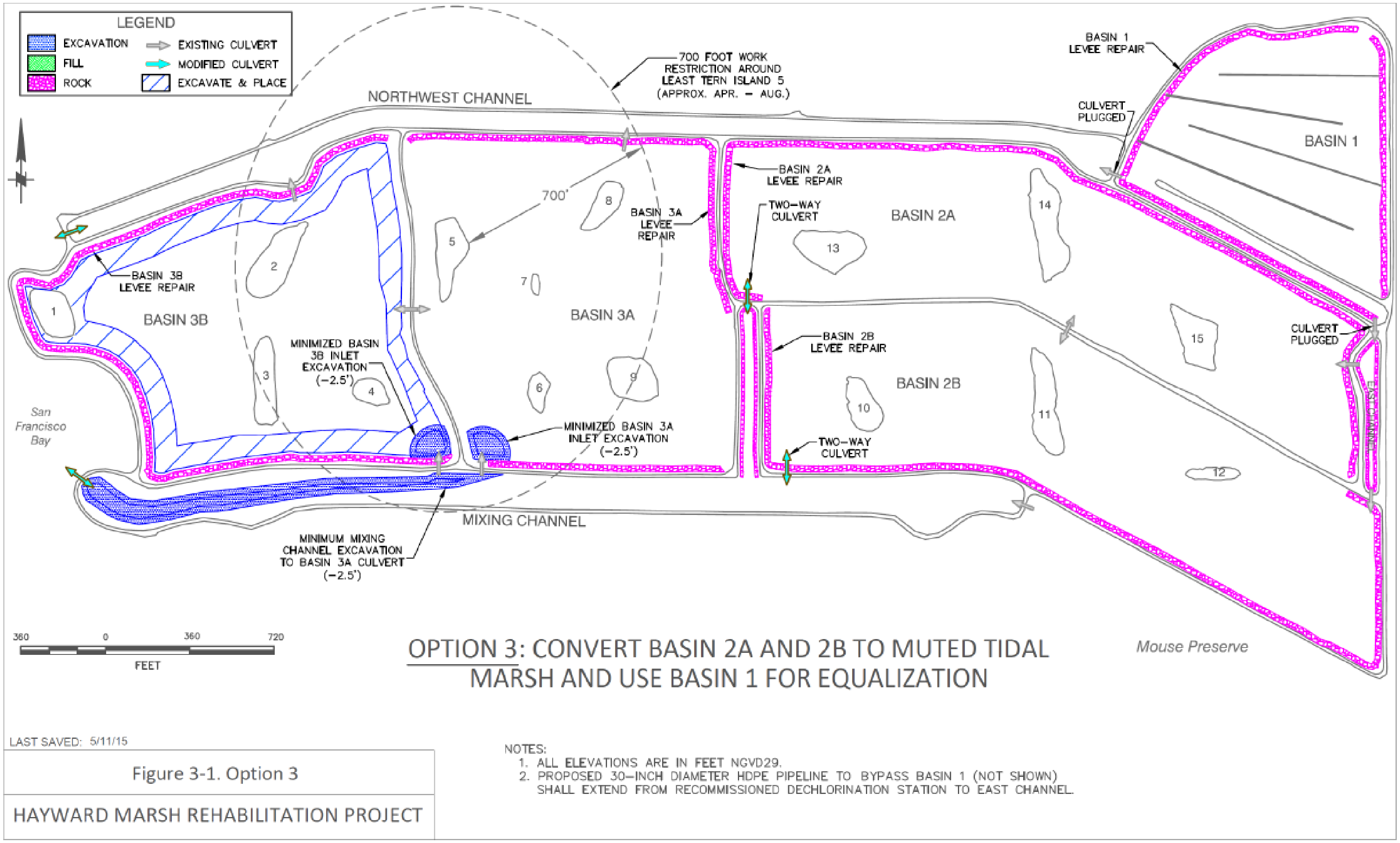
It may be in USD's interest to retain its NPDES permit for Hayward Marsh, maintaining its ability to discharge to that site as long as possible. In either case, in the long term, USD would reduce its dependency on Hayward Marsh, gradually decrease effluent flows into the marsh by implementing other discharge options. With increasing flows anticipated in the future, these Hayward Marsh restoration options as effluent management options would be a partial solution and low viability due to the following challenges: ongoing management, operations and maintenance of the marsh, clarifying the long-term objectives with EBRPD and resource agency stakeholders, and collaboration with multiple agencies whose objectives for the marsh have varied from those of USD. However, the Hayward Marsh option to convert Basin 1 to EQ storage would be the most feasible sub-option since the basin already exists and minimal wetland losses would be associated with such a project. Collaboration with EBRPD and other agencies would still be required for the Basin 1 EQ option.

Figure 2-13: Hayward Marsh Baseline Restoration Option



Source: Hayward Marsh Rehabilitation Options Study (RMC 2015d)

Figure 2-14: Hayward Marsh Basin 1 Equalization Option



Source: Hayward Marsh Rehabilitation Options Study (RMC 2015d)

2.3.2.3 Reconfiguration of Hayward Marsh

Regardless of USD's future participation in the Hayward Marsh, EBRPD is currently evaluating long-term options for the marsh. Recently, EBRPD purchased additional land adjacent to the east side of the Hayward Marsh. One potential option that is being considered is the construction of a multi-benefit ecotone slope project that would provide, habitat, shoreline resilience, and water quality benefits. A similar project was recently implemented by Oro Loma Sanitary District as described in Section 2.2.3. The project at Oro Loma Sanitary District receives a small amount of treated (for ammonia removal) secondary wastewater effluent that flows from a storage pond through a seepage berm. Currently, the treated wastewater that flows through the seepage berm is collected and returned to the treatment plant. Long-term, the goal is to allow the seepage to flow directly into the bay.

If EBRPD decides to pursue an ecotone slope project, it may be possible to incorporate continued effluent discharge from USD. EBRPD would like to convert one of the existing freshwater treatment cells, Basin No. 2B, into a tidally influenced cell. If USD continues to discharge to the marsh, this would potentially require meeting the current Hayward Marsh NPDES permit effluent total ammonia limits with just flow through Basin Nos. 1 and 2A. Woodard & Curran was tasked with evaluating whether this proposed configuration is feasible and if adding aeration to Basin Nos. 1 and 2A would be sufficient to reduce the total ammonia levels to the target anticipated in the upcoming NPDES re-negotiated permit. A technical memorandum documenting this analysis is included as **Appendix A**: Hayward Marsh Reconfiguration – Ammonia Reduction Projection Technical Memorandum.

Based upon the review of the historic performance of the Hayward Marsh, Basin Nos. 1, 2A and 2B, are currently reducing the total ammonia concentration during warm weather conditions to nearly 1 mg/L. There may be an opportunity to push the entire flow through Basin Nos. 1 and 2A in the proposed configuration. However, because it is difficult to model all of the ammonia reduction pathways taking place and therefore the impact of reduced retention time on those pathways, more field data would be needed to predict this with certainty.

In order to achieve consistent ammonia reduction within a pond system, complete mixing is recommended to keep the biomass in suspension and promote adequate nitrifier growth. A partially mixed system could result in significant zones of low DO reducing the overall nitrification efficiency and ammonia reduction. Another factor that can inhibit nitrifier growth in a pond system is the low food to microorganism (F:M) ratio. In a post-secondary treatment pond system, the F:M ratios are low; in a partially mixed system, there is the added difficulty of insufficient opportunities for food and microorganisms to come into contact.

Based on these factors and on discussions with aerator manufacturers, the best option for a complete mix system is using diffused air, comparable to a more conventional aeration basin. However, the shallow depths of Basin No. 1 and 2A preclude the application of diffused aeration equipment. Mechanical surface aerators are the only option for the physical characteristics of these basins, but in order to get complete mixing with surface aeration, it would require a significant number of aerators (fifteen 60-HP aerators) that would cost over $1M purchase and to install. The energy cost to operate in this mode will be substantial. While it would be expected that the total ammonia would be reduced over and above that which is currently happening, the manufacturers would offer no guarantee of meeting the target value, especially in the winter months.

Alternatively, a nitrification filter bed could be constructed at the inlet to Basin No. 1 or 2A. Literature suggests that adding an attached-growth media (such as a nitrification filter bed) to the pond system could yield additional ammonia reduction (Cites, Middlebrooks, Bastien, and Reed, 2014). The media, approximately 1 to 2 feet in depth, provides a surface for the nitrifiers to grow (improving the food to microorganism ratio) as well as greatly increases the uniformity with which dissolved oxygen is added into the entire flow of the system, improving the mixing conditions. Wetland effluent is recycled back to the filter bed with a recycle ratio determined based on maintaining oxygenation throughout the profile of the filter bed. There have been successful installations of nitrification filter beds in at least 3 other free water surface wetlands in the U.S. that resulted in effluent concentrations of total ammonia between 0 to 6 mg/L-N (starting from an influent of 20 mg/L-N) even in winter conditions (Crites, Middlebrooks, Bastien, and Reed, 2014).

Ideally, if a satisfactory level of ammonia removal can be achieved during warm weather, USD may be able to continue the existing wet weather operation of the Hayward Marsh, which would maintain USD's existing hydraulic capacity in the EBDA system.

Based on the preliminary analysis, there is the potential for a reconfigured Hayward Marsh to provide ammonia removal up to the current performance. Implementation of this alternative would require on-going coordination with EBRPD, including defining the infrastructure modifications required. Additional analysis would be required to further develop/refine ammonia removal estimates from a reconfigured Hayward Marsh. A scope of work for reconfiguring the marsh has not been defined, therefore for the purposes of this study it is estimated that implementing this reconfiguration would have a capital cost similar to the cost of converting Basin 1 to EQ, which is $15 million, which would be shared between USD and EBRPD. Reconfiguring the Hayward Marsh would be a <u>partial</u> solution and <u>low</u> viability due to the following challenges: ongoing management, operations and maintenance of the marsh, clarifying the long-term objectives with EBRPD and resource agency stakeholders, and collaboration with multiple agencies whose objectives for the marsh have varied from those of USD.

2.3.3 Increase EBDA Hydraulic Capacity

2.3.3.1 Acquire Additional EBDA Capacity

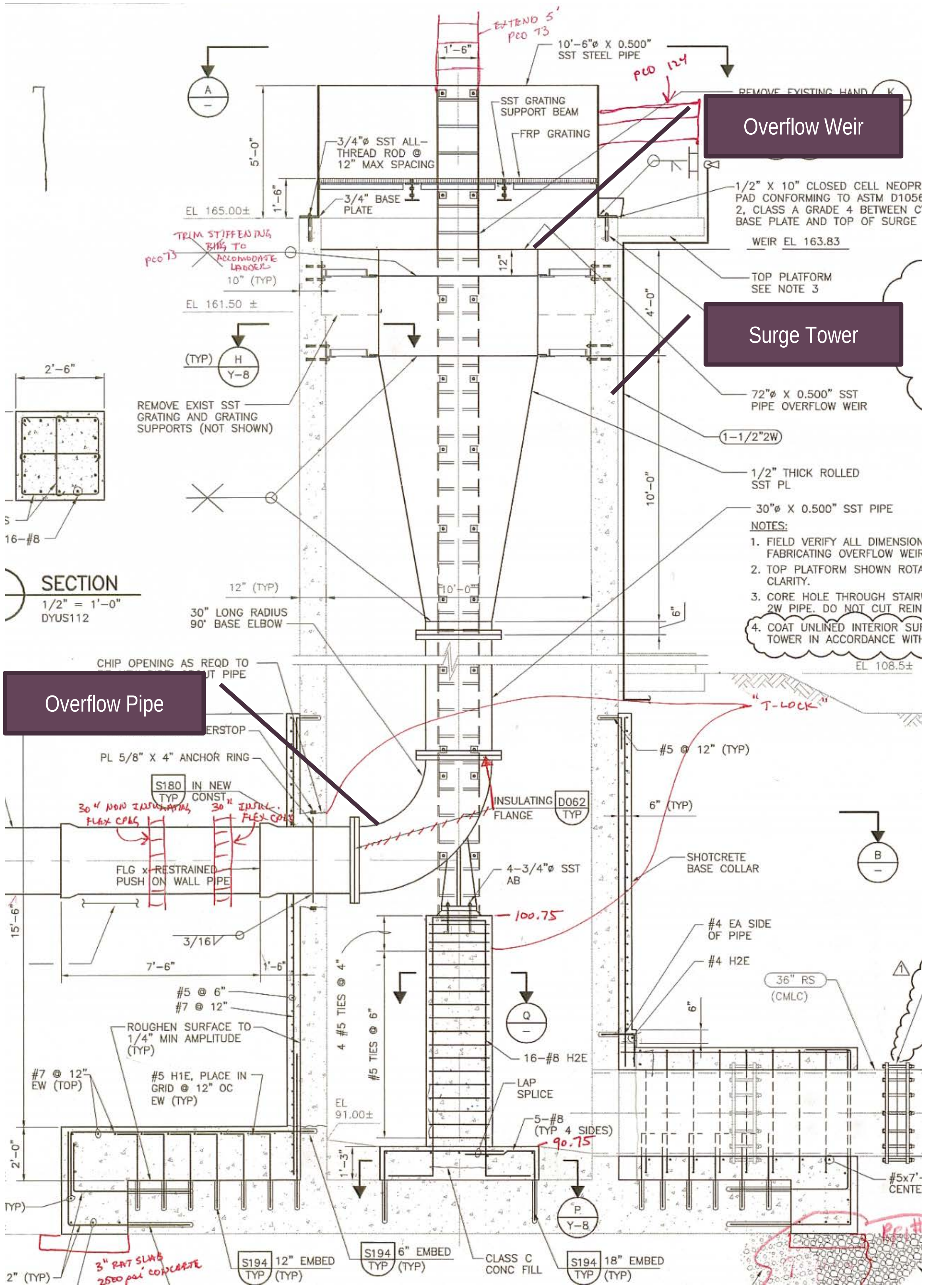
Under this option, USD would expand its permitted discharge to the EBDA system, either through an increase of EBDA's hydraulic capacity by making infrastructure improvements or coordinating with other EBDA agencies to purchase additional discharge capacity. EBDA is a Joint Exercise of Power Agency (JEPA) and the joint agreement under which it operates is set to be renewed in 2020. If additional capacity were allocated to or removed from any agency, EBDA would be responsible for any additional flow monitoring and modeling necessary to assist in making these potential changes to the system. Because of the hydraulic limitations in the EBDA system, with regard to capacity at AEPS, acquiring additional EBDA capacity would be a partial solution with low viability due to the coordination required amongst the EBDA agencies many of which are facing similar wet weather capacity challenges. Equalization at the Hayward Ponds, in coordination with the City of Hayward, appears to be the best method to accomplish an effective increase in EBDA capacity for those 2 agencies.

2.3.3.1.1 Infrastructure Improvements

Implementation of infrastructure improvements to the EBDA system could enable an increase in the EBDA system's hydraulic capacity. This project would rely upon work done recently by EBDA on its system capacity. It is anticipated that this option(s) would involve a major investment in infrastructure and rely on extensive collaboration with multiple EBDA partner agencies.

