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April 1, 2019

BY EMAIL

Hope Smythe
Executive Officer
Regional Water Quality Control Board, Santa Ana Region
3737 Main Street, Suite 500
Riverside, CA 92501

Re: Proposed Seawater Desalination Plant at Huntington Beach Alternative Intake Site Feasibility Analysis

Dear Ms. Smythe:

We write on behalf of Poseidon Resources (Surfside) LLC ("Poseidon") in regard to the Santa Ana Regional Water Quality Control Board's ("Regional Board") feasibility analysis under Water Code section 13142.5(b), and specifically the request for additional information related to the feasibility of alternative offshore sites for the proposed Huntington Beach Desalination Project's ("Project") 1-millimeter ("mm") wedgewire screened ocean intake.

In response to the Regional Board's recent request for information, Poseidon submits additional information in the enclosed Appendix RRRRR.[1] As set forth herein and in Poseidon's prior submittals to the Regional Board, the proposed 1-mm wedgewire screened seawater intake site (i.e., Site E) is the best available site feasible, and there is no evidence that the alternative sites identified and evaluated are environmentally superior to Site E. As such, the Regional Board has sufficient information to make its determination that the proposed Project including offshore intake Site E complies with Water Code section 13142.5(b) ("Section 13142.5(b)") and the Desalination Amendment to the California Ocean Plan ("OPA").

Regional Board staff's desire for additional analysis of the feasibility of alternative offshore seawater intake sites appears to be based on the assumption that "[a]vailable data do not support proposed intake site (station E) as the best location to minimize intake and mortality."[2] To the contrary, the evidence in the Regional Board's administrative record, including the Neutral Third-Party Review ("NTPR") reports commissioned by the Regional Board staff, confirm that neither alternative Site D2 nor Site U2 is environmentally superior to Site E.

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[1] In addition to the Regional Board's February 4, 2019 letter, Appendix RRRRR provides responses to the Regional Board's February 27, 2019 and March 11, 2019 correspondence.

[2] See Regional Board Staff's February 8, 2019, Informational Presentation to the Regional Board.

Section 13142.5(b) and the OPA require a finding that a proposed desalination facility uses the best available site, design, technology and mitigation measures "feasible" to minimize the intake and mortality of all forms of marine life. Although the Water Code does not define "feasible," the California Court of Appeal has held that "feasible" as used in Water Code section 13142.5(b) has the same meaning as "feasible" in the California Environmental Quality Act ("CEQA"). (See *Surfrider Foundation v. Regional Water Quality Control Board* (2012) 211 Cal.App.4th 557, 582.) In addition, in its 2015 update to the OPA, the State Board, consistent with Poseidon's own request at the time, amended the OPA to expressly include the same definition of "feasible" as used in CEQA and by the Court of Appeal. (OPA, p. 54.) Therefore, "feasible" in the Water Code and the OPA context means "capable of being accomplished in a successful manner within a reasonable period of time, taking into account economic, environmental, social, and technological factors." (*Ibid.*)

Surfrider also confirmed that the set of criteria described in the OPA—*i.e.*, "site, design, technology, and mitigation measures"—is to be read collectively, and no one criteria is more important than another. (See 211 Cal.App.4th at pp. 576-577.) In turn, Chapter III.M.2.a(2) of the OPA incorporates *Surfrider* and confirms that the Regional Board must not narrowly focus on the "best available site" inquiry to the exclusion of the OPA's remaining criteria.[3] Accordingly, it would be clear violation of the OPA and the Water Code for the Regional Board to focus exclusively on one factor, and fail to examine the combination of factors in making its final decision.

In this regard, not only was Site E previously analyzed under Section 13142.5(b) and permitted for use as part of the Project by the Regional Board in 2012,[4] but Site E also was extensively analyzed by the California State Lands Commission ("State Lands") in its 2017 Final Supplemental Environmental Impact Report ("FSEIR") for the Project.[5] Through the FSEIR

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[3] Pursuant to Chapter III.M.2.a(2) of the OPA: "The regional water board shall first analyze separately as independent considerations a range of feasible alternatives for the best available site, the best available design, the best available technology, and the best available mitigation measures to minimize intake and mortality of all forms of marine life. Then, the regional water board shall consider all four factors collectively and determine the best combination of feasible alternatives to minimize intake and mortality of all forms of marine life. The best combination of alternatives may not always include the best alternative under each individual factor because some alternatives may be mutually exclusive, redundant, or not feasible in combination."

[4] The Regional Board's 2012 NPDES permit and Water Code section 13142.5(b) compliance determination states: "Pursuant to Water Code Section 13142.5(b), the Regional Water Board finds that there are no better alternative and feasible sites available for the Facility and that the HBGS site is the best available site feasible to minimize intake and mortality to marine life during operations of the Facility." (See p. F-24.)

[5] The Interagency Permit Sequencing Framework Agreement provides the framework for how the Regional Board and State Lands will coordinate on environmental review for the Project. With respect to the Regional Board, the Agreement provides: "the Regional Board agrees that, except as otherwise required by CEQA, in developing its draft Tentative Order it can rely on the 2010 City of Huntington Beach certified Subsequent Environmental Impact Report (SEIR) in combination with CEQA analysis prepared and approved by the State Lands Commission [e.g., the FSEIR] in its evaluation of Poseidon's

process, State Lands evaluated not only potential impacts from the use of 1-mm screens, but also alternative sites of the intake and screens, as required by the OPA. The Sacramento Superior Court recently upheld the FSEIR, confirming that State Lands performed the requisite analysis. (See Latham & Watkins LLP January 16, 2019 Letter, Att. A at p. 14.) As a responsible agency under CEQA, the Regional Board is required to consider the analysis of the Commission's SEIR on the issue of alternative sites and other environmental consideration.

The selection of Site E as the best available site feasible for the Project represents the culmination of years of analysis by multiple agencies, including a six-month NTPR process initiated by the Regional Board, which considered multiple lines of evidence utilizing larval entrainment data that the Regional Board has been relying on to evaluate the Project since the NPDES permit was first approved in 2006. The Regional Board's administrative record concludes there is no statistical difference in entrainment among the Proposed Site E and alternatives Sites D2 and U2[6]. This determination is logically supported by the fact that all three intake sites are the same distance and depth from shore and located in relatively close proximity along the nearshore. All three sites encompass the same soft bottom, sandy habitat with no distinguishing habitat characteristics among the three sites. In these circumstances, the OPA should not be read to mandate assessing Sites E, D2, and U2 as different sites, given that the locations are within close proximity in a biologically homogenous area. The similarities in marine life sampling results at these locations demonstrate that they are better considered sub-points within one "site"—i.e., "the general onshore and offshore location" in and around Huntington Beach. (OPA, § III.M.2.b.)

Nevertheless, other factors compel exclusion of Sites U2 and D2 from further analysis. The implementation of an intake at Site U2 clearly conflicts with OPA directives, including that screened intakes "shall" be located a maximum distance "<u>feasible</u>" from marine protected areas. (OPA, § III.M.2.B ["To the extent feasible, surface intakes shall be sited so as to maximize the distance from a MPA or SWQPA."].) Because the intake site proposed by Poseidon is clearly "feasible," the Ocean Plan requires that any surface intake be sited in Poseidon's proposed location, rather than Site U2. Further, the implementation of an intake at Site D2 requires the intake to be sited adjacent to the mouth of the restored Huntington Beach wetlands, creating a high risk to sensitive species in the area. (Appx. JJJJJ-1 at p. 16.)[7] As such, there is no

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proposed seawater intake and discharge technology modifications for the purposes of complying with CEQA."

[6] Dr. Raimondi's report concludes that the proposed intake site E ranks best based on the Standardized Larval Concentration ("SLC") line of evidence and third when the Mean Larval Concentration ("MLC") and SLC are equally weighted and merged into one line of evidence. When statistical significance testing is applied, as recommended by Dr. Raimondi throughout the NTPR process, there is no ecological difference between stations E, D2, and U2 and therefore no scientifically justifiable rationale for re-locating the site of the proposed 1-mm screened seawater intake. Lacking any statistical differences among the three remaining sites, there is no scientific confidence that an intake at the alternative sites would result in less actual entrainment over the operational life of the HBDP. (Appendix JJJJJ-1 at p. 1.)

[7] Please see January 2019 report commissioned by California Coastkeeper Alliance raising concerns about potential entrainment impacts to species associated with the Huntington Beach Wetlands Complex.

scientifically justifiable rationale for re-locating the site of the proposed 1-mm screened seawater intake currently located at Site E. (See *id.* at p. iii.).

The Board's administrative record supports the finding that all three intake sites share the same habitat and do not have statistically different levels of entrainment. This scientific and statistical fact, standing alone, conclusively establishes that none of the three sites can be classified as the "best," to the exclusion of the other two, with respect to levels of entrainment. Sites or other alternative proposals that are scientifically and statistically the same in their performance cannot be excluded from the category of "best" with respect to a particular factor, such as site, under the Water Code and the OPA.

Nonetheless, Poseidon has conducted the alternative offshore 1-mm wedgewire intake feasibility analysis requested by Regional Board staff. This extensive analysis concludes that Site E represents the best available site feasible due to the technical, economic, environmental, and social constraints associated with Sites D2 and U2. (See Appx. JJJJJ-2 at pp. 15-16.) For instance, Sites D2 and U2 are technically infeasible because new industrial infrastructure would have to be located on the beach. (*Id.* at p. 15.) The location of infrastructure on the beach would, in turn, render the sites socially infeasible due to impacts to major tourist-generating events that occur on the beach and other beach closures. (*Id.* at p. 16.) Sites D2 and U2 are also economically infeasible because the increased construction would add at least $474 million to total Project costs. (*Id.* at p. 15.) Further, development and construction at these sites could take up to thirteen years, as opposed to 4.5 years for Site E. Thus, constructing a new intake system at Sites D2 or U2 cannot be successfully accomplished in a reasonable timeframe. (See *ibid.*)

Given that robust evidence and technical analyses in the record demonstrate that alternative Sites D2 and U2 would result in greater impacts—without any scientifically justifiable environmental or ecological benefit—than Site E, the Regional Board should find that Site E represents the "best available site feasible" under Section 13142.5(b). The Regional Board has all the information necessary to make its Section 13142.5(b) determination.

We appreciate the Regional Board staff's careful and continued consideration of Poseidon's application and look forward to the Regional Board's planned July 1, 2019 release of the tentative order.

Sincerely,

Christopher Garrett
by JKR

Christopher W. Garrett
of LATHAM & WATKINS LLP

cc: Teresita Sablan, Esq.
Scott Maloni, Poseidon Water

Enclosure: Appendix RRRRR

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Part I

Response to Regional Board's February 4, 2019 Request for Additional Information Regarding 1-mm Wedgewire screened ocean intake Alternative site Feasibility Assessment

Response to Regional Board's February 4, 2019
Request for additional Information

CEQA

Brine Diffuser

Request 1: *Additional analysis is needed for marine sediments that will need to be moved in advance of the linear cradle placement for the diffuser. The worst-case scenario (i.e., the possibility that all excess sediment will not be side-cast) must be analyzed to address the incremental increase in air emission impacts that may result if excess sediment needs to be transported offsite for disposal.*

Water Boards staff requests that Poseidon include the worst-case scenario approach and provide additional analysis to address the potential impacts associated with the towing and onshore disposal of the 200-300 cubic yards of sediment from "leveling" activities. The analysis should include a discussion of air quality impacts from increased emissions, anchoring impacts for additional vessels in the diffuser sediment-leveling area, and greenhouse gas impacts for additional onshore and offshore vehicle movement. Additionally, Appendix BBBBB-2 analyzes only one additional day of air emissions, which does not address the possibility of onshore sediment disposal occurring at a different time from riprap transport in the air emissions analysis. This possibility should be included in the air emissions analysis. Water Boards staff also requests clarification as to whether the disposal of sediments could occur simultaneously with the last day of dredging and wedgewire screen construction (the scenario for maximum daily construction emission) and provide any associated data analysis.

Additional information is also needed for the statement that the 2017 Supplemental EIR "overestimated" the amount of side-casted materials. In response to State Lands Commission's comment regarding calculations for change in emissions, the response memo states that "the amount of excavated and side-casted materials [1,000 to 3,300 cubic yards] required for the installation of the wedgewire screen were overestimated in the 2017 Supplemental EIR". Water Boards staff requests that Poseidon provides the additional information that supports the statement that the amounts were overestimated in the 2017 Supplemental EIR or adjust the emission calculations to include the additional emissions for the derrick barge.

Response 1: It should be noted that it is unlikely that the sediment from leveling activities would be disposed of on land, because it would result in schedule and cost implications when compared to side casting. The 2017 Final Supplemental EIR (FSEIR) analyzed a scenario in which the excess material would be loaded on support barges and taken to the POLB for disposal in an appropriate land-based facility. As stated in Section 2.4.5.4, Wedgewire Screen Installation, on page 2-28 of the FSEIR:

Dredging would occur to the level of the header invert (10 days dredging), and pile foundations would be driven for the screens (requiring about 3 days for the pile driving). Approximately 1,000 to as much as 3,300 cubic yards of soils would be excavated, depending on the final design phase geotechnical confirmation, and side-cast near the construction area, where they would have highly similar sediment composition to those in the side-cast footprint. Suspended sediments excavated would likely be redistributed by ocean currents. If excess material cannot be re-used or relocated on the ocean floor, it will be loaded on support barges and taken to the POLB for disposal in an appropriate land-based facility.

As a primary matter, the FSEIR does evaluate a worst-case scenario. As stated above in the FSEIR, this small amount of sediments *"will be side-cast near the construction area, where they would have highly similar sediment composition to those in the side-cast footprint."* Onshore sediment disposal is considered unlikely for up to 3,300 cubic yards and will only be necessary if the sediments cannot be re-used or re-located. As such, this represents the worst-case scenario analyzed in the FSEIR. The 1,000 – 3,300 cubic yards of seafloor sediments is intentionally broad and also reflects the conservative, worst-case excavation scenario.

Therefore, the criteria air pollutant and greenhouse gas (GHG) emissions associated with this activity were already captured in the 2017 FSEIR . Although the excavation described above is associated with the installation of the WWS, and not the diffuser, the 2017 FSEIR provided conservative (i.e., high), worst-case estimates of seafloor excavation, which would capture sediment removal associated with the new linear diffuser installation. The criteria air pollutant and GHG emission calculations are provided below.

The disposal of sediment on land would require haul truck and marine vessel trips to deliver this material. The construction emissions from worker trips, haul truck trips, construction equipment use, and marine vessel transportation are anticipated to be the same as analyzed for the Lease Modification Project diffuser in the 2017 FSEIR. To accommodate 300 cubic yards of sediment export, one tug boat and barge roundtrip would be required. A typical barge has a capacity of 1,750 tons. Assuming that the sediment has a 100% water saturation rate, sediment could have a density of 83.4 pounds per cubic foot or 1.13 tons per cubic yard.[1] Therefore, 300 cubic yards of sediment would be approximately 339 tons, which would require one tug boat and barge roundtrip. Based on default values for haul truck capacity in the California Emissions Estimator Model (CalEEMod) Version 2016.3.2, transporting the sediment to the Port of Long Beach would result in approximately 20 round trips by haul trucks (see Attachment A for calculation details). Per Request 1, it was assumed that sediment removal and transport would occur on a separate day than riprap transport.

To provide the emissions associated with one tug boat round trip and haul truck trips during construction of the new linear diffuser, information from the same marine vessel emission calculation methodology was used (*Emission Estimation Methodology for*

[1] https://clu-in.org/download/contaminantfocus/sediments/sediment-characterization-doern13.pdf

Commercial Harbor Craft Operating in California [CARB 2004]). Similarly, the same assumptions for tug boat engine characteristics, fuel sulfur content (0.0015% or 15 parts per million [ppm] sulfur diesel fuel to comply with the Harbor Craft Fuel Regulation [CARB 2008]), speed (following the Santa Barbara Channel Vessel Speed Reduction Trial [SBCAPD 2014]), and travel distance used in the 2017 FSEIR were used in this analysis. The maximum daily emissions for construction of the Lease Modification Project were identified to occur on the last day of dredging in the 2017 FSEIR. The construction emissions for this day are shown in Table 1, Estimated Daily Maximum Construction Emissions – New Linear Diffuser Sediment Removal, and are provided with the maximum daily construction emissions in the 2017 FSEIR.

