Feasibility Assessment of Carlsbad Desalination Plant Intake and Discharge Alternative 21

CDP Report of Waste Discharge Appendix DDD

Poseidon Water

Carlsbad Desalination Plant

Carlsbad, CA

November 20, 2017

Executive Summary

Poseidon Water (Poseidon) has been contracted HDR to evaluate the intake/discharge alternatives available to Poseidon in preparation for the stand-alone operation of the Carlsbad Desalination Plant (CDP) once the Encina Power Station (EPS) ceases operation. HDR and TWB Environmental Research and Consulting (TWB) prepared this feasibility assessment to evaluate intake/discharge Alternative 21. The intake and discharge modifications evaluated under Alternative 21 are shown in Figure ES-1.

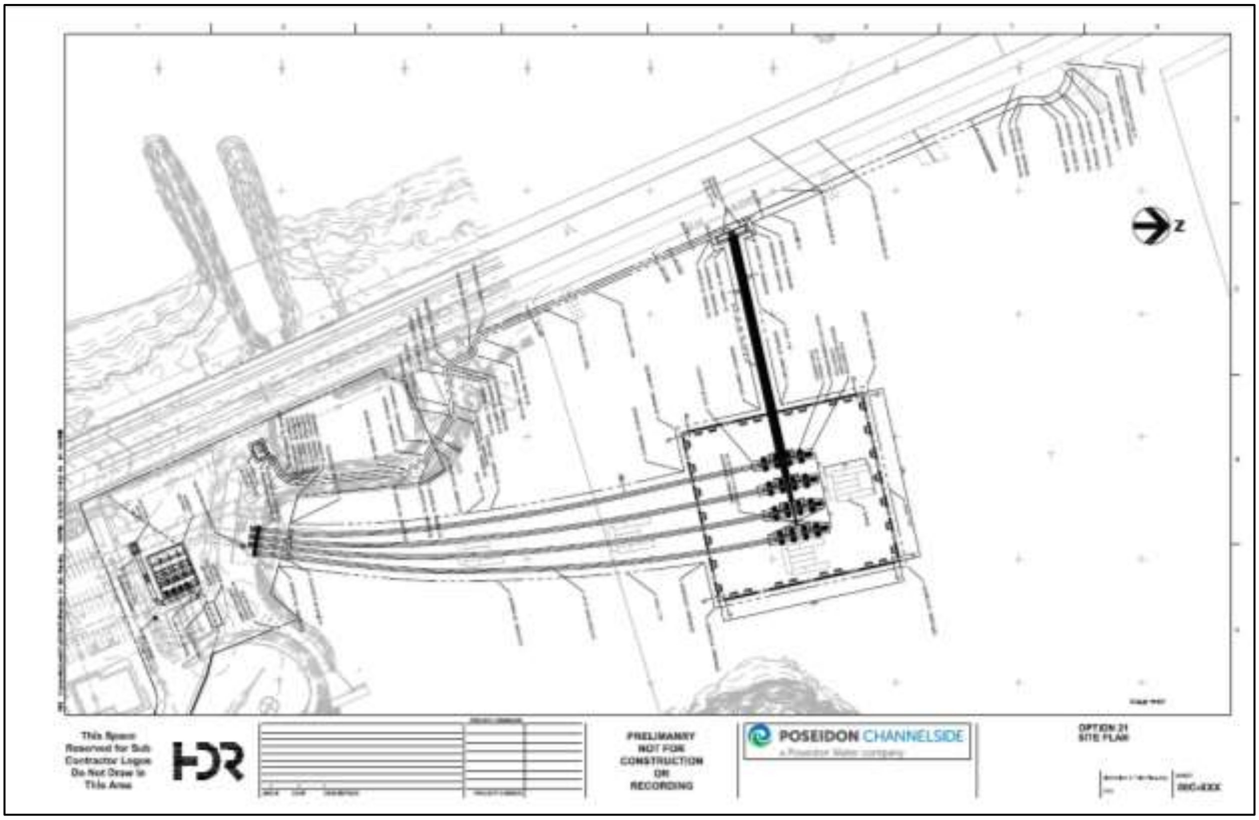

Figure ES-1. Conceptual layout of Alternative 21 Lagoon-based 1-mm passive wedgewire screens with flow augmentation for long-term stand-alone operation, plan view.

With this alternative, the submerged passive wedgewire screen (WWS) arrays are located within the Agua Hedionda Lagoon (Lagoon). The WWS will be connected to intake pipelines that will be connected to the existing intake tunnels. A new wet well/fish-friendly pumping structure will be constructed adjacent to the existing EPS tunnels. The wet well/fish-friendly pumping structure will transfer maximum of 298 MGD of screened seawater from the existing tunnels to the existing CDP intake pump station (IPS) and the fish-friendly flow-augmentation pumps. The CDP IPS will transfer maximum of 127 MGD of screened seawater to the CDP for processing. The fish-friendly flow augmentation pumps will transfer up to 196 MGD of screened seawater to the existing discharge channel for brine dilution (flow augmentation).

Since Alternative 21 involves complex construction in a marine estuary, up to five years may be required to secure the necessary permit and approvals, complete final engineering design, select a contractor, amend the Water Purchase and Operation and Maintenance Agreements, secure financing, and construct, commission, and startup the intake and discharge modifications. Local permitting efforts are expected to be complete in 2018; state and federal permit efforts would not be completed until 2020. Final design would be completed in 2021, with and estimated construction completion by 2023.

Table ES-1 summarizes the feasibility of Alternative 1, Alternative 15, and Alternative 21. The greatest feasibility concerns are associated with the technical aspects of Alternative 21. The use of narrow-slot WWS in a low-energy marine environment constitutes an operational risk since there are no performance data on such installations as proposed for this alternative. The technical challenges of implementing 1-mm WWS in the Lagoon translate into operation risks that could compromise the reliability of the CDP. In the absence of full-scale performance data, the use of WWS (active or passive) in the Lagoon also represents a significant risk to a key design feature of the CDP, which is to provide the San Diego region with a highly-reliable water supply through the use of proven technology.

The schedule for permitting, design, and construction of Alternative 21 in the Lagoon is estimated to take up to five years. During this five-year period, the CDP would need to operate in interim stand-alone mode to ensure uninterrupted delivery of potable water to the San Diego County Water Authority.

The environmental impact of Alternative 21 is greater than the other intake/discharge alternatives that are still under consideration (Alternative 1 and Alternative 15) since it requires construction in the Lagoon with an associated loss of benthic habitat. Impingement mortality is assumed to be zero and since entrainment is proportional to flow, entrainment mortality is assumed to be the same for all three alternatives.

The estimated capital and operation and maintenance (O&M) costs for Alternative 21 are substantially higher than other intake/discharge alternatives evaluated. The increased cost is associated primarily with the marine construction and greater O&M costs associated with the removal of biofouling and accumulated debris on the surface of the screens and inside the intake laterals.

When considering all the feasibility criteria, Alternative 21 is not the preferred intake/discharge alternative for the stand-alone operation of the CDP once the EPS ceases operation. More than any other criterion, the uncertainty and risk surrounding the operational performance of an intake technology in an application for which no performance data are available drive the conclusion that Alternative 21 is not feasible for the CDP. Alternative 21 has the potential to introduce reliability issues that that could impair the operation of the CDP. These concerns can be generally parsed into the following three categories: 1) the use of an existing intake technology in an unproven application, 2) the use of a technology that will require boat or barge access for cleaning and maintenance, and 3) the use of a technology that requires a cleaning/maintenance method (manual cleaning by divers) which is a higher-hazard approach than other land-based intake screen technologies.

Conclusions

The following are the conclusions and findings presented in this feasibility assessment:

- The use of an existing intake technology in an unproven application represents a technical risk to the reliable operation of the CDP
- The cleaning and maintenance requirements are high due to uncertainty relative to performance of narrow-slot WWS in the Lagoon
- The cleaning of the intake laterals via pigging creates challenges associated with debris management and meeting the terms of the Water Purchase Agreement regarding allowable days offline
- The schedule for permitting, designing, and constructing a structure in the Lagoon will take up to 5 years – longer than alternatives that do not require construction in the Lagoon
- The total environmental impact is greater than other alternatives due to the permanent loss of benthic habitat in the Lagoon
- The cost is greater than other alternatives due to requisite in-water construction and increased maintenance anticipated

Table ES-1 presents a summary of the feasibility assessment of Alternative 21. It also compares the environmental impact, cost, and schedule aspects of Alternative 21 to the other Alternatives under consideration (Alternatives 1 and 15). Table ES-1 indicates that Alternative 21 has a greater total environmental impact (related principally to the permanent loss of benthic habitat in the Lagoon), a higher cost (capital and annualized), and a longer schedule. For those reasons, Alternative 21 is not feasible given the alternatives available.

Table ES-1. Summary of Feasibility Assessment				
Feasibility Criteria	**Impact Assessment Method**	**Alternative**		
		1	**15**	**21**
Environmental Impact		**Impacted Area (Acres)**		
Intake	APF calculated per Appendix E of the Staff Report/SED to the Ocean Plan Amendment using a 95% confidence bound for an assumed 100% mortality of all forms of marine life entrained by 127 MGD CDP process water with an APF of 35.76 acres and 171 MGD flow augmentation with an APF of 47.68 acres after accounting for a 1% credit for 1 mm screening technology.	83.44	83.44	83.44
	Potential mortality associated with the operation of the fish return system.	0.93	0.85	0
Discharge	Area within the BMZ potentially exposed to a salinity in excess of 2 ppt over natural background salinity.	18.51	18.51	18.51
Construction	Permanent footprint of intake/discharge components within lagoon.	0.10	0.10	4.2
Total Environmental Impacts (Acres)		**102.98**	**102.90**	**106.15**
Cost	Capital Cost	**$49,000,000**	**$53,400,000**	**$58,800,000**
	Annualized Cost (Capital and O&M)	**$7,860,000**	**$8,200,000**	**$11,030,000**
Schedule	Expected Operation Date of Ocean Plan Compliant Intake and Discharge Facilities	**2021**	**2021**	**2023**
Conclusion	Overall Feasibility Assessment	**Feasible**	**Feasible**	**Infeasible[1]**

[1] Significant operational reliability concerns, environmental impacts to Lagoon, significant increase in capital and O&M costs for minimal reduction in marine life mortality

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1. Introduction

The Carlsbad Desalination Plant (CDP) is currently permitted to produce up to 56,000 acre feet per year (AFY) equivalent to 50 million gallons per day (MGD) average flow. of desalinated water while operating in conjunction with the Encina Power Station (EPS) by using the power plant's cooling water discharge as its source water. The planned retirement of the EPS at the end of 2018 will result in the need to retrofit the CDP for a transition to stand-alone operation.

There is also potential to increase the rated CDP capacity to realize the improvements in reverse osmosis membrane production capabilities since the original CDP approvals. The membrane technology advances enable the CDP to increase potable water output from an annual average of 56,000 AFY (maximum production rate of 54 MGD) to an annual average of 62,000 AFY (maximum production rate of 60 MGD) with minimal plant improvements. Therefore, this feasibility assessment assumes the maximum production rate of 60 MGD.

Poseidon previously evaluated 20 alternative intake/discharge designs. Appendix II (an addendum to the original CDP Intake/Discharge Feasibility Study) was prepared by HDR on August 12, 2016 and included an intake/discharge alternative utilizing wedgewire screens (WWS) in Aqua Hedionda Lagoon (Lagoon). In that analysis, the WWS array was located within approximately 100 ft of the existing EPS intake structure to provide the submergence required for the WWS and to minimize use conflicts with the Carlsbad Aquafarm.

At the September 27, 2016 meeting with the San Diego Regional Water Quality Control Board (RWQCB), staff requested that Poseidon evaluate the Lagoon WWS alternative in more detail. Subsequently, a technical memo prepared by HDR (Appendix SS - Technical Memorandum: Feasibility of Cylindrical Wedgewire Screens in Agua Hedionda Lagoon) was submitted on October 31, 2016 with a more detailed review of a potential Lagoon WWS alternative. The results presented in the technical memo were also presented to staff in person during the November 2, 2016 RWQCB meeting.

This current feasibility assessment of CDP intake/discharge Alternative 21 is in response to the RWQCB's October 13, 2017 request for a feasibility assessment of WWS and inlet laterals located in the Lagoon to provide seawater for processing at the CDP and for brine dilution purposes (flow augmentation). Feasibility criteria considered in this assessment include technical, schedule, environmental, operational reliability and cost considerations.

2. Description of Alternative 21 Intake/Discharge Modifications

A. General

The intake and discharge modifications evaluated under Alternative 21 are shown in Figure 1**Error! Reference source not found.**. The submerged passive WWS arrays are located within the Lagoon. The screens will be connected to intake pipelines that will be connected to the existing intake tunnels. A new wet well/fish-friendly pumping structure will be constructed adjacent to the existing tunnels. The wet well/fish-friendly pumping structure will transfer up to 298 MGD of screened seawater from the existing tunnels to the existing CDP intake pump station (IPS) and the fish-friendly flow-augmentation pumps. The CDP IPS will transfer up to 127 MGD of screened seawater to the CDP for processing. The fish-friendly flow augmentation pumps will transfer up to 196 MGD of screened seawater to the existing discharge tunnel for brine dilution (flow augmentation).

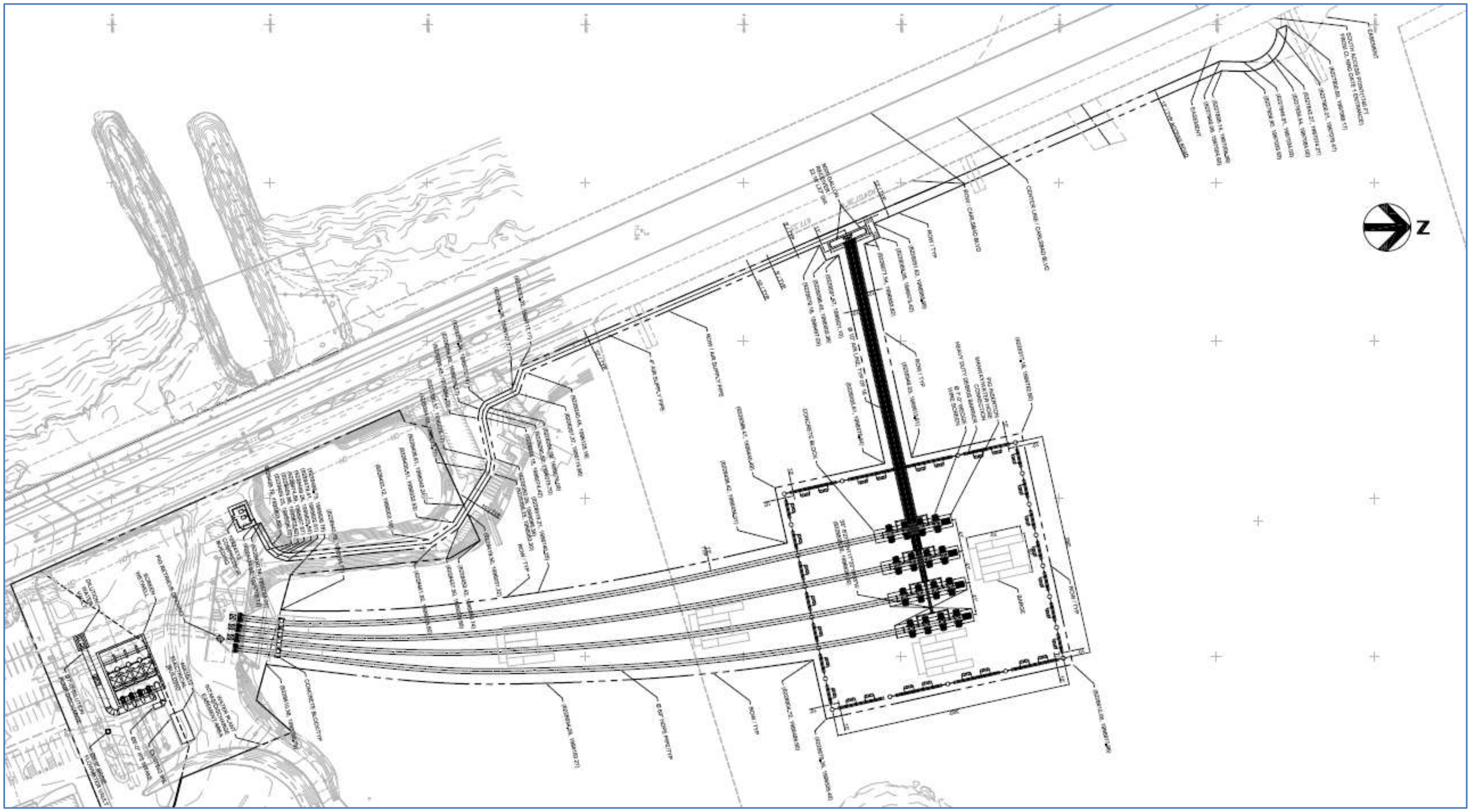

Figure 1. Conceptual layout of Alternative 21 Lagoon-based 1-mm wedgewire screens with flow augmentation for long-term stand-alone operation, plan view.

B. Schedule

i. Interim Stand-Alone Operation

The CDP would rely on interim stand-alone operation until the intake and discharge modifications are ready for commercial operation. This would be accomplished through: 1) the use of the existing traveling water screens and cooling water pumps at the EPS, 2) new traveling water screens to match EPS screening requirements and pumps that are installed solely to bridge the gap between when the EPS facilities are no longer available and when the new Ocean Plan-compliant intake and discharge facilities are ready to go into service, or 3) a combination of existing and new screens and pumps.

ii. Ocean Plan Compliance

As noted in Table 1, the intake and discharge modifications contemplated under Alternative 21 are expected to achieve full Ocean Plan compliance within five years of the RWQCB approval of the Renewed Order and Water Code Determination.

Table 1. Project schedule for intake/discharge Alternative 21.

Project Implementation Requirements		Expected Completion Date
Local Permits and Approvals	CEQA compliance	2018
	City of Carlsbad Precise Development Permit Amendment	2018
State Permits and Approvals	Regional Water Board NPDES Permit Renewal and Water Code Determination and 401 Water Quality Certification	2018
	California Coastal Commission Coastal Development Permit Amendment	2018
	State Lands Commission Lease Amendment	2018
Federal Permits and Approvals	NEPA review	2019
	Army Corps of Engineers 404 Permit	2020
	NMFS/NOAA Biological Opinion	2020
Pre-Construction	Final engineering design, contractor selection, amendment of Water Purchase and Operation and Maintenance agreements, and financing	2021
Construction	Construction, Commissioning, and Startup of intake and discharge system modifications	2023
Operation	Commercial operation of intake and discharge system modifications	2023

3. Site

New structures would be constructed in the Lagoon to support the arrays of WWS. The WWS arrays would be located approximately 800 feet from the existing intake trash rack at a Lagoon floor depth of 20 feet below MLLW. This location was selected to provide the greatest potential for exposure to tidal-related sweeping currents. This location also provides the submergence required for the WWS. The WWS arrays would be surrounded by a floating debris boom. Detail of the placement and bathymetry survey is provided in Attachment A.

Four 63-in diameter intake pipelines (laterals) would convey the withdrawn water from the WWS arrays to a new wet well west of the existing IPS. The intake laterals would be laid on the Lagoon floor and ballasted with concrete collars. The WWS arrays at the end of each lateral would be supported/anchored by concrete gravity bases. The new wet well onshore would function as a common plenum from which SWRO process water flow would be drawn by the existing pumps at the IPS and from which augmentation flow would be drawn by fish-friendly axial flow pumps. A total flow of 298 MGD would be withdrawn: up to 127 MGD through the process water side and up to 196 MGD through the flow augmentation side.

Feedwater and flow augmentation water for the CDP would be withdrawn through the new WWS arrays from the Lagoon; there would be no change from the current source waterbody. The new WWS array would require significant in-water construction activity, most of which would be accomplished from a derrick barge moored in the Lagoon.

Brine from the CDP would be mixed with augmentation flow in the existing EPS discharge tunnel and ultimately discharged to the Pacific Ocean. There would be no change in the receiving waterbody nor would the discharge plan require any structural modification to the existing EPS discharge pond or ocean outfall. A general schematic of the Alternative 21 intake/discharge layout is provided in Figure 2.

An amendment to the lease agreement would be required from NRG for the Lagoon installation site. Based on the dimensions of the design (and allowing 5 feet on each side of installed equipment), a lease of approximately 4.2 acres would be required for the intake laterals, the Lagoon-based WWS arrays, and the floating debris boom.

Under this option, approximately 298 MGD of seawater would be withdrawn directly from the Lagoon – up to 127 MGD for processing by the CDP and up to 196 MGD for brine dilution. At potential maximum production, approximately 60 MGD of the diverted seawater would be converted to fresh water which would be piped to the San Diego County Water Authority's delivery system in the City of San Marcos. The remaining flow (up to 67 MGD) would be returned to the EPS discharge tunnel for blending with seawater prior to discharge to the Pacific Ocean. The discharge would consist of brine produced by the reverse osmosis (RO) process (up to 60 MGD) and treated backwash water from the pretreatment filters (up to 7 MGD). The salinity of the discharge prior to dilution would be approximately 65 ppt (67 ppt with no backwash water included), whereas the average salinity of the ambient seawater in the vicinity of the discharge channel is 33.5 ppt. Poseidon is proposing an initial dilution of the brine to a maximum of 42 ppt in the discharge pond prior to discharge to the Pacific Ocean. This would be accomplished by mixing the CDP discharge with 171 MGD of the seawater withdrawn from Pacific Ocean for flow augmentation purposes. The combined CDP discharge and dilution water flow rate would be approximately 238 MGD. As compared to the existing project operations, the CDP operations described above could achieve up to a 10% average annual increase in fresh drinking water production while reducing total quantity of seawater required for processing and flow augmentation purposes.

The Desalination Amendment (at III.M.3.d) provides that the discharge shall not exceed a daily maximum of 2.0 parts per thousand (ppt) above natural background salinity measured at the edge of the brine mixing zone (BMZ) 200 meters (656 feet) seaward of the end of the outfall channel (SWRCB 2015). Over the last 20 years, the natural background salinity at the closest reference site (Scripps Pier) has measured a minimum salinity of 30.4 ppt, maximum salinity of 34.2 ppt, and an average salinity of 33.5 ppt (Jenkins 2016). Therefore, under average conditions, the discharge shall not exceed a daily maximum of 35.5 ppt at the edge of the BMZ (200 meter [656 foot] radius).