For example, the EBDA System Flow Master Plan identified an upgrade to the OLEPS firm capacity as a major infrastructure improvement needed. As discussed previously in **Section 1.1.1**, the hydraulic modeling results predicted that peak flows influent to OLEPS could reach as high as 134 MGD, indicating a firm capacity deficiency of nearly 20 MGD. The upgrades, estimated to cost $10 million, would particularly benefit LAVWMA and San Leandro by providing additional hydraulic capacity downstream of the force main (Carollo 2011). The benefits to USD and the AEPS from this project would be restricted to an improved ability to keep the wet well at OLEPS at a lower operating level. As discussed in Section 1.2, the hydraulic capacity from AEPS is affected by the OLEPS wet well elevation. Another infrastructure issue that currently affects the effective EBDA capacity from AEPS is the surge tower configuration. Currently, there is a gravity diversion to Old Alameda Creek. which is intended for use in the event the EBDA system is at capacity. Discharge to the Old Alameda Creek outfall is permitted for emergency use when EBDA capacity is exceeded. Due to hydraulic variations in the EBDA system, the water level in the surge tower fluctuates up and down continuously; therefore, USD typically operates the AEPS to maintain a 2 to 4-foot buffer between the operating water level and the spillway elevation. The buffer prevents flow from spilling over the top of the surge tower during non-emergency conditions. Operating AEPS with the buffer essentially reduces the operational hydraulic capacity. It may be possible to reconfigure the surge tower to provide additional height so that AEPS can operate closer to its maximum capacity, while still providing a buffer between the operating elevation and the spillway elevation. However, based on discussions with USD staff, there is some concern that raising the surge tower height may have unanticipated consequences on the EBDA pipeline; therefore, this option is not being considered further at this time. Options to maximize the operation of the surge tower should be considered if the AEPS is rehabilitated or relocated in the future. For example, a passive overflow pipe could be installed in the surge tower, to alleviate concerns with having an operating water surface too close to the top of the surge tower. USD has previously implemented this approach at the Irvington Pump Station. The configuration at the Irvington Pump Station is shown in Figure 2-15. Eliminating the need for a surge tower operational buffer would help USD maximize the capacity of the AEPS to pump into the EBDA system and thereby help to minimize the required effluent storage volumes.

Figure 2-15: Example Surge Tower Passive Overflow at Irvington Pump Station



2.3.3.1.2 Coordination with EBDA Agencies to Purchase Capacity

As an alternative to increasing the EBDA system's total capacity, USD may also purchase additional capacity from other EBDA agencies. Infrastructure improvements, in this case, would be more limited compared to that needed for increasing the entire EBDA system's capacity, but coordination with the other agencies may be a challenge. Note that USD is the first agency to discharge upstream into the EBDA system. All other EBDA agencies discharge downstream of USD and may potentially be affected by USD's discharge to EBDA, as demonstrated in **Figure 1-2**.

Based on the results of the recent hydraulic modeling of the EBDA system, even if USD purchases additional capacity from other EBDA agencies, the existing hydraulic constraints of the EBDA pipeline would prevent USD from taking full advantage of additional allocated capacity from EBDA. Therefore, USD has decided not to pursue the purchase of additional capacity from the EBDA system at the current time.

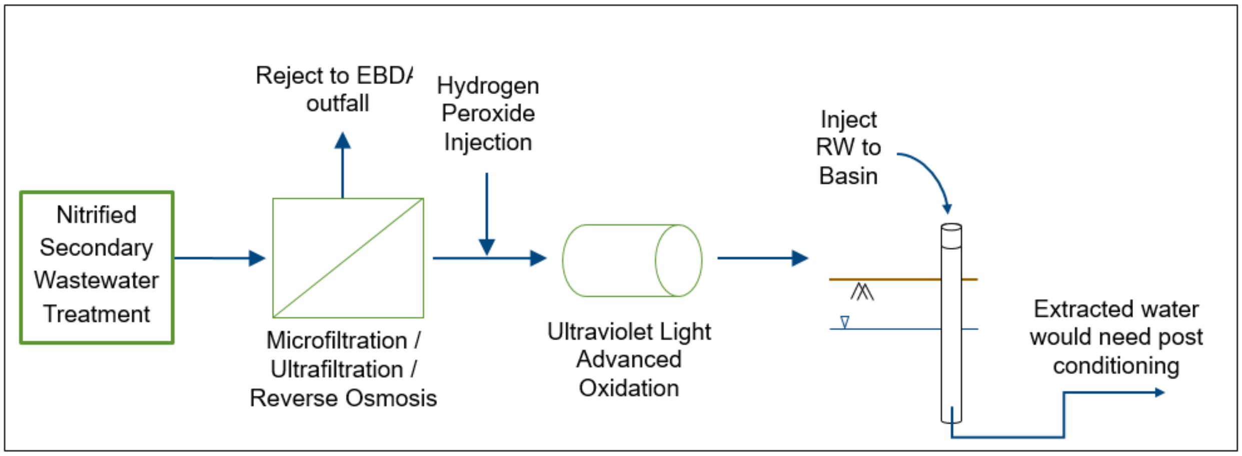
Increasing USD's EBDA hydraulic capacity is potentially a <u>partial</u> solution but is ranked as <u>low</u> viability due to the existing hydraulic restrictions and the cost of infrastructure improvements required to remove the restrictions.

2.3.4 Wet Weather Recycled Water Use (i.e., IPR)

Under this option, treated effluent would undergo advanced treatment and be injected into groundwater basins for indirect potable use. However, the RO concentrate reject from the advanced treatment process would still be released to EBDA. There are various flowrate options under this scenario, ranging from 5 to 20 MGD. RMC previously assisted USD and ACWD in a Recycled Water Feasibility Study (2016), where the 5 MGD option was explored. Costs for the project are estimated to be $1,770-1,980/AF. Of that cost, $540/AF is attributed to secondary process improvements that would be necessary to reduce nutrient levels to those appropriate for feeding into the new advanced water purification facility and to comply with groundwater recharge regulations. For any project alternatives greater than 5 MGD, purchase of extracted groundwater would need to be coordinated with other agencies in the region.

Given the limit on the flow rate associated with this option, this option is considered as a <u>partial</u> solution. It is ranked with <u>low</u> viability due to the anticipated level of effort associated with coordinating with other agencies and the lack of urgency in potable supply needed currently. Full implementation of the 5 MGD project is estimated to take approximately 5.5 years according to the Feasibility Study but could take longer depending on the responsiveness of the agencies.

Figure 2-16: Schematic of Local Groundwater Recharge Option



3. NUTRIENT REMOVAL TECHNOLOGIES

In order to further evaluate the alternative for shallow water discharge at Old Alameda Creek or other outfall locations, additional development of nutrient removal process configurations was performed by Woodard & Curran. USD is anticipating future nitrogen removal requirements and is proactively planning upgrades to the Alvarado WWTP to meet these potential limits in conjunction with the need for secondary treatment upgrades and to handle future growth anticipated within the sewer service area. The Enhanced Treatment & Site Upgrade Program considers several alternatives for upgrading the WWTP to meet these dual needs. Additionally, the District is evaluating potential nutrient removal upgrades that could be undertaken in the near-term to provide some immediate nutrient reduction and are compatible with the long-term program. Near-term improvements are expected to take place within the next five years.

3.1 Nitrogen Removal Technology Overview

Biological nitrogen removal is typically a two-step process. In the first step, ammonia is oxidized to nitrate, which is referred to as nitrification. Nitrification is carried out by autotrophic organisms in an aerobic environment. In an activated sludge system, this occurs in the aerobic zone of the aeration tanks. The growth rate of the autotrophic organisms is very temperature-dependent and much slower than heterotrophic organisms, which are primarily responsible for the biodegradation of organic matter (e.g. cBOD, BOD, bCOD, etc.). Therefore, the solids retention time in the aerobic zone of the activated sludge system needs to be longer for nitrifying systems than for BOD-only removal and can vary based on seasonal temperature differences in the wastewater.

The second step in biological nitrogen removal is denitrification, in which nitrate is reduced to nitrogen gas and released to the atmosphere. In an activated sludge system, this reaction occurs in an anoxic environment where dissolved oxygen is not present. The heterotrophic organisms in the mixed liquor of the anoxic zone will utilize the oxygen in the nitrate for the biodegradation of organic matter, resulting in the release of nitrogen gas.

The above-described mechanisms for biological nitrogen removal are the most common in municipal wastewater treatment. However, there are other mechanisms used for nitrogen removal that can involve fixed film treatment processes, nitritation, and deammonification. These mechanisms are incorporated into well-established and innovative nitrogen removal configurations and are further described in the following technology overviews.

The following BNR technologies were investigated as part of this task:

1. Modified Ludzak-Ettinger Process (MLE) and 4-Stage Bardenpho
2. Step Feed
3. Intensification
    a. Membrane bioreactor (MBR)
    b. Integrated fixed-film activated sludge (IFAS)

   c. Moving bed bioreactors (MBBR)
   d. Biomag
   e. Membrane aerated biofilm reactor (MABR)

4. Separate Stage
   a. Biological activated filter (BAF)
   b. Denitrification Filter

5. Granular Activated Sludge (GAS)

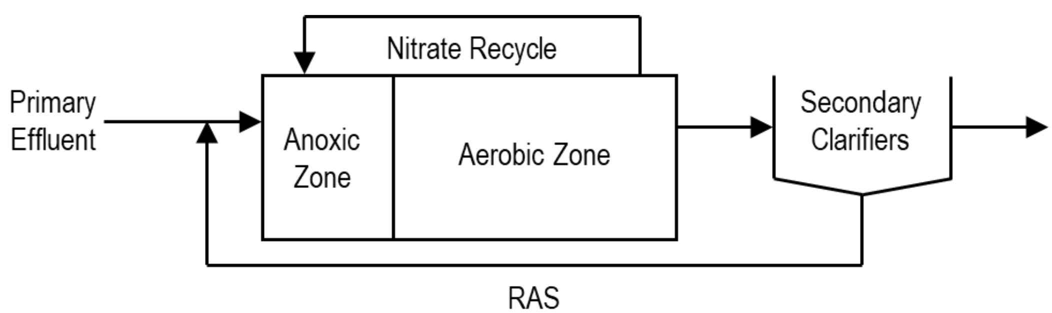
6. Sidestream Treatment of Recycle Flows from the Anaerobic Digesters
   a. Anammox
   b. Post-Aerobic Digestion

The overview includes a basic description of how they work, generalized process configurations, and implementation considerations.

3.1.1 Modified Ludzak-Ettinger (MLE) and 4-Stage Bardenpho

The Modified Ludzak-Ettinger (MLE) process is a modified version of a conventional activated sludge system designed to provide improved biological nutrient removal (BNR). The MLE configuration includes an anoxic zone prior to an aerobic zone, often referred to as a pre-anoxic zone. The aerobic zone provides nitrification of ammonia to nitrite and then nitrate. An internal recycle returns nitrified mixed liquor suspended solids (MLSS) from the end of the aerobic zone back to the anoxic zone for denitrification. **Figure 3-1** shows a typical MLE process schematic.

Figure 3-1: MLE System Process Schematic



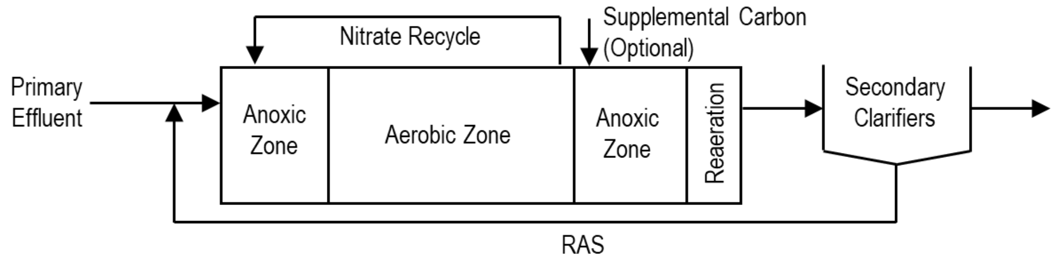
The anoxic zone is located ahead of the aerobic zone so that the organisms can utilize the organic matter in the primary effluent for the denitrification reaction. The internal recycle of mixed liquor provides for increased denitrification, which would otherwise be limited to the quantity of nitrate returned in the RAS. Typical rates for the internal recycle are 2-4 times the forward flow.

A typical ratio of the anoxic volume to the aerobic volume is 30% anoxic and 70% aerobic, but this will vary based on site-specific characteristics. The process can be configured with the flexibility to accommodate changing conditions. For example, step feeding of influent can be performed during wet weather events to keep mixed liquor in contact stabilization mode, and therefore reduce the risk of washout of solids in the clarifiers.

With sufficient influent BOD and anoxic contact time, the MLE process can typically achieve an effluent total nitrogen concentration of 8 to 15 mg/L, depending on the influent characteristics, size of the anoxic zone and internal recycle rate.

The 4-Stage Bardenpho process is similar to the MLE process but is designed to achieve lower effluent total nitrogen concentrations. **Figure 3-2** shows a typical 4-Stage Bardenpho process.

Figure 3-2: 4-Stage Bardenpho Process Schematic



The process includes a second anoxic zone downstream of the aerobic zone for additional denitrification Often, supplemental carbon (e.g., methanol or MicroC) is added to this post-anoxic zone. The post-anoxic zone is followed by a small aerobic zone to reaerate the MLSS before it flows to the secondary clarifiers to prevent issues associated with low dissolved oxygen in the clarifiers.

The 4-Stage Bardenpho process typically can achieve effluent total nitrogen concentrations of 3-5 mg/L. Design and operational considerations for the MLE and 4-Stage Bardenpho processes include:

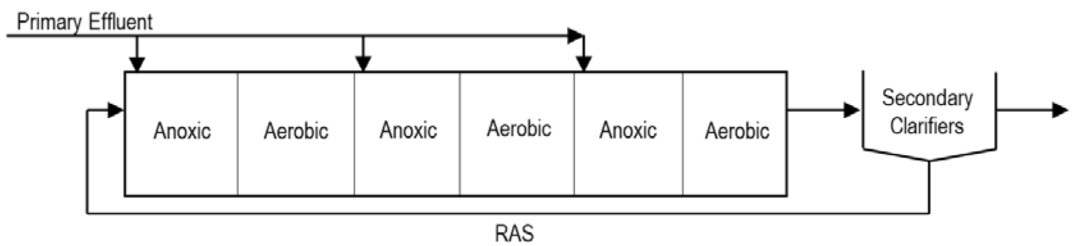
- High operator familiarity due to its similarity to conventional activated sludge systems
- The potential for operational flexibility, including:
  - Step feeding for wet weather management
  - Swing zones (aerobic and anoxic) to change the size of the aerobic zone to accommodate seasonal solids retention time (SRT) changes and maximize volume for denitrification
- Relatively larger footprint required for bioreactors and secondary clarifiers
- Does not require the purchase of proprietary systems or equipment
- These processes are compatible with enhanced biological phosphorus removal (EBPR) and can be designed to include an anaerobic zone ahead of the pre-anoxic zone. When EBPR is included, the MLE and 4-Stage Bardenpho processes are referred to as anaerobic-anoxic-oxic (A2O) or 5-Stage Bardenpho.

- Widely-accepted and utilized processes for BNR in the United States and worldwide with similar size and BNR characteristics as the Alvarado WWTP

3.1.2 Step Feed

Step feed for nutrient removal is an activated sludge system with alternating anoxic and aerobic zones in series. The process is operated as a plug-flow system, and a fraction of the primary effluent is fed to each of the anoxic zones. **Figure 3-3** shows a typical Step Feed process. As needed, supplemental carbon can be added to the anoxic zones. The number of steps and size of the zones is dependent on the primary effluent wastewater characteristics and effluent nitrogen target. The process can achieve effluent total nitrogen concentrations as low as 3-5 mg/L.