Table 1
Estimated Daily Maximum Construction Emissions – New Linear Diffuser Sediment Removal (pounds/day unmitigated)

	VOC	NOx	CO	SOx	PM_{10}	$PM_{2.5}$
Emissions During One Day of the New Diffuser Installation with Sediment Removal[1]	1.81	15.39	9.17	0.03	0.76	0.44
Typical Daily Construction Emissions from 2017 FSEIR [2]	*3.22*	*19.70*	*15.14*	*0.02*	*1.26*	*0.80*
2017 FSEIR – Lease Modification Project [3]	*6.71*	*73.85*	*34.52*	*0.15*	*4.26*	*2.13*
Threshold	75	100	550	150	150	55
Threshold or 2017 FSEIR Emissions exceeded?	No	No	No	No	No	No

Source: Attachment A.
Notes: [1] Assumes one tug boat and barge roundtrip from the Port of Long Beach and 20 haul truck roundtrips. A typical barge has a capacity of 1,750 tons (U.S. ACOE 2018). Assuming that the sediment has a 100% water saturation rate, sediment could have a density of 83.4 pounds per cubic foot or 1.13 tons per cubic yard. Therefore, 300 cubic yards of sediment would be approximately 339 tons. Therefore, the sediment removal would be accommodated by one round trip from the Port of Long Beach by a tug boat and barge.
[2] Shows emissions analyzed for a normal work day during diffuser and wedgewire screen intake installation in the 2017 FSEIR.
[3] Maximum daily construction emissions occurred on the last day of the dredging phase of construction of the Lease Modification Project in the 2017 FSEIR.

As indicated in Table 1, the tug boat and haul truck trips associated with construction of the new linear diffuser, including simultaneous construction of the wedgewire screen (WWS) intake, would not exceed those of the Lease Modification Project analyzed in the 2017 FSEIR and would not exceed any of the SCAQMD construction thresholds, because these emissions were captured in the 2017 FSEIR. As such, impacts would be less than significant.

The GHG emissions from the disposal of sediment on land are shown in Table 2, Estimated Annual Construction GHG Emissions – New Linear Diffuser Sediment Removal. This estimate assumes simultaneous construction of the WWS intake and disposal of sediment on land as a worst case scenario; the same as was assumed in the 2017 FSEIR.

Table 2
Estimated Annual Construction GHG Emissions – New Linear Diffuser Sediment Removal

Modifications	MT CO_2E	Amortized Annual Emissions[1]
2017 FSEIR – Lease Modification Project [2]	71.64	1.43
Linear Diffuser Sediment Removal Emissions [3]	3.53	0.07

Source: Attachment A.
Notes: MT = metric tons; CO_2E = carbon dioxide equivalent
[1] Emissions amortized over the 50 year project life time.
[2] Includes emissions from simultaneous construction of the WWS intake from the Lease Modification Project analyzed in the 2017 FSEIR.
[3] Assumes one tug boat round trip and 20 haul truck round trips during one construction day for new linear diffuser sediment removal.

Construction-related GHG emissions would not represent a long-term source of GHG emissions. Additionally, as stated in the 2017 FSEIR the GHG Plan requires the offset of 100% of the construction-related GHG emissions. Therefore, the sediment removal associated new linear diffuser's GHG emissions would remain below SCAQMD's 10,000 MT CO2E per year threshold, would be offset to net zero GHG emissions, and would not conflict with any applicable GHG regulations. Therefore, impacts would be less than significant.

Finally, as clarification to the statement that the 2017 FSEIR overestimated the amount of side casted materials, a more accurate statement would be that the FSEIR included highly conservative assumptions, primarily that a 12-foot-deep trench would be required for the installation of the WWS. Although the 12-foot-deep trench is associated with the installation of the WWS, and not the diffuser, the 2017 FSEIR provided conservative (i.e., high), worst-case estimates of seafloor excavation, which would capture sediment removal associated with the new linear diffuser installation. Poseidon's Engineering, Procurement & Construction ("EPC") team believes that as the design is optimized as part of the final design the depth of the trench can be reduced to reduce the dredging quantity which would remain well within the overall envelope of impact analysis provided in the FSEIR. Poseidon expects the criteria air pollutant and GHG emissions will not exceed those analyzed in the SLC FSEIR.

1-mm Screen Intake Site Feasibility

Timing

Request 2: *Appendix JJJJJ-2 estimates that permitting for an intake at station U2 or D2 would take 5 years and that permitting for station E would take 15 months. Given that a Section 404 permit, Coastal Development Permit, SLC lease, and easement for state beach impacts need to be obtained for station E. Based on the estimate for the time needed for a Section 404 permit and under the assumption that the state permitting process would run concurrently, it is unclear why the estimate for station E is not also 5 years, and further, how the 15-month estimate for station E was reached. Water Boards*

staff requests that Poseidon clarify why a different estimate for permitting time was calculated for station E.

Response 2: The justification for the proposed project's 15-month schedule is based on the Interagency Permit Sequencing Framework Agreement, which is included as Appendix B to the FSEIR. Based on the terms of that Agreement, Poseidon anticipates completion of RWQCB and CCC permitting processes before the end of 2019. Subsequent actions in accordance with the Agreement should reach completion milestones in mid-2020. As the previous analysis notes, processing of a Nationwide Permit by the Army Corps is anticipated to run concurrent with those approvals, hence a 15-month time frame for Army Corps permit completion.

As previously noted, due to the limited impacts on waters of the U.S., it is anticipated that modifications at Intake station E would fall under a Nationwide Permit process. Conversely, construction of a new seawater intake system at either of the alternative sites would be the first to be installed along the coast of California since the intake for the Santa Barbara Desalination Plant and would involve substantially greater activities within Waters of the U.S. An individual permit would likely be pursued by the Army Corps, and thus is the basis for the "worst-case" processing schedule assumption for those options.

Economic

Request 3: *Water Boards staff requests clarity regarding the estimate for construction costs for station E, and if/how these costs were deducted from the $1 billion estimate for total project costs. Additionally, Water Board staff requests that a more detailed breakdown of cost estimates for stations E, U2, and D2 be provided. For stations D2, E, and U2 please provide the following: a. Details of design and construction cost estimates, including but not limited to design/sizing calculations of the intake structure and pipeline, pump station, and pipeline connecting to the Project. Please include assumptions, hydraulic profile, and cite sources of information for the design and cost estimate calculations. Water Boards staff also requests the revised report include any and all equations and data used for the calculation details for the parameters mentioned above, as well as the discharge pipeline b. Calculations to demonstrate impact of additional cost for an intake at station D2 or U2 on the cost of water to Orange County Water District. The cost needs to be compared to the cost of water at the proposed intake.*

Response 3: Please see the attached cost comparison table between the Proposed Project intake site (i.e., site E) and the alternative intake sites D2 and U2 (Attachment B).

The estimated cost of the proposed Project is based on fixed-price offers provided to Poseidon during the Engineering, Procurement & Construction ("EPC") and Operations & Maintenance ("O&M") international competitive selection process. The Huntington Beach Project's Preferred EPC Team includes Kiewit and Acciona and the pricing provided by the EPC Team was consistent the level of detail required for an owner to

select a contractor based on fixed-price bids. The pricing was based on engineering drawings and process equipment selection developed by the EPC Team during the preparation of the fixed-price offer. The cost estimates to move the WWS intake to alternatives sites D2 or U2 are based on a conceptual design level of detail to determine the feasibility of an alternate intake alignment. At the conceptual design level, higher contingencies are warranted until the engineering is advanced to a higher level of detail.

The estimated $17M cost of the WWS initially reported in Appendix JJJJJ-2 was a 2017 cost estimate and has been updated to 2018 dollars in Attachment B to $17.6M. The cost of the WWS is incorporated into the estimated $1 billion total cost for the proposed Project and is the same WWS estimated cost for the three intake sites being evaluated.

For alternative intake sites D2 and U2, the initial details of the design and $215 million construction cost estimates can be found in Attachment 2 to the Highlands Construction memo incorporated in Appendix JJJJJ-2. Based on the February 4, 2019 comments from RWB staff, Highlands Construction has revised the construction costs estimates from $215 million to $197 million based on a partial reduction in the alternative intake sites pipeline diameter from 14 to 12 feet. The support for the 12-foot pipeline diameter and cost estimates can be found in Response 12 of this document and within Attachments B and F.

To derive the Total Project Cost estimate Poseidon included additional costs necessary to deliver the project on a project finance basis, including:

- Additional development costs including costs for permitting, construction management and administration;
- Additional costs for property taxes and title insurance;
- Capitalized Interest for the respective construction period (72 months for D2 and U2) including a 6-month contingency, and;
- Reserves and financing fees to project finance the project with an investment grade credit rating structure

The enclosed alternative intake site cost comparison table (Attachment B) concludes that the total cost (i.e., financing and direct and indirect costs) to build the WWS intake at the proposed site is $93 million. In comparison, constructing the WWS at either alternative site D2 or U2 is estimated to cost $474 million, over $380 million more or an increase of approximately 410%. As such, moving the WWS intake to either D2 or U2 will increase the unit cost of water by over $600 per acre foot, an increase of 33%.

The cost information that was prepared in 2017 was based on connecting a new intake pump station into existing Power Generating Station infrastructure including the existing intake pump station wet well, the existing intake tunnel, the existing 14 foot diameter concrete pipeline and the existing concrete intake tower located approximately 2,400 ft west of the Power Generating Station forebay. Please refer to Attachment C which shows the details for the new intake pump station and Attachment D which shows the details of the existing Power Generating Station intake infrastructure. The cost information from 2017 did not include modifications to the existing infrastructure as previous inspections and condition assessments have shown that the infrastructure is in

a good state of repair and modifications are unnecessary. The cost for the new intake pump station as shown in Attachment C would be the same for either location E or locations D2/E2 and this cost is part of the 2017 cost information provided by the EPC Team. For locations D2/U2, the new cost estimate provided with this response assumes that a new intake line would be connected to the existing 14 foot diameter concrete intake pipe and that a new pipe would be installed to a new concrete tower of a similar size to the existing concrete intake tower. The cost to install the WWS assembly is considered the same for either location E or location D2/U2. The cost to install the discharge diffuser is the same for either location E or location D2/U2 and does not factor into the selection process for the intake location.

The hydraulic profile for the desalination plant is provided in Attachment E. The assumptions used in calculating head loss for the new intake pipeline at locations D2/U2 are included in the information provided in the response to Question 12. These assumptions include allowable head loss at the new intake pump station, assumptions for marine growth and assumptions for pipe roughness coefficients used in standard engineering calculation for head loss in pipe.

Environmental

Request 4: *Given that the Project will have negative effects on air emissions, as identified in the 2017 Supplemental EIR, it is unclear how an intake at station U2 or D2 would have significantly different emissions than the proposed intake. Water Boards staff requests clarification on how, or if, an intake at station U2 or D2 would result in significantly higher air emissions.*

Response 4: As clarification, the Feasibility Report does not state that emissions are "significantly different" – it merely state that NOx emissions "would contribute further to the exceedance of this SCAQMD construction threshold", which is a factual statement supported by the analysis provided in the Feasibility Report.

Request 5: *Station E is the closest station to the currently operating ocean intake for the Huntington Beach Generating Station. Accordingly, station E likely has already been experiencing entrainment effects from the existing intake. Water Boards staff requests that Poseidon provide information on how the currently operating intake may have affected larval concentrations at station E, in comparison to stations U2 and D2.*

Response 5: Sampling at Station E was outside the hydraulic zone of influence of the HBGS intake for the safety of the sampling equipment, vessel, crew, and the power plant intake itself (See RWB application Appendix A4). If sampling occurred within the zone of influence, the potential for the net to be drawn into the intake structure would be too high and potentially result in the loss of the sampling equipment, damage to the sampling vessel, injury to the sampling crew, damage to the intake structure, or any combination of these. For these reasons, the operating intake likely had no impact on the larval concentrations calculated from the sampling effort.

Furthermore, as noted by T. Reeder (Senior Engineering Geologist, Santa Ana Regional Water Quality Control Board) during a January 31, 2019 conference call with Poseidon and again during the February 8, 2019 informational presentation before the Santa Ana Regional Water Quality Control Board, ambient currents from up-coast of the proposed project area would induce larval dispersal equally to each of the three candidate intake sites. It has been Poseidon's position that this concept of equal larval dispersal amongst the three candidate intake sites applies equally to larvae sourced from any coastal habitat located up-current of the project area. This is further evidenced by the statistical analyses, and the statistical difference between the candidate sites at the standard scientific significance level of $p = 0.05$. Please refer to Part II (Response to the Regional Board's February 27th Additional Comments regarding Feasibility of an alternative site for 1-mm wedge wire screened ocean intake), Response 3.

Request 6: *It is unclear why the mouth of the Santa Ana River and the hard/rocky substrate associated with Huntington Beach Pier and Orange County Sanitation District's riprap armoring on the outfalls structures are designated as sensitive habitat in Appendix JJJJJ-1. Water Boards staff requests that Poseidon provide scientific analysis or information to support classifying these areas as sensitive habitat.*

Response 6: By virtue of the tidal wedge, the Santa Ana River mouth facilitates the mixing of fresh and ocean waters, consistent with the estuary definition in the Water Quality Control Plan for Inland Surface Waters, Enclosed Bays, and Estuaries of California. The Huntington Beach Pier and Orange County Sanitation District outfall riprap represent nearly all of the high-relief, hard substrate within the area. Reefs, natural and artificial, are characterized by their relief or the extent their hard structures rise from the seafloor to create complex habitat. Specifically, "*the wastewater discharge structure (outfall) represents one of the largest artificial reefs along this coastal region and supports communities typical of hard substrates*" (OCSD 2000). Estuaries and rocky reefs (artificial or natural) are uniformly considered estuarine habitat due to their comparatively higher biological productivity and scarcity than open-coast, soft bottom habitat.

Request 7: *In Table 2 of Appendix JJJJJ-1, there are several species that are not listed in Table 1 (e.g., Cancer oregonsis). Water Boards staff requests that additional information be provided to* clarify why *certain species from Table 1 are not in Table 2.*

Response 7: Table 1 of Appendix JJJJJ-1 was captioned to indicate that only the 25 most common taxa caught amongst the seven sampling stations were included. Table 2 of Appendix JJJJJ-1 represented only those taxa captured amongst the three candidate sites (U2, E, and D2). Therefore, taxa that did not appear in Table 1 that were in Table 2, such as *Cancer oregonsis*, were not among the 25 most abundant taxa caught amongst the seven sampling stations. Likewise, taxa included in Table 1 that were not listed in Table 2 did not occur at one of the three candidate sites (U2, E, or D2) or were not identified, e.g. larval fish fragment or larvae, unidentified yolksac. Unidentified larvae categories were not informative for the analysis presented in Table 2 as they could have

included any and all of the rest of the identified taxa. *Chromis punctipinnis* was one identified taxon listed in Table 1 and not in Table 2 because all collections of this taxon occurred at Stations U4, O2, and O4 only.