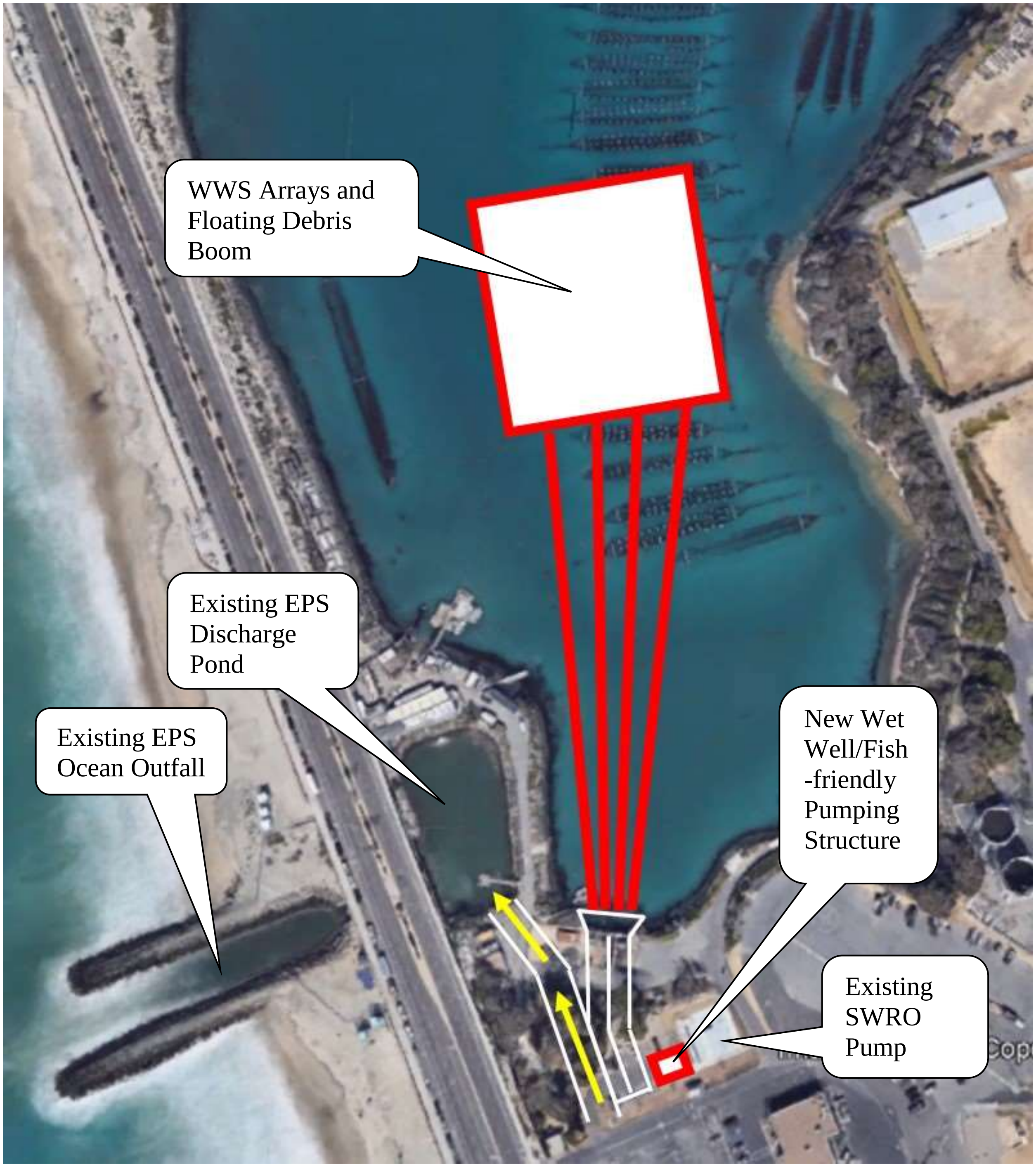

Figure 2. Alternative 21 Schematic. CDP with Lagoon-based 1-mm wedgewire screens and flow augmentation

4. Design

The following sections describe the major components and hydraulic design of intake/discharge Alternative 21, consisting of the submerged intake laterals, the WWS arrays, and other miscellaneous items. These facilities are identified on the conceptual drawing provided as Attachment B.

A. Design of Major Project Components

i. Intake Laterals

The intake system will be comprised of four 63-in diameter intake laterals (3 + 1 standby) that are each approximately 800 ft. long. The pipe material will be high-density polyethylene (HDPE) which provides corrosion resistance and a slick internal surface to discourage the settlement of fouling organisms. Installation of the HDPE laterals will be by floating the assembled pipe into place, ballasting the pipe with concrete collars (which will also serve to anchor the pipe to the Lagoon floor), and finally flooding the pipe with seawater to submerge the pipe on the Lagoon floor.

The offshore end of each lateral will include a 100-ft long, 63-in diameter super duplex stainless steel header. Each header will include riser connections for four WWS units. The trash racks will be removed from the existing intake and the four laterals will be connected into the intake structure such that the intake is only able to withdraw water from the laterals. Each of the laterals will be equipped with an access port on the upstream end of each lateral to accommodate cleaning and maintenance. The connection to the existing intake will be designed to accommodate debris removal as described in Section - Intake Pipe Cleaning below. Hydraulic calculations have been performed by HDR. However, a CFD and/or physical modeling has not been performed as of yet.

ii. Wedgewire Screens

Two different types of WWS were considered for Alternative 21: active and passive. Active screens provide mechanical cleaning and passive screens contains no mechanical components. Both types of screens are described below. Screens were evaluated assuming a maximum intake capacity of 298 MGD, with 1-mm slot widths and through-slot velocity that cannot exceed 0.5 feet per second (ft/sec). Cut sheets for each WWS type are provided in Attachment D. Section - Intake Screening Technology provides detail on the two WWS technologies evaluated; this section, however, describes the general design of the WWS intake and is insensitive to the WWS technology selected.

The WWS would be mounted on the risers from the header. Each of the four laterals will have four WWS, for a total of 16 WWS. The four laterals will be in a 3+1 arrangement (total of four intake laterals with one that can be taken out of service) with each lateral and header generally oriented north to south. The WWS on each lateral header would be oriented perpendicular (east to west) to the header in order to maximize exposure to tidal-related sweeping currents.

Each WWS array would be comprised of four 84-inch diameter WWS with 1mm slot widths (Figure 3). Screens would be spaced per vendor recommendations and equipped with an air burst cleaning system. The air burst cleaning system would rely of the natural ambient tidal sweeping currents to carry liberated debris away from the screens. Given the concerns over the use of copper nickel screening material (i.e., potential for leaching copper into the water) the screens would be fabricated from super-duplex stainless steel. The screens would be cleaned regularly by divers to control biofouling on the screens.

The screens are designed to maintain a through-slot velocity of 0.5 ft/sec or less under all expected operating conditions. The concept design includes a fouling factor of 15%, meaning that under a clean condition, the design through-slot velocity would be 0.43 ft/sec with one of the laterals out of service. All 16 screens would be operable when the CDP enters long-term standalone operational mode, meaning the through-slot velocity would be well below 0.5 ft/sec.

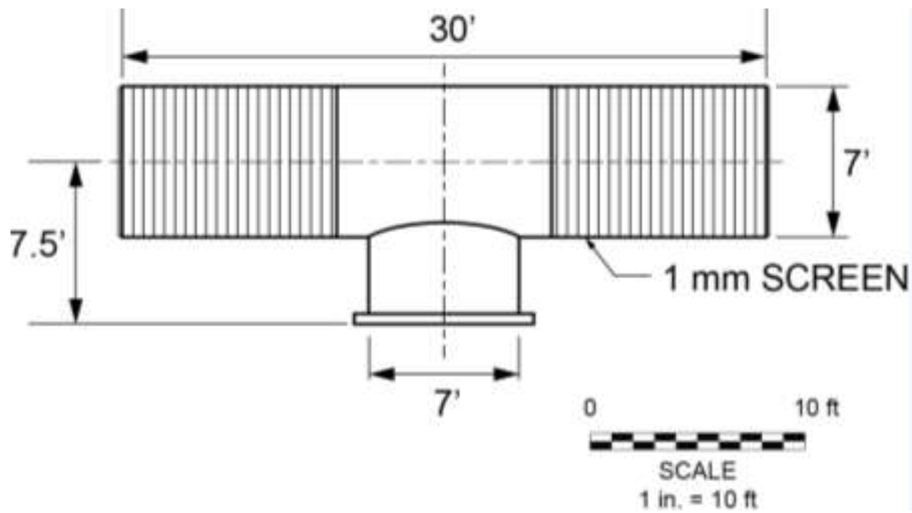

Figure 3. 84-in diameter cylindrical wedgewire passive screen proposed for CDP intake/discharge Alternative 21.

B. Hydraulic Design

Various hydraulic calculations were completed to evaluate Alternative 21; they are described in greater detail below. The primary criteria for the hydraulic calculations included:

- Peak intake flow rate: 298 MGD
- Maximum through-slot velocity: 0.5 ft/sec
- Water surface elevation (maintain gravity flow from intake to pump stations)
- Mean lower low water (MLLW) level: -2.3 feet per NGVD29 survey datum

The hydraulic calculations are provided in Attachment C with descriptions summarized in the following sections.

Figure 4 illustrates the hydraulic profile through the intake system with one lateral out of service (three 63-in Average Outside Diameter [OD] HDPE pipes in service) and an assumed amount of biofouling allowed without impacting maximum desired intake flow capacity. During the preliminary design phase, the optimum diameter for the laterals would be determined to balance hydraulic with marine growth assumptions. Specific hydraulic considerations for each primary component are described in the following sections.

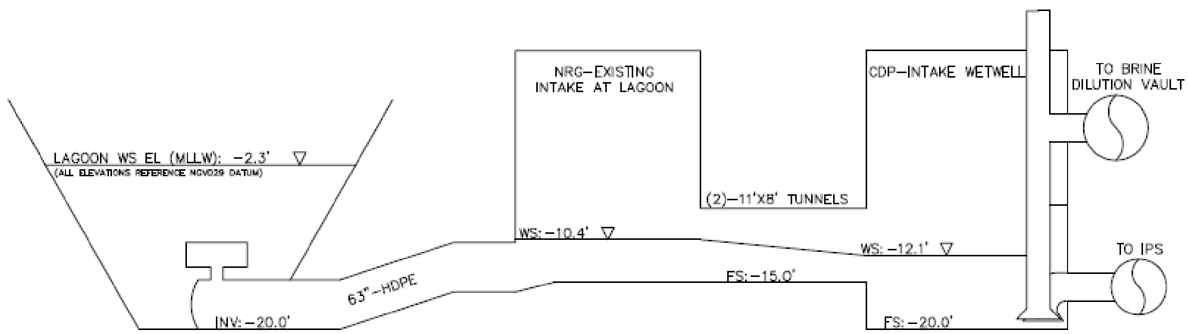

Figure 4. Worst-case hydraulic profile for Alternative 21.

i. Wetwell Structure

Hydraulic calculations were completed to determine the minimum wet well/fish-friendly pumping structure depth necessary to allow intake flow by gravity under variable operating conditions. The other dimensions, including pump sizing, pump submergence requirement, and pipeline connections are described for Alternatives 11-14.

The hydraulic calculations indicated that the use of both existing EPS intake tunnels provides hydraulic benefits and redundancy; therefore, both intake tunnels were assumed to be necessary for optimal hydraulic performance. The size and number of intake laterals was also shown to have a significant impact on hydraulics (Table 2 and Table 3). The worst-case operating condition was with one intake lateral out of service.

The presence of a potential hydraulic jump (velocity change) at the transition between the intake laterals and the new wet well/fish-friendly pumping structure limited the design choices. The axial flow, fish-friendly pumps require 6 ft of submergence (plus an additional 2 ft of water column buffer) for proper operation.

The head loss impacts of biofouling in the WWS and the intake laterals were also evaluated. Fouling of the WWS (assumed to be 15%) had only minor impacts of the determination of the wet well/fish-friendly pumping structure depth. Fouling of the intake laterals had a significant impact on the number and size of pipelines due to the increased roughness factor and the reduced cross-sectional area available for passing flow. For determination of the wet well/fish-friendly pumping structure depth, a C-factor of 100 was assumed (a C-factor of 140 is typical for new HDPE) which resulted in minimal loss of capacity with one lateral out of service.

Based on the criteria described above and the calculations provided in Attachment C, a wet well/ fish-friendly pumping structure elevation of -20 feet is necessary.

ii. Intake System

The intake system consists of the WWS, the intake laterals, and existing intake tunnels. The maximum head loss allowable through the intake system was calculated to be approximately 10 feet using the following upstream and downstream criteria.

- Upstream: MLLW water surface elevation of -2.3 feet
- Downstream: Elevation of -12 feet, consisting of:
  - 6 feet of water column above pump intake at -20 feet
  - Approximately 2 feet of buffer
  - Prevention of hydraulic jump at the transition from the intake tunnels to the pump station wet well

HDR evaluated multiple variations of intake laterals, lateral diameter, lateral material, number of intake tunnels used and amount of biofouling along with considerations for redundancy as described in Section - Redundant Screens and Laterals. To meet the desired criteria, the preferred arrangement is for a 3+1 (total of four intake laterals with one that can be taken out of service without violating the 0.5-ft/sec through-slot velocity), 63-in diameter HDPE pipeline configuration for the Lagoon intake system proposed for Alternative 21.

iii. Biofouling Impacts on Hydraulics

Biological growth (biofouling) is a significant issue when evaluating hydraulics for the CDP facilities that convey ocean water. Marine organisms are expected to attach to all wetted facilities, increasing surface roughness and decreasing carrying capacity. Therefore, both hydraulics calculations and provisions for maintenance are necessary to prevent capacity restrictions during operations. Maintenance provisions are discussed in Section - Operation and Maintenance. Table 2 summarizes the calculated allowable biofouling thickness on the intake system conveyance facilities before intake capacity becomes restricted. These calculations indicate that biofouling in the laterals can reach a thickness of almost 6 in with all four laterals in service before capacity becomes restricted enough to impact the plant’s design capacity.

Table 2. Biofouling impacts on the hydraulic design of CDP intake/discharge Alternative 21.

Description	Laterals in Service	
	3	4
Calculated Allowable Bio-growth Thickness (in)	2.82	5.81
Calculated Pipeline Velocity (fps)	9.67	9.17
Resulting Pump Station Water Surface Elevation (ft)	-12.61	-12.61

Note: In the absence of site-specific data, the evaluation of fouling rates, blinding issues, cleaning frequencies, and screen replacement frequencies should be considered estimates.

C. Redundancy

Redundancy for the proposed flow-augmentation, fish-friendly pumping structure was evaluated and summarized in Alternatives 11 -14, so only the intake system portions within the Lagoon are considered here. Redundancy of the WWS and the intake laterals was considered as described in the following sections.

i. Redundant Screens and Laterals

One extra intake lateral with four screens would be provided for redundancy. Providing redundancy of an entire lateral has distinct benefits from an operation and maintenance (O&M) perspective and from a constructability perspective.

With a redundant lateral, full plant production would be unaffected if a lateral needed to be taken out of service, (except in the cases where pigging a lateral is required). In addition, there is a small buffer which allows some biofouling to occur without restricting the intake capacity (Table 2). With the proposed Alternative 21 design, isolating a lateral to take it out of service can also be done from shore with stop logs. Various arrangements were considered that provided redundancy between 20 and 50% depending on the number of laterals included. Table 3 summarizes the ranges of redundancy considered for the intake laterals.

Table 3. Evaluation of various redundancy schemes for the CDP Alternative 21 intake laterals.

Lateral Configuration	Nominal Diameter, (in)	Internal Diameter (in)	Screens Per Lateral/Number of Redundant Screens	Total Screens
1+1	96	96	12	24
2+1	72	72	6	18
3+1	63	59.7	4	16
4+1	52	49.6	3	15

Relative to the configurations evaluated in Table 2, the 3+1 configuration was selected because it results in a pipe diameter that is commercially available and can be welded with standard equipment aids in minimizing the proposed Alternative 21 capital costs. Configurations that rely on fewer laterals require pipe diameters that are less readily available and are not weldable with standard equipment. The 3+1 configuration also translates well to the existing intake structure which has 4 intake trash rack bays into which each of the four 63-in diameter pipes will fit.

D. Operation and Maintenance

Maintaining submerged facilities has a significant impact on their operability. Therefore, multiple screen and intake system maintenance methods were considered. The selected methods are summarized in the following sections. In addition, the O&M-related project components not described elsewhere (e.g., the pipe cleaning system, the floating debris boom in the Lagoon, the submersible camera for WWS inspection, and the barge for boom and WWS maintenance) are described in greater detail in this section.

i. Wedgewire Screens

Bio-fouling and free-floating debris are two separate concerns when considering screen cleaning. Several methods for addressing the maintenance requirements for both bio-fouling and floating debris are described below.

a. Biofouling

The passive screens are assumed to be cleaned in place by divers that will be based on a floating barge. Visual inspections will occur periodically using a submersible camera to determine cleaning requirements. An entire lateral would be isolated to clean all screens along a lateral at one time. The screen exterior and interior would be cleaned as follows:

- Exterior—Divers would use a combination of manual cleaning with brushes and hydro-blasting using pressurized water spray nozzles on the external surfaces of the screens. The seawater used for hydro-blasting would pass through one of the adjacent screens prior to use. Biofouling debris removed from the exterior of the screens would remain in the Lagoon. Accumulated biofouling debris (as well as any accumulated silt, sand, and sediment) near the screens will be removed periodically via suction dredging from a maintenance barge. The dredged material would be discharged to a tank mounted on the barge that would filter the material from the water using siltation curtains before returning the water to the Lagoon.

- Interior—Both manual cleaning and hydro-blasting would be used in the internal surfaces of the screens. Divers would enter the screen via hatches (likely at one of the endcaps). Any biofouling debris that has released from within the screen would be removed using a trash pump. The trash pump would discharge to a tank mounted on the barge that would filter the biofouling debris from the water using siltation curtains before returning the water to the Lagoon. Solids collected would then be dewatered and hauled offsite for disposal.

Screen cleaning would occur as frequently as necessary to ensure the screening system is able to meet the CDP's intake requirements. Under typical operating conditions, the expectation is that the screens would be cleaned once a month (12 cleanings annually). During challenging conditions such as winter storm events or algal blooms, more frequent cleaning may be required to manage free-floating debris that may collect on or near the WWS (see following section).

b. Free-Floating Debris

The nature of free-floating debris is different than biofouling on the screen face. Some free-floating debris can be liberated by airburst cleaning or it could be swept from the screen face by ambient sweeping currents. An airburst system would be used to attempt to dislodge debris that may collect on screens. The airburst system would consist of two receiver tanks (5,000 to 10,000 gallons), two air compressors with structure, and conveyance piping. Manual cleaning of the screens by divers would be conducted as needed during the monthly screen cleaning events to remove floating debris that may accumulate on the screens.

A floating debris boom/curtain around the intake screens would block floating debris from entering the screening area. The floating debris boom extends from the surface three feet down into the water. The debris boom would be a solid barrier rather than a mesh to avoid marine life impacts. The debris boom will act as a stand-off zone to prevent the public from entering the screened area where airbursting will occur. Portions of the floating debris boom would be adjustable to allow for surface vessel entrance/exit to the protected area. The boom would also have to be maintained by manually removing floating debris that may accumulate. Biofouling should not impact the effectiveness of debris boom, so cleaning is likely to be infrequent.

An air-burst system and floating debris boom have been included in the Alternative 21 intake concept. In addition to these control approaches, challenging conditions may require additional cleaning efforts. Therefore, in addition to the monthly biofouling cleaning events (during which free-floating debris can also be cleared from the WWS), additional cleaning events are anticipated to effectively manage free-floating debris during challenging conditions.

The following constitute the principal threats to WWS operation during challenging conditions:

- During winter storms, dislodged kelp can enter the Lagoon, coalesce into large mats, and threaten WWS operation. EPS operators have acknowledged the potential influx of kelp as a debris management issue. There is potential for long pieces of kelp to wrap around the WWS.
- Free-floating eelgrass can impinge on screen faces. If not swept away by currents or airbursting, large influxes of eelgrass have potential to occlude screening surface area.
- Sand, silt, and sediment can accumulate near the WWS. To prevent the potential for ingestion, periodic suction dredging from a maintenance barge would be required.

Due to these additional uncertainties related to the operational performance and debris handing capabilities during challenging conditions, additional cleaning efforts are anticipated. These additional cleaning efforts may include manual cleaning/debris removal by divers, use of the maintenance barge for collecting floating debris, and use of the maintenance barge for suction dredging accumulated debris that has settled near the WWS. Therefore, annual costs reflect cleaning efforts in excess of the monthly WWS cleanings for controlling biofouling.

ii. Intake Pipe Cleaning

Two methods were considered for removing biofouling expected to accumulate on the internal surfaces of the intake laterals. Both physical and chemical methods were considered and are described further in the following sections.

a. Physical Pipe Cleaning

Pipe pigging (Figure 5) was evaluated as a primary method for removing biofouling from within the intake laterals. Pigging would be conducted quarterly and will require a shutdown of 1 day (per lateral) for each pigging event (i.e., a total of 16 pigging events per year).

The following provides details of how the cleaning would be performed and the facilities that are required to conduct the work. The frequency of pigging would be dependent on the amount of biofouling that can be allowed without impacting intake system capacity (see Section - Biofouling Impacts on Hydraulics).

Figure 5. Pig insertion for physical pipe cleaning (image courtesy Ridge Runner Pipeline Services).

Pipe pigging would be done in an offshore to onshore direction, moving from the WWS towards the Lagoon shoreline. Pigging in this direction ensures that the debris removed from the pipes' internal surfaces can be efficiently collected; pigging in an onshore to offshore direction would make collection of the debris more difficult. The pig would be launched from a barge and the water pressure to drive the pigging process would come from a barge-mounted pump taking suction from an intake lateral (so that pumped flow has been screened through the WWS).

HDR evaluated alternatives pig launching locations including downstream of the screens (which would require manual cleaning of the WWS header), through a top-mounted access hatch at the terminus of the WWS header (design provisions must ensure hatch operability after seawater submergence), through a blind flange at the terminus of the WWS header (design provisions must prevent sedimentation which could prevent access to the flange). For the last two alternatives in which the pig would be inserted upstream of the screens, design provisions (e.g., diver-installed inflatable plugs) must be included to prevent debris from being forced in to the screens during pigging operations.

Debris removed by pigging and additional flushing water would be directed to the discharge pond. Existing stop logs in the existing tunnels will be used to divert the flow and debris into the existing discharge tunnel and ultimately into the pond. The pig would be retrieved onshore trough an opening in the deck.

The management of the pigging debris will be accomplished through two separate means: 1) hydraulic sorting (settling) of solids based on particle size and velocities in the discharge pond and 2) temporary physical barriers (silt curtains). Each is described in more detail below.

A temporary barrier will be installed in the discharge pond (in a north-south orientation) to extend the flow path of the pigging discharge considerably. This extended flow path will offer greater retention time and increase the opportunity for settling of suspended solids. Pigging would be conducted while the plant is offline. The only flow entering the discharge pond during the pigging operation would be the volume of the pipeline being cleaned, which would significantly reduce the velocity and increase the retention time and settling rate in the discharge pond.

In addition to the hydraulic sorting provided by the barrier, the use of a temporary silt curtain will provide a physical filtration barrier to control discharge of the smaller suspended particulates. The temporary silt curtain will be designed to be used only during pigging operations; during normal operations, the silt curtain will be removed.

Dredging of the discharge pond will be conducted as needed to remove any accumulated debris. Dredging operations would be designed to comply with the California Ocean Plan Water Quality Objectives.