Figure 3-3: Step Feed Process Schematic



Design and operational considerations for the Step Feed process include:

- Step feed can be advantageous during high flow events because of the relatively lower MLSS concentration at the end of the step feed zones
- Internal recycle pumping is not required
- Supplemental carbon may be required for downstream anoxic zones
- Limited flexibility in the modification of the zones to accommodate changing conditions
- More complex than MLE process due to needed flow split control and aeration control in each aerobic zone

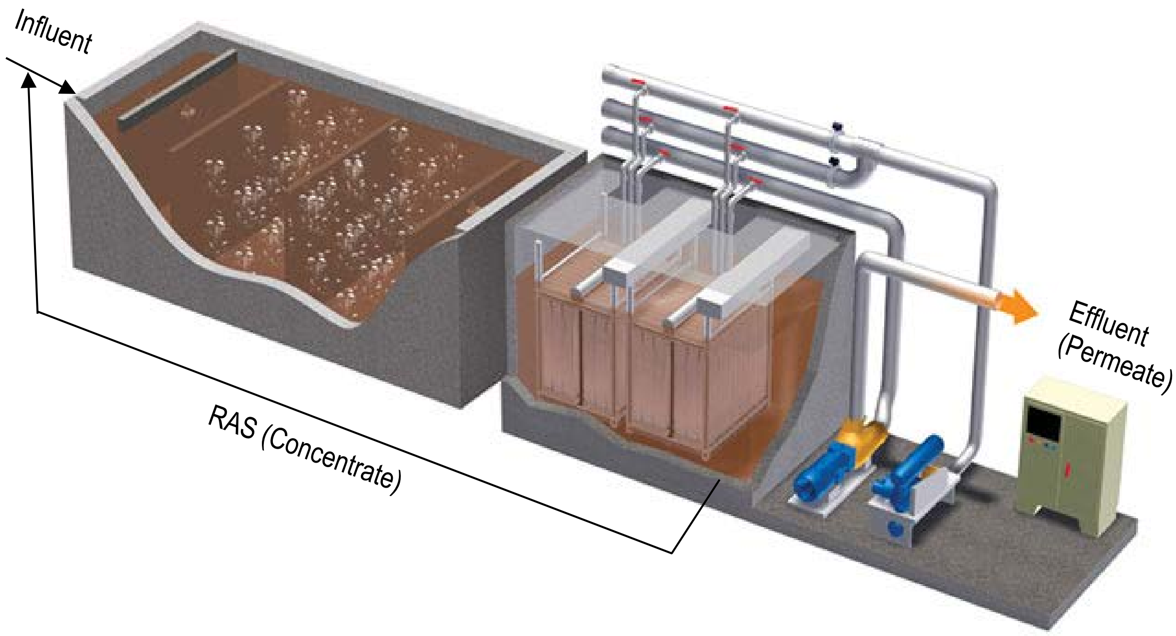
3.1.3 Intensification Technologies

Intensification technologies include processes that allow more treatment in a similar volume when compared to MLE or 4-stage Bardenpho nitrogen removal processes. This is typically accomplished through changes to the solids separation process and/or incorporation of a fixed film process. These processes are often proprietary.

3.1.3.1 Membrane Bioreactors (MBR)

Membrane Bioreactors (MBR) are an activated sludge process that utilizes membranes for solids separation rather than gravity settling in secondary clarifiers. This allows for a much higher MLSS concentration in the bioreactors, which results in a higher treatment capacity per bioreactor volume and an overall smaller footprint for the process. The bioreactors can be configured in combinations of anoxic and aerobic zones similar to MLE and 4-Stage Bardenpho processes. **Figure 3-4** shows a typical MBR system process diagram.

Figure 3-4: MBR System Process Schematic



Source – www.water-aerator.com / Suez Zenon

MLSS from the bioreactors enters the MBR tank and two streams exit, RAS/WAS and effluent (permeate). The effluent passes through the membrane, while solids are retained on the upstream side of the membrane and recycled back to the process or wasted. Membranes typically retain particles, including microorganisms, of about 0.1-micron diameter and larger. MBRs allow for higher MLSS concentrations, in the range of 8,000 to 10,000 mg/L, since gravity separation of solids is not a limiting factor. Higher MLSS concentrations allow for longer solids retention time (SRT) with a smaller tank footprint. Operational issues most often include biofouling of the membranes which is addressed by regular air scouring, backwashing, and quarterly to semiannual acid or caustic cleaning.

Nitrogen removal in an MBR is similar to what can be achieved by MLE and 4-Stage Bardenpho processes.

Design and operational considerations for an MBR include:

- Relatively smaller bioreactor footprint due to the ability to run higher MLSS concentration
- Relatively smaller solids separation footprint than secondary clarifiers
- Eliminates the risk of solids washout from secondary clarifiers

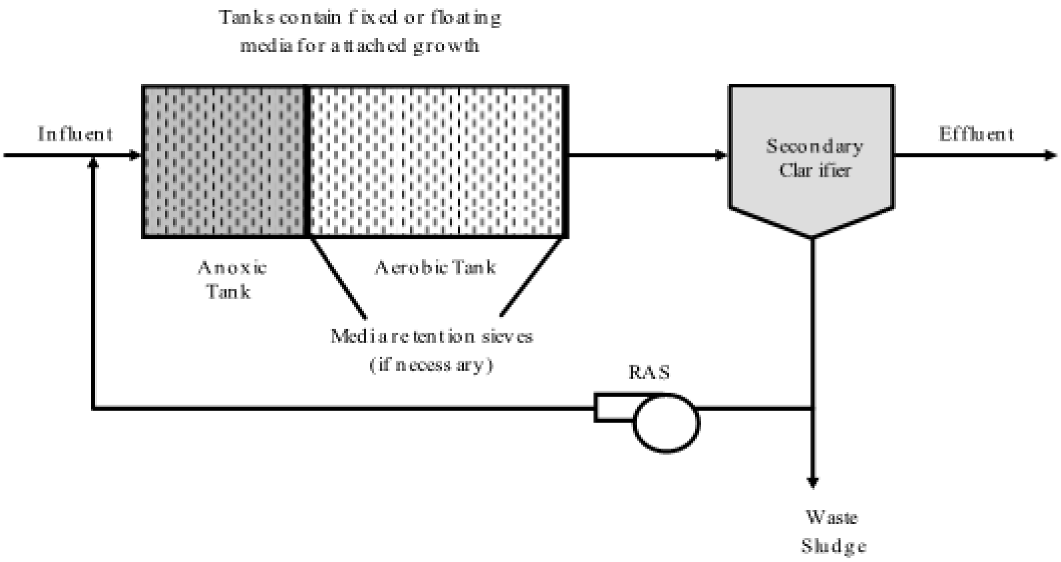
- High effluent quality with low turbidity and suspended solids appropriate for Title 22 reuse
- Requires fine screening (2 mm perforated plate preferred) to protect membranes
- Highly automated process
- High capital and operational costs
- Additional facilities and chemicals required for membrane cleaning and maintenance
- MBR Blowers required for scouring of membranes in addition to the aeration blowers resulting in relatively more air needed for the process
- Relatively more pumping is required with effluent permeate pumping
- Peak flows are limited by membrane capacity
- Many existing installations in the United States and worldwide with similar size and BNR characteristics as the WWTP
- Non-standardized systems from various manufacturers may require pre-selection

3.1.3.2 Integrated Fixed-film Activated Sludge and Moving Bed Bioreactor

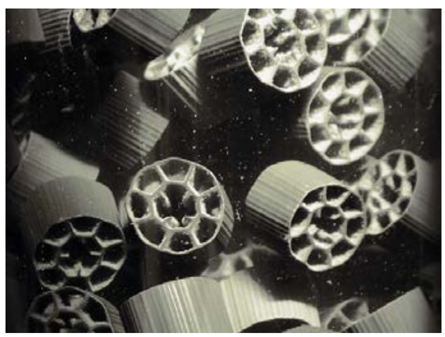
Integrated fixed-film activated sludge (IFAS) and moving bed bioreactor (MBBR) systems incorporate attached growth/fixed film into the BNR process. The bioreactors can be configured in combinations of anoxic and aerobic zones similar to MLE and 4-Stage Bardenpho processes. IFAS systems are activated sludge systems combined with fixed film media typically added to the aerobic zone for both the anoxic and aerobic zones to effectively increase the effective MLSS concentration and SRT. MBBR systems are similar to IFAS, but do not use suspended activated sludge and, therefore, do not require a return activated sludge (RAS) line.

Figure 3-5 includes a process schematic of a typical IFAS system and a close-up view of an example of one of the types of attached growth media.

Figure 3-5: IFAS System Process Schematic and Media Photo



Source – EPA Municipal Nutrient Removal Technologies Reference Document, 2008



Source: http://www.degremont-technologies.com/cms_medias/jpg/meteor-activecell.jpg

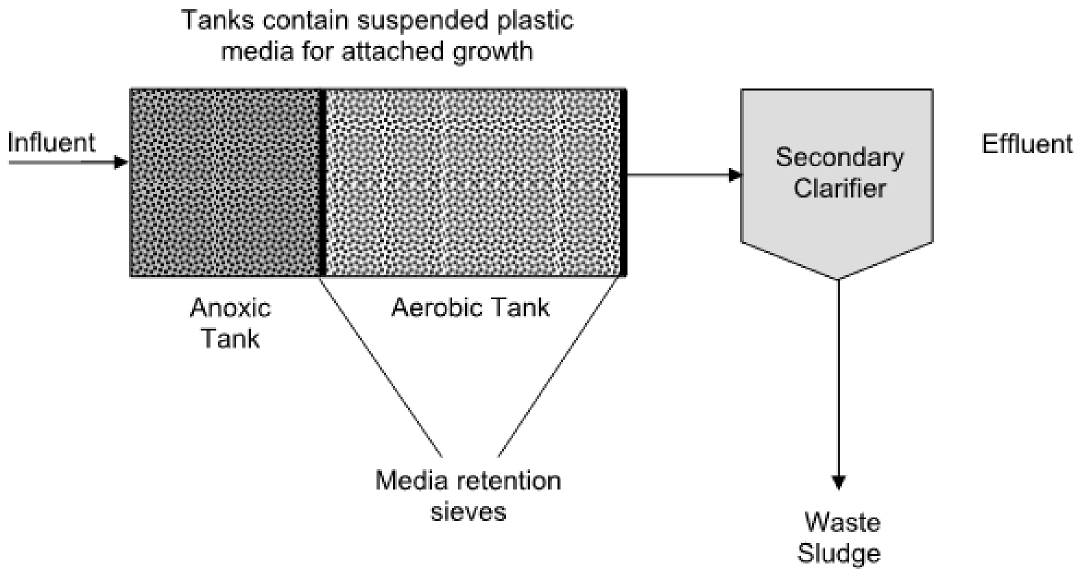
The fixed film media is lightweight with a high surface-to-volume ratio to maximize attached growth. The media typically consists of free-floating sponges, plastic discs, or small plastic cylinders with internal fins providing surface area for biofilm growth. The suspended media provides an advantage for slow-growing bacteria, including nitrifiers, by retaining the biology in the bioreactor and resulting in long SRTs. Biofilm attached to the media sloughs off and is wasted with the suspended activated sludge.

The fixed film media remains in circulation within the bioreactors and is retained by the use of screens at the effluent of the bioreactor tanks or is completely enclosed in cages. Coarse or fine bubble aeration is used to keep the media in suspension in aerobic zones and shearing of the media may deliver increased oxygen transfer. Slow-speed, submersible mixers are used to maintain suspension in the anoxic zones.

Nitrogen removal in an IFAS system is similar to what can be achieved by MLE and 4-Stage Bardenpho processes.

MBBR systems may be used in the mainstream process and can also be utilized in sidestream reactor processes, including the anammox process discussed later in this document. **Figure 3-6** includes a process schematic of a typical MBBR system.

Figure 3-6: MBBR System



Source – EPA Municipal Nutrient Removal Technologies Reference Document, 2008

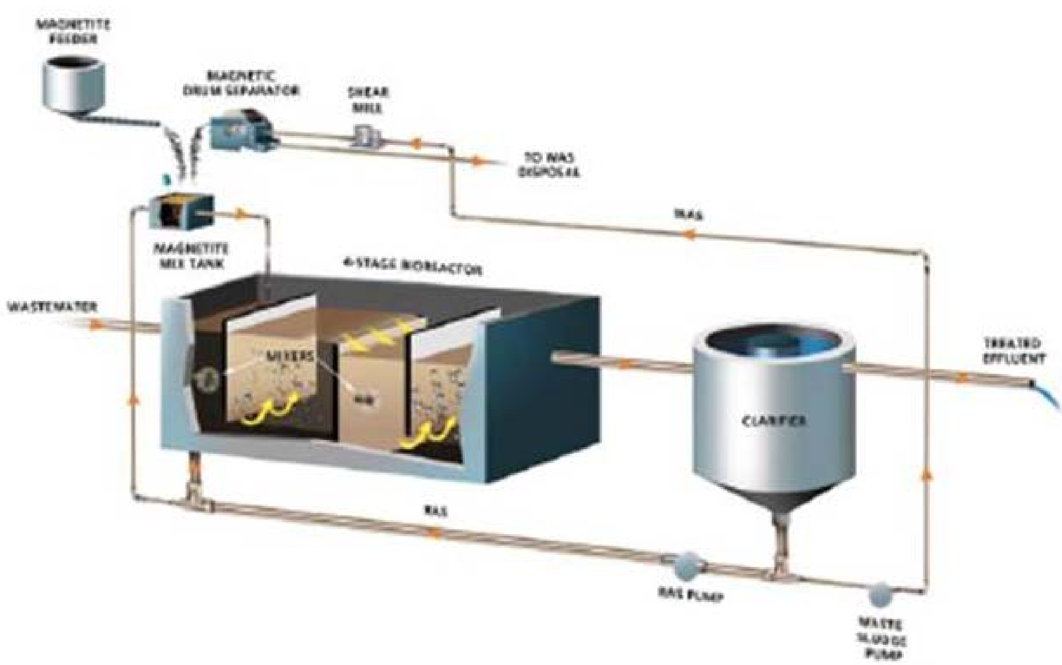
Design and operational considerations for IFAS and MBBR systems include:

- Relatively smaller bioreactor footprint due to the presence of fixed film
- Relatively lower solids loading to secondary clarifiers and less risk of washout
- Less prone to toxic upsets
- Media requires additional maintenance over conventional suspended growth and media management in out-of-service tanks needs to be considered
- Additional headloss through bioreactors due to media
- Requires additional mixing and pumping to prevent movement to and accumulation of media at the effluent end of the bioreactor
- Requires fine screening of influent (3-6 mm)
- Requires additional aeration over conventional activated sludge system for suspension and mixing of media
- Differences in systems from various manufacturers may require pre-selection

3.1.3.3 BioMag

BioMag is a proprietary technology that uses a ballast in the activated sludge process to increase the specific gravity of the biological floc to enhance settling, allowing for higher secondary clarifier loading rates. This, in turn, allows for more treatment within the same bioreactor footprint when compared to conventional activated sludge systems because systems can be operated at higher MLSS concentrations. **Figure 3-7** shows a process schematic of the BioMag system.

Figure 3-7: BioMag System Process Schematic



Source – Evoqua conceptual proposal

The ballast is magnetite, an inert iron ore, that is blended with mixed liquor or RAS in a feed tank. The ballasted mixed liquor flow through the bioreactor tanks, which can be configured for BNR, typically in an MLR or 4-Stage configuration, and to the secondary clarifiers where the solids settle out. The majority of the sludge is returned to the bioreactor via RAS, while magnetite is sheared from WAS and recovered for reuse in the process. Magnetite must be periodically added to the system; 100% recovery is not possible.

Nitrogen removal in a BioMag system is similar to what can be achieved by MLE and 4-Stage Bardenpho processes.

Design and operational considerations for the BioMag process include:

- Relatively less bioreactor volume than a conventional activated sludge system, similar to an MBR

- Relatively less secondary clarifier volume than a conventional activated sludge system – very high secondary clarifier loading rates are possible
- Existing clarifiers may require mechanism modification to scraper-type mechanisms to accommodate BioMag sludge
- Excellent settleability of sludge due to the density of ballasted floc and very low effluent TSS in the effluent
- Accommodates peak flows better than conventional or MBR systems
- High capital and materials cost
- Requires addition of ballast material and equipment to add and separate from sludge
- Fine screening of RAS required
- Potential for abrasion in mechanical equipment from long-term ballast use unproven
- Innovative, proprietary technology by Evoqua with relatively few installations operating more than five years
- Unproven at the scale required for the Alvarado WWTP

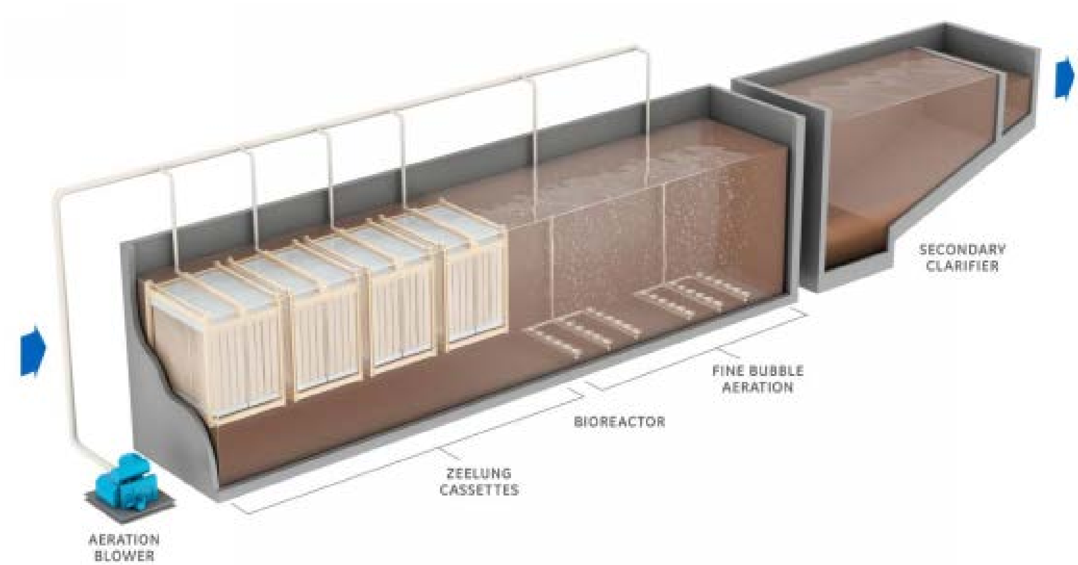
3.1.3.4 MABR

The membrane-aerated biofilm reactor (MABR) is another innovative, proprietary process designed to intensify the amount of treatment that can be done in a given footprint when compared to conventional activated sludge systems. It is also designed to save energy through increased oxygen transfer efficiency in the biological reactors.