Request 8: *Appendix JJJJJ-1 describes how nursery grounds for Giant Sea Bass (Stereolepis gigas) have been identified near the head of the submarine canyon off Newport Beach and how, out of the three intake locations, station D2 is in closest proximity to the nursery grounds. Young-of-the Year (YOY) Giant Sea Bass have been found only on the sandy soft bottom areas located within 500 meters of the mouth of submarine canyons; station D2 is located approximately 4000m NW of the mouth of the canyon (Benseman and Allen, 2018). Larval settlement (from planktonic phase) takes place when Giant Sea Bass YOY are 10-21 mm long in total length (Benseman and Allen, 2018). Individuals of this size would be large enough to avoid entrainment through a surface intake using 1.0 mm wedgewire screens and likely would not be subject to impingement if the intake velocity is 0.5 feet/second. In addition, the growth rate for YOY GSB is 1.23 mm/d (Benseman and Allen, 2018), further supporting the case that they are not likely to be vulnerable to entrainment. Based on the information above, and the analyses provided in Appendix JJJJJ-1, Water Boards staff does not agree that station D2 will have a negative effect on GSB. Water Boards staff requests clarification as to how an intake at station D2 would result in higher intake and mortality of GSB than intakes at stations U2 or E.*

Response 8: Section III.M.2.b.(3) states: Analyze the feasibility of placing intake, discharge, and other facility infrastructure in a location that avoid impacts to sensitive habitats and sensitive species. Section III.M.2.b. (4) states: *Analyze the direct and indirect effects on all forms of marine life* resulting from facility construction and operation, individually and in combination with potential anthropogenic effects on all forms of marine life.* Section III.M.2.b. (4)

Appendix JJJJJ-1 did not state that an intake at potential site D2 would entrain Giant Sea Bass, only that of the three sites under consideration, site D2 posed the greatest potential (based on distance) risk to Giant Sea Bass. While we agree the size of Giant Sea Bass larvae nearing the end of their planktonic phase would generally be larger than the 1.0-mm slot width of the wedgwire screens, the Ocean Plan Amendment does not provide any consideration of the entrainment minimizing effects of the 1-mm WWS. In fact, the OPA expressly provides only a 1% “credit” (i.e., reduction on the APF calculation) for the use of 1.0-mm slot width WWS. This constraint was noted in the Los Angeles Regional Water Quality Control Board’s June 25, 2018 comment letter on the West Basin Municipal Water District’s Ocean Water Desalination Project drafts Environmental Impact Report as follows:

> *In 2013, the Final Report by the Expert Panel Ill on Intake Impacts and Mitigation concluded that, while screens on surface intakes can be effective for reducing entrainment of larger larvae, when all life stages are considered, screens reduce entrainment mortality by less than one percent. Chapter III.M.2.e.(1)(a) of the Ocean Plan allows for the regional water board to apply a one percent reduction to*

the APF acreage calculated for seawater desalination projects to account for the reduction in entrainment of all forms of marine life when using a 1.0 mm slot size screen on a surface intake. The DEIR should be revised to state that the Ocean Plan does not allow for additional reductions in APF acreage to credit for use of a 1.0 mm slot size screen. Changing this amount of credit would require amending the Ocean Plan. For additional explanation of the one percent credit provided by the Ocean Plan, please see pages H-423-426 in Appendix H of the Final Staff Report to the Desalination Amendment to the Ocean Plan.

In this regard, we do not agree with Staff's February 4, 2019 comment that moving the intake to station D2 will "not have a negative effect" on Giant Sea Bass in comparison to sites E and U2. At a minimum entrainment of plankton at alternative site D2 would potentially reduce the available forage needed by the recently settled Giant Sea Bass.

Request 9: *Appendix JJJJJ-1 notes that station D2 is closest to the mouth of the Huntington Beach wetlands. However, because of the Southern California countercurrent, it seems just as likely that larvae coming from those wetlands would end up at an intake at station E as D2, depending on tides, weather, and currents. In addition, Appendix JJJJJ-1 also states the station U2 is the closest of the three stations to a Marine Protected Area (MPA), the Bolsa Chica Ecological Reserve. While station U2 is located 3.1 miles from the Bolsa Chica MPA, it is located downcurrent from the MPA because of the Southern California countercurrent, which runs northwest along the shoreline of much of the Southern California Bight. Water Boards staff requests that Poseidon provide detailed information on how the Southern California countercurrent would affect entrainment of larvae from the Huntington Beach wetlands intakes at stations U2, E, and D2.*

Response 9: Due to its location, larvae from Huntington Beach Wetlands (HBW) would be more susceptible to entrainment at site D2 as the more concentrated plume of larvae from the HBW passes over site D2 when currents are from either direction.

The potential transport of larvae from the Bolsa Chica MPA or the Huntington Beach wetlands would not be affected by the California Countercurrent (CCC) as it is not the dominant factor in alongshore transport over the inner shelf (water depth < 20 m). The CCC is a broader scale current on the mid-outer shelf and is driven by large scale pressure gradient forces. Over the inner shelf, currents are driven mainly by highly variable local winds. This decoupling is described in Rasmussen, L. 2018. Technical Memorandum: San Pedro Bay Nearshore and Shelf Currents (Attachment 4 of Appendix JJJJJ-1). Rasmussen (2018) Figures 4 and 7 illustrate the differing physical forcing and resulting currents.

The mean flow over the inner shelf is to the ESE, from Bolsa Chica toward the Huntington Beach wetlands (Figure 8, Attachment 4 of JJJJJ-1). At typical alongshore current velocities of 10-20 cm/sec (in either direction), larvae would be transported between either of the larval sources and any of the proposed intakes in a day or less. Therefore, the prospect of transport of viable larvae would be more or less equal

regardless of intake location. The typical range of current velocities in this area can be seen in Figure 6 and 8 in Attachment 4 of JJJJJ-1. At 10-20 cm/sec (0.10 - 0.20 m/sec), the transport rate would be 0.36 - 0.72 km/hour. The main difference for larval entrainment between sites would be the dispersal of larvae after leaving the source. Significant (and unpredictable) diffusion of larvae would have taken place at all sites except D2 for larvae originating at HBW. D2 could also be expected to receive larvae from HBW with currents from either direction, at any speed.

Furthermore, the Regional Board's evaluation of the project is bound by the requirements in the OPA. OPA Section III.M.2.b. (7) states, "*Ensure that the intake and discharge structures are not located within a MPA or SWQPA …* to the extent feasible, surface intakes shall be sited so as to maximize the distance from a MPA or SWQPA." Merriam-Webster defines distance as *"the degree or amount of separation between two points, lines, surfaces, or objects."* The word "shall" is a directive. Siting the 1-mm screened intake at stations D2 or U2 does not comply with the OPA when another feasible site (i.e., station E) is available.

Social

Request 10: *Appendix JJJJJ-2 concludes that it is socially infeasible to construct an intake at stations U2 or D2 due to impacts on recreation and access. However, no information is provided on whether construction of an intake at station E would impact recreation and access. Water Boards staff requests that Poseidon provide information on what impacts to beach access, beach usage, and annual beach events would result from construction of an intake at station E.*

Response 10: The description of the construction means and methods, timeframes and impacts of constructing the 1-mm wedgewire screen are addressed in the State Lands Commission's 2017 FSEIR. Offshore installation of the 1-mm WWS at the proposed intake site is expected to last 3 months and would not require any staging or impacts to the beach. Please see Part III (Response to the Regional Board's March 11th Additional Comments regarding Feasibility of an alternative site for 1-mm wedge wire screened ocean intake), Response 9, for an expanded discussion.

Technological

Request 11: *Appendix JJJJJ-2 is inconsistent in a number of ways with the similar consideration of offshore construction issues raised and addressed during the Independent Science and Technical Advisory Panel (ISTAP) when it reviewed the feasibility of an offshore subsurface intake gallery (SIG). The ISTAP looked at two construction methods (i.e., trestle and float-in) to construct a much larger SIG just a little farther offshore than where Poseidon's proposed intake pipes would be located. The ISTAP found that it would be technically feasible to construct the SIG using either construction method. The SIG involved a slightly deeper site (i.e., 42 feet below the water surface versus 30 feet for the single intake pipe), though the SIG site was subject to similar continuous long-period ocean swells as Poseidon has described for the intake*

pipe locations. Water Boards staff requests that Poseidon address the feasibility of using a modified version of the ISTAP's float-in method to install the intake pipe, or perhaps a combined version of the float-in and trestle methods, in which a trestle structure/construction platform could be built at the offshore end of the existing intake pipe to be used as a staging location for the offshore installation.

Response 11: The construction considerations in Appendix JJJJJ-2 are consistent with, and informed by, the conclusions reached by the Coastal Commission's Independent Scientific & Technical Advisory Panel ("ISTAP"). In fact, the specific question regarding the feasibility of moving the seafloor infiltration gallery ("SIG") closer to shore and utilizing the "float-in" construction method was addressed by the ISTAP in 2015 (see Phase 2 report on page 29).

On June 23, 2015, ISTAP Panel member Bob Bittner, who's firm specializes in the design of innovative marine structures and who suggested the "float-in" construction approach to the Phase 2 Panel, provided the following response to the Coastal Commission staff's inquiry about the feasibility of moving the SIG location closer to shore: *"The cost-savings would not be significant and actually for the float-in option, I feel that putting it closer to shore could actually increase the cost because as you move closer into shore the waves start to drag on the bottom and they get higher which causes problems for the construction."*

Poseidon's construction contractor's conclusions are aligned with the ISTAP; the float-in construction method is not technically or economically feasible for the alternative 1-mm screened intake sites and such a project could not be successfully completed in a reasonable period of time. Please refer to Part II (Response to the Regional Board's February 27th Additional Comments regarding Feasibility of an alternative site for 1-mm wedge wire screened ocean intake), Response 5, for an expanded discussion.

Request 12: *Appendix JJJJJ-2 evaluated installing a 14-foot (168-inch) diameter intake pipe. This appears to result in overstated construction impacts and constructability concerns, as it appears the pipe could be substantially smaller than described. Research by Water Boards staff indicates that 8-8.5 foot-diameter pipelines have been used elsewhere for intake structures at desalination facilities that require similar or greater volumes of seawater than the proposed Project. Additionally, Poseidon's proposed intake design for the Carlsbad Desalination Project includes intake screens that are sized and designed to meet the 0.5 feet/second through-screen velocity requirement, although the water velocities inside the pipes connecting those screens to the facility would be just over 7 feet/second. If the same sizing formula were applied to the Project, a pipe pulling 106 MGD at -7 feet/second would be significantly smaller (i.e. , about 64 inches in diameter instead of 168 inches). Referring again to the above-referenced ISTAP report, Water Boards staff notes that installing the SIG would have involved placing a series of 54-inch diameter pipes perpendicular to the primary wave direction, which would be similar to Poseidon's placement of a 64-inch intake pipe. Appendix JJJJJ-2 additionally states that its intake pipe would need to be 10-14 feet in diameter due to net positive suction head requirements and to take into account future*

marine growth. Water Boards staff proposes to work with Poseidon to add additional inches to the diameter, if needed, to account for reduction in the diameter due to biofouling within the pipe, using industry standards for this type of suction pipeline in a marine environment. None of these concerns appears to have prevented Poseidon from proposing the smaller intake pipes at the Carlsbad Desalination Project. In fact, the smaller diameter pipes at that facility (and presumably at Huntington Beach) could be cleaned through pigging, which is not a viable method for a 14-foot pipe.
Water Boards staff request that Poseidon provide the following with respect to the intake pipes:
• The minimum suction pipe diameter needed.

Response 12: Poseidon has reviewed the sizing of the pipeline that would be required for the offshore option (from location E to new location D2 or U2) and has been able to refine the pipe sizing based on the calculations. In determining the size of the pipe, the total head loss was calculated for the entire piping system (intake screens to the new intake pump station located at the Plant site) for the full range of tide levels. The minimum net positive suction head (NPSH) for the intake pumps were compared to the total head loss calculated for the intake system assuming 6 inches of marine growth in the large diameter piping. For the offshore alternative which would connect the existing concrete tower (location E) to the new location D2 (or U2), a 12 foot diameter pipe would be required. The calculations assumed that a concrete pipe was used for pipe internal pipe dimensions and the Hazen-Williams coefficient. For the onshore alignment that would tie-in to the existing 14 foot diameter intake pipe on the beach, the required pipe diameter would be as follows:

a. Onshore portion – 5719 ft, 14 ft diameter (6 inches of marine growth)
b. Offshore portion – 2598 ft, 12 ft diameter (6 inches of marine growth)

A summary of the calculations and justification for the 12-foot minimum pipeline diameter is provided in Attachment F.

The reference to the intake piping size at the Carlsbad Desalination Plant is unclear. If the comment refers to the future intake system to be placed in the Agua Hedionda Lagoon, the intake system design is still at the conceptual phase and the piping diameter has not been finalized. However, based on the engineering evaluation performed to date on the new intake system, the diameter of the intake piping is expected to range between 80 and 90 inches depending on further evaluations on the expected thickness of marine growth. For pipe diameters up to 2.5 meters (98 inches), a piping solution incorporating high-density polyethylene (HDPE) is possible, and HDPE is planned for Carlsbad. For the Huntington Beach evaluation, the required piping diameter exceeds the maximum size available for HDPE. Concrete piping was therefore considered for Huntington Beach. In addition, the Carlsbad Desalination Plant proposed intake system has provisions for frequent pigging with the ability to control turbidity caused by the pigging operation by directing the wash water into in an engineered discharge pond. For the alternate piping arrangement for an intake location at D2 (or U2), the added complexities required for launching and/or retrieving pigs for large diameter pipe in the open ocean are unknown and therefore have not been

considered in the engineering or cost evaluation. Such complexities would include sizing of the concrete vaults required for launching and receiving the pig, provisions for turbidity control in the open ocean and pumping requirements to propel the such a large pig. Poseidon recognizes the difficulty in pigging a large diameter pipeline such as a 12 foot diameter pipe. Based on discussions with companies specializing in pigging pipes, Poseidon has confirmed that that a pig of the required size could be fabricated for this application but solutions to address the technical challenges listed above (launching/retrieving in the open ocean, turbidity control, and pumping requirements to propel the pig have not been fully developed.

Request 13: *Appendix JJJJJ-2 concludes that installing onshore air burst systems for intakes at stations U2 and D2 is technically infeasible. Water Boards staff requests additional information clarifying why onshore air burst systems would be required for intakes at stations U2 and D2 and why self-cleaning screens (as currently proposed at station E), manual cleaning by divers, use of a boat-based airburst system, or pigging could not be installed instead. Water Board staff note also that the 2017 Supplemental EIR only addresses manual cleaning by divers and boat-based airburst system cleaning and does not address the need for an onshore-based system. Use of an onshore-based airburst system would require additional CEQA analysis. Additionally, if air burst systems are required at those stations, it seems likely that an air burst system would be necessary for an intake at station E as well. Water Boards staff requests information on the technical feasibility of installation of an onshore air burst systems for station E.*

Response 13: Appendix JJJJJ-2 did not conclude that shore-based airburst systems are infeasible. Instead, JJJJJ-2 concludes that if the 1-mm screened ocean intake is located at site D2 or U2 then the decision to utilize a shore-based airburst system would require that the facilities be placed on the beach in order to be close enough to the WWS to be effective. An airburst system located on the plant site would be approximately 1.5 miles away from the 1-mm screens if located at U2 or D2 and too far to effectively perform. This is not the case for locating the intake at the proposed site (i.e., station E). A shore-based air burst system could be located on the plant site and effectively clean screens given the close proximity of the proposed intake site vis-à-vis intake sites D2 and U2.

The enclosed correspondence [Attachment G] from Acciona, Poseidon's Huntington Beach Preferred O&M provider, explains that a shore-based air burst system located on the facility site is feasible at station E due to the relative close proximity of the WWS compared to the alternative sites D2 or U2.

The State Lands Commission's 2017 FSEIR evaluated a boat-based airburst approach because it was considered by Poseidon to be the most environmentally impactful alternative. Both the boat-based airburst approach and shore-based airburst approach would require energy to operate the air burst equipment. However, the boat-based approach would result in additional GHG and criteria air pollutant emissions associated with boat operations and worker trips. Boat-based air bursting would occur 6 times per year, with a 1-hour, one-way vessel trip. These emissions were captured in the 2017 FSEIR.

A shore-based airburst system could feasibly operate at the plant site for the proposed project (intake site E), as described in Attachment G. Therefore, operation of a shore-based airburst system, for intake site E only, would not result in the technical, social, or environmental impacts associated with construction on the beach. The location and means of cleaning the screens will ultimately be based on the requirements of the WWS manufacturer and best available industry practices. However, the 2017 FSEIR analyzed the worst-case scenario for intake site E through consideration of the boat-based airburst approach.

If the WWS screens were located at site D2 or U2 a boat-based airburst system could be utilized but that consideration does not change JJJJJ-2's conclusion that construction and operation of a facility with the WWS screens located at site D2 or U2 cannot be accomplished in a successful manner in a reasonable period of time taking into consideration technical, social, environmental and economic considerations. As described in the feasibility report, the relocation of the intake to site U2 or D2 would be infeasible from a scheduling perspective. Implementation of the intake relocation, which includes permitting and construction, would take approximately 13 years to complete, and therefore cannot be completed in a reasonable timeframe. The relocation of the intake to site U2 or D2 would be economically infeasible, as it would significantly increase the project construction costs without any quantifiable environmental benefit to justify the added costs. Construction of the offshore intake site at U2 or D2 would result in the permanent loss of a small amount of benthic habitat. Construction of a trestle for marine work would involve disturbance and preclusion of access to beach and parking lots. Offshore and onshore construction would introduce significant visual changes to a visually sensitive area. The marine trestle required for offshore construction could experience structural damage from large high wave energy, tsunami events, or seismic induced risks including damage or inoperability. Sensitive receptors would be exposed to construction noise for a significantly longer amount of time as compared to the construction associated with the intake site E system. The construction of intake site D2 could create visual or noise impacts to annual events at Huntington Beach, including the Great Pacific Airshow and the U.S. Open of Surfing. While it is not anticipated that these events would significantly delay construction, the construction activities could impact the activities, some of which are major tourist-generating and revenue-generating events for the City of Huntington Beach.