The additional need to use the discharge pond for management of pigging debris and the subsequent need to periodically dredge the discharge pond for removal of the accumulated pigging debris adds to the requirements for having the CDP to be off-line and contributes to a reduction of the overall reliability of the CDP and will result in an O&M cost increase when compared to alternative not requiring additions of new intake piping requiring pigging.

b. Chlorine Injection

Chlorine injection is not being considered for removal of biofouling within the intake laterals due to the potential for chlorine being released into the Lagoon. Although the use of chlorine injection as a screen biofouling control and intake pipeline control is commonplace for other seawater intakes around the world, Poseidon assumes that chlorine injection will not be permissible. For this reason, Alternative 21 includes a maintenance approach that constitutes best practice for controlling biofouling in the absence of features designed to minimize biofouling (copper nickel screen material and chlorine injection).

E. Alternative 21 Design Summary

Table 4 below provides a summary of the temporary stand-alone and Ocean Plan-compliant intake modifications for Alternative 21.

Table **5** provides an overview of the design criteria used for the Ocean Plan-compliant Alternative 21.

Table 4. Summary of principal project components for Alternative 21 relative to the CDP operational status.

Operational Status	
Interim Stand-Alone	**Ocean Plan-Compliant**
New (temporary) pumps and piping connection to provide 298 MGD to the existing discharge tunnel upstream of the IPS pump station	Four 63-inch (or larger, pending chlorine use determination) HDPE laterals
Existing or new (temporary) traveling water screens to match EPS screening requirements	Four 7-foot diameter passive WWS per intake lateral (16 total)
Electrical building	Intake lateral connections to existing inlet structure and improvements necessary for pipeline maintenance
	Airburst system consisting of two compressors, two air receivers, associated electrical and associated piping to WWS
	Floating debris boom/curtain
	Barge for screen and pipeline maintenance
	Flow-augmentation fish-friendly pumps and piping connection to discharge tunnel
	Flow-augmentation fish-friendly wetwell with connection to both intake tunnels
	Wetwell connection to the existing IPS

Table 5. Summary of design criteria for the Ocean Plan-compliant Alternative 21.

Description	Value	Unit
Design Capacity	298	MGD
Laterals in Operation	3	#
Screens per Lateral	4	#
Minimum Lateral Inside Diameter (ID)	57.9	in.
Percent Effective Screening Area	36	%
Allowable Fouling	15	%
Maximum Through-slot Velocity	0.5	ft/sec
MLLW NGVD29 datum	-2.3	feet

5. Technology

A. Intake Screening Technology

Two different types of WWS were considered for Alternative 21: active and passive. Active screens provide mechanical cleaning and passive screens contains no mechanical components. Both types of screens are described below. Screens were evaluated assuming a maximum intake capacity of 298 MGD, with 1-mm slot widths and through-slot velocity of 0.5 ft/sec or less. Cut sheets for each WWS type are provided in Attachment D.

i. Active Screens

Active WWS have rotating screen sections and stationary external and internal brushes that reduce the need for manual cleaning and can be made of nearly any material desired. The system cleans both the inside and outside surfaces. An active screen is illustrated in Figure 6 below.

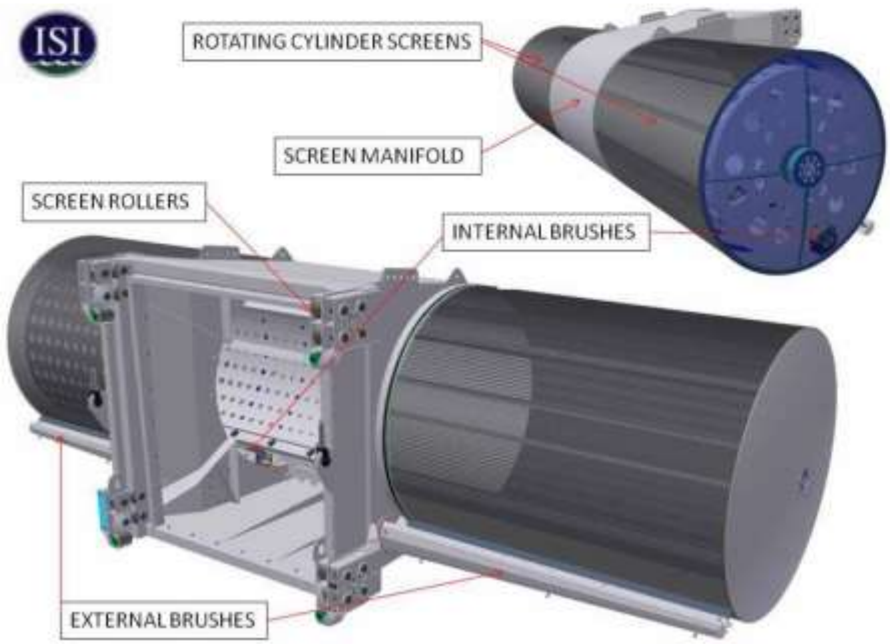

Figure 6. Rotating, brush-cleaned wedgewire screen (active screen) from Intake Screens, Inc. (image courtesy ISI).

According to the manufacturer, a total of 16, 7-foot diameter screens are recommended to meet the design criteria (flow rate, slot width, through-slot velocity, and redundancy). However, there are no operating performance data for a comparable intake system (narrow slot width and large flow) in seawater. Intake Screens, Inc. (ISI) is the only known manufacturer of rotating, brush-cleaned screens. ISI has many installations in fresh water, but their experience with such screens in seawater has been very limited. To date, ISI has one full-scale seawater installation at the Exploratorium (a science museum) in San Francisco Bay (FIG). The intake flow rate is 2 MGD which is screened by one screen. The intake includes two (one for redundancy) 36-in diameter drum screens (Figure 7). The screens are 28 in long, have 1.75-mm slots, are fabricated of 316L stainless steel, and are track-mounted to allow frequent inspection. Screens are typically inspected weekly. Consequently, there are no commercially-comparable existing installations of this technology; therefore, there is no reliable application data that would support establishing the design and operation requirements for use of an active screen at the CDP.

In the absence of comprehensive operational performance data in seawater, it may be difficult to solicit bids from such vendors that would have to meet reliability/performance guarantees set by the Engineer, Procure, Construct (EPC) contractor or the multiple proposal requirement of the Water Purchase Agreement (WPA). Given the scale of the costs involved and virtually nonexistent application data, the active screen technology is not a feasible option for use with the CSD Plant

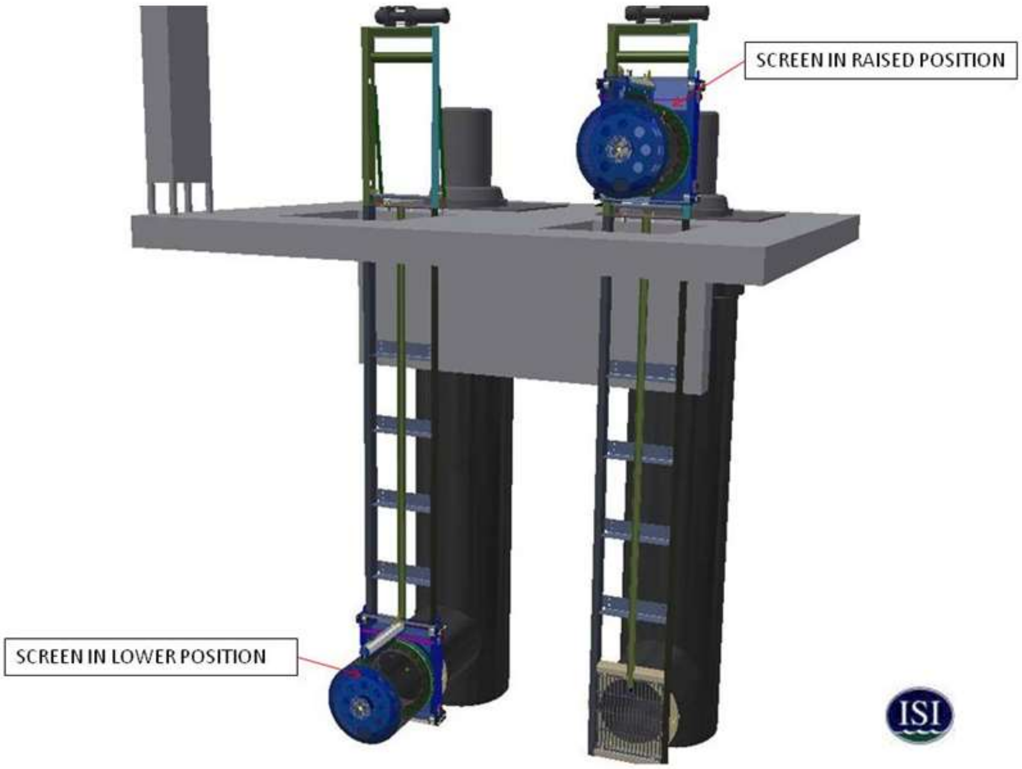

Figure 7. Schematic of the 2 MGD Exploratorium seawater intake in San Francisco Bay (image courtesy ISI).

ii. Passive Screens

Passive WWS are stationary cylindrical screens that have no moving parts. Passive WWS have been used in seawater applications and are manufactured by several different companies (Figure 8). However, like active screens, there are very few installations that use 1-mm slot widths in seawater and none that could be identified at the scale that would be needed for the CSD facility. An Aqseptence (formerly Bilfinger and Johnson Screen) WWS was used for conceptual design purposes. The limited application data to support development of optimal design and O&M requirements for 1-mm slot passive WWS in a low energy environment contributes to lower feasibility as it adversely impacts the reliability of the use. According to the manufacturer, a total of 16, 7-foot diameter screens were recommended to meet the design criteria (flow rate, slot width, through-slot velocity, and redundancy).

Figure 8. Passive wedgewire screen being installed (image courtesy Aqseptence)

An airburst system (further described in Section - Operation and Maintenance and illustrated below in Figure 9) is recommended with passive screens to clear free-floating debris that may collect on the screen. Airburst will not remove bio-growth, so manual cleaning is necessary for bio-growth removal from inside and outside of the screen. An access hatch is provided at either the top and/or side of the screen to provide access to the screen interior.

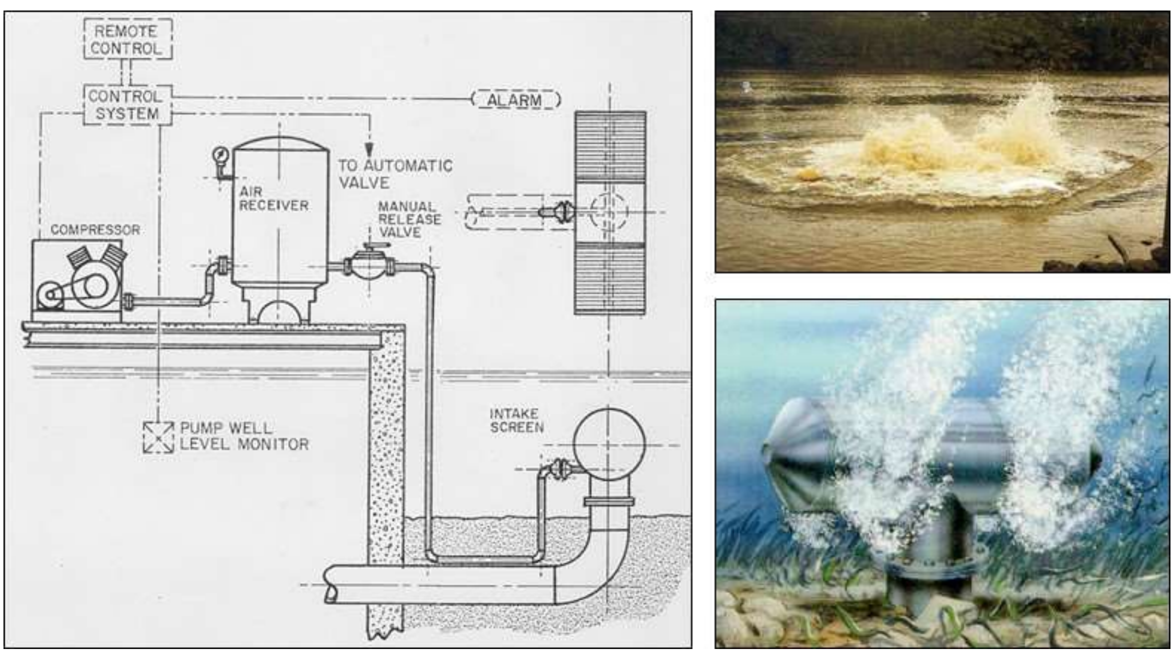

Figure 9. Airburst cleaning system. Clockwise from left: airburst typical design, airburst from surface, airburst at screen (images courtesy Aqseptence).

Airbursting would be conducted on one lateral at a time with each of the four screens on a lateral being burst in sequence. Airbursting will create a turbulent surface boil as shown in the top left image in Figure 9.

iii. Screen Material

The screen material selected is important as it will dictate the biofouling and the corrosion rates. Pilot-studies have been conducted for proposed seawater desalination projects to determine the material best suited to prevent biofouling (see excerpt results in Figure 10). Previous studies (Tenera 2010, Tetra Tech 2016) indicate that copper-zinc alloy results in very little biofouling, but this alloy has been concerns expressed by RWQCB related to potential leaching of copper into the Lagoon. Poseidon has assumed, for the purposes of this feasibility study, that copper zinc alloy is not allowed, therefore, super-duplex stainless steel is being considered for this application. However, super-duplex alloy will require more maintenance for clearing biofouling.

Figure 4.5-2. Biofouling of duplex coupon (#1) from November 16, 2009 to June 17, 2010.

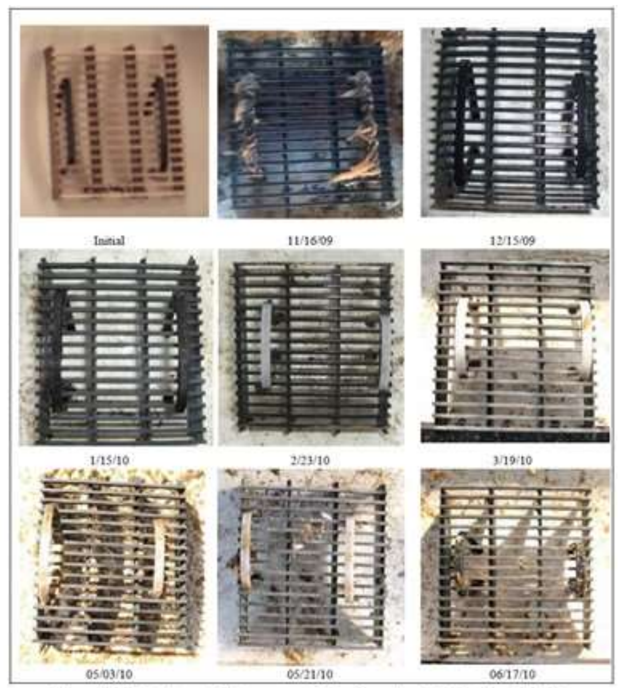

Figure 4.5-4. Biofouling of Z-alloy coupon # 1 from November 16, 2009 to June 17, 2010.

Figure 10. Biofouling over time with duplex stainless steel (left) and a coper-nickel alloy (right) (images from Tenera 2010).

B. Discharge Flow Augmentation Technologies

Flow augmentation at the CDP would be accomplished by drawing additional flow through the WWS to mix with the brine flow generated by the SWRO process. Poseidon has committed to using fish-friendly Ocean Plan-compliant flow augmentation pumps to minimize entrainment mortality. Fish-friendly pumps were originally designed for transferring fish in the aquaculture industry. Such pumps have demonstrated the capacity to transfer fish with little or no injury.

Since their inception, fish-friendly pumps have been used in fish passage and protection facilities to convey fish to a safe release location. There are several types of fish-friendly pumps available, each designed with the common goal of safely transferring live marine organisms. Each fish-friendly pump type employs certain fundamental principles that reduce the potential injury and mortality to fish. To varying degrees, fish-friendly pump designs limit fish exposure to stressors, such as pressure, shear, and impeller blade strike. More specifically, fish-friendly pumps limit fish exposure to:

- dramatic pressure differentials and high rates of pressure change;
- shear forces caused by rapid flow acceleration or deceleration;
- potential for blade strike by limiting the number of blades on the impeller and/or increasing blade thickness; and
- other sources of mechanical injury (e.g., pinching in gaps between the impeller and housing)

The fish-friendly pumps evaluated for this feasibility assessment are described in greater detail below.

i. Fish-friendly Axial Flow Pumps

The Bedford Pumps' fish-friendly axial flow pump consists of an impeller within a pipe driven by a sealed motor (Figure 11). These pumps are smaller in dimension than many conventional pumps and are designed for low heads and high flows. The low head design of the pumps (approximately 5 psi) should minimize the potential for pressure-related injuries. These pumps have been designed and used to safely pass live fish for pumping applications worldwide.

The pump specified for this application has a two-bladed impeller, a pumping capacity of 57 MGD, and is fully submersible. A total of four pumps would be installed with three in service and one as a backup. The model of pump specified for the CDP underwent independent fish survival testing in 2012 and demonstrated that survival was high (Vis and Kemper 2012).

Figure 11. Bedford Pumps axial flow submersible pump: left: general installation arrangement similar to the approach at the CDP, middle: cutaway of the pump, right: photo of pump impeller (images courtesy Bedford Pumps and VisAdvies Ecological Consultancy and Research).

6. Project Schedule

Alternative 21 involves complex construction in marine wetlands. The project completion schedule shown in Table 1 shows up to five years to secure the necessary permits and approvals, complete final engineering design, select a contractor, amend the Water Purchase and Operation and Maintenance Agreements, secure financing, and construct, commission, and startup the intake and discharge modifications.

A. Construction Sequence

The following section provides a description of the construction sequence for each phase of the Alternative 21 intake/discharge system. A conceptual construction schedule is provided in Attachment F.

i. Interim Stand-Alone Construction

The temporary stand-alone phase will be constructed first to maintain operation of the CDP following decommissioning of the EPS (scheduled for the end of 2018). The CDP would continue to rely on interim stand-alone operation until the Ocean Plan-compliant facility is ready for commercial operation. This would be accomplished through: 1) the use of the existing traveling water screens and cooling water pumps at the EPS, 2) new traveling water screens to match EPS screening requirements and pumps that are installed solely to bridge the gap between when the EPS facilities are no longer available and when the new Ocean Plan-compliant intake and discharge facilities are ready to go into service, or 3) a combination of existing and new screens and pumps.

ii. Ocean Plan-Compliant Construction

Construction of the Ocean Plan-Compliant Alternative 21 within the Lagoon requires permits and approvals with a long lead time. The schedule (Table 1) shows up to five years to secure the necessary permit and approvals, complete final engineering design, select a contractor, amend the Water Purchase and Operation and Maintenance Agreements, secure financing, and construct, commission, and startup the intake and discharge modifications. The following conceptual sequence is anticipated, which would be further refined during preliminary design:

- Mobilization
- Dredge Lagoon for lateral installation.
- Provisions for installing a temporary barrier in the discharge pond for debris maintenance purposes and provisions for placement of silt curtains for use during pigging operations.
- Concurrent work
    - Air burst system
    - Lateral and screen installation without connection to intake structure
    - Floating debris boom/curtain
- Plant Shutdown

    - Modify intake structure to receive intake laterals
    - Connect laterals to intake structure
- Commissioning and testing
- Demobilization

7. Feasibility Assessment

A. Technical

The technical evaluation presented herein was prepared to support the feasibility assessment of this alternative. Further refinement would occur during the final design of the intake and discharge modifications.

i. Site Constraints

a. Intake Site

The movement of seawater in and out of the Lagoon is predominantly tidally-generated. In addition, feedback from the existing EPS intake operators indicates that debris loads can be high. The EPS operators remove, on average, nine cubic yards of debris per day from the existing trash rack. The debris consists of kelp (tidally floated into the lagoon), eelgrass, and macroalgae. While the existing EPS trash racks and traveling water screens with spraywash systems are designed to collect, divert, and dispose of such debris, WWS rely on the tidal currents, the use of airburst systems, and manual cleaning by divers to manage such free-floating debris.

To construct this Lagoon-based WWS array, an NRG lease of approximately 4.2 acres would be required for the intake laterals, the Lagoon-based WWS arrays, and the floating debris boom. This area includes an extra 5 ft on all sides of the installed equipment.

In lieu of the debris collection, diversion, and disposal features inherent to the existing EPS intake technologies, HDR has included a floating debris boom around the WWS arrays in the Lagoon. The floating debris boom extends from the surface three feet down into the water. Though this feature will deflect some of the floating debris, it will not provide the same degree of protection from debris as the trash racks and traveling water screens with spraywash systems. Storms and periods of macroalgae blooms may require more frequent airbursting or increased screen inspection and cleaning. In addition, no operational performance data are available for similar installations in marine lagoons. The due diligence effort completed by TWB Environmental Research and Consulting (TWB) to evaluate the performance of WWS in similar seawater installations is summarized below.

Performance of Wedgewire Screens in Seawater

TWB completed a comprehensive search for existing facilities that use WWS in a fully marine environment. The search included:

- Querying the largest vendors of WWS (Aqseptence [formerly Bilfinger Water Technologies and Johnson Screens], Hendrick Screen Company, and Intake Screens, Inc [ISI]) for reference sites that use WWS in seawater
- Reaching out to other industry professionals with expertise in seawater intakes for desalination and power generation facilities

- Reviewing the available literature on pilot-scale WWS testing conducted for proposed seawater desalination facilities in California

Although the vendor-related search indicated that there were some seawater installations of WWS globally, no data were available on their operational performance or maintenance requirements. The vast majority of the seawater WWS installations use slot widths greater than 1 mm and have intake flows under 50 MGD. Approximately half of the seawater WWS installations are fabricated of a copper alloy and the remaining half were stainless steel. Only one was an active screen installation (see Section Active Screens for details on that installation).

The search relying on feedback from industry professionals with expertise in seawater intakes yielded a list of 16 facilities that use WWS in seawater. Table 6 lists the 16 facilities for which at least total flow rate and screen slot width information were available. None of the facilities used screen slot widths of 1.0 mm. Operators at only two of these 16 facilities (indicated in Table 6) were responsive to requests for additional information on the WWS design and operational performance; each of those are described below.

Table 6. Operating seawater intakes using WWS. Facilities in bold and italics were responsive to requests for additional information; brief case studies are provided for these facilities below the table.