The MABR system is different from an MBR because the membranes are not used for solids separation, and clarifiers are still required to separate MLSS. The membrane provides surface area for attached biofilm growth to increase biomass in the bioreactor for nutrient removal. Gas transfer of oxygen to the biofilm occurs directly through the membrane cords, which are connected to the process air supply system, with greater efficiency than standard fine bubble diffusers. The MABR technology can be incorporated into many BNR tank configurations, including MLE and 4-Stage Bardenpho.

Figure 3-8 shows a process schematic of the MABR system.

Figure 3-8: MABR System Process Schematic



Source – Suez ZeeLung MABR brochure

Nitrogen removal in an MABR is similar to what can be achieved by MLE and 4-Stage Bardenpho processes.

Design and operational considerations for the MABR process include:

- Relatively smaller bioreactor footprint due to the presence of fixed film
- Relatively lower solids loading to secondary clarifiers and less risk of washout
- Higher oxygen transfer efficiency in aeration tanks resulting in less energy
- Relatively higher capital and maintenance costs expected due to membranes
- Redworms may be a concern with long SRT and low dissolved oxygen (DO)
- Innovative, proprietary technology by Suez with no full-scale installations operating more than five years

3.1.4 Separate-Stage Systems

The following technologies are described as separate-stage nitrogen removal processes and would be used in conjunction with a conventional activated sludge process designed for BOD-only removal or BOD-removal and nitrification, as shown in **Figure 3-9** and **Figure 3-10**.

Figure 3-9: Separate Stage BAF Nitrification and Denitrification Filter Process Schematic

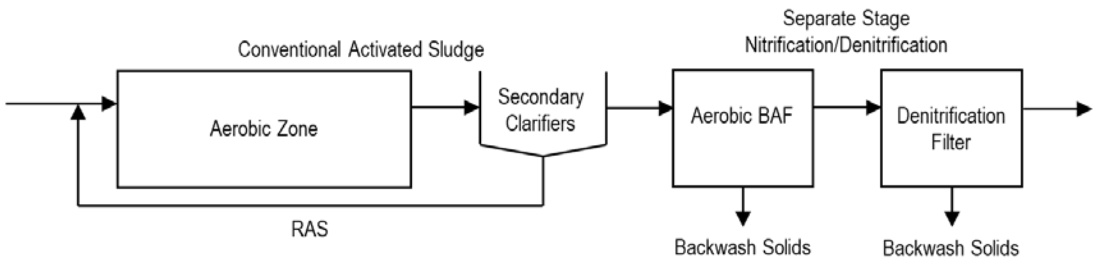
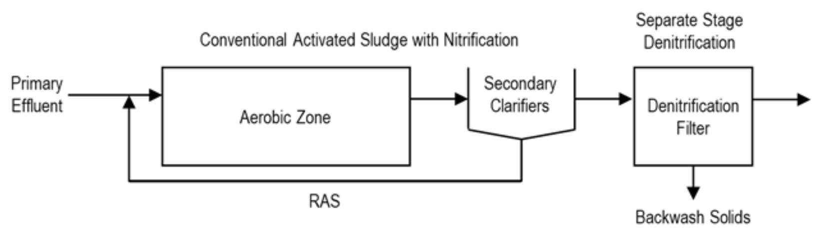


Figure 3-10: Separate Stage Denitrification Filter Process Schematic



3.1.4.1 BAF/Denitrification Filters

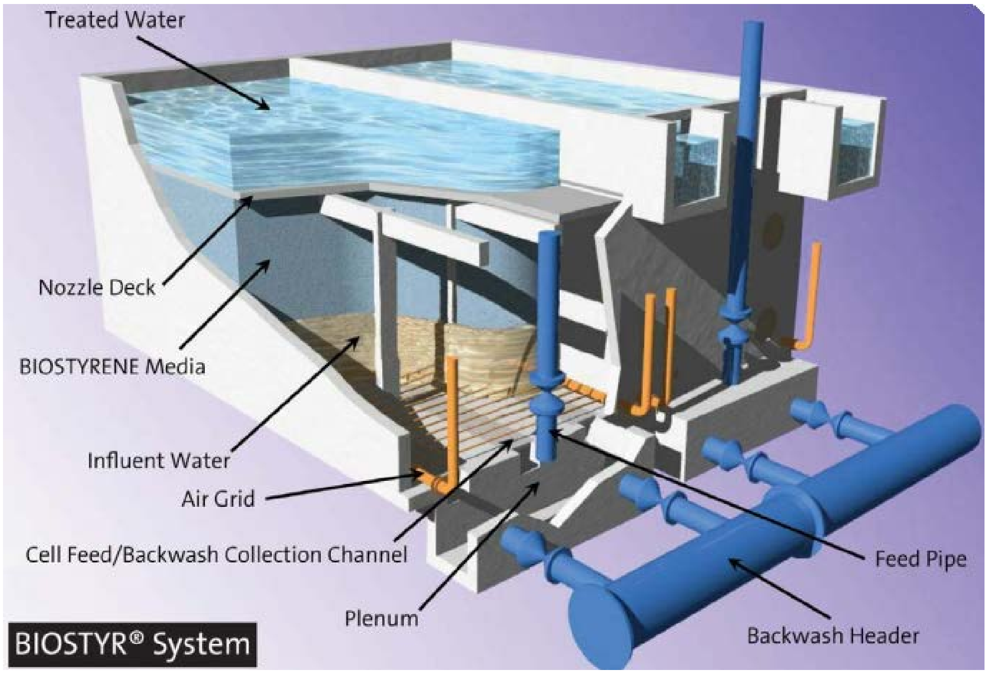
Biological active filters (BAF) have three applications: secondary treatment, separate-stage denitrification, and separate-stage nitrification. Aerobic BAFs require continuous aeration to support the biological growth of nitrifiers on the filter media. It should be noted that nitrification will only substantially occur if soluble BOD is very low in the BAF influent, which is typical of secondary clarifier effluent. Denitrification of the aerobic BAF effluent would be performed with denitrification filters, which are discussed in the following section.

BAFs are available in both upflow and downflow configurations. Upflow configurations can be used aerobically and anoxically and have the advantage of using gravity flow of effluent for backwashing. Downflow configurations typically are used for denitrification and require regular back pulsing of effluent to free nitrogen gas bubbles from the media.

BAF media typically consists of mineral or plastic material, such as polystyrene. Mineral media is generally denser than water making it suitable for downflow applications, while plastic media is less dense and appropriate for upflow configurations.

The example BAF system shown in **Figure 3-11** is an upflow style system utilizing polystyrene beads for media.

Figure 3-11: Upflow Biologically Aerated Filter Process Illustration



Source – Veolia BIOSTYR System Brochure

Intermittent backwashing is a typical requirement of BAF systems, to remove excess biomass from the media surface and maintain acceptable head loss through the system. Backwashing may be accomplished using pumps or gravity flow depending on the BAF configuration. Additionally, aeration combined with backflow of effluent helps to dislodge solids accumulated in the media. Some BAF systems may operate in a continuously backwashed mode, depending on the manufacturer.

Design and operational considerations for the BAF process include:

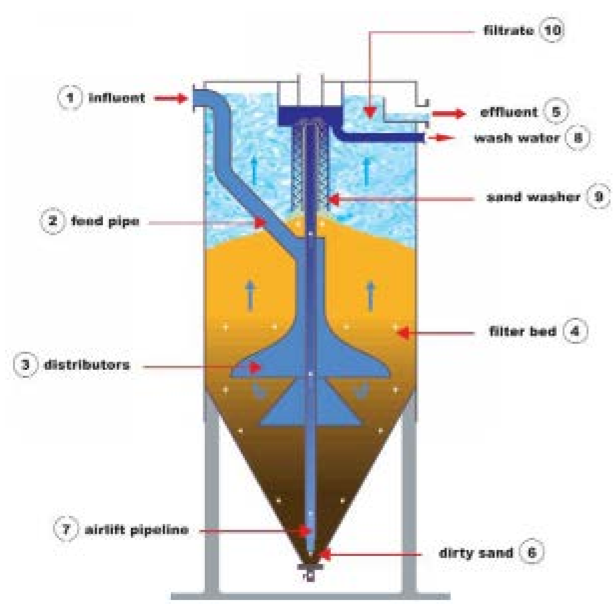
- Small footprint
- Combines BNR and filtration process
- A two-stage BAF system or a BAF combined with a denitrification filter is required to both nitrify and denitrify
- Headloss through media may necessitate additional pumping
- Highly automated process
- Backwash adds additional recycle stream to WWTP
- Differences in systems from various manufacturers may require pre-selection
- A limited number of US installations at the scale required for the Alvarado WWTP
- Due to capacity constraints in existing secondary treatment at the WWTP, the secondary system may still require expansion even with the utilization of a BAF for nitrification

3.1.4.2 Denitrification Filters

Denitrification filters utilize fixed-film biomass to reduce nitrates in secondary clarifier effluent to nitrogen gas. It follows that nitrification in the preceding secondary treatment process or a separate-stage process is a prerequisite to denitrification filter technology. Typically, low concentrations of readily biodegradable carbon sources (rbBOD) in the secondary clarifier effluent leave little carbon for denitrification, and thus denitrification filters require supplemental carbon addition, such as methanol or MicroC.

Denitrification filters are available in two configurations, including upflow and downflow continuous-backwash filters. **Figure 3-12** shows an upflow denitrification filter.

Figure 3-12: Astrasand Upflow Denitrification Filter Process Illustration



Source – EPA Wastewater Management Fact Sheet Denitrifying Filters

In upflow filters, influent flows by gravity upward through the filter, countercurrent to the media. The media is continuously moving and is circulated back to the top of the filter with an airlift pipeline, where the sand-washer scrubs contaminants into a waste line. The treated effluent (filtrate) flows by gravity over a weir. In downflow filters, influent flows by gravity downward through the filter media in a typical filtration mode. Downflow filters use an underdrain to collect effluent, while upflow filters do not. Media is typically granular sand for downflow filters, while upflow filters use fine sand. Downflow filters typically use influent weirs which may entrain DO depending on the design.

In either configuration, the filter media must be periodically backwashed to remove contaminants and nitrogen gas bubbles, which increase head loss through the filter. Denitrification filters are modular in design, where at least one unit is always backwashing while the others continue in a filtration mode, hence the term continuous-backwash filter.

Backwashing includes pumping of effluent backward through the filter media along with air for scouring. Backwash water is sent to the head of the plant for treatment. Filter housings are typically plastic or metal but can be installed in concrete structures as well.

Denitrification filters can reduce nitrate to concentrations to as low as 1 mg/L in the effluent.

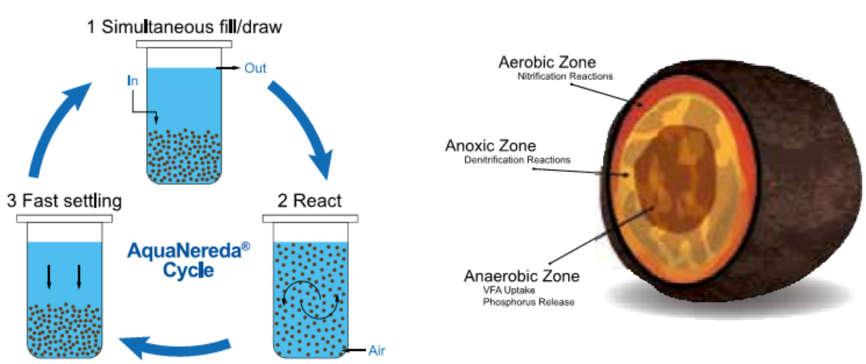
Design and operational considerations for the Denitrification Filter process include:

- Small footprint but requires structures for the filters and pipe gallery
- Combines BNR and filtration process
- Headloss through media may necessitate pumping to the system
- Backwash adds additional recycle stream to WWTP
- Highly automated process
- Additional pumping for backwash and blowers for air scour
- Requires supplemental carbon source for denitrification
- Differences in systems from various manufacturers may require pre-selection
- Limited installations longer than five years at the scale of the Alvarado WWTP

3.1.5 Granular Activated Sludge

The granular activated sludge process utilizes slow-growing and fast-settling biomass that forms bio-granules, which are unlike conventional activated sludge systems that form low-density biological flocs. The granules are formed of multiple layers of biofilm interlaced with biopolymer chains for structural support and do not require a carrier media. Granules are denser and settle faster than typical activated sludge flocs and allow for comparatively higher MLSS concentrations in the bioreactor. Each biofilm layer contains aerobic, anoxic, or anaerobic bacteria, from the outermost to the innermost layer, respectively. Diffusion between the biofilms is responsible for mass transfer of nutrients to each bacterial group. **Figure 3-13** shows a cross-sectional view of a bio-granule.

Figure 3-13: Granular Activated Sludge Batch Cycle Diagram & Bio-Granule Section



Source – Aqua-Aerobics AquaNereda System Brochure

The bio-granule structure allows for simultaneous nitrification, denitrification, and phosphorus removal in addition to BOD and TSS removal without requiring anaerobic or anoxic zones within the bioreactor. The granule structure outer layer protects the anoxic and anaerobic bacterial groups from oxygen poisoning. The outer layer consists of nitrifiers which oxidize ammonia into nitrite and nitrate for the anoxic denitrifiers in the middle layer. The denitrifiers reduce nitrate to nitrogen gas which diffuses outward and is released into the atmosphere. Granular activated sludge has the potential for enhanced biological phosphorus removal with the presence of polyphosphate accumulating organisms (PAOs) in the granule core, which uptake phosphates during the aeration phase. Wasting of sludge provides for a net removal of phosphorus from the bioreactor.

Aqua-Aerobics' Aqua-Nereda technology utilizes granular activated sludge in a batch process format, negating the need for secondary clarifiers, similar to a sequencing batch reactor (SBR). Rather than typical flow-through completely mixed reactors, batch processes use multiple reactors to sequentially fill, react, settle, and draw. Typically, one reactor fills while at least one other is reacting, settling, or drawing (emptying). Sludge wasting is typically performed following the settling phase. Batch systems are scalable due to the modular nature of the reactors, however operational complexity increases with the number of reactors. **Figure 3-13** illustrates the batch cycle as applied to the granular activated sludge process. Primary clarifiers are reported to be optional for the granular activated sludge process.

Startup of the granular activated sludge process is achieved through two methods, incrementally from conventional activated sludge flocs or through seeding from existing granular activated sludge installations. Bio-granule forming bacteria are cultivated through selective wasting of slow settling sludge.

Nitrogen removal in an AquaNereda granular activated sludge system is similar to what can be achieved by MLE and 4-Stage Bardenpho processes.