Additionally, any facilities located on the beach would be subject to site constraints making them technically infeasible including climate change effects like sea level rise and coastal erosion. Temporary and permanent beach-based facilities would also require permits and approvals from land owners like the City of Huntington Beach, which may not be made available thus making the alternatives infeasible. Please refer to Part II (Response to the Regional Board's February 27th Additional Comments regarding Feasibility of an alternative site for 1-mm wedge wire screened ocean intake), Response 8, for an expanded discussion.

Request 14: *Address the feasibility of locating D2/U2 parallel pipelines further inland, instead of under the state beach to minimize beach impacts.*

Response 14: Moving the D2/U2 connection pipelines farther inland would require facilities to be built under Pacific Coast Highway and/or on private property and through local wetlands. All of these options would require easements and other rights that may not be attainable. Construction of the U2/D2 connection pipelines inland from the beach would also result in some of the same environmental and social impacts that would be experienced on the beach. Construction in and under Pacific Coast Highway would require lane closures, resulting in increased traffic, and impediments caused by the presence of construction equipment and materials. Additionally, temporary visual impacts would also occur from the presence of heavy construction equipment and laydown of construction materials along the pipeline alignment. Views of the Pacific Ocean and coastline from the nearby residences and beach users would be temporarily altered or obstructed throughout the duration of construction of the pipeline.

Construction in protected wetlands would be prohibited. Avoiding direct impacts in the wetlands through the use of trenchless methods would introduce risks of hydraulic fracturing, and locating entry and exit pits in areas that are either not protected wetlands, or developed areas would likely be infeasible. Other short-term, indirect effects could occur to biological resources such as sensitive coastal vegetation in the vicinity of construction including noise from construction equipment that could adversely affect wildlife and important wildlife activities such as bird breeding; contaminated stormwater runoff from construction sites could impact the water quality of a nearby wetlands or streams; fugitive dust from construction could cause wetland degradation; vegetation removal that may be required to clear the construction site or staging area could affect the viability of plant communities, thereby decreasing available habitat; and increased human activity in the area could lead to trampling of vegetation or disruption of wildlife behavior.

Request 15: *Water Boards staff requests information that includes the schedule and layout for facility construction for the proposed Project, including any distribution line construction farther inland that may run parallel to shore.*

Response 15: The proposed Project description including anticipated construction schedule is identified in the State Lands Commission's 2017 FSEIR. The construction schedule and durations have not changed since the State Lands Commission's certification of the 2017 FSEIR. Per the response above, constructing the distribution farther line inland to connect to alternative intake sites D2 or U2 is not feasible.

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Part II

Response to the Regional Board's February 27th Additional Comments regarding Feasibility of an alternative site for 1-mm wedge wire screened ocean intake

1. ***CEQA, Diffuser design, Response #1 – SLC staff concerned that previous estimates of air emissions impacts from onshore sediment disposal in their SEIR did not include onshore sediment disposal from construction of the new diffuser design, and that therefore, the worst-case scenario had not been addressed. Dudek's response was that the SEIR adequately covered NOx and other air emission impacts in their original analysis, which was overly conservative in estimating the need for a 12' deep trench for a diffuser and that there would be no additional impacts as the 300 CY from the diffuser construction would be covered under the initial estimates. Additional information/clarification should be added to this response.***

Response 1: As stated in Section 2.4.5.4, Wedgewire Screen Installation, on page 2-28 of the Final 2017 Supplemental EIR (FSEIR):

Dredging would occur to the level of the header invert (10 days dredging), and pile foundations would be driven for the screens (requiring about 3 days for the pile driving). Approximately 1,000 to as much as 3,300 cubic yards of soils would be excavated, depending on the final design phase geotechnical confirmation, and side-cast near the construction area, where they would have highly similar sediment composition to those in the side-cast footprint. Suspended sediments excavated would likely be redistributed by ocean currents. If excess material cannot be re-used or relocated on the ocean floor, it will be loaded on support barges and taken to the POLB for disposal in an appropriate land-based facility.

As a primary matter, the FSEIR does evaluate a worst-case scenario. As stated above in the FSEIR, this small amount of sediments *"will be side-cast near the construction area, where they would have highly similar sediment composition to those in the side-cast footprint."* Onshore sediment disposal is considered unlikely for up to 3,300 cubic yards and will only be necessary if the sediments cannot be re-used or re-located. As such, this represents the worst-case scenario analyzed in the FSEIR. The 1,000 – 3,300 cubic yards of seafloor sediments is intentionally broad and also reflects the conservative, worst-case excavation scenario.

Therefore, the criteria air pollutant and greenhouse gas (GHG) emissions associated with this activity were already captured in the 2017 FSEIR. Although the excavation described above is associated with the installation of the wedgewire screen (WWS), and not the diffuser, the 2017 FSEIR provided conservative (i.e., high), worst-case estimates of seafloor excavation, which would capture sediment removal associated with the new linear diffuser installation. Poseidon's Engineering, Procurement & Construction ("EPC") team believes that as the design is optimized as part of the final design the depth of the trench can be reduced to reduce the dredging quantity which would remain well within the overall envelope of impact analysis provided in the FSEIR. Poseidon expects the criteria air pollutant and GHG emissions will not exceed those analyzed in the SLC FSEIR.

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2. ***Economics, Attachment B, Response #3 – Please remove the term "retrofit" from the Cost Comparison table and provide additional clarification on:***
    a. ***Why costs for WWSs are same for existing intake and alternative intake locations***

Response 2a: The cost to install the wedgewire screen (WWS) at existing location E entails connecting the WWS/manifold assembly to the existing concrete tower. For the alternate location, the conceptual design considers installing a concrete structure of a similar size as the existing concrete structure in order to facilitate future maintenance. The cost to install the WWS/manifold assembly to the new concrete structure would be same as installing the WWS/manifold assembly to the existing concrete structure.

    b. ***Lines 11 and 12 – why direct capital costs are so much higher for moving the intake to U2 or D2 (costs and schedule)***

Response 2b: Please see detail of the cost components included for the Alternative Intake (U2/D2) on the 'Direct Capital Cost Detail' tab; the primary differences include constructing the pipeline, trestle and associated infrastructure for the alternative intakes at U2/D2 instead of utilizing the existing structures at E.

    c. ***Describe the differences between the cost calculations and assumptions used in this document and in the ISTAP report. Both documents are part of Poseidon's submittals, but appear to take very different approaches to identifying expected costs.***

Response 2c: The Coastal Commission's Independent Scientific & Technical Advisory Panel ("ISTAP") Phase 2 report evaluated the estimated cost of the proposed project including both the construction, operation, and maintenance cost for a desalination facility utilizing 1) a Seafloor Infiltration Gallery ("SIG") and 2) an open intake. The ISTAP cost estimate also included a "low cost estimate" and a "high cost estimate". The "low cost estimate" assumed that work could be performed year-round without schedule constraints. The "high cost estimate" assumed that work would be suspended from Memorial Day to Labor Day. In addition, the "high cost estimate" relied on capital and O&M cost provided by Poseidon while the "low cost estimate" incorporated reductions in some cost items based on input from ISTAP panel members and the Coastal Commission staff.

In contrast to the ISTAP cost estimate, the 1-mm wedge wire screened seawater intake alternative site analysis provides a cost comparison between building the 1-mm wedgewire screened intake structure at the existing permitted site as compared to building the same 1-mm wedgewire screened intake structure at alternative sites approximately 1.2 miles up and downcoast. The cost for the balance of the plant and the operation and maintenance cost were not included. The cost alternate site analysis also incorporated a truncated construction schedule to address pile driving work restrictions related to whale migration. The 2017 FSEIR requires that pile driving will

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only be permitted between June and November to avoid potential impacts to gray whale migration.

In terms of the calculation of the estimated direct capital costs, both the approach undertaken for the ISTAP process and the cost estimate Poseidon has submitted for the 1-mm wedge wire screened intake alternative site analysis estimate direct capital costs for building the required intake infrastructure, with the addition of necessary costs related to secure both a turnkey construction and an operating agreement including performance guarantees to construct and operate the infrastructure as well as the costs necessary to project finance the infrastructure with an investment grade capital structure. For the purposes of ISTAP, the capital cost calculation includes the same inputs as was presented in Poseidon's alternative intake site cost estimates. Any apparent differences are presentational and immaterial. The consistent basis used for both calculations includes estimating:

- A construction period for each alternative. (Note that the ISTAP estimate was prepared prior to the whale migration schedule restrictions included in the 2017 FSEIR)
- Direct capital cost for both estimates included indirect cost, insurance, contractor profit and overhead, engineering cost and contingencies appropriate for a Category IV estimate.
    - Direct capital costs estimate for the ISTAP SIG only cost was consistent with a Category IV level of accuracy with an accuracy range from -30% to + 50%. The cost basis for the balance of the plant was consistent with a Category II estimate with an accuracy of +/-10% to 25%. The estimate that Poseidon provided for the the1-mm screened Intake alternative site analysis was prepared at a Category IV level of accuracy.
    - Direct costs are escalated until the period of Financial Close when they are set under the engineering, construction and procurement (EPC) Agreement.
- Costs for capitalized interest during the construction period including a contingency
- Reserves, financing fees and other closing costs necessary to achieve an investment grade capital structure

For presentational consistency, Poseidon has prepared an alternative version of the Alternative Intake Site Cost Estimate table included as [Attachment B] that presents the estimate on the same presentational basis as Table 5.1 in the ISTAP Final Phase 2 Report (Page 52). The presentational differences in Table 5.1 include:

- Engineering costs were shown included in each cost component (while it is shown as a separate line item in Ref. Line #10 in the Alternative Intake Site Cost Estimates)
- Escalation of each capital cost component to period of Financial Close are shown allocated to each cost component (while it is shown as a separate line item in Ref. Line #13 in the Alternative Intake Site Cost Estimate)

- Owner's Project Management and Inspection was shown as a separate line item (While it was grouped with "Development and Construction Costs" in Ref. Line #15 in the Alternative Intake Cost Estimate)
- Intake Pump Station and Screen Retrofit shown as separate line items (See response to Question 2a for more details on Intake Structure costs)
- Reserves are shown as a separate line item (while they were shown grouped with "Financing Fees" in Ref. Line #17 of the Alternatives Intake Site Cost Estimate)
- Diffuser costs were shown as a separate cost line item (the Alternative Intake Cost Estimate does not include a Diffuser cost as it reflects costs for the Intake)
- The ISTAP analysis included Decommissioning Costs (these costs are not included in the Alternative Intake Site Cost Estimate)

3. ***Environmental, Response #5 –***
    a. ***Sampling at E conducted outside of the hydraulic influence of the existing AES intake.***
    b. ***Water Board staff clarified that the intent of the question (#5) was to ask that if there were no intake at Station E, would the biological community look different at E, especially when compared to Stations U2 and D2. Eric responded that evidence from otter trawls conducted as part of the annual monitoring for the AES generating station indicate that E has higher fish densities than locations sampled approximately 1 km up- or down-coast. He added that fish tend to congregate along outfalls. Please provide this additional clarification to the response and provide references (e.g., trawl data reports, journal articles referenced during the conference call regarding this issue).***

Response 3: The owners of the Huntington Beach Generating Station (SCE and AES) conducted demersal fish sampling using a 7.6-m otter trawl net annually in the summer from 1991-2016, with some breaks between 2001 and 2006. These surveys were in compliance with their National Pollutant Discharge Elimination System permit. The breaks were during a period when their permit only required the survey in select years rather than annually. During all years, the same stations and sampling parameters (net size, net mesh, trawl time) were the same. MBC Applied Environmental Sciences conducted the survey each year, usually during one day in August (MBC Applied Environmental Sciences 1991 through 2015). Therefore, there was also institutional consistency in the sampling program. The catch totals from each Annual Receiving Water Monitoring Report are summarized in Table 1 below. The annual reports were submitted to the Santa Ana Regional Water Quality Control Board each year by the current generating station owner. This represents the total number of fish caught in the net each year at each of the three permit-designated sampling stations.

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Table 1
Annual Total Trawl Caught Fish Abundance Offshore the Huntington Beach Generating Station

Year	Upcoast	Discharge	Downcoast
1991	131	25	3
1992	3,419	774	2,217
1993	125	15,237	174
1994	602	1,098	379
1995	31	46	1,719
1996	805	159	21
1997	347	445	239
1998	528	225	1,180
1999	40	17	25
2000	207	210	907
2001	245	250	126
2004	54	45	66
2007	40	41	27
2008	96	59	41
2009	47	65	64
2010	122	25	43
2011	1,089	5,861	107
2012	246	182	198
2013	588	536	1,098
2014	1,814	1,569	691
2015	72	80	103
2016	81	65	98
Grand Total	10,729	27,014	9,526
Mean	487.681818	1,227.90909	433
SE	167.295865	718.252807	131.384502

Source: MBC Applied Environmental Sciences 1991 through 2015
Notes: Annual total trawl caught fish abundance at each of three sites sampled each year offshore the Huntington Beach Generating Station. The large catch in 1993 included Northern Anchovy, White Croaker, and Queenfish, the three long-term dominant taxa.

Figure 1 depicts the mean annual catch with standard error bars. It demonstrates the fish community near the discharge (which is proximate to the intake) was more abundant overall, with some interannual variability demonstrated over time. This is consistent with what Helvey and Dorn (1981) observed, wherein fishes aggregated near an operating seawater intake.

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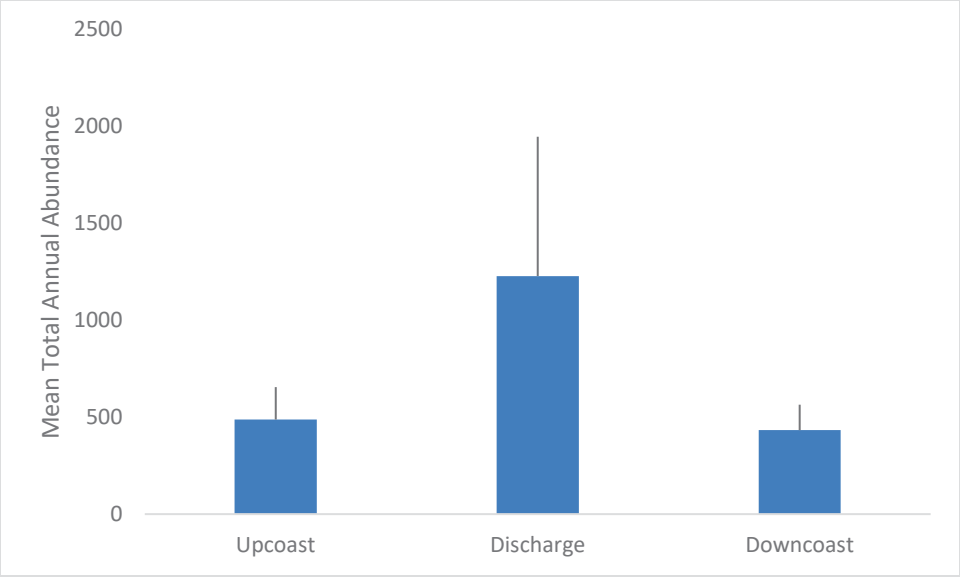

Figure 1. Mean total annual abundance of trawl-caught fishes at each of the three sites sampled each year in the summer for the Huntington Beach Generating Station. Standard error bars are included to denote variability.

4. ***Environmental, Response #6 – Please remove references to outfall and riprap as "sensitive habitat" and to lower Santa Ana as "estuarine habitat." The lower Santa Ana River, while it is listed in the Santa Ana River Basin Plan as a tidal prism, it is not listed as having the estuarine beneficial use.***

Response 4: Please see the revised language in Part I (Response to Regional Board's February 4, 2019 Request for Additional Information Regarding 1-mm Wedgewire screened ocean intake Alternative site Feasibility Assessment), Response 6.