Facility and Industry	Location	Intake Flow Rate (MGD)	Slot Size (mm)	Install Date
Bocamina Unit 2 - Power	***Chile***	***285.0***	***3***	***2015***
Bocamina Unit 1 - Power	***Chile***	***133.0***	***3***	***2015***
Beckton Gateway - Desal	London	211.7	3	2010
Ras Al Khaimah - Desal	UAE	196.5	3	2006
Aluminium Bahrain, Calciner and Marine	Bahrain	190.0	Not Provided	2009
Alba - Desal	Bahrain	126.8	6	2001
Galilah - Desal	UAE	36.0	3	2010
Voestalpine - Iron Processing	USA	23.0	3	2015
Khor Fakkan - Desal	UAE	15.0	3	2010
Burrup - Desal	Australia	15.0	3	2003
Radwa Farm - Desal	KSA	14.1	6	2008
Jeddah - Desal	KSA	14.0	6	2008
Sur - Desal	***Oman***	***13.2***	***5***	***2015***
Kalba - Desal	UAE	9.0	3	2008
Fujairah Port - Desal	UAE	2.3	3	2005
Exploratorium - Cooling	USA	2.0	1.75	2012

Bocamina Power Station - Enel

Site: Enel uses passive WWS on cooling water intakes for two power plants (Bocamina Units 1 and 2) on the Chilean coast (Pacific Ocean). The power plants are in Coronel Bay (south of Concepcion, Figure 12) and have intakes that are 200-250 m (656-820 ft) from shore and 5-6 m (16-20 ft) deep. Unit 1 has a 2-m (6.6-ft) diameter pipeline and Unit 2 has a 3-m (9.8-ft) diameter pipeline.

Screens: Table 7 provides the intake flow rates and numbers of screens. The screens have 3-mm slot widths, are constructed of Z-Alloy (a proprietary copper-nickel mix used by Aqseptence/Bilfinger/Johnson), are designed for a 0.15-m/sec (0.5-ft/sec) through-slot velocity, and were manufactured and installed in 2015. Each Unit's intake includes an offshore platform above the intake terminus – the air burst system is housed on the platform.

Maintenance: Enel uses a custom air burst system rather than the system supplied by the vendor. Air bursting is done daily. Divers manually clean the screens (exterior and interior) every 6-8 months. A chlorine system was supplied by the vendor, but distribution of the chlorine over the screen surfaces is not uniform. It was unclear whether the intake pipelines included provisions for pigging.

Changes to Design They Would Consider: Operators recommend better detail on required screens welds as well as QA/QC of screen manufacturing. They recommended including isolation valves on the screens to prevent having to shut down entire plant to clean screens. They recommended including a better chlorine injection system to effect more uniform distribution of chlorine.

Figure 12. Location of Bocamina Power Station Units 1 and 2 in Coronel Bay, Chile.

Table 7. Bocamina cooling water intake flow rates and number of screens.

	Flow Rate		
Bocamina Unit	m^3/hr	MGD	# Screens
1	21,000	133	7
2	45,000	285	14

<u>Sur Desalination Plant - Veolia</u>

Site: Veolia uses passive WWS on the intake of a seawater desalination plant (Sur) on the Oman coast (Gulf of Oman, Indian Ocean). The desalination plant is located on the east coast of Oman (Figure 13). Sur was built in two phases; Phase 1 uses a subsurface intake, Phase 2 uses offshore WWS. The Phase 2 WWS intake is located 400 m (1,312 ft) offshore at a depth of 10 m (32.8 ft). The screens are 1.2 to 1.3 m (4 to 4.25 ft) off of the seafloor and have not experienced any sand ingestion issues. The intake pipeline is HDPE with an outside diameter of 1,200 mm (47.2 in) and an inside diameter of 1,086 (42.8 in).

Screens: There are two screens, each with 5.0-mm slot widths (Figure 14). The screens are constructed of Super Duplex Uranus 52N, are designed for a 0.1-m/sec (0.33-ft/sec) through-slot velocity, were manufactured in 2014, and were installed in 2015. Each screen is rated for 2,500 m^3/hr (15.9 MGD) for a total intake capacity of 32.8 MGD. The screens are 1,250 mm (49 in) in diameter and 4,303 mm (14.1 ft) long. The T-stem is 815 mm (32 in) in diameter.

Maintenance: Veolia uses an airburst system comprised of two compressors and two receivers (all onshore). Air piping to the screens follows the intake pipeline alignment and is 180 mm (7.1 in) in diameter. Receivers are charged to 8-10 bar and the receiver tank capacity is approximately 3 m^3. The valves and actuators to release an airburst are on land. Both screens are burst concurrently and bursting occurs every hour.

A chlorination distribution system also delivers chlorine to the screen faces (Figure 15). The screens are shock dosed at 10 mg/L (10 ppm) using calcium hypochlorite delivered at 290 L/hr (1,839 gal/day). The chlorine injection system effected poor distribution across the screen faces leading to periodic fouling events. The operator custom modified the injection system and performance has been better.

Veolia also conducts manual cleaning by divers. Manual cleaning is conducted quarterly, though the frequency may decrease with the improved distribution of chlorine across the screen faces. Diver manually clean the external and internal screen surfaces with a high-pressure water gun which has been deemed to be very effective. The external screen surfaces are cleaned during every quarterly manual cleaning event, while the internal surfaces are cleaned every other quarterly manual cleaning event. The inside of the screen is accessed via a hinged door in the screen endcap.

The intake system includes provisions for pigging the pipeline, though no pigging has been completed to date (after two years in operation). Divers inspect the pipeline during each quarterly manual screen cleaning event. Pipeline is 1.2 m (~4 ft) diameter. The pipeline is HDPE laid in trench 1.5 m (~5 ft) below seabed.

Changes to Design They Would Consider: The original design did not include surface buoys/a standoff zone. They felt this was important to prevent damage to screen by anchors and to provide an exclusion area to prevent capsizing subsistence fishermen in small boats near the intake during airburst cleaning events.

Figure 13. Location of Sur Desalination Plant, Oman.

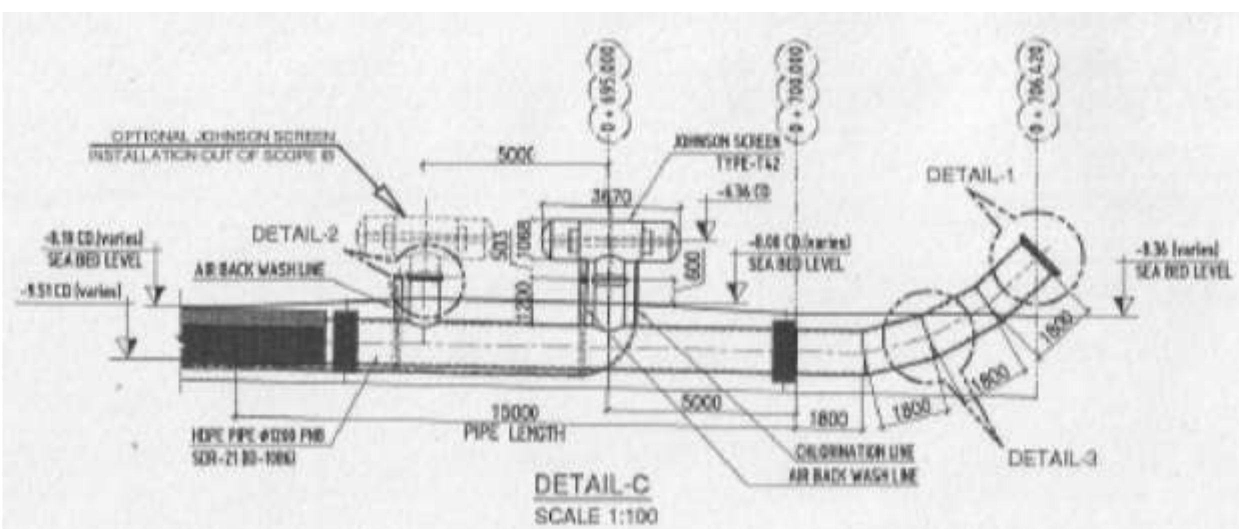

Figure 14. Wedgewire screens used at Sur Desalination Plant, general arrangement.

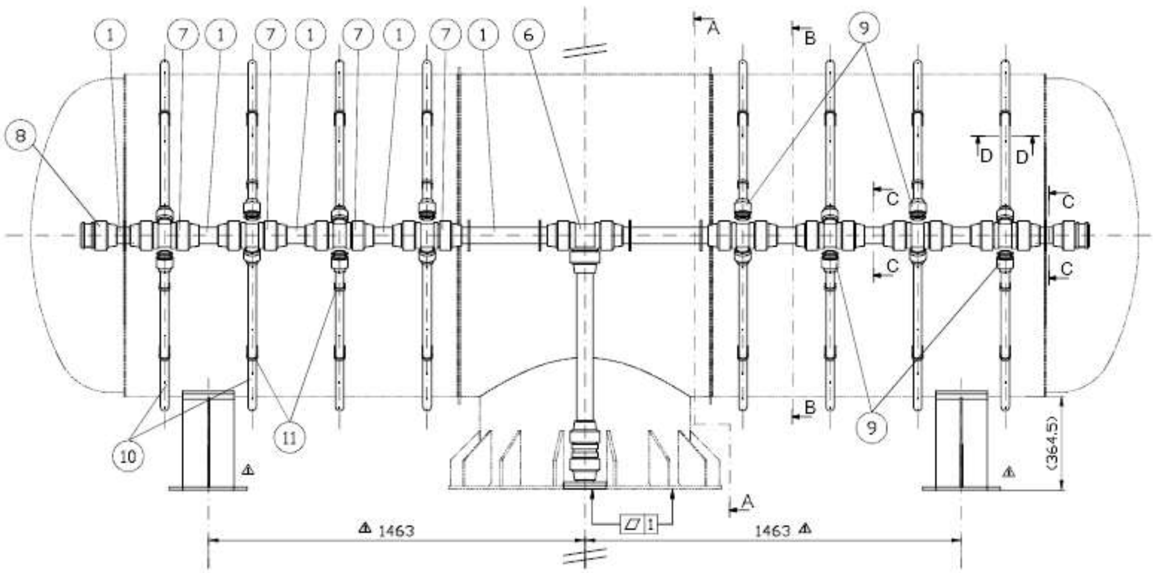

Figure 15. Detail view of the chlorine injection system surrounding the WWS used at the Sur Desalination Plant.

b. Discharge Site

The use of flow augmentation at this site does not present any technical constraints. There is sufficient space available to install a wet well/fish-friendly pumping structure between the existing EPS intake tunnels and the SWRO IPS from which process and dilution flows can be drawn.

ii. Equipment

a. Intake Equipment

Although WWS are commercially available, as described above in Section - Intake Screening Technology, there are no data readily available on the performance of narrow-slot screens in marine environments. As a result, the biggest technical concern with the use of WWS in the Lagoon is the lack of information on the performance of narrow-slot WWS in a marine environment. A WWS with 1-mm slots has the potential to become clogged quickly under certain conditions. This presents a level of operational uncertainty relative to the debris management performance of WWS in the Lagoon, whereas with travelling screens there is data to support performance assumptions. Section - Performance of Wedgewire Screens in Seawater above provides additional detail on the two operational intakes that use WWS in similar seawater applications.

In the absence of full-scale performance data, the use of 1 mm WWS (active or passive) in the Lagoon represents a significant operational risk for the CDP. As described above in Section - Operation and Maintenance, the debris management approach for ensuring reliable operation of the WWS includes monthly manual cleaning events. In addition to the monthly biofouling cleaning events (during which free-floating debris can also be cleared from the WWS), additional cleaning events are anticipated to effectively manage free-floating debris during challenging conditions.

Relative to the pigging equipment for cleaning the intake laterals, it is anticipated that quarterly pigging will be required. During the quarterly pigging events, the CDP must be offline; therefore, pigging is estimated to require that the CDP is offline for a total of 16 days. The current Water Purchase Agreement (WPA) with the San Diego Water Authority allows only ten days offline; therefore. the CDP would be in violation of its WPA. In addition, the management of pigged debris poses a technical challenge. Pigging creates a large volume of water mixed with removed biofouling growth from the intake laterals' internal surfaces. Although periodic dredging of the discharge pond is proposed to remove accumulated pigged debris, there is a risk that when the CDP is brought back online after each pigging event, debris could be resuspended and discharged to the Ocean. Handling of the pigged debris, therefore, has potential to result in NPDES-related compliance issues.

b. Discharge Equipment

The use of flow augmentation will require the installation of fish-friendly axial flow pumps and the related piping to route the dilution flow to the existing discharge tunnel at the EPS. There are several types of axial flow fish-friendly pumps commercially available and one (Bedford Pumps or equal) has been recommended for this application. As such there are no technical constraints relative to the equipment.

B. Schedule

The schedule to complete Alternative 21 is given in Table 1. It includes the time required to secure the necessary permit and approvals, complete final engineering design, select a contractor, amend the Water Purchase and Operation and Maintenance Agreements, secure financing, and construct, commission, and startup the intake and discharge modifications.

The schedule for permitting, design, and construction of Alternative 21 in the Lagoon is estimated to take up to 5 years. During this 5-year period, the CDP would need to operate in interim stand-alone mode to ensure uninterrupted delivery of potable water to the San Diego County Water Authority and the residents and businesses of San Diego County.

C. Environmental

The screened surface intake under consideration would be located within Agua Hedionda Lagoon; therefore, the source water for the CDP will remain the same as under the current co-located operation. Both feedwater and augmentation flow for the CDP would be withdrawn through a new 1-mm WWS array. Organisms that could be potentially impacted by the surface water intake include those occurring near the water withdrawal point in the Lagoon. Previous entrainment sampling indicates that gobies and blennies are the dominant taxa.

i. Impingement

Impingement is the pinning of larger organisms against the screen mesh by the flow of the withdrawn water. The magnitude of impingement losses for any species from intake operation is a function of the involvement of the species with the intake (number or proportion impinged) and the subsequent mortality of those organisms (referred to as impingement mortality or IM).

Intake velocity is commonly accepted to be the strongest predictor of impingement. Furthermore, a through-screen velocity of 0.5 ft/sec or less has been identified for being protective of impingeable sized fish. Per the Desalination Amendment language at 2.d.(1)(c)iv., the State Water Resources Control Board (SWRCB) has prescribed a through-screen velocity no greater than 0.5 ft/sec in order to minimize impingement at surface water desalination intakes.

The WWS in the array for the Alternative 21 intake/discharge structure are designed as passive screens with a through-slot velocity that is 0.5 ft/sec or less. The WWS would meet the Desalination Amendment requirement for minimizing impingement at the wet well/fish-friendly pumping structure for the CDP. Impingement mortality is assumed to be zero.

ii. Entrainment

Entrainment is the passage of smaller organisms through the screening slots. The magnitude of entrainment losses for any species from intake operation is a function of the involvement of the species with the intake (number or proportion entrained) and the subsequent mortality of those organisms as they pass through the process equipment (referred to as entrainment mortality). Entrainment mortality is assumed to be 100% for the organisms entrained into the feedwater flow. Similarly, entrainment mortality is assumed to be 100% in the flow augmentation system, although the system has been designed to maximize survival to the greatest extent possible (e.g., fish-friendly pumps, conveyances designed for minimal turbulence and shear).

Per the Desalination Amendment language at 2.d.(1)(c)ii., the SWRCB has prescribed screens with 1-mm mesh in order to reduce entrainment at surface water desalination intakes. In accordance with the Desalination Amendment, Poseidon has selected a 1-mm slot width for the lagoon WWS.

Based on intake-related entrainment through the SWRO feedwater system (127 MGD), the calculated APF is 35.76 acres. Based on intake-related entrainment through the flow augmentation system (171 MGD), the calculated APF is 47.68 acres. The total APF associated with a combined flow of 298 MGD is 83.44 acres using the methodology set forth in Appendix E of the Staff Report for the Desalination Amendment after accounting for a 1% credit for 1 mm screening technology.

The Desalination Amendment also requires that the applicant estimate the mortality caused by each of the stressors that could potentially contribute to entrainment mortality in the flow augmentation system is discussed in the sections below. Notwithstanding the expected high rate of survival of all forms of marine life exposed to the cumulative effects of the flow augmentation system, for the purposes of demonstrating to the RWQCB that this technology provides a comparable level of intake and mortality of all forms of marine life to that of the multiport diffuser system, Poseidon has conservatively assumed the worst-case outcome -- 100% mortality of all organisms passing through the flow augmentation system.

iii. Brine Mixing Zone

The brine mixing zone (BMZ), for the CDP is a 200-meter (656 foot) semi-circle originating from the terminus of the discharge channel in the Pacific Ocean. Outside of the BMZ, salinity cannot exceed 2 ppt over ambient background salinity. The benthic area encompassed by the BMZ would be approximately18.51 acres.

iv. Benthic Habitat Impacts

Agua Hedionda Lagoon is a coastal estuarine system comprised of three connected water bodies: the Inner, Middle, and Outer Lagoons. The Lagoon was originally a natural, seasonal estuary that was frequently closed to the Pacific Ocean. The Outer Lagoon was opened permanently to the Pacific Ocean in 1954 to provide cooling water flow to the EPS which went online the same year. The intake for the EPS and co-located CDP is located at the southernmost end of the Outer Lagoon. The inlet and portions of the Outer Lagoon are dredged approximately every two years to maintain the basin for cooling water purposes.

The Outer Lagoon has a diversity of habitat utilized by various lifestages of marine organisms. The types of habitat include sand, mud, eelgrass, rock revetment, and dock pilings.**Error! Reference source not found.** Recent eelgrass surveys have been completed to inform ongoing dredging operations. Figure 16 shows a recent survey (Merkel and Associates 2015). To the greatest extent, the Alternative 21 WWS arrays were sited to avoid impacts to the existing eelgrass beds in the outer Lagoon.

The Alternative 21 WWS array would require significant in-water construction activity in the Lagoon. Construction would be done from a derrick barge moored in the Lagoon. Anchoring of the derrick barge would create only temporary benthic and turbidity-related impacts.

The installation of the WWS arrays and the four intake laterals would result in the permanent loss of benthic habitat. Based on the dimensions of the installation (and allowing 5 feet on each side of installed equipment), the benthic footprint would be approximately 4.2 acres (Figure 17).

v. Relative Comparison of Environmental Impacts

Table 8 presents a summary of the environmental impact of Alternative 21. It also compares the environmental impact of Alternative 21 to the other Alternatives under consideration (Alternatives 1 and 15). Table 8 indicates that Alternative 21 has a greater total environmental impact (related principally to the permanent loss of benthic habitat in the Lagoon). For that reason, Alternative 21 is not the environmentally superior alternative. Impingement mortality is assumed to be zero and since entrainment is proportional to flow, entrainment mortality is assumed to be the same as for all other intake/discharge alternatives evaluated.

Table 8. Summary of feasibility assessment for Alternatives 1, 15, and 21.

Table 8. Summary of Feasibility Assessment				
Feasibility Criteria	**Impact Assessment Method**	**Alternatives**		
		1	**15**	**21**
Environmental Impact		**Impacted Area (Acres)**		
Intake	APF calculated per Appendix E of the Staff Report/SED to the Ocean Plan Amendment using a 95% confidence bound for an assumed 100% mortality of all forms of marine life entrained by 127 MGD CDP process water with an APF of 35.76 acres and 171 MGD flow augmentation with an APF of 47.68 acres after accounting for a 1% credit for 1 mm screening technology.	83.44	83.44	83.44
	Potential mortality associated with the operation of the fish return system.	0.93	0.85	0
Discharge	Area within the BMZ potentially exposed to a salinity in excess of 2 ppt over natural background salinity.	18.51	18.51	18.51
Construction	Permanent footprint of intake/discharge components within the lagoon.	0.10	0.10	4.2
Total Environmental Impacts (Acres)		**102.98**	**102.90**	**106.15**
Cost	Capital Cost	**\$49,000,000**	**\$53,400,000**	**\$58,800,000**
	Annualized Cost (Capital and O&M)	**\$7,860,000**	**\$8,200,000**	**\$11,030,000**
Schedule	Expected Operation Date of Ocean Plant Compliant Intake and Discharge Facilities	**2021**	**2021**	**2023**
Conclusion	Overall Feasibility Assessment	**Feasible**	**Feasible**	**Infeasible[1]**

1. **Significant operational reliability concerns, environmental impacts to Lagoon, significant increase in capital and O&M costs for minimal reduction in marine life mortality**

Figure 16. 2015 post-dredge eelgrass survey in Outer Lagoon of Agua Hedionda Lagoon (image from Merkel and Associates 2015).

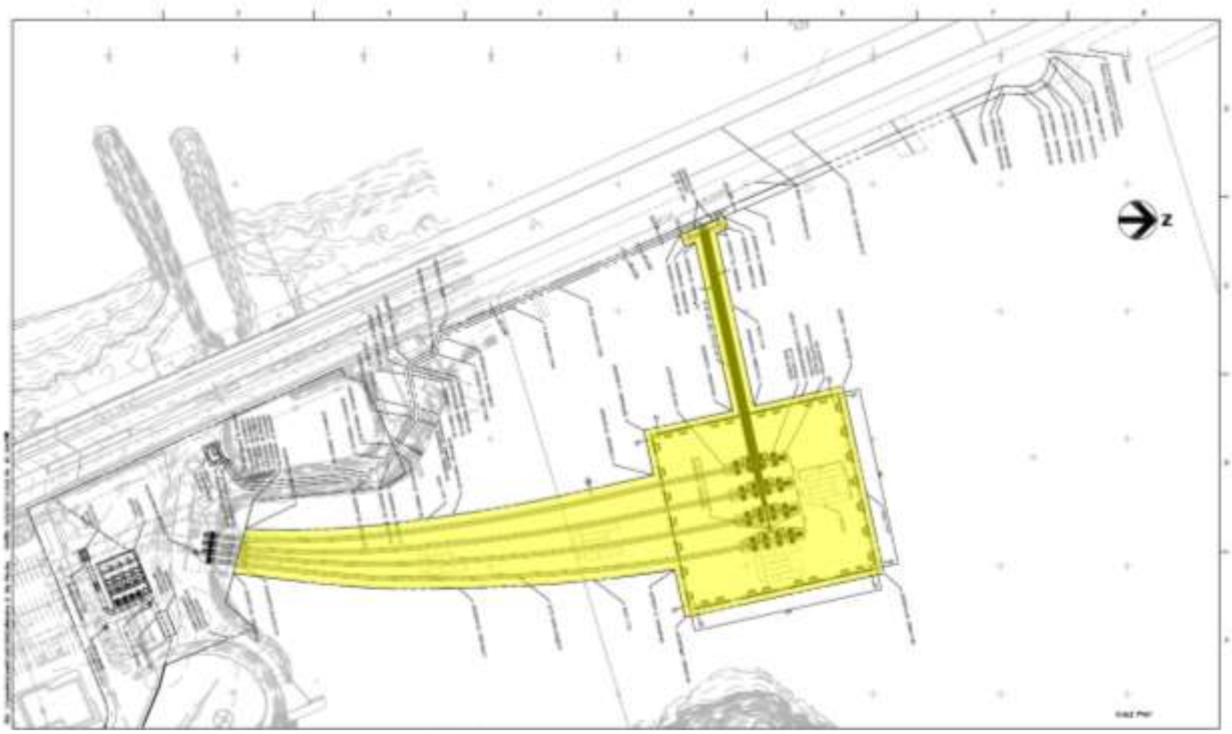

Figure 17. Benthic footprint (shaded yellow) of Alternative 21 in the Lagoon.