Design and operational considerations for granular activated sludge systems include:

- Smaller bioreactor footprint
- Batch process bioreactor configuration with no secondary clarifiers and return sludge lines
- No separate reactor zones for aerobic/anoxic conditions
- Can be operated with higher MLSS concentration (8,000 to 12,000 mg/L) with low SVI due to granule density
- Highly automated process
- Claimed reduction of chemical use for BNR and EBPR
- Requires fine screening (6mm perforated plate) upstream of reactors
- Innovative, proprietary technology provided by Aqua-Aerobics in the United States with no full-scale installations operating at the scale of the Alvarado WWTP. Vendor literature

suggests over 40 full-scale plants in operation or under design worldwide with a few in the design/construction phase with average flow capacities greater than 20 mgd

- Currently, on-going research for application as a non-batch process; granular activated sludge may be susceptible to shear forces from return sludge pumping, which could damage granules and kill anoxic/anaerobic bacteria

3.1.5.1 Granular Activated Sludge Selection with Hydrocyclones

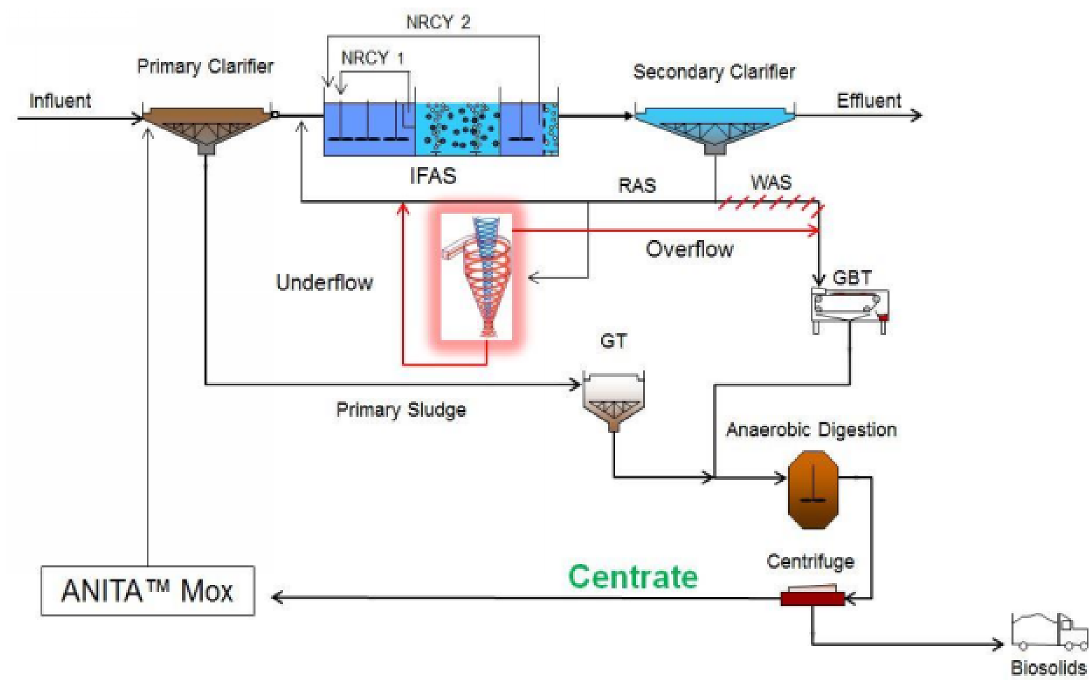
One manufacturer, World Water Works, markets a technology for selectively targeting growth of granular activated sludge. Coined a gravimetric selection technology, the InDENSE system utilizes hydrocyclones to separate low-density sludge flocs from higher density bio-granules in the mixed liquor. The hydrocyclones operate on a portion of the RAS and include two effluent streams; underflow, which retains the higher density particles, and overflow, which includes the less dense sludge flocs. The overflow stream is rejected to sludge processing and replaces the WAS stream in a conventional activated sludge system. The majority of flow into a hydrocyclone exits via the overflow. To maintain fractional wasting of sludge a small portion of RAS is pumped through the hydrocyclones, while the rest is pumped directly to the biological reactors. Several hydrocyclones are typically operated in parallel to allow for varying rates of sludge wasting. Each hydrocyclone operates within a narrow range of pressure and flow to maintain cyclonic action necessary for sludge density classification. To accommodate changes in flow requirements hydrocyclones may be put in and out of service as necessary. Figure 3-14 shows a bank of eight parallel hydrocyclones installed in an existing activated sludge plant to select granular activated sludge and waste less dense sludge flocs.

Figure 3-14: Parallel Hydrocyclone System

Source – World Water Works InDense System Brochure

Figure 3-15 shows an activated sludge process diagram utilizing IFAS, anaerobic digestion, an MBBR system for nitrogen removal in centrate, and hydrocyclones for selection of granular activated sludge. Note the typical secondary clarifier WAS line is replaced with the hydrocyclone overflow. The biological reactors in this example also include two internal nitrified recycle (NRCY) lines.

Figure 3-15: Granular Activated Sludge Process Diagram with Hydrocyclone



Source – "Improving Settleability and Enhancing Biological Phosphorus Removal through the Implementation of Hydrocyclones", Welling, 2015.

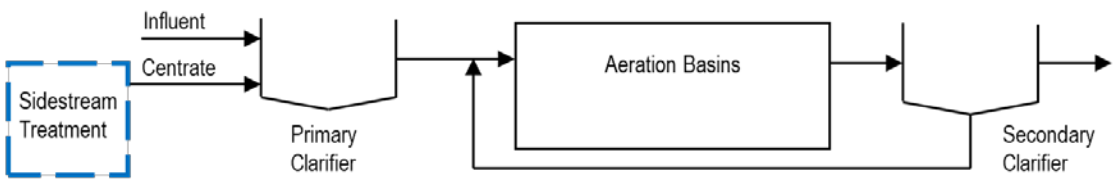
Design and operational considerations for granular activated sludge systems with hydrocyclones include:

- Relatively simple integration into existing WWTP infrastructure
- Selects against poor settling filamentous organisms
- Enhances biological phosphorus removal by selecting for phosphorus accumulating organisms (PAOs)
- Improves SVI by increasing biomass density and selecting for denser floc and granules
- Innovative, proprietary technology provided by World Water Works in the United States with at least one full-scale installation operating at a similar scale to the Alvarado WWTP

3.1.6 Sidestream Treatment

The following technologies are defined as sidestream processes to treat centrate from dewatered anaerobic digester effluent and reduce the total nitrogen load to the main biological process. These sidestream treatment processes would be operated in conjunction with the existing conventional activated sludge system at the WWTP, as shown in **Figure 3-16**, or in combination with any of the above-listed BNR processes.

Figure 3-16: Sidestream Treatment Process Schematic



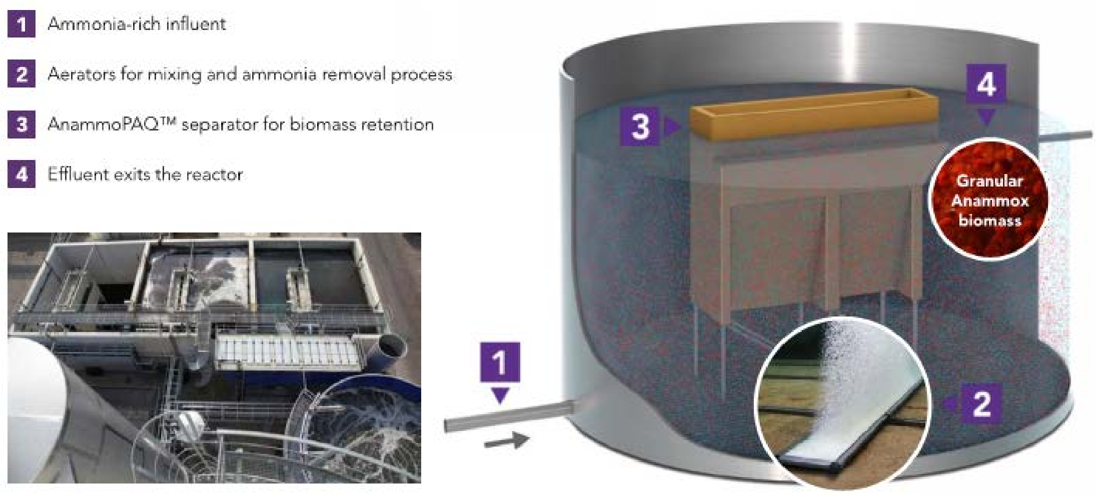
3.1.6.1 Anammox

The Anammox processes rely on two types of microorganisms: ammonium oxidizing bacteria (AOB) and anammox bacteria. The processes are typically aerated to provide the aerobic AOB group with oxygen for conversion of ammonium to nitrite (nitritation). The AOB group may be suspended growth within the process tank or grow as an aerobic layer above the anoxic anammox layer in the biofilm, depending on the process. The Anammox group are autotrophs that grow as a biofilm on either granules or MBBR media and convert ammonium and nitrite to nitrate and nitrogen gas, which is released to the atmosphere, under anoxic conditions. This step in the process is referred to as deammonification. The Anammox media or granules are retained within the sidestream reactor using screens or other devices at the effluent end of the tank.

The Annamox bacteria use inorganic carbon rather than organic carbon for growth, eliminating the need for influent BOD or supplemental carbon for the system. Because only nitritation is achieved rather than full nitrification, the oxygen requirements also are reduced. Because of these two advantages over conventional BNR, this process continues to be the focus on on-going research and development. There are several commercial process configurations that are designed for deammonification, and two are briefly highlighted here:

Ovivo-Paques developed the AnammoPAQ process using granular biomass and provides mixing and oxygen to the system using fine bubble diffusers. **Figure 3-17** shows anammox sidestream reactor for treating centrate from dewatered digester effluent, high in ammonia. The AnammoPAQ bioreactor uses a gravity separator to retain the granular anammox biomass within the tank.

Figure 3-17: AnammoPAQ Process Schematic



Source – Ovivo AnammoPAQ Process Brochure

Veolia's ANITA Mox process uses MBBR technology with plastic carrier media to provide surface area for biological growth. A pilot study at the USD Alvarado WWTP showed that approximately 70% of total inorganic nitrogen (TIN) could be removed from the centrate, thereby decreasing the average final effluent TN load by approximately 30%.

Design and operational considerations for the Annamox processes include:

- Reduces nitrogen loading to the secondary treatment process
- No external carbon source required
- Requires additional process equipment, including pumps and blowers, and tankage in addition to secondary equipment
- Smaller bioreactor footprint
- Potential savings in power, sludge generation, chemical use compared to traditional BNR options
- Innovative, proprietary technology with limited full-scale installations operating at the scale of the WWTP
- Differences in systems from various manufacturers may require pre-selection

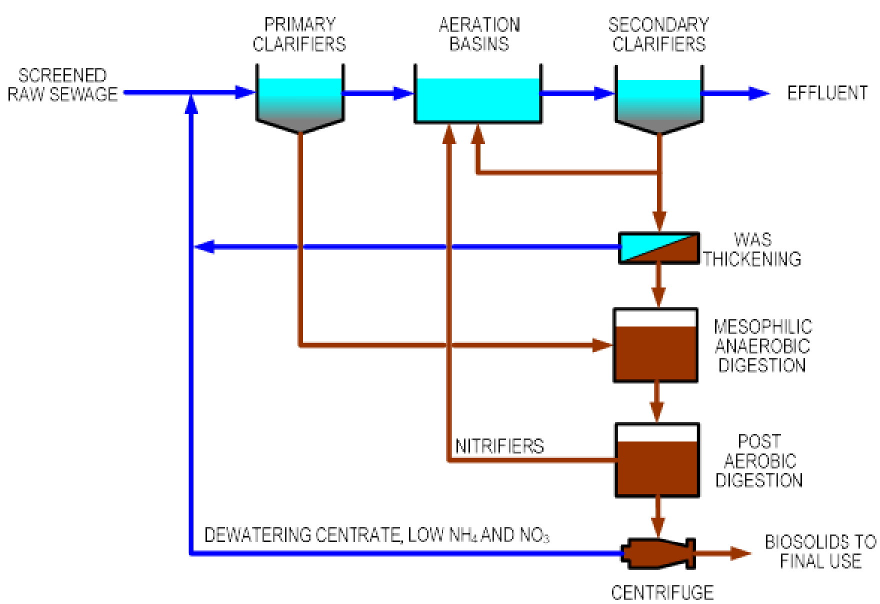
3.1.6.2 Post-Aerobic Digestion

Post-aerobic digestion is an advanced phased digestion process, where anaerobic digester effluent is further digested aerobically to reduce nitrogen and volatile solids loading to the secondary treatment system. Post-aerobic digesters can be operated in several different modes, including cyclic aeration and continuous aeration. For nitrogen removal, cyclic aeration provides for alternating aerobic and anoxic conditions in the post-aerobic digester. Cyclic aeration requires means to both aerate and anoxically mix the post-aerobic digester. Nitrification occurs within the post-aerobic digester during the aerobic phase, when high concentrations of ammonia are oxidized to nitrite and then nitrate. Denitrification occurs during the anoxic phase when nitrate is reduced to nitrogen gas for respiration and released to the atmosphere.

Post-aerobic digestion has the added benefit of providing additional volatile solids reduction (VSR) over anaerobic digestion alone. Volatile solids generally include organic solids, or those containing carbon. Aerobic digesters are not typically covered, and much of the VSR is accomplished by oxidation of organics to carbon dioxide gas. No methane is produced in aerobic digesters as methanogens are not present.

Figure 3-18 shows a process diagram with anaerobic and post-aerobic digestion, including bioaugmentation of the aeration basins with nitrifiers from the post-aerobic digester. Bioaugmentation is a means of seeding the secondary treatment process with a population of nitrifiers to enhance nitrification within the bioreactors. **Figure 3-19** shows an example installation of an aerobic digester.

Figure 3-18: Post Aerobic Digestion Process Schematic



Source – Menniti et al, (2010) Combining Mesophilic Anaerobic Digestion with Post-Aerobic Digestion to Enhance Volatile Solids Reduction and Reduce Sidestream Ammonia.

Figure 3-19: Aerobic Digestion

Source – www.ovivowater.com

Design and operational considerations for the post-aerobic digestion process include:

- Reduces nitrogen loading to the secondary treatment process
- Suitable retrofit for an existing process with anaerobic digestion
- No external carbon source, alkalinity, or chemicals required

Requires additional process equipment, including pumps and blowers, and tankage in addition to secondary equipment

4. CONCLUSIONS AND NEXT STEPS

4.1 Options Summary

Table 4-1 provides a summary of the effluent management options considered, their viability, and the extent to which they can provide a solution to future effluent storage requirements.

Table 4-1: Summary of Effluent Management Options

Alternative	Agency Coordination/ Complexity	Storage Volume / Flow Discharge Available	Complete solution?	Planning Level Costs	Implementation Timing	Viability
Influent Flow Reduction						
Inflow/Infiltration Reduction	USD Collection System team	~0 MG	Minor	N/A	On-going	Low
Conveyance System Storage	USD	Additional 1.8 MG @ Irvington, 2 MG @ Newark	Minor	~$10 M – $30 M, each basin [1]	3 – 5 years (based on current CIP)	Moderate
Satellite Treatment & Disposal	USD; potentially EBDA, Water Board	Reduces flow by up to 1.7 MGD	Partial	$58M (2010 dollars; includes treatment and distribution)	May be part of provisions to implement RW projects by 2020	Low
WWTP Onsite						
Equalization Storage						
EQ Basin East of WWTP	USD, ACFCD, ACWD, Army Corps, Water Board	Up to 20 MG	Partial to Full	$90 M [2]	5 years or more for permitting; potential partnership with ACFCD	Low
EQ Basin for site drainage flows	USD	2.6 MG (Plant stormwater)	Partial	$5.5 M (FY 2018 CIP)	Within next few years if on USD property	Low

[1] Costs estimated from ongoing predesign effort for storage basin at Newark Pump Station.
[2] Cost from the Secondary Treatment Process Improvements (CAS Option 3, Hazen and Sawyer).