5. ***Technological, Response #11 – Please clarify that it isn't technologically infeasible to use a float-in method for constructing the intake at one of the alternative sites, but that there are other more cost effective and better methods for constructing the intake offshore. Include why reduction in pipe diameter from 14' – 12' does not change this finding of relative infeasibility.***

Response 5: The "float-in" method that was reviewed by ISTAP consisted of "floating-in" a pre-assembled cofferdam frame with 22.5 foot long sheet piles attached to the cofferdam. ISTAP further assumed that the pre-assembled cofferdam would be placed in 42 feet of water in the coastal area of Huntington Beach considered in the ISTAP report as "high-energy shoreline with high wave heights and accompanying strong long-shore currents." After setting the cofferdam frame into position at the location, sheet piles would be driven to a depth of 21.5 feet. ISTAP assumed that the excavation could

be advanced 12.5 feet beneath the seabed leaving 9 feet of sheet piles beyond the bottom depth of the excavation for toe resistance. In order for Poseidon to utilize this float-in approach for the installation of a large diameter pipeline from the existing location E to new location D2/U2, changes would be required beyond what was evaluated by the ISTAP panel. The depth of the excavation required for a large diameter pipeline would need to be deeper than what was assumed for the SIG to allow for placement of bedding and sufficient cover for scour protection over the pipeline.

Reducing the pipeline diameter from 14-12 feet does not have a material effect on the cost of materials and installation and does not change construction means and methods. Considering a 12-foot diameter pipeline, for example, the requirements for pipe bedding and scour protection over the crown of the pipe would add another 4 to 6 feet of excavation depth. Assuming that an additional 1 to 2 feet of embedded sheet for toe-resistance is required for the deeper excavation, the overall length of the sheet piles would approximate 28 to 30 feet. These longer sheet piles (28 to 30) would be placed on the pre-fabricated cofferdam and the entire assembly would be positioned in only 30 feet of water closer to the high energy surf zone. Poseidon and its contractor are not aware of where a “float-in” approach has been used in a similar application as to what would be required to construct the pipeline in such shallow water, in the open ocean and adjacent to the high-energy surf zone (and such an application is significantly different than what the ISTAP panel considered for the feasibility of using the “float-in” approach for SIG). Therefore, Poseidon is unable to state that using the float-in approach “isn’t technically infeasible.” The marine construction work required to build an alternate intake would represent a significant portion of the construction and schedule risk for the project. As such, Poseidon’s EPC team would only select an available and proven approach where the cost and schedule risk could be quantified. The “float-in” approach for placement of a pipeline from existing location E to alternate locations D2/U2 would not meet this standard as previously highlighted by ISTAP Panel member Bob Bittner, who's firm specializes in the design of innovative marine structures. Mr. Bittner stated during the ISTAP process that *"The cost-savings would not be significant and actually for the float-in option, I feel that putting it closer to shore could actually increase the cost because as you move closer into shore the waves start to drag on the bottom and they get higher which causes problems for the construction [emphasis added]."*

Finally, regardless of the construction approach, attempting to site the 1-mm wedge wire screened intake at alternative sites U2 or D2 would result in a substantially longer implementation schedule (up to 13 years to develop, permit, construct, and commission), substantially greater social impacts (e.g., beach closures and associated economic impacts on tourism), avoidable short-term and long-term environmental impacts (e.g., noise and air emissions, permanent loss of benthic habitat) and at substantially greater cost ($474M in additional costs) without any scientifically justified environmental benefit.

6. ***Technological, Response #12 – Please provide additional clarification for the large diameter pipe needed for the intake (12-14’), including justification for why an alternative intake would need to T-off of either***

the existing intake pipe shoreward of, or under PCH, or the offshore intake tower, in order to move the intake and connecting pipeline to either Station U2 or D2 (provide assumptions used).

Response 6: In evaluating the alternative locations (D2/U2) instead of location E, Poseidon considered multiple technical factors including the following:

1) Technical Feasibility
   a. Locations E, D2, and U2 are located in approximately 30 feet of water and approximately 1500 feet to 1900 feet from the shore and are subject to wave forces including waves heights that may exceed 20 feet during extreme northern hemisphere storms.
   b. Work offshore involving pile driving is subject to a truncated working schedule due to gray whale migration restrictions.
   c. The intake pump station for the desalination plant will be located on the existing power generating facility site and will tie-into existing power generating facility infrastructure. Head loss calculations for NPSH restrictions for the intake pumps must take into account head losses through existing infrastructure as well as the new intake system including marine growth.

2) Utilization of Industry Standard Construction Practices
   a) The Huntington Beach Desalination Project, like the Carlsbad Desalination Project, is anticipated to be funded using a mix of tax-exempt debt and private equity. Rating agencies will evaluate whether the project can be considered investment grade by conducting a risk assessment which includes a thorough independent engineering evaluation of the process technology, a review of the O&M risks and an evaluation of the EPC cost estimate and schedule risks, all of which could affect the viability of the project. Construction approaches for the intake that are outside of industry standard and accepted practices increase the risk profile of the project and impact the ability to obtain both a cost competitive EPC price and project financing. At a minimum, using approaches outside of the available and proven methods associated with industry standard construction practices would be expected to raise the cost of both attaining a fixed-price/turnkey EPC contract (in the form of higher insurance, contingency and engineering costs) and raise the weighted-average cost of capital of the project (in the form of higher costs of debt and equity due to the increased risk profile).

3) Operation and Maintenance Issues
   a) The O&M phase of the project will span 30 years and will utilize WWSs to comply with the OPA.
   b) WWSs will require regular cleaning including air-bursting to remove debris attached to the screens. To the extent possible, the ability to clean the

screens using land-based systems is preferable to boat-based maintenance operations as boat-based systems are dependent on calm sea conditions.

c) Marine growth will need to be considered over the term of the O&M phase.

4) Impacts to Community including Beach Access and Traffic
   a) The existing power generating power station infrastructure includes two 14 foot diameter concrete pipelines that connect the forebay at the southern boundary of the facility's property with existing intake and discharge concrete structures located offshore.
   b) Based on a condition assessment completed in 2018, the pipelines are in good condition. Additional pipeline crossing under Pacific Coast Highway can be avoided if the existing pipelines are utilized.
   c) To the extent possible, existing infrastructure should be utilized to avoid prolonging the construction schedule to reduce impacts to beach recreation and traffic.

In evaluating the options to utilize location D2 (or U2), Poseidon first considered connecting location D2 to the existing concrete intake tower utilizing a new pipeline that would be installed on the seabed. This approach would largely avoid impacting beach recreation and traffic as it would utilize the existing intake infrastructure and construction could be performed using floating equipment. Preliminary pipe sizing calculations determined that a large diameter pipe would be required for the 1.2-mile extension (between 10 to 14 foot diameter) to avoid pump cavitation. (A revised engineering evaluation showed that a 12 foot diameter pipe would be used). Poseidon first considered using an HDPE pipeline that could be floated to location and sunk into position. Upon a closer review of this approach, there were several technical obstacles that were identified which caused this approach to be eliminated including the size limitations of HDPE (8.2 foot maximum diameter for commercially available pipe), wave loading on the a 1.2 mile pipe aligned parallel to the shore (perpendicular to the incoming wave direction), potential impacts due to scour and seabed movement and the likely impact on wave characteristics along the surfing beach caused by the pipelines profile (parallel to the beach for over 1.2 miles). Refer to GHD memo provided as Attachment H.

Poseidon also evaluated burying a pipeline from location E to location D2/U2. This approach would allow for the installation of a 12 foot diameter pipeline to meet the required NPSH requirements and would avoid subjecting the pipe to wave loading and would also avoid altering the wave characteristics. The industry standard construction practice of placing a buried pipeline offshore includes dredging the pipe excavation using an open cut method or placement utilizing a trestle with sheet piles. The open cut method was first evaluated as this could be performed entirely offshore with floating

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equipment and would largely avoid impacts to beach recreation and traffic. An ocean rated barge would excavate material using a large clam shell and would either side cast the material or load onto support barges for either ocean disposal or land-based disposal. Based on the expected marine soil conditions and the proximity of the trench alignment to the surf zone, maintaining the excavation prior to placement of the pipe would be challenging, leading to massive quantities required for excavation. Open cut excavation using methods described above are more applicable for areas well outside of the surf zone where the excavation side slopes are less susceptible to sloughing from wave action. Poseidon also considered burying the pipeline using trestle construction. This approach would incorporate the use of sheet piles which would greatly limit the amount of dredging required. Upon a closer review of the sheet pile option, it became apparent that this approach also had serious technical issues. Due to the pile driving restrictions during gray whale migration, the timeframe to install the trestle approach makes this approach unworkable. In addition, the prospect of placing a trestle 1.2 miles parallel to the coast (perpendicular to the wave loading) which would remain in place for multiple winter storm seasons represents a significant construction and schedule risk which would be flagged in the independent technical review by financial institutions. There would also be a significant impact to beach recreation as multiple access trestles would be required to connect the 1.2-mile trestle to the shore for movement of personnel and materials. The approach of burying a new pipeline from location E to alternate location D2/U2 was also eliminated. Refer to the HCSLLC memo provided as Attachment H for a more detailed discussion on viability of the offshore trestle.

After the offshore approaches to connect location E to D2/U2 were eliminated due to technical consideration, Poseidon evaluated a hybrid approach that would entail both a land-based pipeline and an offshore pipeline. The land-based reach of the pipeline would tie-in to the existing 14 foot diameter concrete intake pipeline on the beach and would traverse the beach parallel to the shore using open-cut construction techniques. Sheet piles would be used to maintain vertical side slopes to minimize the construction footprint. By connecting to the existing pipeline on the beach, a crossing of a large diameter pipeline under a major highway (Pacific Coast Highway) could be avoided. The pipeline would then turn a right-angle on the beach and then traverse through the surf zone using trestle/sheet pile construction techniques. The trestle would be placed perpendicular to the shore and would be much shorter than was anticipated for the offshore alignment described above. This hybrid approach would utilize industry standard and accepted construction practices for placement of pipelines and the risk associated with these construction methods are no different than normal pipeline work for the onshore portion and for the trestle portion traversing through the surf zone.

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While this hybrid approach does cause impacts to beach recreation and traffic, there are ways that the impacts can be reduced such as scheduling the land-based construction during the winter season. The trestle work (which involves pile driving), however, could only be performed during the limited work window available to avoid gray whale migration, which includes the summer months. .

For this hybrid approach, Poseidon also considered provisions in the conceptual design for maintaining the pipeline. Concrete vaults were included at the 90 degree turn point where the pipeline heads offshore and at the offshore location D2 or U2. The vaults would allow entry into the pipeline for maintenance and inspection and would also provide locations to launch and retrieve pigs for cleaning the pipeline. A land-based air burst station was also considered in the conceptual design and was included at the vault located at the 90 degree turn point. For location E, which is much closer to the Project site, air bursting of the wedgewire screens is possible using air compressors and receivers located at the power generating station. In order to provide land-based air burst capability for location D2/U2, the air burst station would be placed at the 90 degree bend vault location as the distance to location D2/U2 is too great for a shore-based system located at the power generating site.

Also provide additional information and justification (references, calculations) for the estimate of 6" of biofouling that is likely to occur within the intake pipe even with the 1 mm WWSs.

Poseidon's assumptions for marine growth are independent of the screen sizing for the inlet screens. The marine growth that accumulates in the pipe will be composed predominantly of marine invertebrates that would be small enough to entrain through a 1-mm slot width. Although large macrofouling organisms (e.g., adult mussels) would not entrain through the 1-mm wedgewire screen, the early life stages of these macrofoulers would. After entrainment through the screen, these early life stages settle and begin to grow into adults using the intake flow to provide a steady supply of planktonic food. The macrofouling organisms that pose the greatest risk to seawater intake systems are hard macrofouling organisms such as mussels and barnacles. The planktonic early life stages of mussels and barnacles measure less than 500 µm (0.5 mm), meaning they would entrain through a 1-mm slot. The successful settlement of these hard macrofoulers depends in large part on the development of early successional fouling layers comprised of smaller fouling organisms (i.e., microfoulers). The typical progression of biofouling in a seawater pipe would begin with the settlement of organic material and bacteria (biofilm), followed by algae, and later by the larger, more problematic hard macrofouling organisms such as mussels and barnacles.

Poseidon's assumption on the thickness of marine growth was based on observations from the 2018 condition assessment of the power generating facility's existing intake/discharge infrastructure (Attachment I). Intake piping onshore was measured by divers and was found to range from 2 inches to 8 inches thick. The offshore pipeline was inspected using a remotely operated underwater vehicle (ROV) equipped with sonar. The sonar allowed the ROV to check for damage to the pipe and to estimate the thickness of marine growth and bottom sediments. The operators of the ROV estimated the marine growth to approximately 3 inches thick with bottom sediments up to 2 feet. Based on the measured thicknesses of marine growth by divers for the onshore intake piping and the estimates from the ROV for the offshore pipeline, Poseidon used a thickness of up to 6 inches of marine growth for the design basis in calculating head loss through the new pipe alignment to location D2/U2 and was based on actual observed information plus a factor of safety. The conceptual design also included provisions for pigging the 12 foot diameter portion of new pipeline in the event that the assumption of 6 inches of marine growth was proven to be insufficient during the 30-year O&M period for the desalination plant. The condition assessment provided a data point for marine growth for a 14 foot diameter pipe; no data point was available for a 12 foot diameter pipeline. The estimate for marine growth for the 12 foot diameter pipe is based on the assumption that it would behave similar to the 14 foot diameter pipe but without actual measured data for a 12 foot pipeline, there is more risk in this assumption than for the 14 foot diameter pipeline.

7. ***Technological, additional clarification to add to Responses #12 and/or #14 – Please discuss (1) whether other configurations for extending the intake to Stations U2 or D2 other than parallel to the beach onshore or offshore of the proposed HBDP have/have not been considered (e.g., intake extending diagonally from the existing onshore intake structure) and why; and (2) whether an intake could be constructed at either of the alternative intake locations using two or more smaller diameter pipes instead of one large pipe, which could be constructed of HDPE, and could be jacked under the beach/offshore and cleaned using a pigging method. Please discuss and provide justification why or why not this option was/was not considered when assessing an alternative intake location at Stations U2 or D2.***

Response 7: In evaluating options to utilize alternate locations D2/U2, trenchless methods were not initially considered because other approaches were available that Poseidon viewed as less risky from a cost and schedule perspective. Please refer to the response to Question 6 above. But based on the Water Board's inquiry on using a diagonal alignment from the Plant site to the alternate location and/or using installing smaller diameter intake pipes using trenchless methods, Poseidon has revisited a trenchless approach for the intake line. Poseidon engaged a specialized consulting

firm, Sub Terra, Inc. that focuses on tunneling and underground engineering. Poseidon used Sub Terra previously for consulting advice during the construction of the Carlsbad Desalination Project when Poseidon's contractor experienced difficulties tunneling a section of the land-based conveyance pipeline.

In evaluating the feasibility of using trenchless methods for the Huntington Beach desalination intake, Sub Terra compared the requirements for the site with previous micro-tunnel projects performed by experienced contractors, the availability of specialized equipment and materials and the geotechnical conditions present at the site. There are several Contractor's in the US with the experience required, however, the maximum distance previously micro-tunneled in the US is just over 3,000 feet using seven intermediate jacking stations to jack an 84-in diameter casing pipe through competent soils. This is significantly smaller than the inside diameter required for this project and the welded steel pipe that was jacked on this 3,000 foot run (the longest that has been performed in the US) would not be suitable for conveyance of seawater; an inner carrier pipe, made of suitable material, would need to be installed inside the casing pipe. Alternative casing pipe materials, e.g., polymer concrete pipe, could be used but there is limited experience installing this type of pipe at the diameter required and, to our knowledge, no experience on micro-tunnel runs of the length required for this project.

Two pipelines would therefore likely be required where corrosion resistant carrier pipes were installed inside a jacked casing pipe. There are also limitations regarding carrier pipe materials; for example, using HDPE would limit the carrier pipe to 2.5 meter (8-feet) inside diameter installed in a minimum 10-ft diameter steel casing pipe. Forming an 18-inch concrete lining would require using an 11 to 12-foot diameter casing pipe in each run to match the flow in a single 12-foot inside diameter concrete pipe installed by surface methods.