D. Economic

A summary of the estimated capital costs is provided in Table 9. This cost includes permitting, construction, construction management, insurance, rent, post-commissioning empirical entrainment study, legal fees, interest, debt service, underwriting, O&M reserve, and the outstanding equity fee.

A summary of the estimated annual costs is provided in Table 10. This includes costs associated with the temporary stand-alone facility as well as the Ocean Plan-compliant facility.

The annual costs for the temporary stand-alone facility include:

- Power for flow-augmentation
- Screen maintenance and power
- Electrical building HVAC
- Inlet structure bar rack and screen debris removal and disposal
- Inlet tunnel and wet well maintenance (biofouling control)

The annual costs for the Ocean Plan-compliant facility include:

- Airburst system maintenance and power
- WWS cleaning 12 times per year – requires divers

- Lateral maintenance four times per year – requires divers, pigging crew, and debris disposal.
- Discharge pond dredging for debris maintenance
- WWS replacement – anticipated every 10 years
- Barge operation and maintenance to support WWS operation

The estimated capital and O&M costs for Alternative 21 are substantially higher than other intake/discharge alternatives evaluated. The increased cost is associated primarily with the marine construction and greater O&M costs.

Table 9. Capital cost estimate for Alternative 21 intake/discharge.

			June 2017 Estimate
			Alternative 21
Construction Period			
Operation Date			
Permitting and 30% Design			$5,100,000
Intake/Outfall Construction			$40,201,000
Construction Management			$4,100,000
Construction Insurance			$1,000,000
Construction Rent			$510,000
Post Construction Entrainment Study			$1,200,000
Subtotal			**$52,111,000**
Transaction Costs, legal			$1,175,017
Capitalize Interest			$2,518,859
Additional 6 Mo Debt Service Reserve			$1,717,895
Debt Underwriting			$481,757
Additional 1 month O&M Reserve			$412,668
Outstanding Equity Fee			$348,980
Total Project Cost			**$58,766,176**

Table 10. Annual cost estimate for Alternative 21 intake/discharge.

	June 2017 Estimate		
Improvement Phase	**Temporary Stand-Alone**	**OPA Compliance**	**Total**
Annual Costs			
Construction Debt Charge			$3,435,789
Construction Equity Charge			$1,640,302
Additional O&M Charge			$5,952,010
Total Annual Costs			**$11,028,101**

8. Conclusion

HDR has prepared this feasibility assessment of intake/discharge Alternative 21 at Poseidon's request to describe the modifications required to accommodate the transition of the CDP to long-term stand-alone operation in compliance with the Ocean Plan. For purposes of Chapter III.M., "feasible" is defined as *"capable of being accomplished in a successful manner within a reasonable period of time, taking into account economic, environmental, social, and technological factors."* This report evaluates each of these feasibility criteria and below we summarize how each relates to Alternative 21. Table 11 summarizes the costs, schedule, and environmental benefits of Alternatives 1, 15, and 21.

As outlined in this report, the greatest concerns are with the technical aspects of Alternative 21. The use of narrow-slot WWS in a low-energy marine environment constitutes an operation risk since there are no performance data on such installations as proposed for this alternative. The technical challenges of implementing 1-mm WWS in the Lagoon translate into operation risks that could compromise the reliability of the CDP. In the absence of full-scale performance data, the use of WWS (active or passive) in the Lagoon also represents a significant risk to a key design feature of the CDP, which is to provide the San Diego region with a highly-reliable water supply through the use of proven technology.

The schedule for permitting, design, and construction of Alternative 21 in the Lagoon is estimated to take up to 5 years. During this 5-year period, the CDP would need to continue interim stand-alone mode to ensure uninterrupted delivery of potable water to the San Diego County Water Authority.

The environmental impact of Alternative 21 is greater than Alternative 1 and Alternative 15 since it requires construction in the Lagoon with an associated loss of benthic habitat. Impingement mortality is assumed to be zero and since entrainment is proportional to flow, entrainment mortality is assumed to be the same as for all three alternatives.

The estimated capital and O&M costs for Alternative 21 are substantially higher than other intake/discharge alternatives evaluated. The increased cost is associated primarily with the marine construction and greater O&M costs associated with the removal of biofouling and accumulated debris on the surface of the screens and inside the intake laterals.

The following are the conclusions and findings presented in this feasibility assessment:

- The use of an existing intake technology in an unproven application represents a technical risk to the reliable operation of the CDP
- The cleaning and maintenance requirements are high due to uncertainty relative to performance of narrow-slot WWS in the Lagoon
- The cleaning of the intake laterals via pigging creates challenges associated with debris management and meeting the terms of the Water Purchase Agreement regarding allowable days offline

- The schedule for permitting, designing, and constructing a structure in the Lagoon will take up to 5 years – longer than alternatives that do not require construction in the Lagoon
- The total environmental impact is greater than other alternatives due to the permanent loss of benthic habitat in the Lagoon
- The cost is greater than other alternatives due to requisite in-water construction and increased maintenance anticipated

Table 11 presents a summary of the feasibility assessment of Alternative 21. It also compares the environmental impact, cost, and schedule aspects of Alternative 21 to the other Alternatives under consideration (Alternatives 1 and 15). Table 11 indicates that Alternative 21 has a greater total environmental impact (related principally to the permanent loss of benthic habitat in the Lagoon), a substantially higher cost (capital and annualized), and a longer schedule. For those reasons, Alternative 21 is not feasible given the alternatives available.

When considering all the feasibility criteria, Alternative 21 is not the preferred intake/discharge alternative for the stand-alone operation of the CDP once the EPS ceases operation. More than any other criterion, the uncertainty and risks surrounding the operational performance of an intake technology in an application for which no performance data are available drive the conclusion that Alternative 21 is not feasible for the CDP.

Table 11. Alternatives 1, 15, and 21 intake and discharge modifications – comparison of costs, schedule, and environmental benefits.

Table 11. Summary of Feasibility Assessment				
Feasibility Criteria	**Impact Assessment Method**	**Alternative**		
		1	**15**	**21**
Environmental Impact		**Impacted Area (Acres)**		
Intake	APF calculated per Appendix E of the Staff Report/SED to the Ocean Plan Amendment using a 95% confidence bound for an assumed 100% mortality of all forms of marine life entrained by 127 MGD CDP process water with an APF of 35.76 acres and 171 MGD flow augmentation with an APF of 47.68 acres after accounting for a 1% credit for 1 mm screening technology.	83.44	83.44	83.44
	Potential mortality associated with the operation of the fish return system.	0.93	0.85	0
Discharge	Area within the BMZ potentially exposed to a salinity in excess of 2 ppt over natural background salinity.	18.51	18.51	18.51
Construction	Permanent footprint of intake/discharge components within lagoon	0.10	0.10	4.2
Total Environmental Impacts (Acres)		**102.98**	**102.90**	**106.15**
Cost	Capital Cost	**$49,000,000**	**$53,400,000**	**$58,800,000**
	Annualized Cost (Capital and O&M)	**$7,860,000**	**$8,200,000**	**$11,030,000**
Schedule	Expected Operation Date of Ocean Plant Compliant Intake and Discharge Facilities	**2021**	**2021**	**2023**
Conclusion	Overall Feasibility Assessment	**Feasible**	**Feasible**	**Infeasible**[1]

1. **Significant operational reliability concerns, environmental impacts to Lagoon, significant increase in capital and O&M costs for minimal reduction in marine life mortality**

9. References

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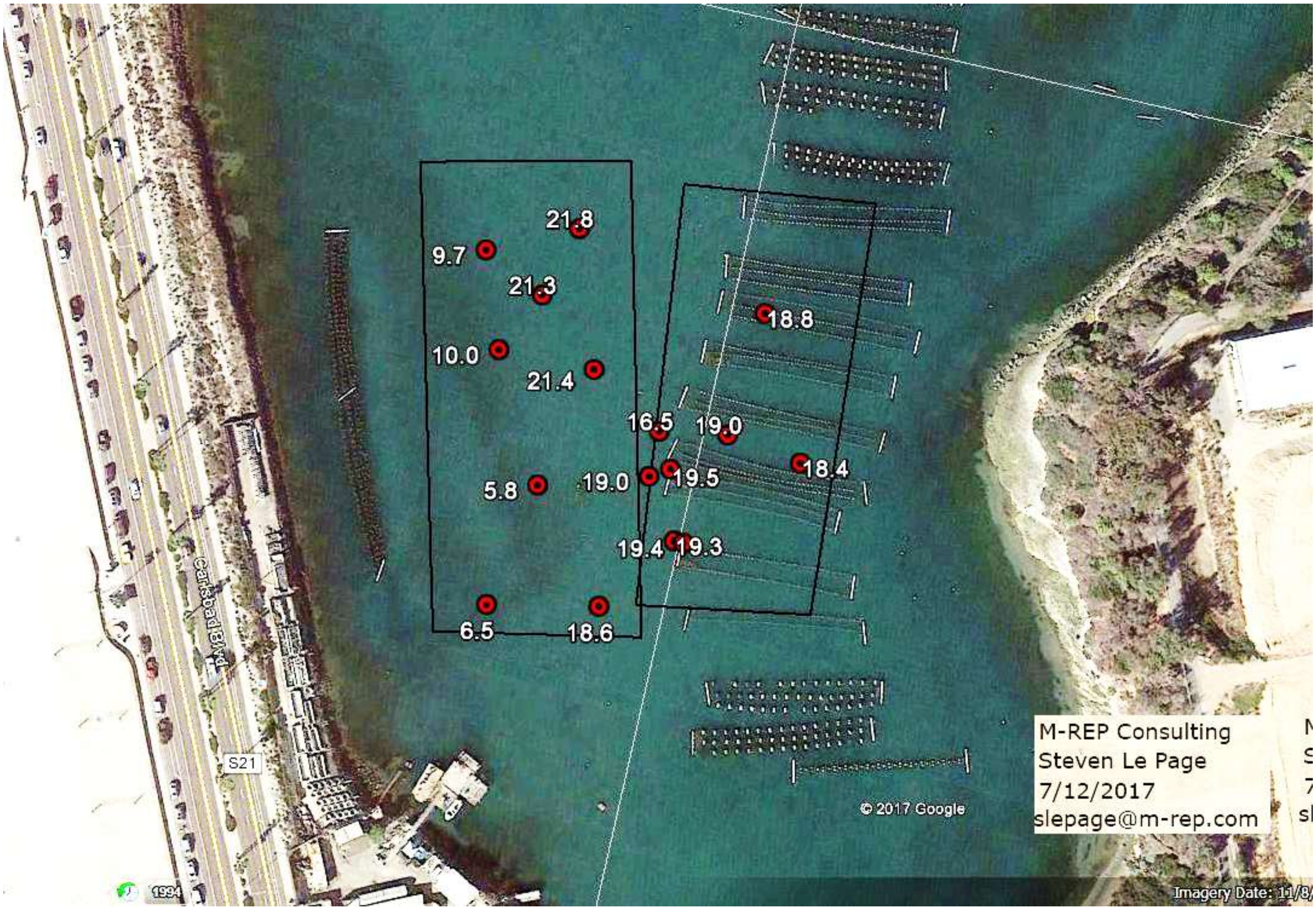
Attachment A - Agua Hedionda Lagoon Depth Measurement Maps and Tables

Agua Hedionda Lagoon Depth (Ft.) at MLLW



Agua Hedionda Lagoon Station Location

(Note from July 12th, 2017, are notated numerically followed by the letter (b) while the station locations from June 7, 2017, are notated only numerically.)

Table 1. Station Locations, Time on station, and calculated MLLW for each station in Agua Hedionda Lagoon on June 7, 2017

Station	GPS Location	Time	Depth Observed (ft.)	Calculated MLLW (ft.)	Correction Used (Table 2)
1	N 33° 08' 27.6" W 117° 20' 19.7"	13:00	24.00	21.76	-2.24
2	N 33° 08' 27.3" W 117° 20' 20.9"	13:05	11.83	9.65	-2.18
3	N 33° 08' 27.3" W 117° 20' 20.2"	13:09	23.42	21.25	-2.17
4	N 33° 08' 26.5" W 117° 20' 19.5"	13:12	23.58	21.42	-2.16
5	N 33° 08' 26.5" W 117° 20' 20.4"	13:15	12.08	9.95	-2.13
6	N 33° 08' 25.4" W 117° 20' 19.8"	13:19	7.85	5.76	-2.09
7	N 33° 08' 24.5" W 117° 20' 18.9"	13:22	20.66	18.61	-2.05
8	N 33° 08' 24.3" W 117° 20' 20.0"	13:26	8.54	6.51	-2.03

Table 2. NOAA/NOS/CO-OPS published tide levels for La Jolla for June 7, 2017

Published Tidal Levels for La Jolla	
Time	Observed Tide
13:00	2.24
13:06	2.18
13:12	2.16
13:18	2.09
13:24	2.04
13:30	2.00

Table 3. Station Locations, Time on station, and calculated MLLW for each station in Agua Hedionda Lagoon on July 12, 2017.

Station	GPS Location	Time	Depth Observed (ft.)	Calculated MLLW (ft.)	Correction Used (Table 4)
1b	33° 8'25.2"N 117°20'18.3"W	10:52	22.7	19.3	3.4
2b	33° 8'25.2"N 117°20'18.2"W	10:54	22.8	19.4	3.4
3b	33° 8'25.7"N 117°20'18.7"W	10:57	22.4	19.0	3.4
4b	33° 8'25.8"N 117°20'18.5"W	10:58	23.0	19.5	3.5
5b	33° 8'26.1"N 117°20'18.7"W	10:59	20.0	16.5	3.5
6b	33° 8'26.2"N 117°20'18.0"W	11:00	22.5	19.0	3.5
7b	33° 8'26.1"N 117°20'17.2"W	11:01	21.9	18.4	3.5
8b	33° 8'27.3"N 117°20'17.9"W	11:06	22.4	18.8	3.6

Table 4. NOAA/NOS/CO-OPS published tide levels for La Jolla for July 12, 2017

Published Tidal Levels for La Jolla	
Time	Observed Tide
10:48	3.3
10:54	3.4
11:00	3.5
11:06	3.6
11:12	3.6

Attachment B - Conceptual Drawings

HDR C:\pwworking\west01\d0218932\Alternative 21 Site Plan.dwg PRINTED: 11/16/2017 8:23:22 AM BY: CGAUTHIE

This Space Reserved for Sub Contractor Logos Do Not Draw in This Area

HDR

ISSUE	DATE	DESCRIPTION
-	-	-

PROJECT MANAGER

PROJECT NUMBER

PRELIMANRY NOT FOR CONSTRUCTION OR RECORDING

POSEIDON CHANNELSIDE
a Poseidon Water company

OPTION 21
SITE PLAN

Alternative 21 Site Plan.dwg
XXX

SHEET
00C-XXX

Attachment C – Hydraulics Calculations

	No Redundant Screen per Lateral						One Redundant Screen per Lateral					
Description	**4(3+1) - 63" - HDPE**		**3(2+1) - 72" - HDPE**		**2(1+1) - 96" - HDPE**		**4(3+1) - 63" - HDPE**		**3(2+1) - 72" - HDPE**		**2(1+1) - 96" - HDPE**	
Overal Capacity (MGD)	299	299	299	299	299	299	299	299	299	299	299	299
Laterals in Operation	3	4	2	3	1	2	3	4	2	3	1	2
Flow Rate per Lateral(MGD)	99.67	74.75	149.50	99.67	299.00	149.50	99.67	74.75	149.50	99.67	299.00	149.50
Flow Rate per Lateral(CFS)	154.22	115.66	231.33	154.22	462.65	231.33	154.22	115.66	231.33	154.22	462.65	231.33
Lateral Size (in)	59.7	59.7	96	96	96	96	59.7	59.7	96	96	96	96
Velocity Per Lateral (fps)	7.93	5.95	4.60	3.07	9.20	4.60	7.93	5.95	4.60	3.07	9.20	4.60
No. of Screens per Lateral	4	4	5	5	12	12	5	5	6	6	13	13
Screen Length (ft)	14	14	14	14	14	14	14	14	14	14	14	14
Screen Diameter (ft)	7	7	7	7	7	7	7	7	7	7	7	7
Screen Area (ft^2)	307.88	307.88	307.88	307.88	307.88	307.88	307.88	307.88	307.88	307.88	307.88	307.88
Screen Effective Area (36%)	110.84	110.84	110.84	110.84	110.84	110.84	110.84	110.84	110.84	110.84	110.84	110.84
Screen Through Velocity (fps)	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Screen Fouling Allowable %	**30.43**	**47.82**	**16.52**	**44.34**	**30.43**	**65.21**	**44.34**	**58.26**	**30.43**	**53.62**	**35.78**	**67.89**
Velocities At 15% Fouling (fps)	**0.41**	**0.31**	**0.49**	**0.33**	**0.41**	**0.20**	**0.33**	**0.25**	**0.41**	**0.27**	**0.38**	**0.19**

Description	**4(3+1) - 63" - HDPE**	
Overal Capacity (MGD)	299	299
Laterals in Operation	4	3
Flow Rate per Lateral(MGD)	74.75	99.67
Flow Rate per Lateral(CFS)	115.66	154.22
Lateral Size (in)	59.7	59.7
Velocity Per Lateral (fps)	5.95	7.93
No. of Screens per Lateral	4	4
Screen Length (ft)	11.5	11.5
Screen Diameter (ft)	7	7
Screen Area (ft^2)	252.90	252.90
Screen Effective Area (36%)	91.04	91.04
Screen Through Velocity (fps)	0.50	0.50
Screen Fouling Allowable %	**36.48**	**15.31**
Velocities At 15% Fouling (fps)	**0.37**	**0.50**

Wedge Wire Screens (3+1 Lateral Configuration)				
Description	**One Channel**		**Two Channels**	
Flow Rate (MGD)	299	299	299	299
Laterals in Operation	3	4	3	4
Total Number of Screens (4 Per Lateral)	12	16	12	16
Percent Allowable Screen Biofouling w/ 7ft DIA L=14ft screens @ 0.5 fps	30.43	47.82	30.43	47.82
Percent Allowable Screen Biofouling w/ 7ft DIA L=11.5ft screens @ 0.5 fps	15.31	36.48	15.31	36.48
HDPE Pipe 63" DIA - 59.7" I.D. (3+1 Lateral Configuration)				
Description	**1 - Channel**		**2 - Channels**	
Number of Influent Channels	1	1	2	2
Hazen Williams C - Factor	100	100	100	100
Allowable Bio Growth Thickness (in)	**0.35**	**3.63**	**2.33**	**5.38**
Effective Diameter (in)	59.00	52.44	55.05	48.94
Pipe Velocity (fps)	8.12	7.71	9.33	8.85
Maximum Froude Number Calculated	1.00	1.00	1.00	1.00
Critical Depth WL (ft)	-10.70	-10.70	-12.06	-12.06
MLLW (ft)	-2.30	-2.30	-2.30	-2.30
Pipe Friction Loss (ft)	5.89	5.89	8.08	8.08
Bar Rack Water Elevation (ft)	**-8.19**	**-8.19**	**-10.38**	**-10.38**
Channel Friction Loss (ft)	2.51	2.51	1.68	1.68
Pump Station Water Elevation (ft)	**-10.70**	**-10.70**	**-12.06**	**-12.06**

Hazen Williams (Full Circular Pipe)

$$h_f = \frac{4.727 * L_{(ft)} * Q_{(CFS)}^{1.8519}}{C^{1.8519} * D_{(ft)}^{4.87}}$$

Manning's (Open Channel)

$$h_f = \frac{0.4504 * L_{(ft)} * Q_{(CFS)}^{2} * n^2}{A^2 R^{4/3}}$$

With No Biofoulng and C=100

Configuration	Flow Rate (CFS)	Pipe Diameter (in)	Pipe Area (ft^2)	Pipe Length (ft)	C-Factor	Pipe Friction Loss (ft)	Velocity (fps)	Total Minor K	Total Minor Losses (ft)	Screen Headloss (ft)	Total Headloss (ft)	Water Elevation at Trash Rack (ft)	Limit Elevation for Sub Critical with Single Channel (ft)
3+1	154.22	59.70	19.44	900	100	3.84	7.93	1.7	1.66	0.08	**5.58**	-7.88	-9
4	115.66	59.70	19.44	900	100	2.25	5.95	1.7	0.93	0.08	**3.27**	-5.57	-9
2+1	231.33	72.00	28.27	900	100	3.26	8.18	1.7	1.77	0.08	**5.11**	-7.41	-9
3	154.22	72.00	28.27	900	100	1.54	5.45	1.7	0.79	0.08	**2.41**	-4.71	-9
1+1	462.65	96.00	50.27	900	100	2.90	9.20	1.7	2.24	0.08	**5.22**	-7.52	-9
2	231.33	96.00	50.27	900	100	0.80	4.60	1.7	0.56	0.08	**1.45**	-3.75	-9

With 4" Biofoulng and C=100

Configuration	Flow Rate (CFS)	Pipe Diameter (in)	Pipe Area (ft^2)	Pipe Length (ft)	C-Factor	Pipe Friction Loss (ft)	Velocity (fps)	Total Minor K	Total Minor Losses (ft)	Screen Headloss (ft)	Total Headloss (ft)	Water Elevation at Trash Rack (ft)	Limit Elevation for Sub Critical with Single Channel (ft)
3+1	154.22	51.70	14.58	900	100	7.73	10.58	1.7	2.95	0.08	**10.77**	-13.07	-9
4	115.66	51.70	14.58	900	100	4.54	7.93	1.7	1.66	0.08	**6.28**	-8.58	-9
2+1	231.33	64.00	22.34	900	100	5.79	10.35	1.7	2.83	0.08	**8.71**	-11.01	-9
3	154.22	64.00	22.34	900	100	2.73	6.90	1.7	1.26	0.08	**4.07**	-6.37	-9
1+1	462.65	88.00	42.24	900	100	4.43	10.95	1.7	3.17	0.08	**7.68**	-9.98	-9
2	231.33	88.00	42.24	900	100	1.23	5.48	1.7	0.79	0.08	**2.10**	-4.40	-9

Fouling All Around (in)	Configuration	Flow Rate (CFS)	Pipe Diameter (in)	Pipe Area (ft^2)	Pipe Length (ft)	C-Factor	Pipe Friction Loss (ft)	Velocity (fps)	Total Minor K	Total Minor Losses (ft)	Screen Headloss (ft)	Total Headloss (ft)	Water Elevation at Trash Rack (ft)	Limit Elevation for Sub Critical with Single Channel (ft)
3.63	4	115.66	52.44	15.00	900	100	4.24	7.71	1.7	1.57	0.08	**5.89**	-8.19	-8.52
0.35	3+1	154.21	59.00	18.98	900	100	4.06	8.12	1.7	1.74	0.08	**5.89**	-8.19	-8.52
5.38	4	115.66	48.94	13.07	900	100	5.92	8.85	1.7	2.07	0.08	**8.08**	-10.38	-10.71
2.33	3+1	154.21	55.05	16.53	900	100	5.69	9.33	1.7	2.30	0.08	**8.08**	-10.38	-10.71