Alternative	Agency Coordination/ Complexity	Storage Volume / Flow Discharge Available	Complete solution?	Planning Level Costs	Implementation Timing	Viability
Shallow Discharge						
Breakpoint Chlorination + Old Alameda Creek	USD, Water Board	Up to 10 MG	Partial	Low Capital & High O&M; Not Developed	3 – 5 years for design, construction, and permitting	Low
Early Action Nutrient Removal + Old Alameda Creek						
Alternative 1: Sidestream Nutrient Removal for Centrate	USD, Water Board	Dependent on negotiations RWQCB; permitting analysis underway	Partial to Full	\$20.8 M [1]	4 – 5 years for design, construction, and permitting	Moderate
Alternative 2: Full Flow Nutrient Removal	USD, Water Board	Dependent on negotiations RWQCB; permitting analysis underway	Full	\$23.2 M [2]	7 years for design, construction, and permitting	High (recommended approach)
Alternative 3: Parallel MBR	USD, Water Board	19 MGD	Partial	\$93M	3 – 5 years for design, construction, and permitting	Low

[1] Cost from the Secondary Treatment Process Improvements (CAS Option 2 – Phase II, Hazen and Sawyer).

[2] Only a fraction of the Secondary Treatment Process Improvements (CAS Option 2 – Phase I, Hazen and Sawyer) is attributable to early action nutrient removal. That fraction is estimated at 10%, or \$23.2 million, and is estimated to result in sufficient nutrient removal to permit increased shallow water discharges to Old Alameda Creek.

Alternative	Agency Coordination/ Complexity	Storage Volume / Flow Discharge Available	Complete solution?	Planning Level Costs	Implementation Timing	Viability
Alternative 4: Parallel MLE	USD, Water Board	12 MGD	Partial	$88M	3 – 5 years for design, construction, and permitting	Low
Shallow Discharge – Eden Landing Marsh	USD, State Coastal Conservancy, Army Corps, Water Board	0.5 MGD (max)	Minor	$6.9M (based on Oro Loma EQ/levee project cost)	5 years for permitting	Low
Offsite						
Equalization at Hayward	USD/EBDA	Greater than 200 MG	Partial to full	$10.4M	3 to 5 years; Requires coordination from EBDA partners	Low
Baseline Restoration of Hayward Marsh	USD, Water Board, EBRPD	20 MGD	Partial	$20.1M	More than 5 years to complete construction	Low
Basin 1 EQ at Hayward Marsh	USD, EBRPD State Lands Commission	30 MG	Partial	$15M total, $5.75M of which is for Basin 1 Conversion	3 to 5 years; Requires coordination from EBRPD	Low
Reconfigure Hayward Marsh for Nutrient Removal	USD, Water Board, EBRPD State Lands Commission	20 MGD	Partial	$15M, assumed to be similar to Basin 1 EQ Project Cost	5 years; Requires coordination with EBRPD and permitting	Low

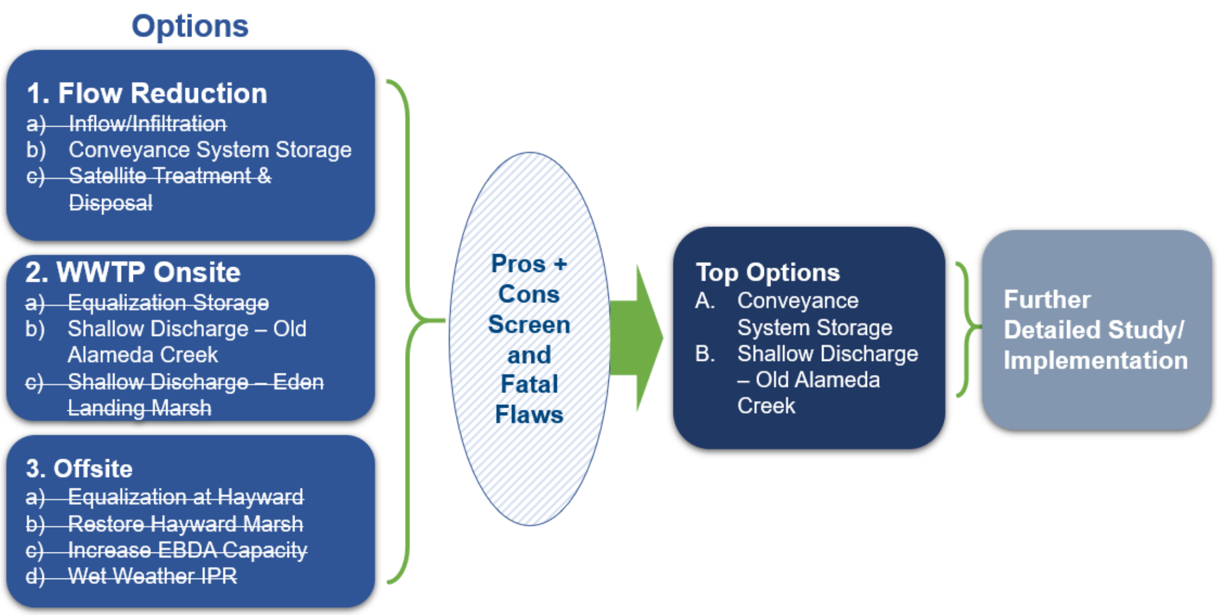
Alternative	Agency Coordination/ Complexity	Storage Volume / Flow Discharge Available	Complete solution?	Planning Level Costs	Implementation Timing	Viability
Increase USD Share of EBDA Capacity	USD/EBDA and Partners	Limited	Partial	Unknown	2+ years to coordinate with EBDA agencies; Longer for infrastructure improvements	Low
Wet Weather IPR	USD/ACWD/ Regional Agencies, Water Board	Up to 5 MGD without regional coordination	Partial	$80M (2016 dollars; includes treatment and distribution)	At least 5 to 6 years	Low

4.2 Most Viable Options (Moderate Viability or Better)

Based on the information presented in Chapter 2 and 3, options ranked with at least moderate viability were carried forward for further evaluation. The most viable options (with Moderate viability or better) are the following (also shown in **Figure 4-1**:

- Conveyance System Storage
- Shallow Water Discharge: Early Action Nutrient Removal + Old Alameda Creek

Figure 4-1: Option Screening Results



The first option is a flow reduction option. The second option is an onsite option at the WWTP. There were no offsite options identified with moderate viability or higher. A pared down summary of the most viable options is presented in **Table 4-2.** If the actual capacity of the AEPS is conservatively estimated at 39.8 MGD, only a shallow water discharge at Old Alameda Creek alternative would provide a complete solution for the peak flow of 73.3 MGD in 2058. Multiple options may be implemented to provide the estimated effluent management capacity required. However, if full flow nutrient removal is required as part of the Bay Area-wide approach to nutrient management, it will increase the viability and flow/volume capacity of the shallow water discharge options. Equalization at Hayward, in conjunction with EBDA partners, remains as a viable alternative to compare to alternatives herein where USD is in the lead position of implementation.

Table 4-2: Summary of Most Viable Management Options

Alternative	Agency Coordination/ Complexity	Storage Volume / Flow Discharge Available	Complete solution?	Planning Level Costs	Implementation Timing	Viability
Influent Flow Reduction						
Conveyance System Storage	USD	Additional 1.8 MG @ Irvington, 2 MG @ Newark	Minor	~$10 M – $30 M, each basin[1]	3 – 5 years (based on current CIP)	Moderate
WWTP Onsite/Adjacent						
<u>Early Action Nutrient Removal + Old Alameda Creek Shallow Water Discharge</u>						
Alternative 1: Sidestream Nutrient Removal for Centrate	USD, Water Board	Dependent on negotiations RWQCB; permitting analysis underway	Partial to Full	$20.8 M [2]	4 – 5 years for design, construction, and permitting	Moderate
Alternative 2: Full Flow Nutrient Removal	USD, Water Board	Dependent on negotiations RWQCB; permitting analysis underway	Full	$23.2 M [3]	7 years for design, construction, and permitting	High (recommended approach)

[1] Costs estimated from ongoing predesign effort for storage basin at Newark Pump Station.
[2] Cost from the Secondary Treatment Process Improvements (CAS Option 2 – Phase II, Hazen and Sawyer).
[3] Only a fraction of the Secondary Treatment Process Improvements (CAS Option 2 – Phase I, Hazen and Sawyer) is attributable to early action nutrient removal. That fraction is estimated at 10%, or $23.2 million, and is estimated to result in sufficient nutrient removal to permit increased shallow water discharges to Old Alameda Creek.

4.3 Next Steps

Suggested next steps are described below. O&M costs were not quantified as part of this analysis, and it is recommended that this be included in future evaluations. However, given the limited number of viable options and the unique characteristics of each, including capital cost, O&M costs are not expected to be a significant factor in alternative selection.

4.3.1 Shallow Water Discharge

The Enhanced Treatment & Site Upgrade Program proposes implementing upgrades to improve secondary process performance as soon as possible. Additional nutrient removal capability as indicated through ongoing evaluation of future nutrient watershed permits would also be implemented concurrently. These upgrades are discussed in more detail in Chapter 3 of the Enhanced Treatment & Site Upgrade Program. Incorporating multiple benefits such as improved process performance, Title 22 recycled water production, and other benefits would need to be factored in to increase the viability of the early action nutrient removal options given their capital cost. USD has had favorable discussions with RWQCB staff regarding possibly permitting an increased wintertime discharge to Old Alameda Creek during high flow periods, along with early action nutrient removal; the next steps are underway and include developing technical studies and, if appropriate, a permit application.

USD, in conjunction with Woodard & Curran, is developing more defined technical documentation regarding discharge to Old Alameda Creek. This documentation will include analyses defining:

- Frequency of discharge to Old Alameda Creek after discharge to Hayward Marsh is no longer possible
- Projected water quality of the discharge based on the implementation timeline of process upgrades

If accepted, the RWQCB would be granting USD an exception to the current shallow water discharge prohibition on the basis that USD would be providing an "equivalent level of environmental protection"[1] to San Francisco Bay due to nutrient removal. This technical proposal is expected to be submitted to the RWQCB in September 2019.

In the meantime, USD will continue to work with EBRPD on the transition of Hayward Marsh from facility accepting secondary effluent from USD year-round to a facility used only during wet weather events for equalization and potential discharge in conjunction with Old Alameda Creek.

[1] San Francisco Bay Regional Water Quality Control Board. Order No. R2-2015-0045, NPDES No. CA0038733 Attachment F. November 18, 2015.

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APPENDIX A: EBDA SYSTEM: WET WEATHER, STANDARD OPERATING PROCEDURE: 2017-2018

EBDA SYSTEM: Wet Weather, Standard Operating Procedure: 2017-2018 **Revised: March 5, 2018**

STAGE	STEP	EBDA	UNION SANITARY	HAYWARD	ORO LOMA/CV	SAN LEANDRO	LAVWMA
1 Normal Operations	**1**	Monitor weather forecasts for potential wet weather events (i.e. extreme rainfall events, or several back-to-back storms). Adjust control set points at the OLEPS for wet weather range (5.0' – 7.0') October 15th - April 15th and/or severe wet weather event (HEPS) accordingly.	In anticipation of rain: • USD assesses the availability/condition of the Hayward Marsh. Marsh Flow Set point normally set to 20 mgd. USD 60-inch valve at Hayward: • 60-inch valve at Hayward will be left at the normal % open (Approx. 22% to 25%) & will monitor surge tower level.	In anticipation of prolonged rain: -Monitor plant influent flow. If above 24mgd, event is considered wet weather flow	Monitor weather forecasts and take select tanks out of service as appropriate	Monitor weather forecasts	LAVWMA discharges up to 41.2 mgd to EBDA. Advisory notification to EBDA. After receiving 1/2" rain within 24 hours, LAVWMA Operator will switch to "storm mode" matching the export flow equal to the system influent flow up to 41.2 mgd
2 One Diesel Operating	**1**	Increased flows greater than the capacity of OLEPS electric pumps (approximately 94 mgd) starts the first diesel pump at OLEPS. Request USD to increase diversion to Hayward Marsh up to permitted limit of 20 mgd. Request LAVWMA modulate flow.	When the surge tower level reaches 45.0', USD to notify EBDA that valve will be opened incrementally at approximately 5 mgd of flow change per 5 minutes. USD will adjust valve to target 40-45' in the tower and notify EBDA prior to making changes to valve position.		As plant influent increases, OL/CVSD will start to divert flow to the equalization basin in an attempt to keep flow to a maximum of 69.2 mgd. Current and future weather conditions and equalization capacity will be taken into account.	As Plant influent increases above FFR capacity (approximately 14 mgd) divert additional flow to the equalization basin as available.	Per EBDA request, modulate flow for 2 to 3 hours to provide EBDA system short term operational flexibility[2].
3 Two Diesels Operating	**1**	Increased flows greater than the capacity of one OLEPS diesel pump (approximately 110 mgd) starts the second diesel pump at OLEPS. If two OLEPS diesel pumps are running and one fails, request COH to divert flow to ponds and request OL/CVSD increase diversion to equalization basin for a short term in an attempt to control OLEPS wet well level until COH pond diversion[1]		If two OLEPS diesel pumps are running and one fails, per EBDA: Divert COH flow to ponds[1]	If two OLEPS diesel pumps are running and one fails, OL/CVSD will increase diversion to equalization basin for a short term in an attempt to control OLEPS wet well level until COH pond diversion[1]		
	2	If the OLEPS wet wells begin to rise after the second engine is in service, divert COH flow to ponds[1] If current and future weather conditions indicate a continued wet weather event, inform LAVWMA that the potential exists for an interruptible event and inform USD that the potential exists for the need to use the Old Alameda Creek Discharge.	If two OLEPS diesel pumps are running and additional diversion of flow from OLEPS is needed, USD to throttle the USD 60-inch valve at 5 mgd flow changes per 5 minutes to compensate for decreased pressure in the line due to COH pond diversion. USD staff will not throttle below 45% open if 45' of surge tower level is exceeded.	Per EBDA: Divert COH flow to ponds[1]			
4 System Over Capacity	**1**	If the OLEPS wet well level continues to rise, inform LAVWMA that EBDA is at capacity and to reduce their flow to 19.72 mgd[3]. If the OLEPS wet well level continues to rise after LAVWMA flow is reduced, inform USD that the potential exists for the need to use the Old Alameda Creek Discharge and they should start the two hour prep time.	Contingency Plan when AEPS surge tower is nearing overflow: • Continue to divert 20 mgd to marsh • Use standby Primary Clarifier storage • Use standby Secondary Clarifier storage USD will attempt to manage its storage capacity to ensure that it has two hours to prepare for discharge to the Old Alameda Creek.		Verify operation of the SBS system for near shore unanticipated bypass	Verify operation of the SBS system for near shore unanticipated bypass	Interruptible event begins, EBDA System at capacity[3]. EBDA notifies LAVWMA to reduce flow to at or below 19.72 MGD.
	2	If current and future weather conditions indicate a continued wet weather event and/or OLEPS wet well level still continues to rise, USD wet weather flow diversion to Old Alameda Creek, considered a USD NPDES permitted diversion	When the maximum hydraulic EBDA capacity is reached, USD will attempt to limit flows to OLEPS to 42.9 mgd by utilizing remaining on-site storage and/or potentially using the Old Alameda Creek Discharge. The prep time to begin using the Old Alameda Creek Discharge is approximately two hours.				
	3	If unable to contain flows, advise RWQCB and select: • increased USD wet weather flow diversion to Old Alameda Creek, considered a USD NPDES permitted diversion • near shore discharge at OLEPS/SLEPS, considered an EBDA unanticipated bypass					

Notes:

[1] A per use fee and cost per estimated gallon to be applied for each Hayward Pond diversion.

[2] Intent is to modulate LAVWMA flow for 2-3 hours within 24 hour window to provide short term operational flexibility for EBDA system (high tide, short term equipment failure etc.) and still maintain LAVWMA system overall 24 hour integrity. LAVWMA will reduce pumping under the following guidelines per "LAVWMA Wet Weather Operations Strategy" as follows:

- Wet weather storage shall not exceed 40% of LAVWMA's total storage capacity, and;
- The EBDA flow reduction request does not last for more than 12 hours, and;
- EBDA agrees they can accept higher than normal flows after the reduction period is over so LAVWMA storage volume can be reduced to <20% within 24 hours, and;
- The weather forecast projects a short term (<24 hour) wet weather event.