The feasibility of utilizing a trenchless approach is also determined by the geotechnical conditions of the site. For the Huntington Beach site, there are a number of geotechnical studies that were commissioned by Poseidon and others that provide a good representation of the conditions that would be encountered. These studies include investigations by the Department of Water Resources and Orange County Water District for the evaluation of the Talbert Aquifer, investigations by the Army Corp of Engineers for evaluating potential offshore borrow areas and investigations by Geosyntec to evaluate the feasibility of an offshore collector as part of the assessment of the Seafloor Infiltration Gallery. Based on the available information, the expected geotechnical conditions can be characterized as silty sand, silty fine-grained sand, and silt or clay intervals with thickness of 0.5 to 6 feet. The Talbert Aquifer is approximately 90 feet deep in this area.

Although tunneling and micro-tunneling approaches have been employed around the world for the installation of intakes and outfalls for power stations, wastewater treatment plants, desalination facilities and fish farms, generally these installations have been used in competent sedimentary rock formations or hard rock – which are significantly less challenging than installations in unconsolidated formations like the conditions at the Huntington Beach site.

When evaluating trenchless pipe installations, pipe jacking is often mentioned. Pipe jacking is a general term which describes the method of using hydraulic rams to push pipe along a defined alignment. There are several options to excavate and remove the soil in front of and inside the pipe including

a) Auger boring: This technique uses drag pits or cutters arranged in a pattern on the face of the cutter head that breaks up the soil and transport the soil out of the pipe using an auger; and
b) Micro-tunneling: This technique uses drag pits or cutters arranged in a pattern on the face of the cutter head that breaks up the soil which is mixed with a slurry inside the cutter head and removed by pumping the slurry/soil mixture through slurry pipelines attached to the inside of the pipe. The soil is separated from the slurry, to the extent possible in a separation plant outside of the tunnel.

Pipe jacking would require a deep shaft 60 to 70 feet deep by approximately 40 feet long to accommodate a jacking frame. In the case of a micro-tunneling approach, a substantial area at the surface would be required to support the slurry processing plant and the slurry separation plant. Additional laydown area is required to store pipe sections and other equipment.

The auger boring technique is applicable to above the water table and is limited to runs of about several hundred feet. Both the distance and the requirement to be above the groundwater eliminates this option for Huntington Beach. The micro-tunneling option also has limitations on the length of runs as the frictional force of the formation (especially in unconsolidated sandy/gravelly formations) has to be overcome using lubricants such as bentonite and intermediate jacking stations have to be installed when needed.

Unconsolidated sandy and gravelly soil formations are associated with higher frictional forces. For drives in sandy or gravelly formations, specialized high viscosity bentonite mixtures are required to reduce the frictional forces. In addition, when used in a seawater environment, bentonite slurries present additional challenges in mixing, transporting, and separating the sands and silt. As the bentonite is pumped to the cutting head under pressure, the risk of “frack-out” or the escape of bentonite from the

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formation and being released into the sea is an ever-present environmental risk when using micro-tunneling for application in a sensitive marine environment. This means that micro-tunnels cannot be advanced underwater with less than 15 to 20-ft of cover without a significant risk of releasing slurry and lubricants into the seawater.

As described above, the longest micro-tunnel installed in the US was approximately 3000 feet long and was installed in competent soil without the challenges of unconsolidated sands/gravels using a much smaller diameter pipe. This application required seven intermediate jacking stations. For a micro-tunnel to be considered for the Huntington Beach site, micro-tunnel runs longer than 1500 feet would not be recommended.

The disposal of the spoil from the tunneling operation for a micro-tunneling approach is also a factor when comparing a micro-tunnel approach versus a more traditional open-cut excavation approach. The micro-tunnel approach results in mixing 100% of the soil removed from the tunnel with a bentonite/polymer slurry so that it can be pumped out. The slurry separation plant will be able to recover much of the bentonite, however, 100% of the sands and silts will be coated with slurry and will require disposal at a landfill. In addition, a high percentage of the material requiring disposal will consist of a very high-water content sludge (approaching 50%) which would likely require disposal in a special landfill. For an open-cut excavation approach, excess sand that is excavated could be re-used.

Sub Terra also commented on an option of using horizontal directional drilling (HDD) to install the intake piping. HDD technology would likely be able to overcome the limitations with micro-tunneling but the technology is restricted in terms of the diameter of pipe that can be installed. Sub Terra indicated that the practical maximum pipe diameter for HDD technology is 42 inches. Sub Terra further noted that in order to achieve the volume flow required for the Huntington Beach project approximately 15 holes drilled (and possibly more depending on assumptions used for marine growth for the small diameter pipelines). Poseidon did not request that Sub Terra evaluate the HDD technology further because 15 or more small diameter intake lines would not be a viable solution for the intake system for the Huntington Beach desalination plant. Poseidon does not have any reference data on marine fouling for small diameter pipe at the project site and would not be able to quantify the O&M risk associated with operating a plant based using a non-conventional intake system.

Given the requirements and limitations of the trenchless approaches in unconsolidated sands/silts, Sub Terra evaluated two possible trenchless options for the Huntington Beach Desalination intake:

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Option 1: Install 6,400 foot pipe along the beach (parallel to the shore) and then turn 90 degrees and tunnel 2600 feet under the surf zone using micro-tunneling methods.

Option 2: Install 7200 feet of pipe in a diagonal alignment (from the forebay of the power generating facility to alternate location D2)

For Option 1, it was assumed that two parallel micro-tunnels would only be used to traverse the surf zone from the beach to a location where the cover over the TBM would have at least 15 to 20-feet and the distance of the tunnel drive would be within the 1500 foot or less range. Sub Terra noted the tunnel drive would terminate short of location D2/U2 and that it would be necessary to use traditional methods such as a trestle or working from barges to retrieve the MTBM and to continue the pipeline to its final location. If the alignment of the tunnel were lowered to provide a minimum cover of 20-feet, the intake structure would need to be installed to a depth of at least 30-foot below the seabed, probably deeper, requiring construction of a large sealed coffer dam or (sheet piled shoring structure if all intake work was to be completed using divers working under 50-feet of water). In this case, the required length of micro-tunnel would exceed the recommended length for site geology.

For the 6400 feet alignment along the beach as part of Option 1, Sub Terra recommended that an open cut approach be used. Micro-tunneling could also be used for the 6400 feet section along the beach but this would require dividing the tunnel into three or more drives (with intermediate jacking stations) which would create a similar beach impact when compared to a more traditional open-cut approach while introducing additional construction risk.

For Option 2, it was assumed that the tunnel would start from the power generating station and proceed 7200 feet to location D2/U2. Sub Terra also stated that a similar diagonal tunneling alignment could be considered starting from the beach (connecting to the existing 14 feet diameter intake line) and proceeding to location D2/U2 which would reduce the overall length of the tunnel by approximately 400 ft. At approximately 7200 feet (or 6800 feet if the tunnel were started on the beach west of Pacific Coast Highway by connecting to the existing intake pipe), Option 2 would need to be divided up into multiple shorter drives with each of the shorter drives requiring intermediate jacking shafts along with support equipment for slurry mixing and separation. Several intermediate jacking shafts could be installed on the beach but an additional jacking shaft would require installation offshore with exposure to the surf zone. Installing intermediate jacking shafts in the surf zone would require trestles for access and installation of the cofferdam required for each shaft. Option 2 is also subject to the same constructability issues as Option 1 both with regard to shallow cover and the

release of slurry and lubricants and the requirement for an additional cofferdam at the Intake location if a deep alignment was followed.

After evaluating the technical requirements and limitations for using trenchless intake pipe installation techniques in unconsolidated sands/silts present the Huntington Beach location, Poseidon does not consider the trenchless option as a viable alternative. The limitations of micro-tunneling including maintaining sufficient cover over the tunnel to avoid “frack out”, recommended length of micro-tunnel runs and the requirement to use two parallel tunnels to obtain the required flow and limit head-loss in the intake system result result in an approach that does not offer any advantages over the previously presented “non-tunneling” options but carries significantly more risk. Both micro-tunneling approaches evaluated by Sub Terra also require offshore construction involving sheet piles and probably trestles which would be subject to whale migration schedule restrictions for pile driving.

In the response to Question 6 regarding the assumptions used in recommending the more traditional approach i.e. land-based, open-cut alignment parallel to the shore and trestle construction to install the pipeline from the shore to location D2/U2, Poseidon explained that four factors were considered in making the selection: 1) technical feasibility, 2) utilization of industry standard construction practices, 3) operation and maintenance Issues, and 4) impacts to community including beach access and traffic With the geologic conditions present at the Huntington Beach site, the trenchless option presents significant technical risk. The tunneling risk can be mitigated by reducing the length of the tunneling drives but this requires that more work is performed offshore using traditional trestle/sheet pile construction to retrieve the TBM and to complete the pipe to location D2/U2. The micro-tunneling approach also introduces operation and maintenance issues as the smaller diameter, twin piping would likely require more effort to control marine growth then a single, larger diameter pipeline. Both the micro-tunnel approach and more traditional open-cut excavation have similar impacts to the beach area during construction. As the micro-tunnel approach would involve both tunneling risk and offshore construction risk (trestle/sheet pile), Poseidon would not present this option to rating agencies for the purposes of securing project funding due to the higher construction and schedule risk profile when the option does not have advantages over the industry standard construction practice of open-cut and trestle approach that was described in response No. 6. Please find the Sub Terra memo included as Attachment J.

8. ***Technological, Response #13 – Please include additional information in the submittal as to why rotating brushes could not be used to clean Cu/Ni WWSs and why an onshore air burst system is the preferred method for cleaning these types of screens.***

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Response 8: As noted during the February 26 conference call with staff of the regional and state board, CA State Lands Commission, and CA Coastal Commission, O&M concerns for WWS can be generally parsed in two categories: management of free-floating debris and management of biofouling on the screen surface. The discussion below is parsed into these two principal categories.

Free-floating Debris

When using passive WWS, an airburst system is required regardless of the material used (duplex stainless steel or copper nickel). A boat-based airburst system was evaluated in the 2017 FSEIR because it is considered the worst-case scenario due to the GHG and criteria air pollutant emissions associated with offshore boat trips. A shore-based airburst system, however, would reduce the impact (lower GHG and criteria air pollutant emissions) and, if available at the time of construction, it would be the preferred alternative. The State Lands Commission's 2017 FSEIR evaluated a boat-based airburst approach because it was considered by Poseidon to be the most environmentally impactful alternative. Both the boat-based airburst approach and shore-based airburst approach would require energy to operate the air burst equipment. However, the boat-based approach would result in additional GHG and criteria air pollutant emissions associated with boat operations and worker trips. Boat-based air bursting would occur 6 times per year, with a 1-hour, one-way vessel trip. These emissions were captured in the 2017 FSEIR.

A boat-based airburst system also has the disadvantage of being unavailable during storm events when offshore conditions would preclude vessel traffic. It is also during such storm events that the WWS would be at their greatest risk of clogging with debris.

Appendix JJJJJ-2 did not conclude that shore-based airburst systems are infeasible. Instead, JJJJJ-2 concludes that if the 1-mm screened ocean intake is located at site D2 or U2 then the decision to utilize a shore-based airburst system would require that the facilities be placed on the beach in order to be close enough to the WWS to be effective. An airburst system located on the plant site would be approximately 1.5 miles away from the 1-mm screens if located at U2 or D2 and too far to effectively perform. This is not the case for locating the intake at the proposed site (i.e., station E). A shore-based air burst system could be located on the plant site and effectively clean screens given the close proximity of the proposed intake site vis-à-vis intake sites D2 and U2.

The enclosed correspondence [Attachment G] from Acciona, Poseidon's Huntington Beach Preferred O&M provider, explains that a shore-based air burst system located on the facility site is feasible at station E due to the relative close proximity of the WWS compared to the alternative sites D2 or U2.

A shore-based airburst system could feasibly operate at the plant site for the proposed project (intake site E), as described in Attachment G. Therefore, operation of a shore-based airburst system, for intake site E only, would not result in the technical, social, or environmental impacts associated with construction on the beach. The location and

means of cleaning the screens will ultimately be based on the requirements of the WWS manufacturer and best available industry practices. However, the 2017 FSEIR analyzed the worst-case scenario for intake site E through consideration of the boat-based airburst approach.

If the WWS screens were located at site D2 or U2 a boat-based airburst system could be utilized but that consideration does not change JJJJJ-2's conclusion that construction and operation of a facility with the WWS screens located at site D2 or U2 cannot be accomplished in a successful manner in a reasonable period of time taking into consideration technical, social, environmental and economic considerations. As described in the feasibility report, the relocation of the intake to site U2 or D2 would be infeasible from a scheduling perspective. Implementation of the intake relocation, which includes permitting and construction, is anticipated to take up to 13 years to complete, and therefore cannot be completed in a reasonable timeframe. The relocation of the intake to site U2 or D2 would be economically infeasible, as it would significantly increase the project construction costs without any quantifiable environmental benefit to justify the added costs. Construction of the offshore intake site at U2 or D2 would result in the permanent loss of a small amount of benthic habitat. Construction of a trestle for marine work would involve disturbance and preclusion of access to beach and parking lots. Offshore and onshore construction would introduce significant visual changes to a visually sensitive area. The marine trestle required for offshore construction could experience structural damage from large high wave energy, tsunami events, or seismic induced risks including damage or inoperability. Sensitive receptors would be exposed to construction noise for a significantly longer amount of time as compared to the construction associated with the intake site E system. The construction of intake site D2 could create visual or noise impacts to annual events at Huntington Beach, including the Great Pacific Airshow and the U.S. Open of Surfing. While it is not anticipated that these events would significantly delay construction, the construction activities could impact the activities, some of which are major tourist-generating and revenue-generating events for the City of Huntington Beach.

Additionally, any facilities located on the beach would be subject to site constraints making them technically infeasible including climate change effects like sea level rise and coastal erosion. Temporary and permanent beach-based facilities would also require permits and approvals from land owners like the City of Huntington Beach, which may not be made available thus making the alternatives infeasible.

<u>Biofouling on WWS Surface</u>

If using passive WWS, copper nickel would be the preferred alternative in order to reduce the need for offshore trips to manually clean the screens of biofouling. As with the airbursting discussion above, a passive WWS constructed of super duplex stainless steel was evaluated in the 2017 FSEIR since it is considered the worst-case scenario due to GHG and criteria air pollutant emissions associated with offshore boat trips required for biofouling cleaning.

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Regarding active rotating WWS, there are no performance data on their use in an offshore marine application. For this reason, Poseidon considers the installation of offshore mechanical equipment a substantial operational risk. This concern is shared by Poseidon's preferred O&M provider for the HBDP (Acciona) and was documented in the Response to Comments in the 2017 FSEIR (Part II.7, page II-555). Pending the results of the two-year WWS pilot study being conducted for the Carlsbad Desalination Plant (during which the reliability of active rotating WWS will be evaluated), Poseidon may be willing to reconsider the potential use of active rotating WWS for HBDP.

Lastly, staff asked if active rotating WWS can be fabricated of copper nickel. Although the answer is yes, since both copper nickel and actively rotating WWS are solutions for managing biofouling on the screen surface, typically only one is selected. Therefore, if copper nickel is used, an active rotating WWS would not be needed. Conversely, if active rotating WWS are used, copper nickel would not be needed.

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Part III

Response to the Regional Board's March 11th Additional Comments regarding Feasibility of an alternative site for 1-mm wedge wire screened ocean intake

1. ***Describe how the discharger proposes to clean/control biofouling in the existing intake pipeline or new intake pipeline (with an internal diameter of 12 ft or less) as a stand-alone facility.***

Response 1:

Cleaning of the existing intake system: The existing intake system is composed of a 14-foot diameter concrete pipeline (approximately 2400 feet in length) which connects the existing offshore intake concrete structure to the forebay located on the power generating facility site. The forebay is connected to the existing intake pump station via a 14 foot x 11 foot box tunnel which is approximately 350 feet long. The new intake pump station proposed by Poseidon for the desalination facility would tie into the wet well of the existing intake pump station. A condition assessment was performed on the existing intake system in 2018. The condition assessment found that along the 2400 foot long, 14 foot diameter pipeline there was a consistent layer of marine growth that was approximately 3 inches thick (Attachment I). In the onshore intake system, the condition assessment found that marine growth ranged from 2 inches to 8 inches thick. Based on the information obtained from the condition assessment, Poseidon does not believe that cleaning the 2400 foot long, 14 foot diameter pipeline will be necessary as the existing power generating facility has been operating for a number of years at intake flows approximating the flows that would be required by the desalination facility and that the thickness identified in the condition assessment represents a steady state condition. For the onshore portion of the intake system, Poseidon has assumed that cleaning would be required during the O&M phase of the project based on information from inspections that would be performed on regular intervals. The cleaning would be performed manually by divers.