One Channel

	4	3+1	
Total Intake Flow Rate	299	299	MGD
Total Intake Flow Rate	463	463	cfs
Intake Channel Length	90	90	ft
Intake Channel Bio Growth	4	4	in
Intake Channel Width	10.33	10.33	ft
Intake Channel Height	6.48	6.48	ft
Intake Channel Area	66.94	66.94	ft^2
Intake Channel Wetted Perimeter P	23.29	23.29	ft
Hydraulic diameter d_h	11.50	11.50	ft
Upstream Velocity	1.36	1.36	fps
Channel Velocity Start	6.91	6.91	fps
Channel Velocity End	11.30	11.30	fps
Down Stream Velocity	0.83	0.83	fps
Entrance Minor Loss	0.36	0.36	ft
Exit Minor Loss	1.97	1.97	ft
45-Degree Bend	0.11	0.11	ft
Total Minor Loss	2.43	2.43	ft
Solver Use	2.43	2.43	ft
Solver Use (Difference)	0.00	0.00	ft
Inake Channel Manning (n-coefficient)	0.013	0.013	#
Channel Friction Loss h_f	0.080	0.080	ft
End Channel Water Depth	3.96	3.96	ft
WSL New Intake Struct MLLW	-10.70	-10.70	ft

Channel Start Froude # (< 1 = Subcritical)	0.47	0.47	#
Channel End Froude # (< 1 = Subcritical)	1.000	1.000	#
Critical Depth	3.96	3.96	ft
Critical Water Level	-10.70	-10.70	ft

Two Channels

	4	3+1	
Total Intake Flow Rate	149.5	149.5	MGD
Total Intake Flow Rate	231	231	cfs
Intake Channel Length	90	90	ft
Intake Channel Bio Growth	4	4	in
Intake Channel Width	9.67	9.67	ft
Intake Channel Height	4.29	4.29	ft
Intake Channel Area	41.48	41.48	ft^2
Intake Channel Wetted Perimeter P	18.25	18.25	ft
Hydraulic diameter d_h	9.09	9.09	ft
Upstream Velocity	0.68	0.68	fps
Channel Velocity Start	5.58	5.58	fps
Channel Velocity End	9.17	9.17	fps
Down Stream Velocity	0.49	0.49	fps
Entrance Minor Loss	0.24	0.24	ft
Exit Minor Loss	1.30	1.30	ft
45-Degree Bend	0.07	0.07	ft
Total Minor Loss	1.61	1.61	ft
Excel Macro Use	1.61	1.61	ft
Excel Difference	0.00	0.00	ft
Inake Channel Manning (n-coefficient)	0.013	0.013	#
Channel Friction Loss h_f	0.071	0.071	ft
End Channel Water Depth	2.61	2.61	ft
WSL New Intake Struct MLLW	-12.06	-12.06	ft

Channel Start Froude # (< 1 = Subcritical)	0.46	0.46	#
Channel End Froude # (< 1 = Subcritical)	1.000	1.000	#
Critical Depth	2.61	2.61	ft
Critical Water Level	-12.06	-12.06	ft

Project:	Computed:	Date:
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Job #:	No:	

Plan : 4(3+1) - 63" - HDPE

Total flow rate is 299 MGD

flow rate per pipe : $Q = \frac{299}{3} = 99.67\,MGD = 154.21\,cfs$

Lateral Size : $D = 59.7" = 4.975\,ft$

Number of Screen on lateral : $n = 4$

Screen size : Length : 14 ft

Diameter : 7 ft

Velocity through Screen Limit : $V_L = 0.50$

Velocity in pipe : $V = \frac{Q \cdot 4}{\pi \cdot D^2} = \frac{154.21 \times 4}{3.14159 \times (4.975)^2} = 7.93\,fps$

Screen Area, $A = \pi \cdot D \cdot L = \pi \cdot 14 \cdot 7 = 307.88\,ft^2$

effective (36%) Area, $A' = A \cdot 36\% = 110.84\,ft^2$

Screen Fouling Allowable :

$$\frac{A' - \left(\frac{Q}{n \cdot V_L}\right)}{A'} = \frac{110.84 - \left(\frac{154.21}{4 \cdot 0.5}\right)}{110.84} = 30.44\%$$

Velocity @ 15% fouling :

$$\frac{Q}{n \cdot A' \cdot (1 - 15\%)} = \frac{154.21}{4 \cdot 110.84 \cdot 0.85} = 0.41\,fps$$

HDR	Project:	Computed:	Date:
	Subject:	Checked:	Date:
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	Job #:	No:	

Plan: 4 (3+1) - 63"

Total flow rate: $Q = 99.67\ MGD = 154.21\ cfs$

Pipe diameter : $D = 59.7$ inch $= 4.975$ ft

Pipe Length : $L = 900$ ft

C factor in Hazen Williams : $C = 100$

Pipe Area : $A = \frac{\pi \cdot D^2}{4} = \frac{\pi (4.975)^2}{4} = 19.44\ ft^2$

Friction Loss : Hazen Williams :

$$hf = \frac{4.727 \times L \times Q^{1.8519}}{C^{1.8519} \times D^{4.87}} = \frac{4.727 \times 900 \times (154.21)^{1.8519}}{100^{1.8519} \times (4.975)^{4.87}}$$

$$= 3.836\ ft$$

Total Minor $K = 1.7$. Velocity : $V = \frac{Q}{A} = 7.93$ fps

Minor loss $= K \cdot \frac{V^2}{2g} = 1.7 \times \frac{(7.93)^2}{2 \times 32.2} = 1.66$

Screen head loss is 0.08 ft

Total head loss $= 3.836 + 1.66 + 0.08 = 5.576$ ft

Water Elevation $= -2.3\ ft - 5.576 = -7.876$ ft

Project:	Computed:	Date:
Subject:	Checked:	Date:
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Job #:	No:	

Plan : 4 (3+1) - 63" with 4" Biofouling

Total flow rate : $Q = 99.67\ MGD = 154.21\ cfs$

Pipe diameter : $D = 59.7 - 2\times4 = 51.7\ inch = 4.31\ ft$

Pipe Length : $L = 900\ ft$

C factor in Hazen Williams : $C = 100$

Pipe Area : $A = \frac{\pi \cdot D^2}{4} = \cancel{\pi(51.7)^2} \frac{\pi(4.31)^2}{4} = 14.59\ ft^2$

Friction Loss : Hazen Williams :

$$h_f = \frac{4.727 \times L \times Q^{1.8519}}{C^{1.8519} \times D^{4.87}} = \frac{4.727 \times 900 \times (154.21)^{1.8519}}{100^{1.8519} \times (4.31)^{4.87}} = 7.72\ ft$$

Total Minor $K = 1.7$

Velocity : $V = \frac{Q}{A} = 10.57\ fps$

Minor loss $= K \cdot \frac{V^2}{2g} = 1.7 \cdot \frac{(10.57)^2}{2\times32.2} = 2.95\ ft$

Screen head loss is 0.08 ft

Total head loss $= 7.72 + 2.95 + 0.08 = 10.75\ ft$

Wate Elevation $= -2.3 - 10.75 = -13.05\ ft$

Project:	Computed:	Date:
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Fouling All Around = 3.62 inch = 0.30 ft

configuration : 4

Flow Rate : $Q = 463/4 = 115.66$ cfs

Pipe diameter : $D = 59.7 - 3.62 \times 2 = 52.46$ inch $= 4.37$ ft

Pipe Area : $A = \frac{D^2 \cdot \pi}{4} = 15.00\ ft^2$

Pipe Length : $L = 900$ ft

Haze Williams : $C = 100$

$$hf = \frac{4.727 \times L \times Q^{1.8519}}{C^{1.8519} \times D^{4.87}} = \frac{4.727 \times 900 \times 115.66^2}{100^{1.8519} \times 4.37^{4.87}} = 4.23 \text{ ft}$$

Velocity : $V = \frac{Q}{A} = \frac{115.66 \text{ cfs}}{15\ ft^2} = 7.71$ fps

Minor Losses : $K \frac{V^2}{2g} = 1.7 \times \frac{7.71^2}{2 \times 32.2} = 1.57$ ft

Screen Head Losses is 0.08 ft

Total Headloss = 4.23 + 1.57 + 0.08 = 5.88 ft

Water Elevation = − 2.3 − 5.88 = − 8.18 ft

Project:	Computed:	Date:
Subject:	Checked:	Date:
Task:	Page:	of:
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Plan : One channel

Total Flow Rate : 299 MGD , $Q = 463$ cfs

Intake channel : Length: $L = 90$ ft , Biogrowth : 4 inch = 0.33 ft

Width : $W = 11 - 2 \times 0.33 = 10.34$ ft

Height : $H = -8.18 - (-15 + 0.33) = 6.49$ ft

Area : $A = W \times H = 10.34 \times 6.49 = 67.11 \ ft^2$

Wetted Perimeter : $P = 2H + W = 2 \times 6.49 + 10.34 = 23.32$ ft

Hydraulic diameter : $d_h = 4 \times \frac{A}{P} = 4 \times \frac{67.11}{23.32} = 11.51$ ft

Upstream Velocity : $V = \frac{Q}{(15-8.18) \times 50} = 1.36$ fps

Channel Velocity : Start : $V_1 = \frac{463}{67.07} = 6.9$ fps

end : $V_2 = \frac{463}{10.33 \times 3.99} = 11.23$ fps

Downstream Velocity : $V_3 = \frac{463}{(200-10.68) \times 60} = 0.83$ fps

Entrance Minor : $K_1 \frac{|V_1^2 - V^2|}{2g} = \frac{0.5 \times |6.9^2 - 1.36^2|}{2 \times 32.2} = 0.355$ ft

Exit Minor loss : $K_2 \frac{|V_2^2 - V_3^2|}{2g} = \frac{|11.22^2 - 0.83^2|}{2 \times 32.2} = 1.944$ ft

45° Bend Minor loss : $K_3 \times \sqrt{\frac{30}{90}} \times \frac{V_1^2}{2g} = \frac{0.25 \times \sqrt{\frac{1}{3}} \times 6.9^2}{2 \times 32.2} = 0.107$ ft

Total Minor loss : $0.355 + 1.944 + 0.107 = 2.406$ ft

Manning coefficient : $n = 0.013$

$$hf = \frac{0.4504 \times L \times Q^2 \times n^2}{A^2 R^{4/3}} = \frac{0.4504 \times 90 \times 463^2 \times 0.013^2}{67.11^2 \times (67.11/23.32)^{4/3}} = 0.080 \text{ ft}$$

WSL New Intake Struct MLLM $= -8.18 - 0.080 - 2.42 = -10.68$ ft

End channel Water Depth $= 15 - 10.68 - 4/12 = 3.99$ ft

HDR

Project:	Computed:	Date:
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Plan: 14(3+1) - 63" - HPDE w/ screen length 11.5 ft

Total flow rate is 299 MGD

Flow rate per pipe: $Q = \frac{299}{3} = 99.67\ MGD = 154.21\ cfs$

Lateral Size : $D = 59.7" = \cancel{29}\ 4.975\ ft$

Number of Screen on lateral : $n = 4$

Screen Size = Length : $\cancel{}\ 11.5\ ft$

Diameter: 7 ft

Velocity through Screen Limit : $V_L = 0.50$

Velocity in pipe : $V = \frac{Q \cdot n}{\pi \cdot D^2} = \frac{154.21 \times 4}{\pi \times (4.975)^2} = 7.93\ fps$

Screen Area, $A = \pi \cdot D \cdot L = \pi \cdot 11.5 \cdot 7 = 252.90\ ft^2$

Effective (36%) Area, $A' = A \times 36\% = 91.04\ ft^2$

Screen Fouling Allowable:

$$\frac{A' - \left(\frac{Q}{n \cdot V_L}\right)}{A'} = \frac{91.04 - \left(\frac{154.21}{4 \times 0.5}\right)}{91.04\cancel{}} = 15.31\ \%$$

Velocity @ 15% fouling

$$\frac{Q}{n \cdot A' \cdot (1 - 15\%)} = \frac{154.21}{4 \times 91.04 \times 0.85} = 0.50\ fps$$

Attachment D – Wedgewire Screen Details and Vendor Correspondence

Johnson Screens®

Aqseptence Group, Inc.
1950 Old Hwy 8 NW
New Brighton, MN 55112
USA

Phone +1 651 636 3900
info.water@aqseptence.com
www.aqseptence.com

May 29, 2017

Budgetary Proposal – Johnson Intake System

<u>Carlsbad CA – Desal Intake</u>

Based on:

- (4) 63 inch Intake Pipelines – 100 MGD Each
- (16) Tee Screens - 25 MGD Flow per screen
- Seawater
- 1 mm slot
- 0.5 feet/second maximum slot velocity
- Use 450 foot airline length for Hydroburst

This works out to be (16) T-84HC Screens (see screen sketch and concept layout). This screen in Z-Alloy would cost about $209,200 each. In Duplex, it would be about $189,800 each.

The Hydroburst would be (2) 5,000 gallon systems with (8) 10 inch valves and manifold each. See attached chart. This system would cost about $140,000 each for a fully automated system with 24/7 operation.

Total equipment cost for the Z-Alloy Option is about $3,627,200 and for Duplex about $3,316,800

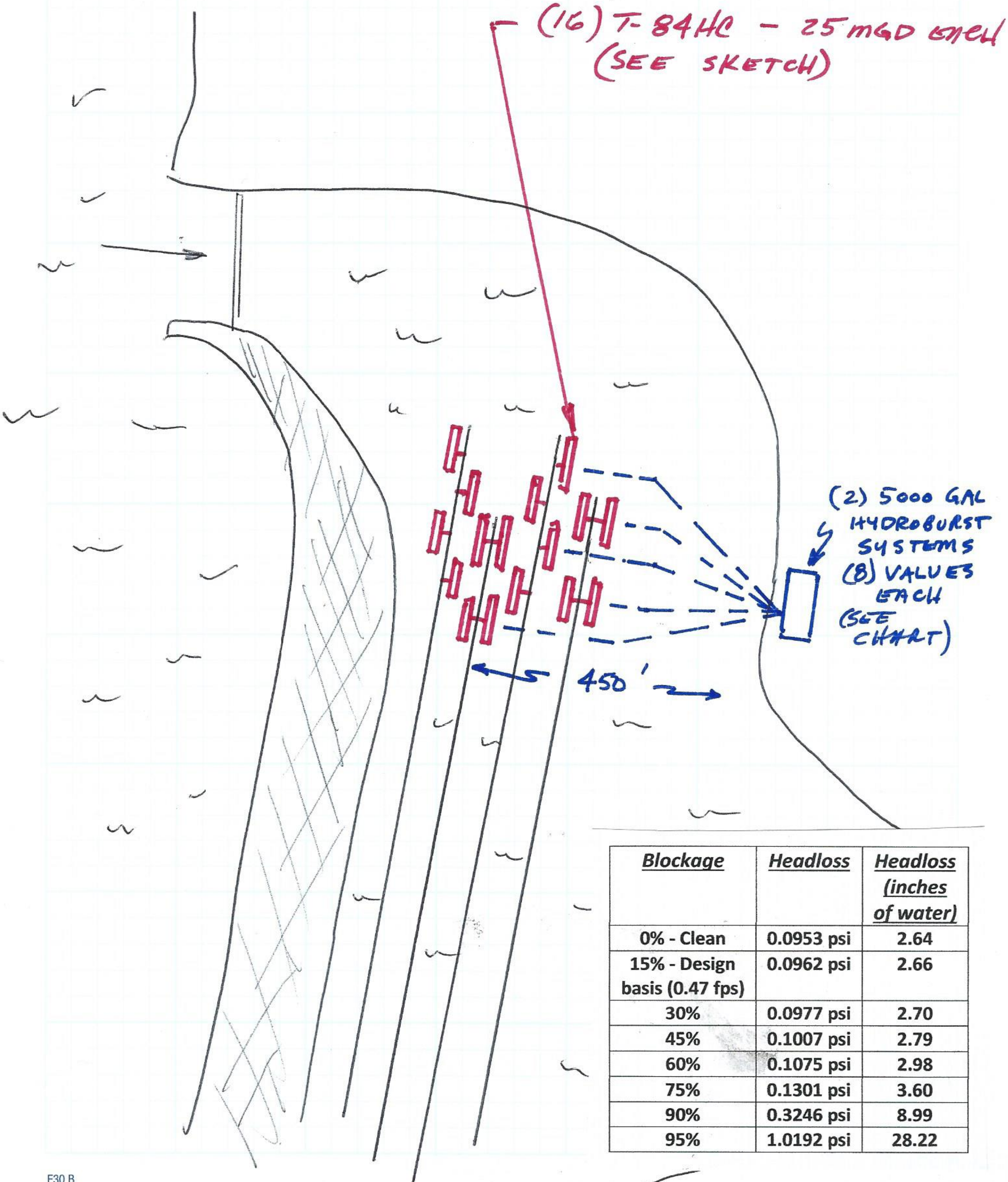
Thanks – Mark

Mark Watson
Eastern Regional Sales Mgr.
<u>Aqseptence – Intake Screen Group</u>

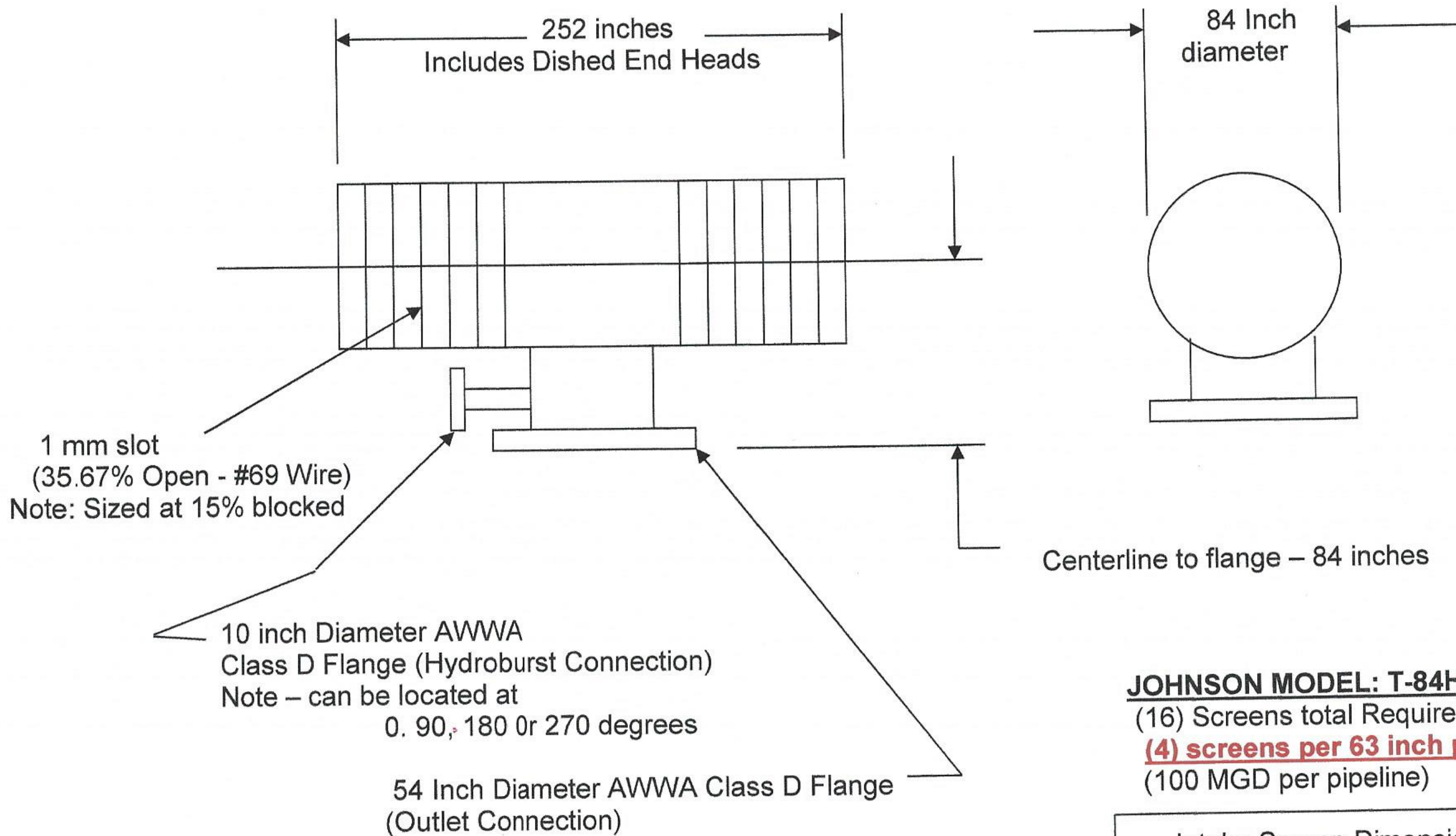
508-347-9309
mark.watson@aqseptence.com

CC: Billy Emmers – Aqseptence
Mark Bell – Aqseptence
Dave Anderson - Aqseptence

SUBJECT			WATER TECHNOLOGIES BILFINGER
CARLSBAD CA	DATE 5-29-17	PROJECT NO.	
FOR	BY M. WATSON	SHEET 1 OF 1	



Blockage	Headloss	Headloss (inches of water)
0% - Clean	0.0953 psi	2.64
15% - Design basis (0.47 fps)	0.0962 psi	2.66
30%	0.0977 psi	2.70
45%	0.1007 psi	2.79
60%	0.1075 psi	2.98
75%	0.1301 psi	3.60
90%	0.3246 psi	8.99
95%	1.0192 psi	28.22

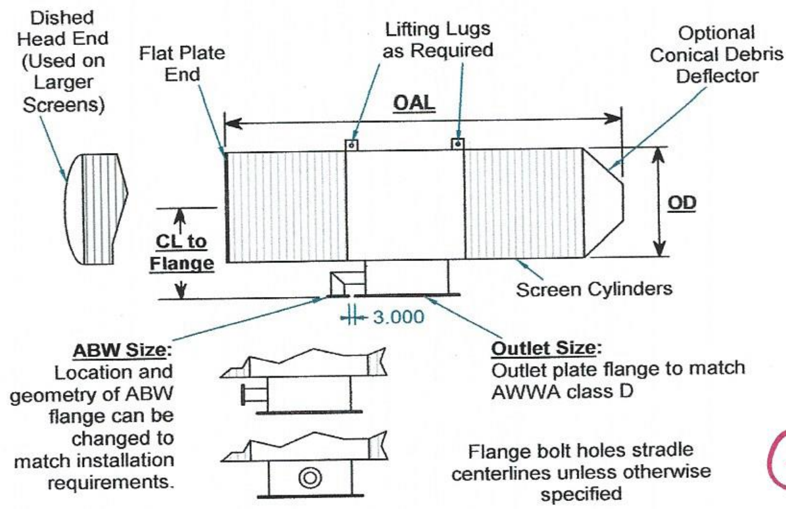


JOHNSON MODEL: T-84HC
(16) Screens total Required
(4) screens per 63 inch pipeline
(100 MGD per pipeline)

Estimated Total Headloss = 0.096 psi
Approximate Weight = 9,710 lbs.
Material = Z-Alloy or Duplex

Design Flow = 400 MGD Total
Actual Slot Velocity = 0.46 fps Max (0.41 Ave)
Design Slot Velocity = 0.5 fps Maximum

Intake Screen Dimensions
Carlsbad Desal Project, CA
Johnson screens™



SCREEN SIZING
BASED ON
15% BLOCKAGE
0.5 fps MAX
1 mm SLOT
(4) 63" PIPES –
100 MGD EACH

Dimension/Sizes	Value	Unit	Comments
Model	T84HC		
OD	84.00	in	Nominal See note 1
OAL	252.00	in	Nominal See note 1
CL to Flange	84.00	in	Nominal See note 1
Outlet Connection Size	54PS		See note 2
ABW Connection Size	10PS		See note 2
Estimated Weight	9710	lbs	
Left End Closure	Dished		
Right End Closure	Dished		

Screen Specifications			
Slot Opening	1.000	mm	
Open Area Percentage	30.32%		w/15% BLOCKED
L/D Ratio	1.000		
Effective Screen Length	84.00	in	
Wire Type	69		

Design:			
Depth	10	Ft	Hydrostatic Load
Collapse Rating	4.34	psi	
Material	ZALLOY		SS or Z-Alloy

Flow Capacities:			
Flow/Screen	17361.11111	GPM	See note 3
Maximum Slot Velocity	0.46	fps	See note 3 w/15% BLOCKED
Average Slot Velocity	0.41	fps	See note 3
Estimated DP/Screen	0.0032	psi	Thru clean screen surface only-See note 4
Estimated DP/Assy	0.0962	psi	Through entire clean assembly - See note 4

@15% BLOCKED

PATENT #
#6,051,131

Notes:

1 Dimensions shown are nominal. All screens are made to order, and can be adapted to a wide variety of installation requirements.