[3] EBDA System at Capacity. EBDA's system has nominal 189 mgd capacity based on tide elevation, timing of combined flows from all members and all equipment operational.

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.....(C) 510-206-3820
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Alameda County Services.....510-577-0500
Sheriff's Dispatcher (working hours).....510-667-7721
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APPENDIX B: HAYWARD MARSH RECONFIGURATION – AMMONIA REDUCTION PROJECTION TECHNICAL MEMORANDUM

COMMITMENT & INTEGRITY
DRIVE RESULTS

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MEMORANDUM

TO: Mark Takemoto
CC: Dave Richardson
FROM: Courtney Eaton
DATE: April 9, 2018
RE: Hayward Marsh Reconfiguration – Ammonia Reduction Projection

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1.1 Purpose

Union Sanitary District (USD) is in conversation with the East Bay Regional Park District regarding potential re-configuration of the Hayward Marsh. USD currently discharges a portion of their secondary wastewater effluent from the Alvarado Wastewater Treatment to the marsh providing the main source of freshwater into the marsh (a free water surface wetland). In the current configuration of the marsh, as shown in Figure 1, Basin Nos. 1, 2A and 2B are freshwater treatment marsh while Basin Nos. 3A and 3B are tidally influenced and brackish.

The current National Pollutant Discharge Elimination System (NPDES) permit for the Hayward Marsh, adopted by the San Francisco Bay Regional Water Quality Control Board (Order No. R2-2011-0058, NPDES Permit No. CA0038636), requires that the influent to the marsh meet BOD and TSS limits and the effluent from the treatment basins, Basin Nos. 2A and 2B meet specified limits for metals (i.e., copper, cyanide and nickel), select organics and total ammonia. The current average monthly limit for total ammonia (34 mg/L-N) is under negotiation with the Regional Board but the target for this study is a total ammonia effluent limit of 1 mg/L-N on an average monthly basis.

The East Bay Regional Park District would like to convert one of the existing freshwater treatment cells, Basin No. 2B, into a tidally influenced cell. This would require meeting total ammonia limits with just Basin Nos. 1 and 2A. Woodard and Curran was tasked with evaluating whether this proposed configuration is feasible and if adding aeration to Basin Nos. 1 and 2A would be sufficient to reduce the total ammonia levels to the target anticipated in the upcoming NPDES permit renewal.

1.2 Current Configuration and Conditions

An important first step in answering the re-configuration question is to understand the current performance of the existing system regarding ammonia reduction. Table 1 summarizes the hydraulic parameters of Basin Nos. 1, 2A and 2B. All flow influent to the marsh currently flows into Basin No. 1 and then splits between Basin Nos. 2A and 2B before recombining in the mixing channel and moving into Basin Nos. 3A and 3B. See Figure 1 for details. Figure 2 shows a close-up view of Basin No. 1, where baffles appear to be creating a serpentine flow pattern through the basin.

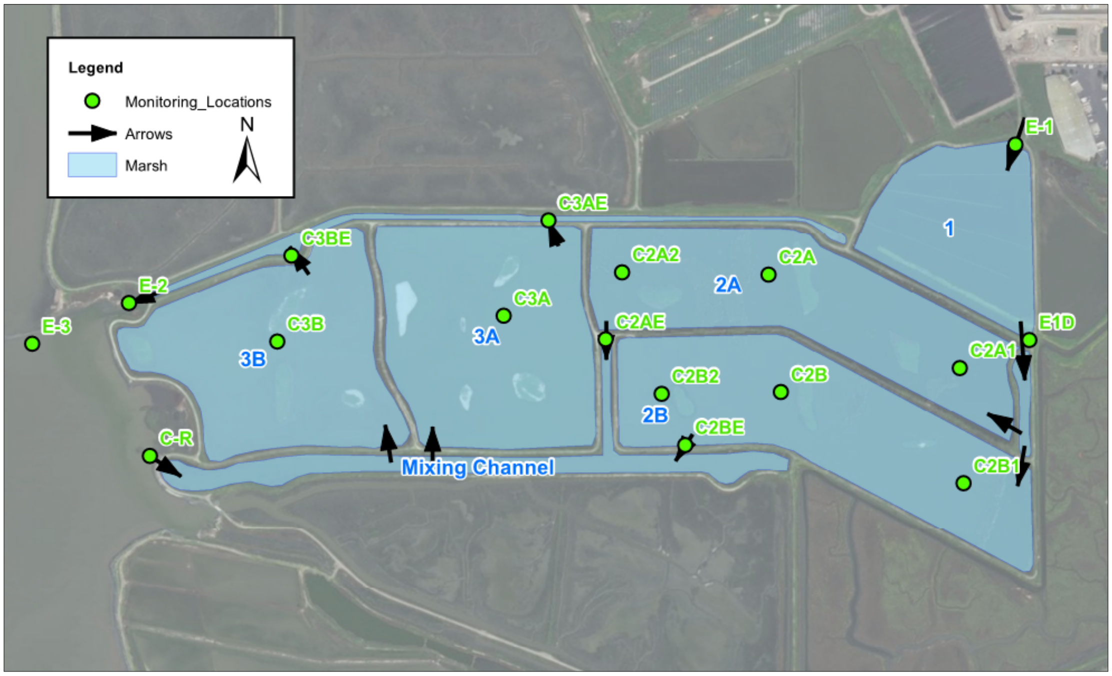


Figure 1. Plan View of Hayward Marsh

Figure 2. Basin No. 1 Detail

Table 1 – Summary of Current Hydraulic Parameters for Freshwater Treatment Basins

	Basin 1	Basin 2A	Basin 2B	Total
Area, ac	15	28	27	70
Average Water Depth, ft(1)	6	4	4	
Approximate Volume, MG	28	37	35	100
Current HRT, days				
Average Day Flow (3.1 mgd)	9.1	23.5	22.8	32(2)
Max Month Flow (4.0 mgd)	7.1	18.3	17.7	25(2)
Max Day Flow (5.2 mgd)	5.4	14.0	13.6	19(2)

(1) Average water depth is based upon the current operation as observed in the field.
(2) Flow is parallel through Basin No. 2A and 2B so the total detention time is based upon volume in Basin No. 1 + either Basin No. 2A or 2B (not both).

Using the NPDES Monthly Operating Reports (MORs) for the Hayward Marsh, average monthly values for flow, influent and effluent ammonia concentration, pH, and temperature were summarized based upon 5 years of data (November 2011 – January 2016). Table 2 summarizes those flow and water quality conditions; no alkalinity data, important to ammonia reduction, was available in the MORs. While average values are important, ammonia concentrations within the marsh system have a strong dependence upon temperature and pH of the water; therefore, it is important to also consider the seasonal aspect of the marsh's performance. Figure 3 shows total ammonia concentrations at key sampling points within the system; locations as noted in Figure 1. Based upon Figure 3, it is apparent that there is a strong climatic influence on the ammonia concentrations discharging from the system. The figure also shows that Basin Nos. 2A and 2B are providing significant total ammonia reductions during certain times of the year. During the warm weather months, average effluent ammonia concentrations from these basins are reaching less than 5 mg/L-N. However, in the winter months, much less reduction in concentration is occurring.

1.3 Ammonia Removal Pathways in Current Configuration

Studies have shown that there are three main mechanisms that could be responsible for ammonia reduction in a pond system (EPA, 2011):

- *Ammonia volatilization (or stripping)* – the rate at which this occurs follows a first-order kinetic mass transfer process and is dependent upon the pH, temperature, hydraulic retention time (HRT) and mixing conditions; at low temperatures and well mixed conditions, stripping will be the main process for ammonia reduction.
- *Assimilation into algal biomass* – the rate at which this occurs will depend upon temperature, organic loading, HRT and wastewater characteristics.
- *Biological nitrification* – low nitrate and nitrite concentrations typically found in pond effluents suggests that nitrification does not account for a significant mechanism of reduction

Table 2 – Summary of Current Conditions for Freshwater Treatment Basins

	Influent (E-1)	Basin 2A (Effluent)	Basin 2B (Effluent)
Influent Flow, mgd			
Average Day Flow	3.1		
Maximum Month	4.0		
Max Day Flow	5.2		
Permitted Peak Hour	20		
Average cBOD, mg/L	7		
Average pH	7.2	8.6	8.8
Average Temperature, deg C	22	17	17
Total Ammonia Average Monthly, mg/L-N			
Basin No. 1 Influent	42		
Average Monthly		15	10
Maximum Day		33	27
Average Monthly Nitrate, mg/L	0.3	0.41	0.31

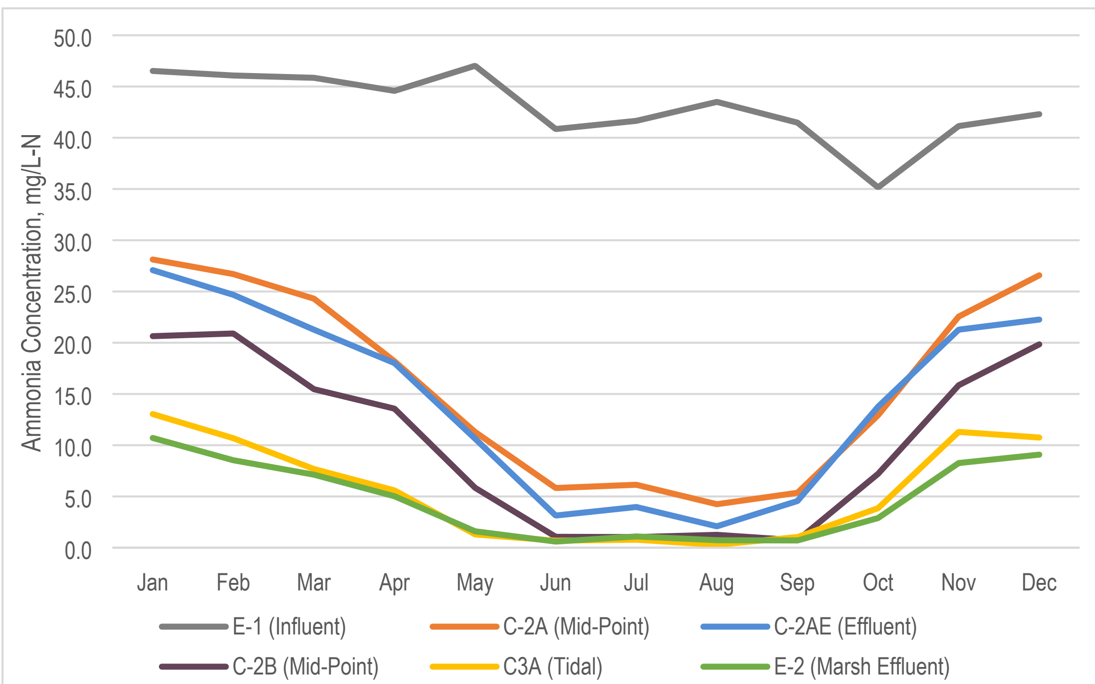


Figure 3. Ammonia Concentration Across the Treatment Basins

Figure 4 illustrates the ammonia removal across the freshwater treatment basins (Basin Nos. 1, 2A and 2B) as well as across the entire marsh system in relation to the average monthly flow, temperature and pH through the basins. The influent flow varies from roughly 1.5 million gallons per day (mgd) to 4 mgd equating to a range in HRT of 25 to 70 days. The longer detention times happen to be occurring during the colder months, when the rate of ammonia reduction is kinetically slower; the longer detention times likely contribute to more ammonia reduction than would typically be realized in winter conditions. The pH is relatively constant within the treatment basins, shifting between 8.5 and 9.5, but well within the alkaline region. Alkaline pH shifts ammonia (pK_a 9.2) towards the unionized form favoring volatilization, or stripping of ammonia.

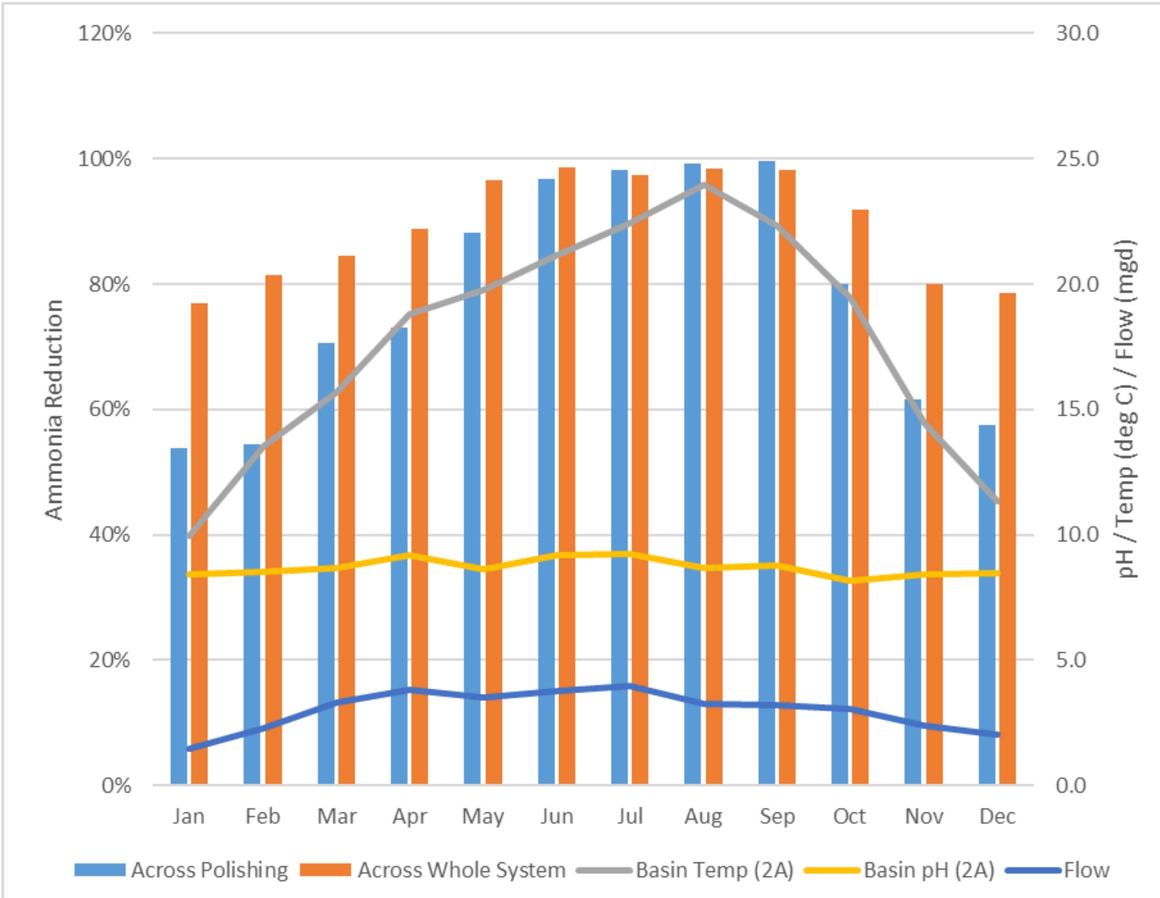


Figure 4. Ammonia Reduction in Relation to Flow, Temperature and pH

While the mechanisms noted above suggest pathways in which ammonia is removed from the system, ammonia can also be released from the system as well. Ammonia that is taken up by algae can be released from algae cells that have settled to the bottom of the pond; as this decay occurs, the ammonia as well as organic nitrogen can be re-released into the water column.

Attempting to quantify the various mechanisms that contribute to ammonia reduction and the degree to which they are occurring is difficult. A first-order kinetic reaction equation is available to model the contribution from volatilization of ammonia based upon HRT (d), temperature and pH (EPA, 2011). This equation (Equation 1) can be used to estimate the contribution from volatilization for Basin Nos. 1, 2A and 2B under the current configuration. Those estimated values are plotted against the actual effluent values in Figure 5.