Cleaning of the new intake system: The new intake system that was proposed by Poseidon to connect to the location D2 or U2, consisted of a 14 foot diameter pipeline along the beach alignment and a 12 foot diameter pipeline that would transition from the beach through the surf zone terminating at the new location D2 or U2. For the 14 foot diameter pipeline along the beach, Poseidon had assumed that the marine growth on the new pipeline would be comparable to the marine growth observed on the existing 14 foot pipeline and that pigging would not be required. For the 12 foot diameter section of the new intake system, information is not available on how much marine growth could be expected and whether pigging would be required. Therefore, it was assumed that the system would have to be designed to allow pigging to be performed if, over the design life of the desalination facility, a layer of marine growth developed beyond what was accounted for in the design for NPSH calculations. The conceptual design incorporated a concrete vault at the right-angle turn where the 14 foot diameter pipeline

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connects to the 12 foot diameter pipeline. The concrete vault on the beach would be used to launch a 12 foot diameter pig which would be retrieved offshore through the new concrete structure that would be constructed at location D2 or U2. When Poseidon proposed the alignment for the alternate location, Poseidon advised that the complexities associated with pigging the large diameter pipeline were not fully addressed and were not included in the estimated cost. Such complexities would include sizing of the concrete vaults required for launching and receiving the pig, provisions for turbidity control in the open ocean and pumping requirements to propel such a large pig. Poseidon recognizes the difficulty in pigging a large diameter pipeline such as a 12-foot diameter pipe. Based on discussions with companies specializing in pigging pipes, Poseidon has confirmed that a pig of the required size could be fabricated for this application but solutions to address the technical challenges listed above (launching/retrieving in the open ocean, turbidity control, and pumping requirements to propel the pig have not been fully developed at this conceptual phase and would require a risk contingency for the O&M phase.

2. ***Provide a facility layout depicting the proposed location of Poseidon's pump station within the existing HBGS facility and include the associated piping of the pump station (include valving, pipe diameters, and other feature of the suction line) all the way to the existing intake pump structure or to the location where the suction line connects to the existing HBGS' intake structure.***

Response 2: The following drawings have been provided in Attachment K:

1. Figure 1 - HB Desalination Plant Model, Underground Piping, IPS Area
2. Figure 2 – 3D Model, Piping New IPS to Existing IPS
3. Figure 3 – AES Facility Circulating Water System Schematic
4. Figure 4 – New Intake Pump Station, Preliminary Drawing
5. Figure 5 – AES Facility Existing Screen Well Cross Section
6. Figure 6 – AES Facility Existing Screen Well Plan View
7. Figure 7 – AES Facility Existing Offshore Concrete Structure
8. Figure 8 – New Pipeline Alignments (offshore and onshore alignments)
9. Figure 9 – WWS Screen Arrangement
10. Figure 10 – Tide Datum Chart, Newport Beach, Newport Bay Entrance

3. ***Calculate the NPSH for an intake pipeline with a smaller internal diameter than 12 feet (i.e. 8 feet). Include for the 12'-14'diameter pipeline configurations, and for smaller diameter intake pipeline configurations, pertinent details regarding the calculations, equations and parameters used and include their corresponding units (i.e. friction***

coefficients, pipe friction loss per 100 feet, pipeline material, specific gravity, vapor pressure, etc.) and cite the sources of data used.

Response 3: NPSH calculations for pipeline diameters smaller than 12 feet is an arbitrary exercise as the technical investigation as confirmed that a minimum 12-foot diameter pipe is required to reliably provide feedwater to the desalination facility. This level of detail for an alternative also fails to provide information necessary to overcome the fact that moving the 1-mm wedge wire screened intake to alternative site D2/U2 cannot be accomplished in a successful manner in a reasonable period of time taking into account technical, social, economic and environmental considerations.

Nonetheless, below is the information requested by the RWB staff.

Head loss calculations for the intake system from the WWS to the intake bell of the of the new intake pump station pumps are provided for the pipeline alignments that were presented in Response 12 in the February 4, 2019 response to Water Board questions. Please see Attachment L for calculations. For each scenario presented below, a new concrete structure of similar dimensions to the existing concrete structure and the WWS assembly as described in the 2017 FSEIR is assumed to be present at the alternate location D2/U2. The calculated head loss for the scenarios evaluated was compared to the maximum allowable head loss which is the head loss determined by the designer of the new intake pump station for the desalination plant to maintain the minimum required water level for the new intake pumps at the lowest tide level.

The scenarios that were analyzed are as follows:

1. Connection from the existing intake concrete structure at location E to alternate location D2/U2 via a new 12 diameter pipeline (6,474 feet in length). The head loss calculated for this scenario was below the maximum allowable head loss.
2. Connection from the existing intake concrete structure at location E to alternate location D2/U2 via a new 10 diameter pipeline (6,474 feet in length). The head loss calculated for this scenario was over the maximum allowable head loss.
3. Connection to the existing 14 feet diameter pipeline (640 feet from the existing power station forebay) to alternate location D2/U2. This scenario consists of a new 14 feet diameter onshore pipeline (5,917 feet long) and a new offshore 12 feet diameter pipeline (2,598 feet long). The head loss calculated for this scenario was below the maximum allowable head loss.
4. Connection to the existing 14 feet diameter pipeline (640 feet from the existing power station forebay) to alternate location D2/U2. This scenario consists of a new 12 feet diameter onshore pipeline (5,917 feet long) and a new offshore 12 feet diameter pipeline (2,598 feet long). The head loss calculated for this scenario was over the maximum allowable head loss.
5. Connection to the existing 14 feet diameter pipeline (640 feet from the existing power station forebay) to alternate location D2/U2. This scenario consists of a new 12 feet diameter onshore pipeline (5,917 feet long) and a new offshore 10 feet diameter pipeline (2,598 feet long). The head loss calculated for this scenario was over the maximum allowable head loss.

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The calculation sheets presented for the scenarios provide details of the equations used, coefficients that were assumed, assumptions for marine growth and bottom sediments and the datum for elevations. Drawings were also presented (see response to Question No. 2) which illustrate pipe diameters and other features considered in the calculations. The drawings are annotated with identification numbers that correspond with the calculations. Estimates for marine growth and sediments in the existing infrastructure were taken from a condition assessment of the existing infrastructure conducted in 2018 (and is provided as part of the response for question 6).

4. ***Provide documentation regarding any other intake pipeline configurations that were considered but have not been presented to Regional Water Board staff, and why those alternate intake pipeline configurations were considered infeasible.***

Response 4: Poseidon has evaluated the following intake pipeline configurations:

a. Utilization of existing infrastructure as described in the 2017 FEIR. This approach is considered feasible.
b. Connection between location E to location D2/U2 using a large diameter pipeline placed on the seabed. This approach was determined to be infeasible. Refer to Part II (Response to the Regional Board's February 27th Additional Comments regarding Feasibility of an alternative site for 1-mm wedge wire screened ocean intake), Response 6.
c. Connection between location E to location D2/U2 using a large diameter pipeline placed beneath the seabed. This approach was determined to be infeasible. Refer to Part II (Response to the Regional Board's February 27th Additional Comments regarding Feasibility of an alternative site for 1-mm wedge wire screened ocean intake), Response 7.
d. Connection between the existing 14 feet diameter intake pipeline on the State Beach to location D2/U2 using a buried, large diameter pipeline along the beach parallel to shore and then from the shore to the new intake location perpendicular to the shore. This approach is considered infeasible due to cost and schedule impacts but if the utilization of location E is precluded by the Regional Water Board, this approach represents the only other practical option.
e. Connection between the Power Generating Station forebay to location D2/U2 using trenchless methods. This approach was determined to be infeasible. Refer to Part II (Response to the Regional Board's February 27th Additional Comments regarding Feasibility of an alternative site for 1-mm wedge wire screened ocean intake), Response 7.

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5. ***Provide additional justification for why a 12-14' diameter pipe is needed for a new intake. Water Board staff have looked at a number of different desalination facilities that are withdrawing similar or greater volumes of saltwater than the proposed HBDP and use one or more pipes that are less than 8' in diameter.***

Response 5: Poseidon's sizing of the piping to connect the existing infrastructure to alternate location D2/U2 was based on developing a solution that would maintain the combined head loss of the entire intake system at a value less than the maximum allowable head loss. The maximum allowable head loss was determined by the designer of the new intake pump station for the desalination plant to maintain the minimum required water level for the new intake pumps at the lowest tide level. The calculations and assumptions are presented in response No. 3 (March 11, 2019 questions), drawings of the existing and proposed infrastructure are presented in response No. 2 (March 11, 2019). The condition of the existing infrastructure including marine growth and sediments is described in response No. 6 (March 11, 2019). Information on how the proposed alternate alignments were selected and the rationale for the assumed thickness of marine growth is presented in response No. 6 (February 26, 2019).
Calculations for various new pipe sizes were presented in response No. 3 (March 11, 2019). Once a pipe size was reached that exceeded the allowable head loss requirement, additional calculations using smaller pipe sizes were not performed.

Furthermore, detailed investigation into multiple pipeline configurations or pipeline diameters smaller than 12 feet is an arbitrary exercise as this consideration fails to overcome the fact that moving the 1-mm wedge wire screened intake to alternative site D2/U2 cannot be accomplished in a successful manner in a reasonable period of time taking into account technical, social, economic and environmental considerations.

6. ***Provide a facility condition assessment report for the existing infrastructure at the AES HBGS that Poseidon intends to use for their proposed operations and include details regarding the remaining use time (years) of those facilities. This report must be prepared by an experienced contractor in the field and signed by a Professional Civil Engineer (include references for the contractor).***

Response 6: Poseidon conducted a condition assessment of the existing intake and discharge infrastructure in February 2018 during a scheduled shutdown of the AES power generating facility. The condition assessment included the use of divers to inspect onshore intake facilities, and the existing offshore concrete structures (intake and discharge) and a remotely-operated underwater vehicle (ROV) to inspect the entire length of the offshore intake and discharge pipeline. Concrete core samples were taken from the offshore concrete structures and from the large diameter concrete pipelines.

The field work was supervised by a California P.E. from GHD and the condition assessment report was signed by the same California registered P.E. Concrete samples were analyzed by Tourney Consulting Group LLC. The diving was performed by Subsea Global Solutions – the same company that has conducted the inspection of the same infrastructure for the AES power generating facility for past several years. The ROV inspection was performed by Hibbard Inshore, LLC under contract to Subsea Global Solutions. The condition assessment, which includes a report prepared by GHD and a report prepared by Subsea Global Solutions is provided as part of Attachment I.

7. ***Please provide justification for why an onshore air burst system, either for the existing intake or at one of the alternative intake locations, is considered necessary when the State Land Commission's October 2017 Final SEIR included removal of the option of using an onshore air burst system from the draft SEIR (https://www.slc.ca.gov/wp-content/uploads/2018/09/PartIII.pdf) as follows:***

Page 2-31, Table 2-6

Option 2: Onshore Air Burst System

…The air burst equipment is typically installed onshore and a small diameter pipe is used to deliver the high pressure air to the screens offshore. Air bursting is typically limited to shoreline and nearshore screen installations and ~~due to the distance of the proposed screens from shore (1,650 feet) it~~ is not considered by Poseidon for the proposed Lease Modification Project ~~a primary alternative at this time~~.

Page 4-55, Lines 27-31:

27 *Maintenance*

28 Maintenance of the wedgewire screened intake would entail~~: 1)~~ periodic manual

29 cleaning by divers (most likely occurring during regularly-scheduled inspections)~~, 2) an~~

30 ~~onshore-based air burst system, or 3)~~ and a boat-based air-burst system (see Section

31 2.4.6.2, *Screen Maintenance*).

Response 7: A boat-based airburst system was evaluated in the 2017 FSEIR because it is considered the worst-case scenario due to the GHG and criteria air pollutant emissions associated with offshore boat trips. A shore-based airburst system, however,

would reduce the impact (lower GHG and criteria air pollutant emissions) and, if available at the time of construction, it would be the preferred alternative.

The State Lands Commission's 2017 FSEIR evaluated a boat-based airburst approach because it was considered by Poseidon to be the most environmentally impactful alternative. Both the boat-based airburst approach and shore-based airburst approach would require energy to operate the air burst equipment. However, the boat-based approach would result in additional GHG and criteria air pollutant emissions associated with boat operations and worker trips. Boat-based air bursting would occur 6 times per year, with a 1-hour, one-way vessel trip. These emissions were captured in the 2017 FSEIR.

An airburst system located on the plant site would be approximately 1.5 miles away from the 1-mm screens if located at U2 or D2 and would be too far to reliably perform. This is not the case for locating the intake at the proposed site (i.e., station E). A shore-based air burst system could be located on the plant site and effectively clean screens given the close proximity to the proposed intake at site E.

If the 1-mm screened ocean intake is located at site D2 or U2 then the decision to utilize an onshore airburst system would require that the facilities be placed on the beach in order to be close enough to the WWSs to be effective. Construction of onshore airburst system facilities on the beach could cost $6-7 million more as compared to constructing an airburst system located on the plant site.

The decision to utilize an onshore or offshore airburst system will be determined during the final design phase of the Project and informed by best industry practices and in compliance with the environmental analysis in the SLC FSEIR. While Poseidon does not consider an onshore airburst system to be essential, it may prove to be the technically superior option and would reduce environmental impacts associated with GHG and criteria air pollutant emissions while avoiding uncertain ocean conditions affecting boat-based maintenance.

However, for the purposes of the 1-mm wedgewire screen intake alterative site analysis, the RWB may choose to assume a boat-based airburst system would be used for all alternative locations. This assumption would reduce costs for the alternative intake sites D2/U2 by $6-7 million and minimize social and recreational beach impacts related to the air burst system, but does not change the underlining finding that there is no scientifically justified environmental benefit from re-siting the 1-mm wedgewire screen ocean intake to D2/U2 and this alternative cannot be accomplished in a successful manner in a reasonable period of time taking into account technical, social, environmental and economic consideration.

8. ***Response 3:***
    a. ***Provide additional information as to why placing an intake at sites D2 or U2 is estimated as costing $215 (reduced to $197) million.***

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Response 8a: Please see detail of the cost components required to place the intake at D2 or U2 included for the Alternative Intake (U2/D2) on the Direct Capital Cost Detail (RWB) tab.

> ***b. The total cost of the intake at station E is listed as $93 million, while an intake at either station D2 or U2 is listed as costing $474 million; please provide an explanation for this large difference in cost.***

Response 8b: Please see detail of the cost components included for the Alternative Intake (U2/D2) on the Direct Capital Cost Detail (RWB) tab; the primary differences include constructing the pipeline, trestle and associated infrastructure for the alternative intakes at U2/D2 instead of utilizing the existing structures at E.

> ***c. Please provide additional information regarding the proposed 6-year construction time frame for placing the intake at station D2 or U2.***

Response 8c: The anticipated construction schedule for the installation of an alternative intake at the D2 or U2 sites is shown in Table 1. The preliminary schedule would commence after permits and approvals are granted. The schedule includes the timing for the onshore construction as well as the offshore work. The offshore work requires the use of vibratory hammers and limited impact pile driving equipment, and therefore the work has been restricted to comply with the whale migration requirements as described in the 2017 FSEIR – June to November. The schedule also includes an 8-month schedule contingency to allow for unknowns and the risk of going past a whale migration work restriction season which would force work to stop until the next allowable work season. Additionally, onshore construction activities would not occur from mid-April to late September, because California least tern might be present in the vicinity of construction activities during these times.

The schedule also illustrates the relationship between the onshore desalination plant construction and construction of the new D2 intake system. The commissioning of the desalination plant can't begin until the intake system is installed. The removal of the trestle can, however, extend past the start of plant commissioning.