2 These are based on the size of the connecting and abw pipe and may vary from the values listed in the technical brochure.

3 Capacities based on use of patented Johnson flow modifier design.

4 Pressure drops below .1 psi should be considered an order of magnitude only, as testing date for these low values is not available.

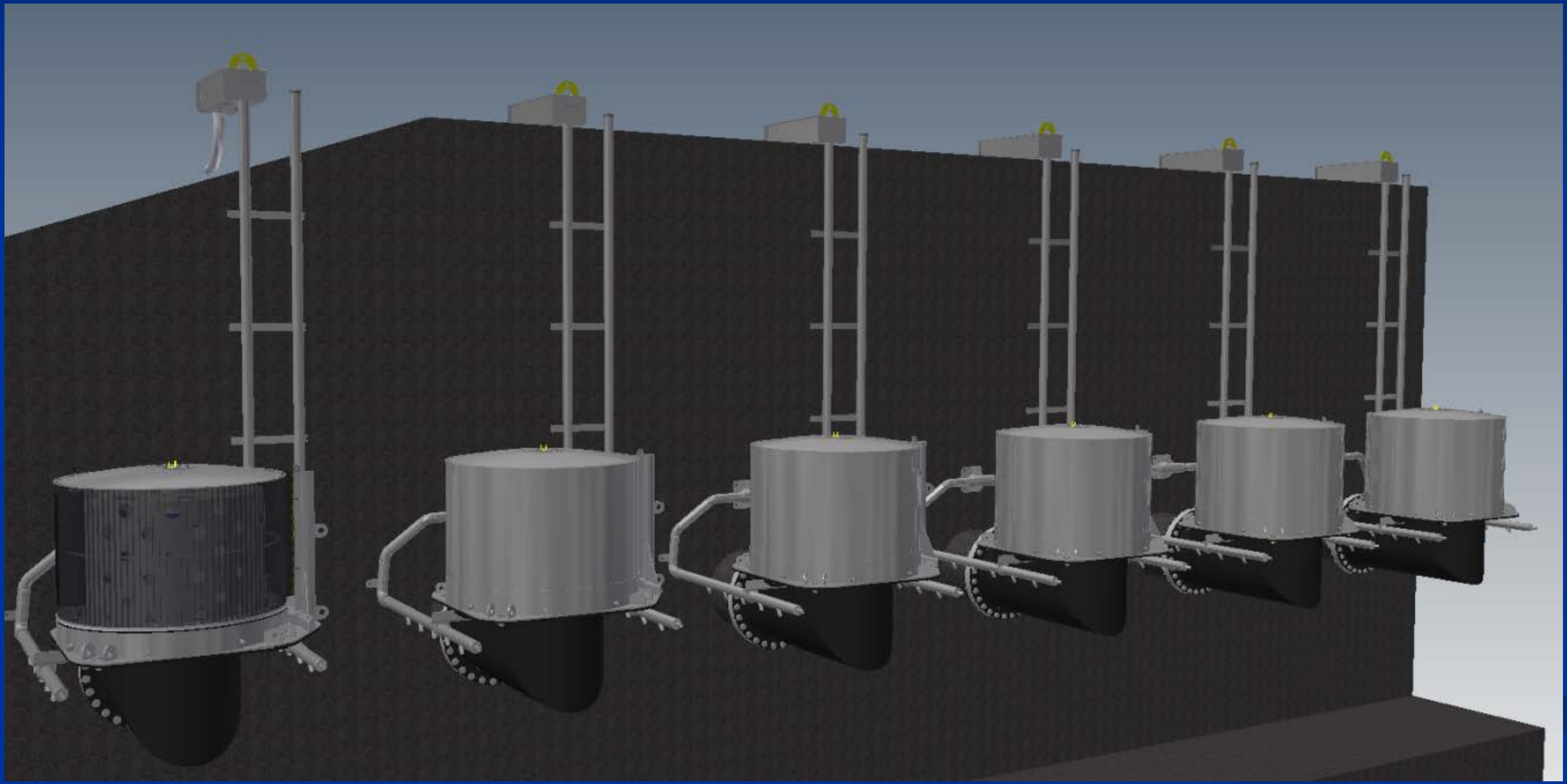
BILFINGER WATER TECHNOLOGIES
1950 Old Hwy. 8, New Brighton, MN 55112
(651)-638-3218 (Phone), (651)-638-3177 (Fax)

Model T84HC

Preliminary Intake Screen Specification
0

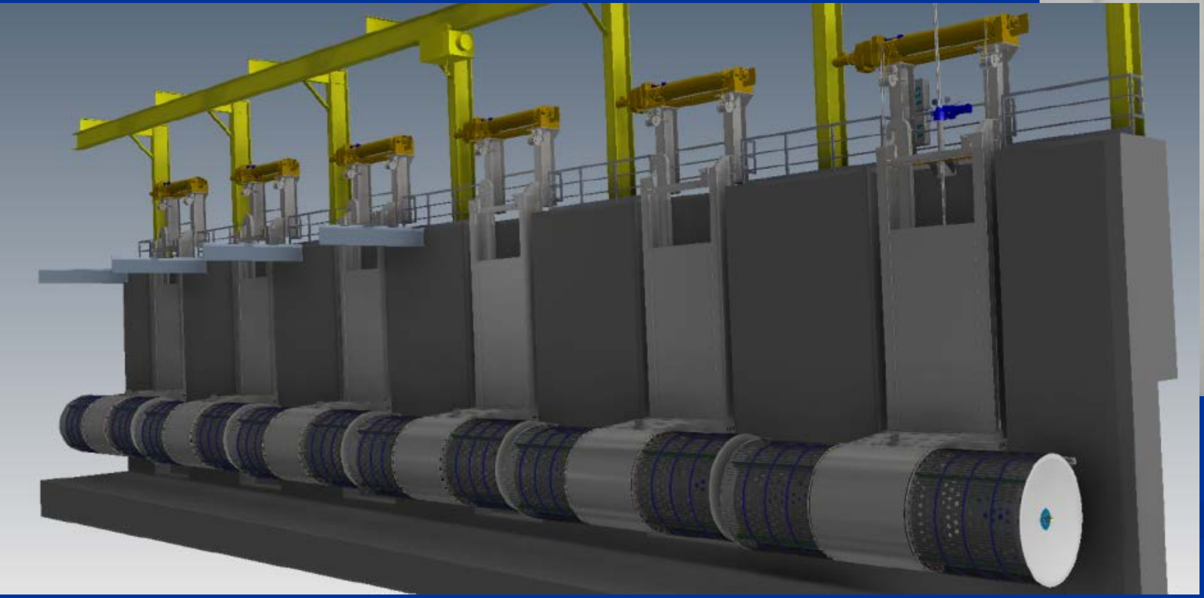
Date:
5/26/2017

Vertical Cylindrical Intake Screen Retrofit Design by ISI



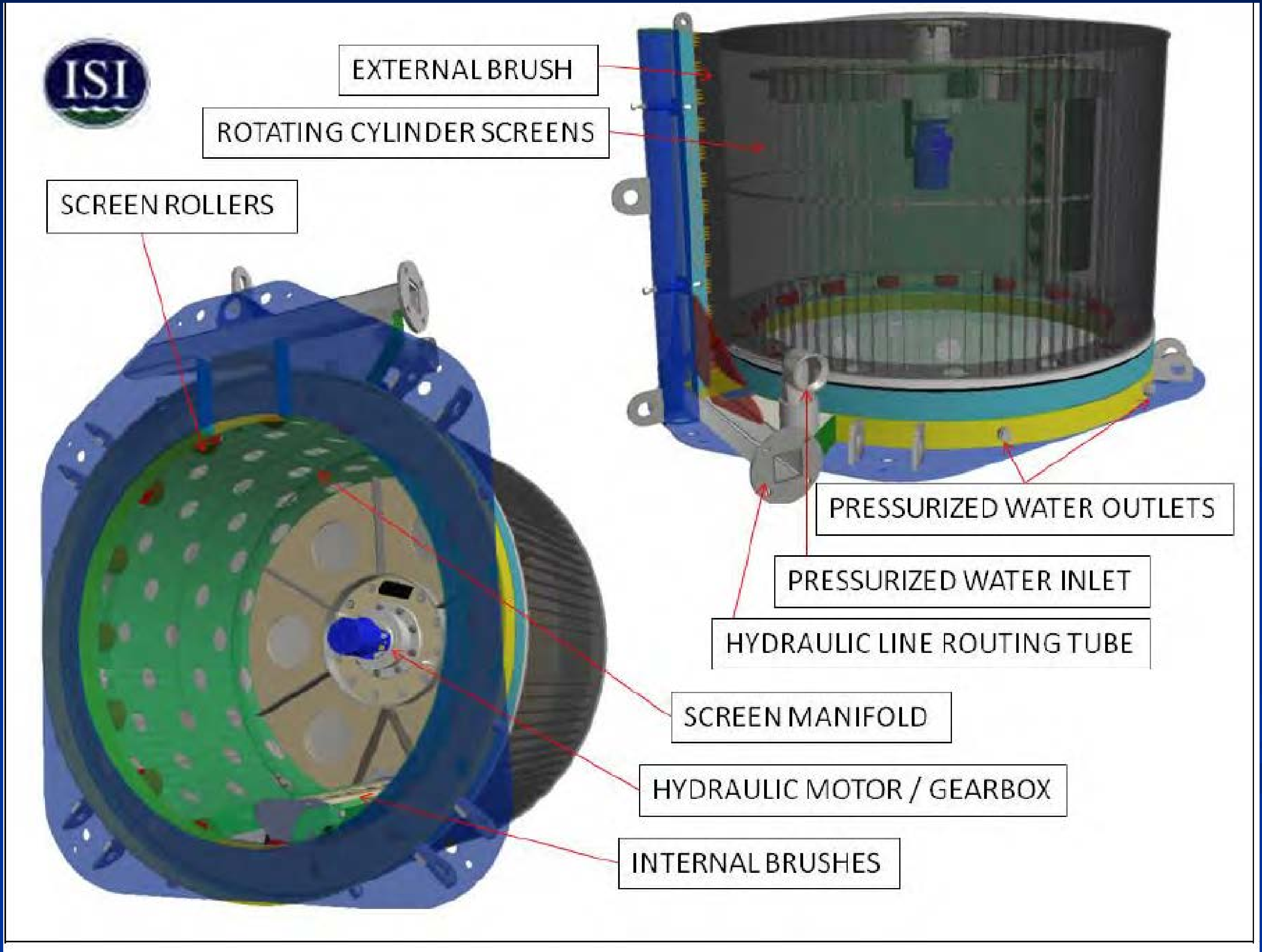
Hydropower Intake Example
Six Screens with Electric Drives

Facility Retrofit with 7-ft Dia. Cylinders in Limited Space

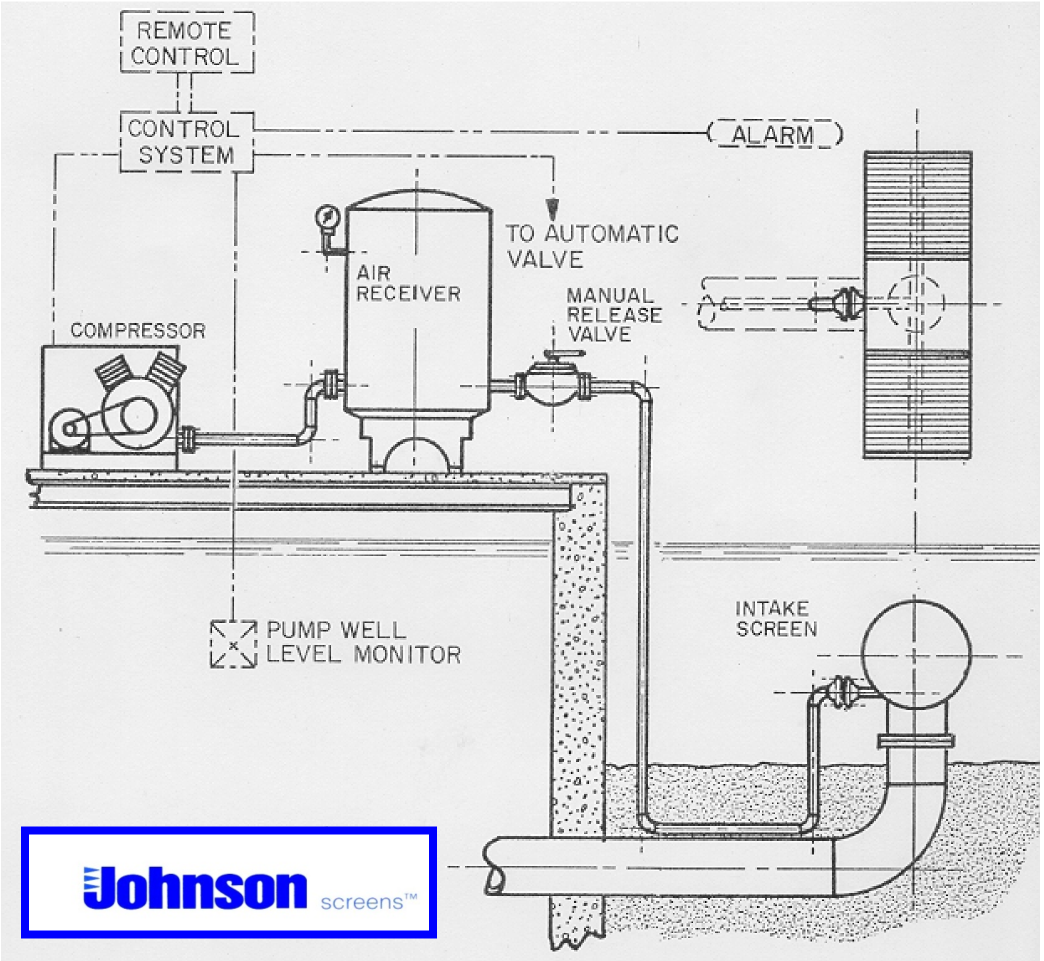


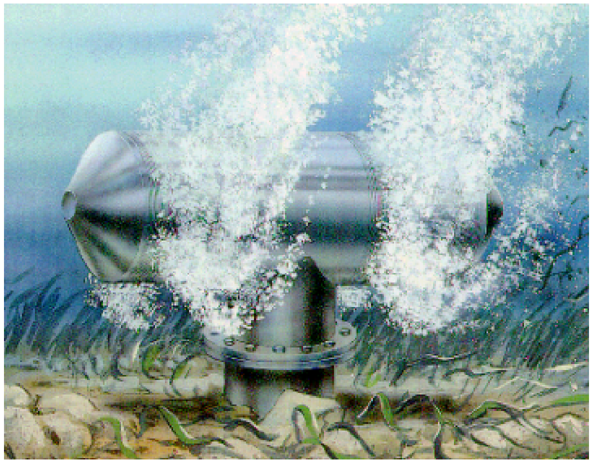
Model Testing at NHC

ISI Brushed Screen Features



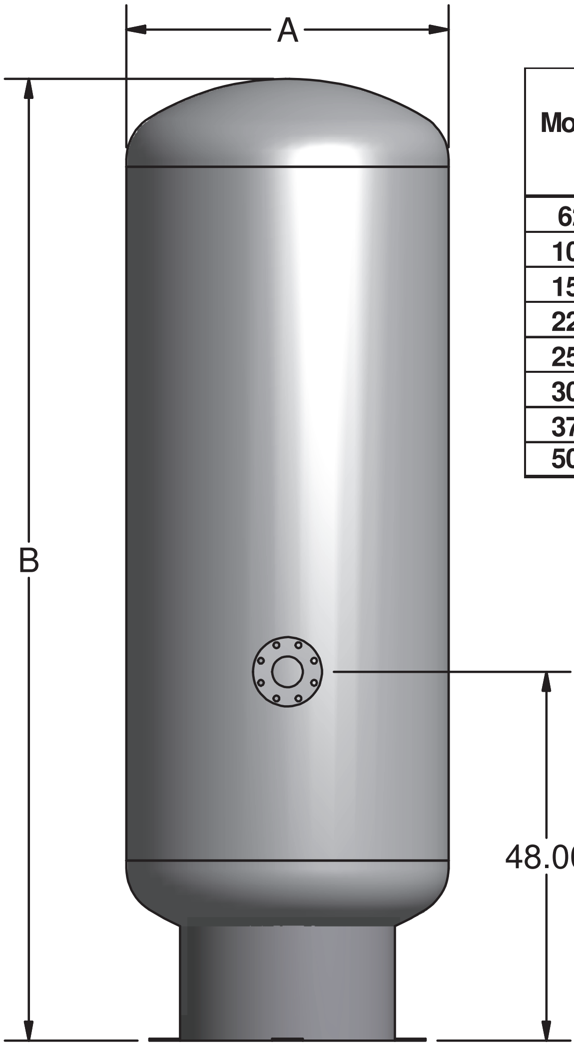
Hydroburst™ Systems





Large Hydroburst Selection Chart

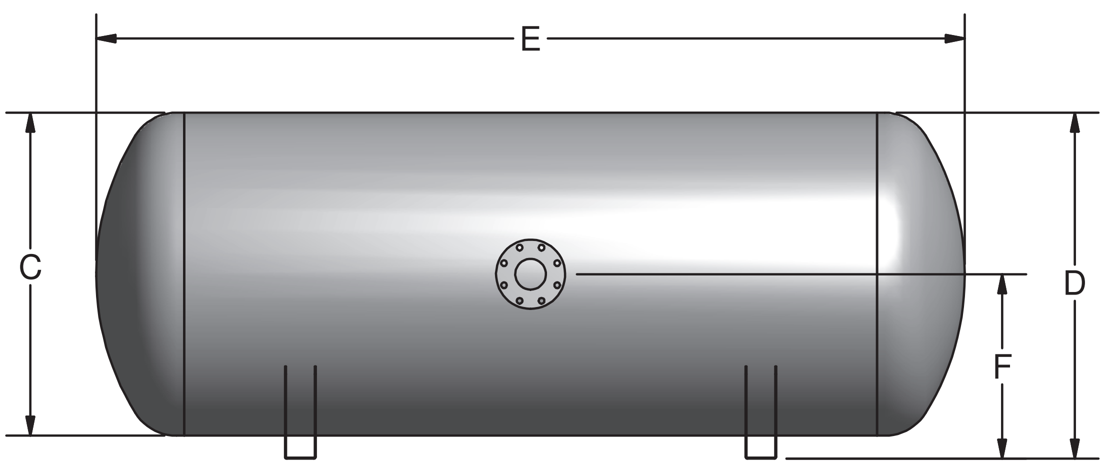
620 Gallon to 5000 Gallon



Vertical Air Receiver
200 PSIG MWP

Model #	Outlet Flange Size	Vertical Air Receiver		Estimated Weight
		Diameter "A"	Height "B"	
620V	4"	42"	125"	1500 LBS
1040V	6"	48"	155"	1900 LBS
1550V	8"	54"	180"	2900 LBS
2200V	8"	60"	203"	3600 LBS
2560V	8"	60"	236"	4200 LBS
3000V	8"	66"	229"	5500 LBS
3750V	10"	72"	240"	6600 LBS
5000V	12"	72"	311"	8600 LBS

Model #	Outlet Flange Size	Horizontal Air Receiver				Estimated Weight
		Diameter "C"	Height "D"	Length "E"	Outlet "F"	
620H	4"	42"	45"	113"	24"	1500 LBS
1040H	6"	48"	51"	143"	27"	1900 LBS
1550H	8"	54"	57"	168"	30"	2900 LBS
2200H	8"	60"	63"	191"	33"	3600 LBS
2560H	8"	60"	63"	224"	33"	4200 LBS
3000H	8"	66"	69"	217"	36"	5500 LBS
3750H	10"	72"	75"	228"	39"	6600 LBS
5000H	12"	72"	75"	299"	39"	8600 LBS

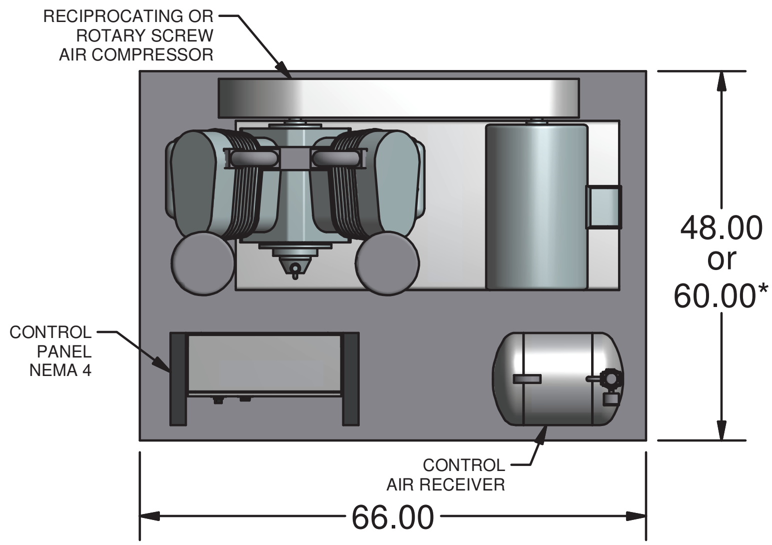


Horizontal Air Receiver
200 PSIG MWP

Johnson Screens

Large Hydroburst Selection Chart

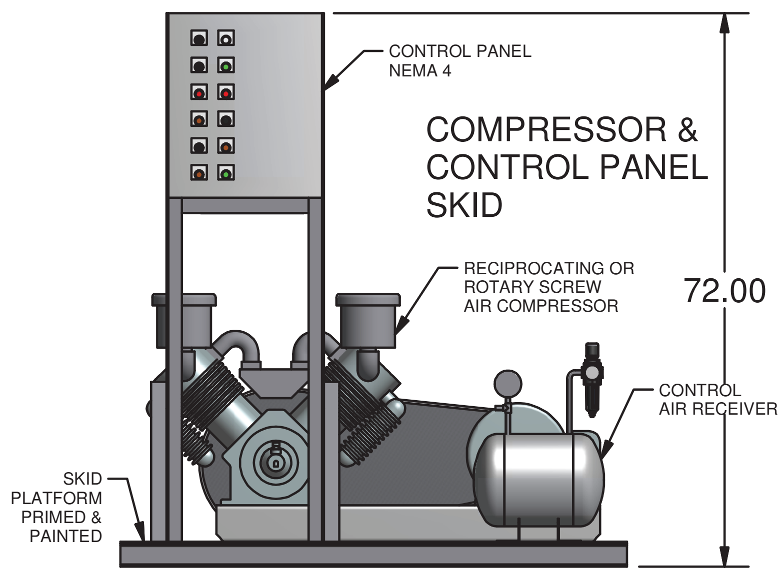
620 Gallon to 5000 Gallon



Air Receiver Size	Compressor Horsepower			
	Maximum Recharge Time			
	15 min	30 min	45 min	60 min
620	15	10	7.5	5
1040	20	15	10	7.5
1550	30	20	15	10
2200	50	25	15	15
2560	50	25	20	15
3000	75	30	20	15
3750	75	40	25	20
5000	100	50	40	25

**FOR COMPRESSOR SIZES BELOW & TO THE LEFT OF THE DOUBLE-LINE, ALL COMPONENTS WILL BE SHIPPED SEPARATELY AS STAND-ALONE COMPONENTS.