Equation 1

$$N_e = N_o e^{-K_T[t + 60.6 * (pH - 6.6)]}$$

Where:

N_e = effluent nitrogen, mg/L
N_o = influent nitrogen, mg/L

N_e = effluent nitrogen, mg/L
K_T = temperature dependent rate constant = $K_{20} (\theta)^{(T-20)}$
K_{20} = rate constant at 20 deg C = 0.0064
Θ = 1.039
t = detention time in the system, d
pH = pH of near surface bulk liquid

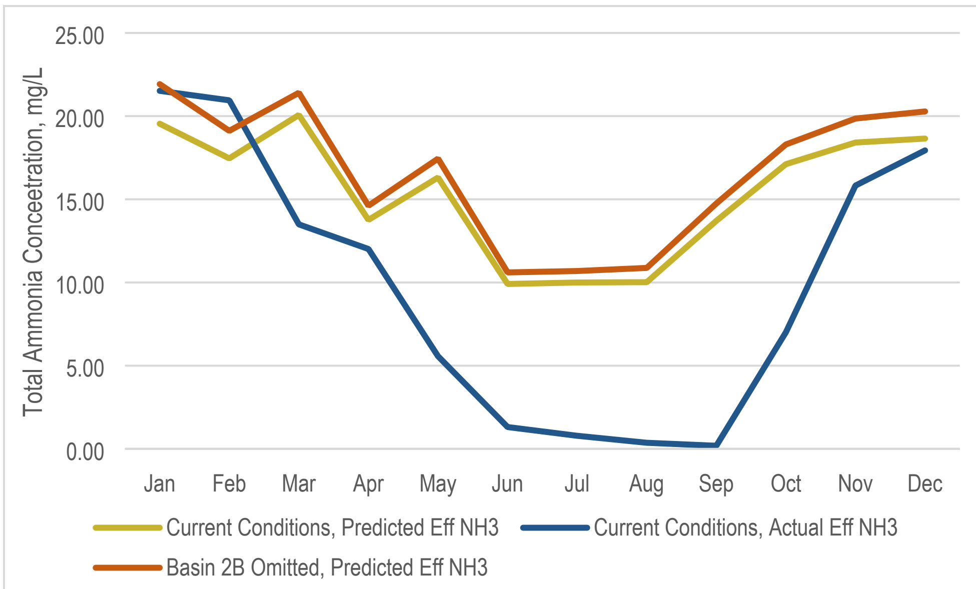


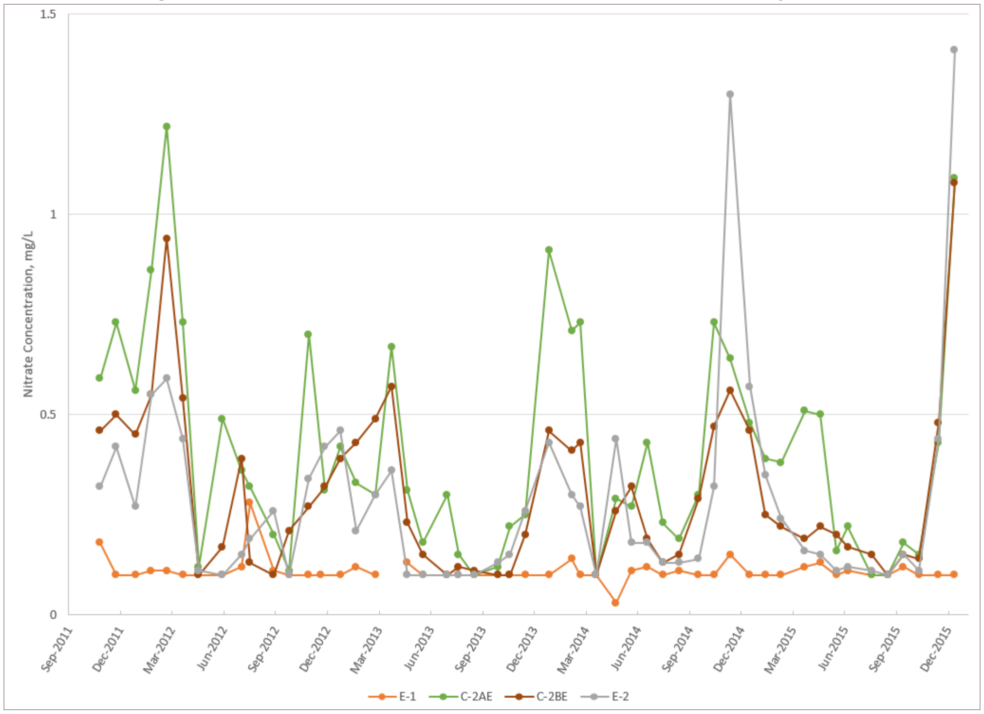
Figure 5. Estimated Effluent Ammonia Concentrations due to Volatilization

From Figure 5, the predicted ammonia effluent concentration due to volatilization (orange line) is significantly (5-10 mg/L-N) higher than the actual effluent ammonia concentration (blue line) observed. From this, it is evident that volatilization is not the only mechanism for ammonia reduction in the marsh system. Uptake by algal biomass is also likely involved as well. Algal growth typically occurs when the HRT of a facultative type pond is greater than 3 to 5 days. Given that the detention time of the smallest basin, Basin No. 1, is between 5 and 9 days, it is possible that there is algal growth within this pond system during the warmer months, though significant algae has not been observed from the surface. Algal growth will contribute to the uptake of ammonia. Additionally, the diurnal affect of algae on dissolved oxygen (DO) will contribute to nitrification. During daylight hours, algae will be a source of DO near the water surface while at night, respiration occurs reducing DO.

Recognizing that nitrification might be occurring to some degree within the system, nitrate concentrations across the ponds were summarized seasonally to determine the potential effect. Figure 6 shows the influent to the marsh (E-1), Basin No 2 effluent (C-2AE, C-2BE) and overall system effluent (E-2) nitrate concentrations. From Figure 6, a slight rise in nitrate concentration occurs in the winter months, potentially indicative of nitrification occurring taking place. This could be related to the better mixing conditions (i.e., higher DO levels) due to increased wind/weather in the winter. In the summer months, it is likely that any nitrate that is forming through nitrification is being denitrified and released causing the nitrate values to drop back to nearly ambient levels.

Unfortunately, there are no well-established models to predict the level of ammonia reduction due to algal growth and natural nitrification within a pond system.

Figure 6. Nitrate Concentrations across the Marsh System



1.4 Achieving Ammonia Reduction in Proposed Configuration

Drawing upon the analysis of the current ammonia reduction profile and possible mechanisms already noted, Woodard & Curran was tasked with determining whether the effluent total ammonia target of 1 mg/L-N could be achieved if Basin No. 2B was converted to a brackish basin and if so, how.

According to Equation 1, the volatilization of ammonia is dependent upon the HRT, pH and temperature of the basin. Therefore, if the HRT is reduced, as shown in Table 3, for the proposed configuration, there is a possibility that the volatilization of ammonia would also be reduced. Using Equation 1, the predicted volatilization could be calculated on a monthly basis for the proposed configuration (i.e., less HRT) and is plotted in Figure 5 (grey line), along with the predicted reduction in ammonia due to volatilization in the current configuration (blue line). As shown in Figure 5, there is very little difference in the predicted ammonia volatilization rate with the reduction of HRT, indicating that kinetically, volatilization is much more dependent upon pH and temperature than shifts in detention times.

Again, from Figure 5, volatilization appears to be provide only a portion of the overall ammonia reducing mechanism, reducing ammonia by approximately 50% in the winter months and approximately 75% in the summer months. It is apparent that other mechanisms are further reducing the ammonia an additional 20-25% during elevated pH and temperature conditions. Unfortunately, there is no predictive way to model those reductions.

Table 3 – Proposed Configuration for Hayward Marsh

	Basin No. 1	Basin No. 2A	Total
Approximate Volume, MG	28	36.5	64.5
Proposed HRT, days			
Average Day Flow (3.1 mgd)	9.1	11.8	21.9
Max Month Flow (4.0 mgd)	7.1	9.1	16.2
Max Day Flow (5.2 mgd)	5.4	7.0	12.4

It is interesting to note, from the ammonia concentration profiles shown in Figure 3, the ammonia concentration at the mid-point of the basin (in either Basin No. 2A or 2B) is nearly the same as the effluent concentration in each basin. This suggests that the majority of the ammonia reduction is occurring within Basin No. 1 and the first ½ of Basin No. 2A or 2B. If that is true as the data suggests, then it is possible that Basin No. 2A has additional 'capacity' for ammonia removal within the basin, at least within the summer months at more favorable temperatures and pH. Just based upon volume, it is possible that 2 times the flow could be routed to Basin No. 2A with potentially the same result. More total ammonia data would need to be collected to create a better profile of ammonia removal within Basin No. 2A to definitively analyze whether this scenario is viable.

During the winter months, however, there is still a significant portion of the total ammonia that would need to be removed by other mechanisms. Given that volatilization and ammonia uptake due to algae growth is low in the colder months, the remaining mechanism for ammonia reduction is nitrification. In order to create appropriate conditions for nitrification during the winter months, added DO, in the form of aeration or increasing the apparent detention time significantly would be necessary.

1.4.1 Addition of Mechanical Aeration

In order to achieve consistent ammonia reduction within a pond system, complete mixing is recommended to keep the biomass in suspension, improve oxygen transfer, and promote adequate nitrifier growth. Ideally, this complete mix system would be followed by a quiescent zone that would promote settling and even include a recirculation loop to aid further in nitrification and denitrification. This is similar to the process in a conventional activated sludge system.

A partially mixed system, on the otherhand, could result in significant zones of low DO reducing the overall nitrification efficiency and ammonia reduction. Another factor that can inhibit nitrifier growth in a pond system is the low food to microorganism (F:M) ratio. In a post-secondary pond system, the F:M ratios are low; in a partially mixed system, there is the added difficulty of insufficient opportunities for food and microorganisms to come into contact. Finally, even in fully aerated conditions, nitrification rates are reduced at lower temperatures. The rate at 16 deg C is roughly 50% of the rate at the optimal temperature of 30 deg C.

Based upon these factors and in discussions with aerator manufacturers, the best option for a complete mix system is using diffused air, comparable to a more conventional aeration basin. However, the shallow sidewater depths of Basin No. 1 and 2A preclude the application of diffused aeration equipment. Mechanical surface aerators are the only viable option given the physical characteristics of these basins. To achieve complete mixing with surface aeration requires a significant number of floating aerators [roughly 30 horsepower (HP) / million gallons (MG)].

Less aeration could be installed, equivalent to the air required to achieve nitrification alone (i.e., 4.6 pounds per day of O_2 for every 1 pound per day of ammonia reduced or roughly 4-8 HP / MG); however, this would then result in a partially mixed system with the issues noted above. While it would be expected that the total ammonia would be reduced over and above that which is currently happening, the manufacturers would offer no guarantee on meeting the target value, especially in the winter months.

Three manufacturers of surface aerators were contacted as part of this study: Solar Bee, Aqua-Aerobics and Blue Frog. Only the latter two would provide a conceptual level design for aeration needed to provide complete mixing of these basins. Only Blue Frog felt that their technology could consistently meet the target effluent requirements, even in the colder months.

Blue Frog offers a hybrid surface aerator with an attached growth media in the form of a submerged net surrounding the aerator. The presence of the attached growth media increases the oxygen transfer rate, improving the food to microorganism ratio and effectively increasing the HRT of the system which greatly enhances nitrifier growth. According to the manufacturer, they may be able to reduce the total ammonia to target levels even with the normal fluctuation of temperature and pH using much less aeration energy than the standard surface aerators in a complete mix system. Additional research into the validity of the manufacturer's claims and performance is needed but this might be a viable mechanical option to explore further.

A summary table of the key components of the aeration equipment proposed is included in Appendix A with their respective equipment cost proposals.

1.4.2 Nitrifying Filter Bed

Literature suggests that adding an attached-growth type media (typically coarse gravel) to the pond system could yield additional ammonia reduction. The media, approximately 1 to 2 feet in depth added to either the influent or effluent of the basin, provides a surface for the nitrifiers to grow (improving the food to microorganism ratio) as well as greatly increases the uniformity with which dissolved oxygen is added into the entire flow of the system, improving the mixing conditions. Wetland effluent is recycled back to the filter bed with a recycle ratio determined based on maintaining oxygenation throughout the profile of the filter bed. Several conditions are required for successful nitrification performance including:

- sufficient alkalinity (10 mg/L alkalinity per 1 mg/L ammonia);
- BOD to TKN ratio of less than 1; and
- maintaining moist media without flooding that creates saturated conditions.

There have been successful installations in at least 3 other free water surface wetlands in the U.S. of a nitrification filter bed that resulted in effluent concentrations of total ammonia between 0 to 6 mg/L (starting from an influent of 20 mg/L) even in winter conditions (Reed, 2014). This is a non-proprietary system developed by Sherwood Reed, a well-known wetlands expert. There are also a number of other proprietary systems available that are based upon the same concept. However, the nitrification filter bed seems to be the simplest in its approach.

1.5 Conclusions and Next Steps

Based upon the review of 5 years of historic performance of the Hayward Marsh as presented above, the existing polishing basins (Basin Nos. 1, 2A and 2B) are currently reducing the total ammonia concentration during warm weather conditions to nearly the target level of 1 mg/L-N. There may be opportunity to push the entire flow through Basin Nos. 1 and 2A in the proposed configuration. However, because it is difficult to model all of the ammonia reduction pathways taking place and therefore the impact of reduced retention time on those pathways, more field data would be needed to predict this with more certainty. It may be that reducing the flow routed to the Basins during the warm weather months will be needed to more consistently meet the target of 1 mg/L-N. This could be better determined with additional data.

While adding traditional surface mechanical aerators seems to not be cost-effective in this application, two fixed-film alternative modifications that have shown promise in increasing the nitrification potential could be explored further: nitrification filter beds and the Blue Frog hybrid aerator. Additional investigation into these technologies would be needed to understand their potential effectiveness for this application.

Woodard & Curran recommends the following next steps:

- Perform additional sampling over a period of 1 to 2 months in a warm weather period, where significant total ammonia reduction occurs. Sampling should be done to be able to create a profile of total ammonia concentration across the length of the flow path in Basin Nos. 1 and 2A. Flow path in Basin 1 is dictated by a series of baffle walls. An assessment of the flow path and potential for short circuiting in both marshes should be assessed. Recommended sampling locations include influent, mid-point and effluent of both basins as well as at quarter and three-quarter lengths along Basin No. 2A. Include sampling for BOD and alkalinity.

- Request additional information from Blue Frog Technologies about their patented technology and the potential for application at the Marsh to understand their predicted performance, lifecycle costs and maintenance requirements.

- Investigate the conceptual performance, feasibility, and capital and operating cost of the addition of a nitrifying bed filter at the influent to Basin No. 2A.

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APPENDIX A

Manufacturer	Type & Number	Power	Additional Equipment	Ammonia Target	Equipment Cost
Blue Frog[1]	Blue Frog horizontal hybrid aerators (5) Yellow Frog efficient horizontal aerators (10)	Blue Frog (3 HP) Yellow Frog (4.25 HP)	Growth Matrix Spokes with SS floating frame (10) attached to Yellow Frog Mixers	2 mg/L[2]	$603,000
Blue Frog[3]	Blue Frog horizontal hybrid aerators (5) Yellow Frog efficient horizontal aerators (14)	Blue Frog (3 HP) Yellow Frog (4.25 HP)	Growth Matrix Spokes with SS floating frame (14) attached to Yellow Frog Mixers	1 mg/L[2]	$774,000
Aqua Aerobics[4]	60 HP Aerators (15) 25 HP Aerator (44)	60 HP 25 HP	None specified	N/A	$1.2 Million

(1) All aerators placed in Basin No. 1.
(2) As provided by the manufacturer.
(3) All Blue Frog / 10 Yellow Frog aerators placed in Basin No. 1; 4 Yellow Frog aerators placed in Basin No. 2A.
(4) 60HP Aerators placed in Basin No. 1; 25 HP Aerators placed in Basin No. 2A.

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