**FOR COMPRESSOR SIZES ABOVE & TO THE RIGHT OF THE DOUBLE-LINE, ALL COMPONENTS WILL BE SKID MOUNTED.

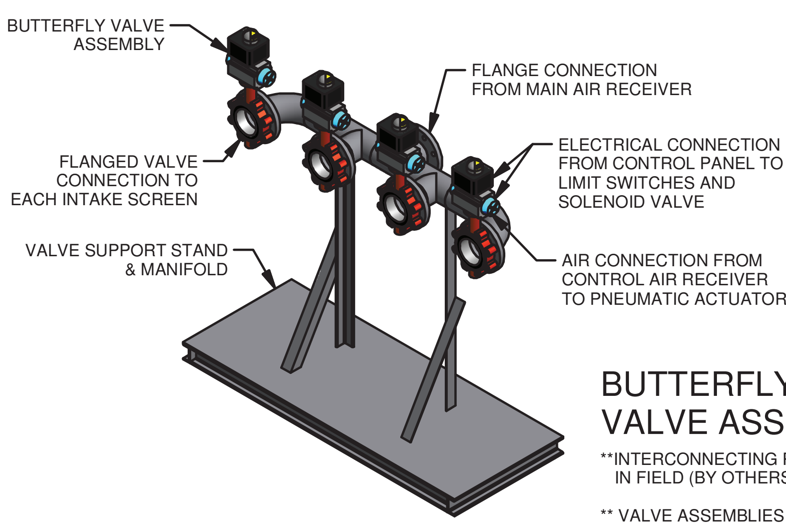


*SKID WIDTH IS 48.00" WITH RECIPROCATING COMPRESSOR & 60.00" WITH ROTARY SCREW COMPRESSOR

Skid Assembly	Estimated Weight	
	Reciprocating	Rotary Screw
5 HP	1000	1750
7.5 HP	1125	1775
10 HP	1150	1800
15 HP	1675	1950
20 HP	1700	2000
25 HP	1725	2075
30 HP	1750	2150

Component	Estimated Weight	
	Reciprocating	Rotary Screw
40 HP	1950	1650
50 HP	2700	2150
75 HP	3500	3150
100 HP	3700	3400
125 HP	3950	5350
	Component	
Control Panel	150	
Control Air Receiver	50	
Valve Assembly*	50	
Skid Platform	350	

**VALVE ASSEMBLY WEIGHT IS BASED ON A 4" SIZE VALVE. FOR EACH INCREASE IN VALVES SIZE, ADD 20 LBS.



BUTTERFLY
VALVE ASSEMBLY

**INTERCONNECTING PIPING & WIRING IN FIELD (BY OTHERS)

** VALVE ASSEMBLIES CAN BE PROVIDED ON A PIPING MANIFOLD WITH SUPPORT STAND OR AS INDIVIDUAL ASSEMBLIES TO BE SITE MOUNTED.

Attachment E - Alternative 21 Engineer's Opinion of Probable Construction Cost

Poseidon Water - Interim Facilities Costs

Carlsbad Desalination Plant - Intake Facility

Preliminary Construction Cost Opinion, June, 2017

Section	Cost
DIVISION 0 - BIDDING REQUIREMENTS, ETC.	$ -
DIVISION 1 - GENERAL REQUIREMENTS	$ -
DIVISION 2 - SITE WORK *Includes: Demolition of existing, Evavation, Shoring, Gravel Fill, Dirt Hauling, Dewatering, and Dumping*	$ 3,265,000
DIVISION 3 - CONCRETE *Includes: Roof Slab, Floor Slab, and Walls (for both Interim Structure and Electrical Building)*	$ 3,130,000
DIVISION 4 - MASONRY *Includes: Concrete Blocks (for Electrical Building)*	$ 45,000
DIVISION 5 - METALS *Includes: Steel Roofing and Misc. (for Electrical Building)*	$ 97,000
DIVISION 6 - WOOD AND PLASTICS	$ -
DIVISION 7 - THERMAL & MOISTURE PROTECTION *Includes: TMP (for Electrical Building)*	$ 34,000
DIVISION 8 - DOORS & WINDOWS *Includes: Stainless Steel Double Door, Stainless Steel Single Door, and Misc./Windows (for Electrical Building)*	$ 32,000
DIVISION 9 - FINISHES *Includes: Pipe Coating and Electrical Building Floor Coating*	$ 24,000
DIVISION 10 - SPECIALTIES	$ -
DIVISION 11 - EQUIPMENT *Includes: (4) 9.5 mm Fish Screens, (4) 200 HP Dilution Pumps, 63 inch Insert Mag Meter, 84 inch Insert Mag Meter, and Misc.*	$ 2,806,000
DIVISION 12 - FURNISHINGS	$ -
DIVISION 13 - SPECIAL CONSTRUCTION	$ -
DIVISION 14 - CONVEYING SYSTEMS	$ -
DIVISION 15 - MECHANICAL *Includes: HDPE Pipe/Fittings, SST Pump Discharge and Fittings, and Misc Pipe Fittings*	$ 718,000
DIVISION 16 - ELECTRICAL *Includes: Transformer, Switchgear, MCC, VFD, PLC, Control Panel, etc.*	$ 2,325,000

Construction Subtotal	**$ 12,476,000**

Additional Cost Items

Mobilization, Bond, and Insurance (6.5 percent)	$ 811,000
General Conditions (9 percent) *Includes: Field Overhead, Construction Supervision, Field Engineering, Project Management, Safety, etc.*	$ 1,123,000
Contractor Profit (15 percent)	$ 1,872,000
Engineering - Design Build (10 percent)	$ 1,248,000
Contingency (25 percent)	$ 4,383,000

Additional Item Subtotal	**$ 9,437,000**

Today's Value Total	**$ 21,913,000**

Poseidon Water - Interim Facilities Costs

Carlsbad Desalination Plant - Intake Facility

Preliminary Construction Cost Opinion, June, 2017

SPEC SECTION		DESCRIPTION	SIZE	QUANTITY	UNITS	UNIT COST	INSTALLATION	TOTAL COST
DIVISION 0 - BIDDING REQUIREMENTS, ETC.								
	Not Used (Incorporated below)							$
						Subtotal:		**$**
DIVISION 1 - GENERAL REQUIREMENTS								
	Not Used (Incorporated below)							$
						Subtotal:		**$**
DIVISION 2 - SITE WORK								
	Demolition of Existing			1	LS	$150,000.00		$ 150,000
	Excavator, Hydraulic, crawler mtd., 1 C.Y. cap = 100 C.Y./hr.	Interim Structures		13,456	BCY	$5.00		$ 68,000
	Shoring	Interim Site		12,851	SF	$150.00		$ 1,928,000
	Gravel Fill, compacted, under floor slabs	Interim Site		233	CY	$63.20		$ 15,000
	Dirt hauling	Hauling of all excavated soils - Interim Site		9,989	CY	$27.75		$ 278,000
	Dewatering Interim($100K)			1	LS	$100,000.00		$ 100,000
	Dump Charges	Assume Soil 100 lb/ft^3		13,485	Tons	$50.00		$ 675,000
	Allowance			1	EA	$51,000.00		$ 51,000
						Subtotal:		**$ 3,265,000**
DIVISION 3 - CONCRETE								
	Slab Interim	Building Slab Interim		1,028	CY	$1,200.00		$ 1,235,000
	Wall Interim	Building Wall Interim		1,263	CY	$1,500.00		$ 1,895,000
						Subtotal:		**$ 3,130,000**
DIVISION 4 - MASONRY								
	Split Face 12" Concrete Blocks			1,120	SF	$40.00		$ 45,000
						Subtotal:		**$ 45,000**
DIVISION 5 - METALS								
	Steel Roofing (include beam, insulation, metal deck etc)	deep rib roofing for new addition		1,125	SF	$50.00		$ 57,000
	Misc			1	LS	$40,000.00		$ 40,000
						Subtotal:		**$ 97,000**
DIVISION 6 - WOOD AND PLASTICS								
	Not Used							
						Subtotal:		**$**
DIVISION 7 - THERMAL & MOISTURE PROTECTION								
	Allowance			1,125	SF	$ 30.00		$ 34,000
						Subtotal:		**$ 34,000**
DIVISION 8 - DOORS & WINDOWS								
	Exterior Stainless Steel Double Door	Electrical Room	8'x10"	1	EA	$6,000.00	$1,200.00	$ 8,000
	Exterior Stainless Steel Single Door	Electrical Room	4'x10"	1	EA	$3,000.00	$600.00	$ 4,000
	Misc to include windows			1	EA	$20,000.00		$ 20,000
						Subtotal:		**$ 32,000**
DIVISION 9 -FINISHES								
	Coating for New Pipes			1	LS	$10,000.00		$ 10,000
	Floor Coating (non skid)			1,125	SF	$12.00		$ 14,000
						Subtotal:		**$ 24,000**
DIVISION 10 - SPECIALTIES								
	Other Allowances			0	EA	$20,000.00		$
						Subtotal:		**$**
DIVISION 11 - EQUIPMENT								
	VERTICAL TURBINE PUMP (200 HP)			4	EA	$200,000.00	$70,000.00	$ 1,080,000
	Fish Intake Center Flow Screens (Interim)			4	EA	$300,000.00	$75,000.00	$ 1,500,000
	63-Inch Insert Style Magnetic Flow Meter			1	EA	$50,000.00	$12,500.00	$ 63,000
	84-Inch Insert Style Magnetic Flow Meter			1	EA	$50,000.00	$12,500.00	$ 63,000
	Miscellaneous	Allowance		1	LS	$100,000.00		$ 100,000
						Subtotal:		**$ 2,806,000**

DIVISION 12 - FURNISHINGS								
	Not Used							$
						Subtotal:		**$**
DIVISION 13 - SPECIAL CONSTRUCTION								
	Not Used			1	EA			$
						Subtotal:		**$**
DIVISION 14 - CONVEYING SYSTEMS								
	Not Used							
						Subtotal:		**$**
DIVISION 15 - MECHANICAL								
	84-Inch HDPE			150	FT	$2,100.00		$ 315,000
	84-Inch 90 - Degree HDPE Bend			1	EA	$6,300.00		$ 7,000
	84-Inch 45 - Degree HDPE Bend			2	EA	$4,200.00		$ 9,000
	72-Inch HDPE			40	FT	$1,800.00		$ 72,000
	48-inch pump discharge SST			40	LF	$4,800.00		$ 192,000
	48-inch 90 elbow SST			4	EA	$14,400.00		$ 58,000
	Miscellaneous Pipe Fittings			1	LS	$50,000.00	$15,000.00	$ 65,000
						Subtotal:		**$ 718,000**
DIVISION 16 - ELECTRICAL								
	MCC	Nema 12 Enclosure; 3-PH, 4 Wire, 65 KAIC, 2000A, 460VAC		4	EA	$40,000.00	$10,000.00	$ 200,000
	200 HP VFDs			4	EA	$40,000.00	$10,000.00	$ 200,000
	PLC Enclosure	Nema 12 Enclosure, Hoffman	90" H x 36" W x 20" D	1	LS	$20,000.00	$5,000.00	$ 25,000
	HVS			2	LS	$100,000.00	$25,000.00	$ 250,000
	Main Transformer 12K-480V			2	LS	$500,000.00	$125,000.00	$ 1,250,000
	INSTRUMENTATION WIRING			1	LS	$40,000.00	$10,000.00	$ 50,000
	Main and Tie			2	LS	$40,000.00	$10,000.00	$ 100,000
	Low Voltage Transformer			1	LS	$10,000.00	$2,500.00	$ 13,000
	Lighting Panel			1	LS	$10,000.00	$2,500.00	$ 13,000
	TELEPHONE/ DATA COMMUNICATIONS			1	LS	$30,000.00		$ 30,000
	Lights, Receptacles and Switches			1	LS	$30,000.00		$ 30,000
	Conduit and Wire			1	LS	$100,000.00		$ 100,000
	Modicon CPU	PLC Hardware		1	LS	$10,000.00		$ 10,000
	Backplane, 16 - slot	PLC Hardware		1	LS	$1,200.00		$ 2,000
	Power Supply	PLC Hardware		1	LS	$2,100.00		$ 3,000
	Ethernet Card	PLC Hardware		1	LS	$4,000.00		$ 4,000
	DI Cards, 32 Channel	PLC Hardware		2	EA	$2,000.00		$ 4,000
	DO Cards, 16 Channel	PLC Hardware		2	EA	$1,200.00		$ 3,000
	AI Cards, 8 Channel	PLC Hardware		3	EA	$3,000.00		$ 9,000
	AO Cards, 4 Channel	PLC Hardware		2	EA	$3,000.00		$ 6,000
	DI Block	PLC - Cable Fast Terminal Blocks		1	EA	$500.00		$ 1,000
	AI Block	PLC - Cable Fast Terminal Blocks		1	EA	$1,500.00		$ 2,000
	AO Block	PLC - Cable Fast Terminal Blocks		1	EA	$600.00		$ 1,000
	DI Cable	PLC - Cable Fast Cables		1	LS	$800.00		$ 1,000
	DO Cable	PLC - Cable Fast Cables		1	LS	$600.00		$ 1,000
	AI / AO Cable	PLC - Cable Fast Cables		1	LS	$2,000.00		$ 2,000
	PLC Software, UNITY			1	LS	$15,000.00		$ 15,000
						Subtotal:		**$ 2,325,000**
						CONSTRUCTION TOTAL:		**$ 12,476,000**
				Mobilization and Insurance (6.5% of Construction Total)				$ 811,000
				General Conditions (9% of Construction Total)				$ 1,123,000
				Contractor Profit (15% of Construction Total)				$ 1,872,000
				Engineering - Design Build (10% of Construction Total)				$ 1,248,000
						Subtotal 1		$ 17,530,000
						Contingency (25%)		$ 4,383,000
						Subtotal 2		$ 21,913,000
						TODAY"S VALUE TOTAL		**$ 21,913,000**

Poseidon Water - OPA Facilities Costs

Carlsbad Desalination Plant - Intake Facility

Preliminary Construction Cost Opinion, June, 2017

Section	Cost
DIVISION 0 - BIDDING REQUIREMENTS, ETC.	$ -
DIVISION 1 - GENERAL REQUIREMENTS	$ -
DIVISION 2 - SITE WORK *Includes: Demolition of existing, Coffer Dam, Dirt Hauling, Dewatering, and Dumping*	$ 1,379,000
DIVISION 3 - CONCRETE	$ 1,177,000
DIVISION 4 - MASONRY	$ 231,000
DIVISION 5 - METALS	$ -
DIVISION 6 - WOOD AND PLASTICS	$ -
DIVISION 7 - THERMAL & MOISTURE PROTECTION	$ -
DIVISION 8 - DOORS & WINDOWS	$ -
DIVISION 9 - FINISHES	$ -
DIVISION 10 - SPECIALTIES	$ -
DIVISION 11 - EQUIPMENT *Includes: Screens and Air Bursting System*	$ 6,138,000
DIVISION 12 - FURNISHINGS	$ -
DIVISION 13 - SPECIAL CONSTRUCTION	$ -
DIVISION 14 - CONVEYING SYSTEMS	$ -
DIVISION 15 - MECHANICAL *Includes: HDPE Pipe, Knife Gate Valves and Misc Pipe Fittings*	$ 2,183,000
DIVISION 16 - ELECTRICAL	$ -
Construction Subtotal	**$ 11,108,000**

Additional Cost Items

Mobilization, Bond, and Insurance (6.5 percent)	$ 723,000
General Conditions (9 percent) *Includes: Field Overhead, Construction Supervision, Field Engineering, Project Management, Safety, etc.*	$ 1,000,000
Contractor Profit (15 percent)	$ 1,667,000
Engineering - Design Build (10 percent)	$ 1,111,000
Contingency (25 percent)	$ 3,903,000
Additional Item Subtotal	**$ 8,404,000**
Today's Value Total	**$ 19,512,000**

Poseidon Water - OPA Facilities Costs
Carlsbad Desalination Plant - Intake Facility
Preliminary Construction Cost Opinion, June, 2017

SPEC SECTION		DESCRIPTION	SIZE	QUANTITY	UNITS	UNIT COST	INSTALLATION	TOTAL COST
DIVISION 0 - BIDDING REQUIREMENTS, ETC.								
	Not Used (Incorporated below)							$
						Subtotal:		**$**
DIVISION 1 - GENERAL REQUIREMENTS								
	Not Used (Incorporated below)							$
						Subtotal:		**$**
DIVISION 2 - SITE WORK								
	Demolition of Existing			1	LS	$100,000.00		$ 100,000
	Dredging and Disposal			8,333	CY	$20.00		$ 167,000
	Debris Booms with Hanging Curtain and Solar Lights			1,040	LF	$300.00		$ 312,000
	Barge			1	LS	$700,000.00		$ 700,000
	Tow Boat			1	LS	$100,000.00		$ 100,000
						Subtotal:		**$ 1,379,000**
DIVISION 3 - CONCRETE								
	Wall	Intake Structure		119	CY	$2,000.00		$ 238,000
	Tremie Concrete Fill	Intake Structure		59	CY	$1,200.00		$ 72,000
	Concrete Blocks for HDPE	50% of Buoyancy Weight of Pipe		578	CY	$1,500.00		$ 867,000
						Subtotal:		**$ 1,177,000**
DIVISION 4 - MASONRY								
	Compressor Building			384	SF	$600.00		$ 231,000
						Subtotal:		**$ 231,000**
DIVISION 5 - METALS								
	Not Used							$
						Subtotal:		**$**
DIVISION 6 - WOOD AND PLASTICS								
	Not Used							$
						Subtotal:		**$**
DIVISION 7 - THERMAL & MOISTURE PROTECTION								
	Not Used							$
						Subtotal:		**$**
DIVISION 8 - DOORS & WINDOWS								
	Not Used							$
						Subtotal:		**$**
DIVISION 9 -FINISHES								
	Not Used							$
						Subtotal:		**$**
DIVISION 10 - SPECIALTIES								
	Not Used							$
						Subtotal:		**$**
DIVISION 11 - EQUIPMENT								
	Fish Screens (Wedge Wire Z-Alloy) + Hydro Burst System			1	LS	$3,630,000.00	$907,500.00	$ 4,538,000
	Fish Screens (Wedge Wire Z-Alloy)	4 Spare Screens		4	EA	$200,000.00		$ 800,000
	10" SS Hydroburst Piping			1800	LF	$300.00	$75.00	$ 675,000
	Submersible Camera			1	LS	$25,000.00		$ 25,000
	Miscellaneous	Allowance		1	LS	$100,000.00		$ 100,000
						Subtotal:		**$ 6,138,000**
DIVISION 12 - FURNISHINGS								
	Not Used							$
						Subtotal:		**$**
DIVISION 13 - SPECIAL CONSTRUCTION								
	Not Used							$
						Subtotal:		**$**
DIVISION 14 - CONVEYING SYSTEMS								
	Not Used							

						Subtotal:		**$**
DIVISION 15 - MECHANICAL								
	63-Inch HDPE			3133	LF	$220.00	$119.15	$ 1,063,000
	60-Inch SDSS Header			467	LF	$1,500.00	$150.00	$ 770,000
	Compressed Air Piping			1800	LF	$100.00	$25.00	$ 225,000
	Miscellaneous Pipe Fittings Piping			1	LS	$100,000.00	$25,000.00	$ 125,000
						Subtotal:		**$ 2,183,000**
DIVISION 16 - ELECTRICAL								
	Not Used							$
						Subtotal:		**$**
						CONSTRUCTION TOTAL:		**$ 11,108,000**
						Mobilization and Insurance (6.5% of Construction Total)		$ 723,000
						General Conditions (9% of Construction Total)		$ 1,000,000
						Contractor Profit (15% of Construction Total)		$ 1,667,000
						Engineering - Design Build (10% of Construction Total)		$ 1,111,000
						Subtotal 1		$ 15,609,000
						Contingency (25%)		$ 3,903,000
						Subtotal 2		$ 19,512,000
						TODAY"S VALUE TOTAL		**$ 19,512,000**

Attachment F - Construction Schedule

Poseidon Water: CDP-Intake Structure

Alternative 21

Phase 1 Construction Schedule

Item	Description	No. Weeks	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
-	Total Length	95																								
1	Design Phase	32																								
2	Permitting	24																								
3	Mobilization	4																								
4	Utility Relocation	8																								
5	Demolition/Dewatering/Excavation/Shoring/Deep Excavation/Hauling/Disposal	16																								
6	Concrete Form/Pour/Cure	20																								
7	Backfill	2																								
8	Mech/Elec Install	16																								
9	Completion	3																								
10	Tie-in	6																								
11	Plant Shut Down	6																								

Phase 2 Construction Schedule

Item	Description	No. Weeks	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
-	Total Length	52																								
1	Design Phase	16																								
2	Design Specific Permitting	24																								
3	Mobilization	2																								
5	Dredging/Hauling/Disposal	4																								
6	HDPE Pipe and Screen Installation	6																								
7	Air Bursting Station	6																								
8	Demolition/Dewatering/Coffer Dam	4																								
9	Concrete Form/Pour/Cure	2																								
10	Completion	2																								
11	Tie-in	1																								
12	Plant Shut Down	2																								