Table 1
Construction Schedule

Phase	Duration (months)	Start	End
Onshore: mobilization	1	Year 1 – Q1	Year 1 – Q1
Onshore: coffer dam construction and tie-in to existing 14 foot pipe	2	Year 1 – Q1	Year 1 – Q1

Table 1
Construction Schedule

Phase	Duration (months)	Start	End
Onshore: placement of sheet piles, excavation, pipe laying, backfill	10	Year 1 –Q4 Year 2 – Q4	Year 2 – Q1 Year 3 – Q1
Offshore: mobilization	1	Year 1 – Q2	Year 1 – Q2
Offshore: trestle construction	9	Year 1 – Q3 Year 2 – Q3 Year 3 – Q2	Year 1 – Q4 Year 2– Q4 Year 3 – Q2
Onshore: construction of vault at pipe turn	1	Year 3 – Q1	Year 3 – Q1
Onshore: construction of manholes	2	Year 3 – Q1	Year 3 – Q1
Onshore: construction of 2,400 foot turn to trestle (placement of sheet piles, excavation, pipe laying, backfill)	4	Year 3 – Q4	Year 4 – Q1
Onshore: demobilization	2	Year 4 – Q1	Year 4 – Q1
Offshore: construction of concrete tower, leveling piles	6	Year 3 – Q2 Year 4 – Q2	Year 3– Q3 Year 4 – Q3
Offshore: disassemble trestle	10	Year 4 – Q2 Year 5 – Q1 Year 6 – Q1	Year 4 – Q3 Year 5 – Q2 Year 6 – Q2
Offshore: set WWS and diffuser	2	Year 4 – Q3	Year 4 – Q3
Schedule contingency	6	Year 5– Q1	Year 5 – Q2
Offshore: demobilization	2	Year 6 – Q2	Year 6 – Q2

The construction, demobilization, startup, and commissioning of the onshore desalination plant would take approximately 39 months. Commissioning would take 6 months and performance testing would take 1 month of the 39 month period. The commissioning of the desalination plant cannot begin until the intake system is installed. The removal of the trestle can, however, extend past the start of plant commissioning.

Based on these estimates, and depending on the project development schedule variables, implementation of an alternative intake at either the U2 or D2 sites would require 10 to 13 years to complete, including five years for permitting and environmental clearances, and six to eight years for construction. In comparison, construction, startup, and commissioning of the desalination plant and intake E system would take

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approximately 39 months to complete and permitting would take approximately 15 months to complete, for a total of 4.5 years.

9. ***Response 10:***
    a. ***The draft response does not appear to be responsive to staff's information request. Please provide more detail other than citing the SLC Final SEIR.***

Response 9:

Intake Site E: Operation of the intake at site E would not require construction on the Huntington State Beach (State Beach). Onshore construction associated with intake site E would be confined to the existing Huntington Beach Generating Station (HBGS) footprint, and areas associated with offsite work (product water transmission pipeline and product water underground booster pump station). These areas are inland of Pacific Coast Highway and not within the State Beach. Therefore, onshore construction and operation associated with intake site E is not anticipated to result in recreation impacts. Offshore construction would be required for the diffuser and WWS installation as part of the operation at intake site E. Diffuser installation would be confined to the area directly above the existing discharge tower, located approximately 1,500 feet offshore. WWS installation work would be conducted from a derrick barge moored above the existing intake tower and would be confined to the area directly surrounding the tower, located approximately 1,650 feet offshore. Offshore construction would not occur within the State Beach. Therefore, operation and construction of the intake at Site E would avoid recreational impacts as it would utilize the existing intake infrastructure.

Intake Site U2/D2: As described in more detail in Response #6 to the February 27th comment above, construction of an intake at U2 or D2 would entail both a land-based pipeline and an offshore pipeline. The land-based reach of the pipeline would tie-in to the existing 14 foot diameter concrete intake pipeline on the State Beach and would traverse the State Beach parallel to the shore using open-cut construction techniques. Loss of beach access and usage would occur within the construction area by limiting or precluding access to the beach and shoreline in the onshore and offshore construction areas. Shoreline access restrictions may occur due to the presence of the trestle structure, and diversion of vehicular, bicycle and pedestrian traffic from the construction and staging areas may also occur. In addition to physical displacement of recreation opportunities within the construction zones, the placement of the construction areas could restrict access and circulation in and around the State Beach area. Furthermore, construction noise in the vicinity of beach areas could disrupt use of these areas by the public due to nuisance noise issues. Installation of sheet pile and trestle construction could have adverse effects on the shore break, affecting recreational activities, including surfing, which is an important recreational asset.

In addition, annual events would occur in the vicinity of or within the footprint of the construction area. The Great Pacific Airshow occurs in October and takes place in Huntington City Beach from Seapoint to Beach Boulevard. Additionally, the U.S. Open of

Surfing takes place at the Huntington Beach Pier and occurs in late July to early August. Both of these events would occur during onshore and offshore construction activities. Onshore and offshore construction could create visual or noise impacts to these events. The construction footprint of intake site U2 may overlap with the U.S. Open, which could limit the event area or create hazards to U.S. Open participants or construction workers.

Onshore and offshore construction of intake site U2 would occur in proximity to the Huntington Beach Pier. Construction activities would create noise and visual impacts which would affect visitor experience, and could result in a decline in tourism in the area.

Construction of either the D2 or U2 alternative intake system would limit beach access during onshore activities. A work area of approximately 100-feet wide by 1000-feet long would include shoring, excavation, pipe laying, backfill, and shoring removal operations. A staging area would be located in a footprint of approximately four acres on the beach adjacent to the work area. The staging area would stage pipe, bedding materials, and shoring materials. These areas would be fenced off to prevent public access. Onshore activities would occur over a 1 year and 8-month period.

Construction of either the D2 or U2 alternative intake system would also limit offshore access during offshore construction activities and during the presence of the construction trestle system. The trestle would be 1,900 feet in length and would be an open structure with steel pipe piles, cap beams, and rail beams to carry gantry-mounted equipment. Although construction activities would only occur outside of the work restriction season, the trestle would remain in place during the entirety of the offshore construction period (except during mobilization and demobilization). Therefore, the trestle would be present for 6 years, although disassembly of the trestle would occur during the last 3 years of this period.

10. Appendix B

a. As mentioned on the phone call with Poseidon, Water Board staff are still unclear how line 12 in Table 1 is calculated. Please provide additional clarification as to why the Direct Capital Cost is $3 million for the existing intake at station E and $38 million for an intake at alternative stations U2/D2.

Response 10a: Line 13 (in the revised Table 1 prepared 3/22/19, updated from the original Line 12) calculates cost escalation between 2018 and the Year of Financial Closing for each intake cost estimate. Specifically for each intake cost estimate we assumed:

- **Proposed Intake E:** Direct Capital Cost in 2018$ ($58.4m) escalated at a compound rate of ***2.5%*** per year through 2020, an additional $3.0m for a total of $61.4m in 2020$

- **Alternative Intake U2/D2:** 2018 Direct Capital Cost ($197.4m) escalated at a compound rate of ***3.0%*** per year through 2024, an additional $38.7m for a total of $236.1m in 2020$

- Higher cost escalation than Proposed Intake E accounts for higher inflation costs expected over period through 2024

b. Lines 4 and 5 in Table 1 show about $50 million of infrastructure costs for station U2/D2 but nothing for E. This discrepancy needs to be explained. Is it the case that all the infrastructure costs at E are covered by the "intake retrofit" line item?

Response 10b: For Proposed Intake E, there is no new infrastructure required as the Project would utilize the existing infrastructure and retrofit it with a new wedgewire screen assembly.

c. Line 15 in Table 1 refers to "capitalized interest during construction." This also needs more explanation as it is unclear why the capitalized interest is 12 times as expensive (over $130 million more) at station U2/D2 compared to E.

Response 10c: The cost of Capitalized Interest During Construction is a function of the level of capital that is required to be financed over the period, as the total capital costs of Alternative U2/D2 is greater than Proposed Intake E, the level of Capitalized Interest is greater. Additionally the following assumptions drive differences in the level of Capitalized Interest required for each intake cost estimate:

- **Proposed Intake E:**
  - **Debt Portion of Intake Costs:** $76m
  - **Construction Period:** 3.25 years
  - **Interest Rate:** 4.75%

- **Alternative Intake U2/D2:**
  - **Debt Portion of Intake Costs:** $389m
  - **Construction Period:** 6 years
    - *Requires 6 years of Capitalized Interest during Construction*
  - **Interest Rate:** 6.0%
    - *Higher interest rate due to financing occurring in the future in an anticipated higher interest rate environment*

d. Table 1 also indicates that costs for station E are in 2020 dollars but station U2/D2 estimates are in 2024 dollars. It is unclear how inflation is calculated or how the estimate for 2024 dollars is calculated. To provide an apples-to-apples comparison, it would be helpful to have all estimates either in terms of 2020 dollars or 2024 dollars.

Response 10d: Table 1 shows the Direct Capital Costs in 2018 dollars for both the Proposed Intake E and Alternative Intake (U2/D2) for comparison purposes. As the

Proposed Intake E and Alternative Intake (U2/D2) have different development and construction schedules which require different timeframes for cost escalation, development, permitting and construction, it is not possible to show the Total Intake Cost Estimate in the same year. For the avoidance of doubt, please note the levels of escalation costs, development costs and capitalized interest inherently cover different periods of time so the Total Intake Cost Estimate cannot be reflected in the same year.

11. Comments on Cost Comparison Spreadsheet

a. Footnote #4 on the Intake Comparison Summary tab in the Excel spreadsheet, refers to line items 57-66 in the Direct Capital Costs Detail tab as the basis for the estimate of approximately $24,313,000 for installation of the WWS assembly. This estimate includes the cost of pick-ups (351/week – total of $229,203), miscellaneous outside equipment rental (1/ls – what is "ls"?, total of $675,000), crew consumables (127,410 man hours – total of $1,274,100), and installation of an onshore air burst system (total, including an onshore building, compressors/air receivers, electrical services, air piping, and land lease costs for $17,624,546), for a total of $24,312,849, which agrees with the rounded estimate in the Intake Comparison Summary tab. However, we request that the following be addressed:

i. The costs for pick-ups (line 57), miscellaneous outside equipment rental (line 58), and crew consumables (line 60) should apply to the entire project, not just the WWS assembly.

Response 11a i: Yes we agree that these standard construction items shown in line 57:59 of the 'Direct Capital Costs (RWB)' tab could be classified as "Other Construction Costs" allocable to the entire project. Please see Reference Line 7 added in the revised table addressing this.

ii. The costs assume that an on-shore air burst system is needed. Please also include costs if instead, manual cleaning by divers or a boat-based airburst system are used to clean and maintain the WWS, especially in light of Poseidon's proposal to use Cu/Ni wedge-wire screens instead of stainless steel

Response 11a ii: Please see response to question #13 (February 27th Comments) above regarding the possible consideration of the land-based airburst system. Furthermore, the manual cleaning costs referred to here are O&M costs that were beyond the scope of Table 1.

iii. Please categorize the costs as follows (including equipment, crew consumables, etc.):

1) ***Construction of pipeline from proposed facility to alternative intake location D2 or U2***

Response 11a iii 1): For detail of Direct Capital Costs components please refer to the color coded legend on the 'Direct Capital Costs (RWB)' tab starting in cell A78.

2) ***Construction including mob/demob of the trestle to build the offshore intake at locations D2 or U2***

Response 11a iii 2): For detail of Direct Capital Costs components please refer to the color coded legend on the 'Direct Capital Costs (RWB)' tab starting in cell A78.

3) ***Construction of the WWS assembly separate from the onshore air burst system***

Response 11a iii 3): For detail of Direct Capital Costs components please refer to the color coded legend on the 'Direct Capital Costs (RWB)' tab starting in cell A78.

4) ***Construction of the onshore air burst system***

Response 11a iii 4): For detail of Direct Capital Costs components please refer to the color coded legend on the 'Direct Capital Costs (RWB)' tab starting in cell A78.

iv. ***Clarify as to how the cost of the onshore air burst system placed at locations D2 or U2 would differ from costs incurred by placing an onshore system at the existing intake location***

Response 11a iv 5): Please see response to question #11 ii) (March 11th Comments) above.

b. ***Intake Comparison Summary Tab – Please provide a detailed cost breakdown for the following line items: Development and Construction Costs, Capitalized Interest During Construction, and Financing Fees and Reserves.***

Response 11b: Please refer to the following assumptions used to prepare the costs for each intake cost estimate:

Development and Construction Costs

i. Development Costs: permitting and development costs estimated by Management, additional costs of $40m included for Alternative Intake
ii. Construction Management Costs: costs estimated by Management, additional $3.2m in costs included for Alternative Intake
iii. Property Tax During Construction: 1% tax rate escalated at 2% over construction period
iv. Title Insurance: $0.45 per $1,000 of Total Intake Cost Estimate

Capitalized Interest During Construction

i. See response to third question under Appendix B questions

Financing Fees and Reserves

i. Fees:
   1) Conduit, Rating Agency, Underwriting Equity Letter of Credit and Advisory Fees: 1.7% of Total Capital Cost Estimate

ii. Reserves
   2) Debt Service: 12 months
   3) Working Capital: 0.1% of Total Intake Cost Estimate
   4) O&M Reserve: 0.5% of Total Intake Cost Estimate

c. Lines 68-70, Direct Capital Cost Detail Tab: Please provide detailed, itemized calculations for the Indirect, Insurance and Overhead Costs.

2. **Response 11c:** The cost estimate that was provided was based on a conceptual design and would not be expected to have an accuracy better than a Category IV estimate (ASTM Standard E2516 -11, Standard Classification for Cost Estimate Classification System) which would range from -30% to +50%. The cost estimate that was prepared by Independent Scientific and Technical Advisory Panel (ISTAP) in evaluating the proposed subsurface infiltration gallery (SIG) in 2015 was also prepared at the Category IV level which was appropriate for the conceptual design for the SIG. When preparing an estimate at this level of accuracy, standard allowances are made based on a fixed percentage of the estimated direct cost for general indirect cost. For the estimate prepared by Poseidon for the alternate pipeline alignment, the following general cost categories were estimated using a percentage of direct cost: 1) indirect contractor cost, 2) insurance and environmental compliance, and 3) contractor overhead and profit. After the design advances from the conceptual phase to a more advance level, the general cost categories described above would be evaluated in more detail and refined either up or down. The percentages that were assumed for these categories are listed in the spreadsheet and below:
   a. **Indirect: 25% of Cost Subtotal** (Please reference Line 68 on the 'Direct Capital Costs (RWB)' tab of the updated Attachment B provided)
   b. **Insurance & Environmental: 5% of Cost Subtotal** (Please reference Line 69 on the 'Direct Capital Costs (RWB)' tab of the updated Attachment B provided)
   c. **Overhead and Profit: 25% of Cost Subtotal** (Please reference Line 70 on the 'Direct Capital Costs (RWB)' tab of the updated Attachment B provided)

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ATTACHMENT A

Air Quality and Greenhouse Gas Emissions Calculations

Vessel 1	
Name	Tug Boat
Vessel Type	Tug Boats
Main Engine Power (hp)	750
Engine Power Range (Low)	501
Engine Power Range (High)	750
Main Engine Power (kW)	559.275
Number of Main Engines	1
Auxiliary Engine Power (hp)	100
Engine Power Range (Low)	51
Engine Power Range (High)	120
Auxiliary Engine Power (kW)	74.57
Number of Auxiliary Engines	1
Main Engine Load Factor	0.5
Auxiliary Engine Load Factor	0.31
Main Engine Model Year	2009
Model Year Range (Low)	2009
Model Year Range (High)	2009
Main Engine Age	11
Main Engine Useful Life	21
Auxiliary Engine Model Year	2009
Model Year Range (Low)	2009
Model Year Range (High)	2009
Auxiliary Engine Age	11
Auxiliary Engine Useful Life	23
Brake Specific Fuel Consumption (g/hp-h)	184
Operational Time (hours/day)	3
Work Days (days)	1
Operational Year	2020

Emissions Factors

Vessel	Engines	ROG (g/kW-h)	NO_x (g/kW-h)	CO (g/kW-h)	SO_x (g/hp-h)	PM_{10} (g/kW-h)	$PM_{2.5}$ (g/kW-h)	CO_2 (g/hp-h)
Tug Boat	Main	0.68	5.1	3.73	0.0028	0.15	0.147	545.6
	Auxiliary	1.18	5.32	3.73	0.0028	0.22	0.2156	545.6

Engine Load Factor

	Tug Boat
Main	0.50
Auxiliary	0.31

Engine Useful Life

	Tug Boat
Main	21
Auxiliary	23

Engine Fuel Correction Factor

Vessel	Engines	ROG	NO_x	CO	SO_x	PM_{10}	$PM_{2.5}$	CO_2
Tug Boat	**Main**	1	0.948	1	1	0.8	0.8	1
	Auxiliary	1	0.948	1	1	0.8	0.8	1

Engine Deterioration Factor

Vessel	Engines	ROG	NO_x	CO	SO_x	PM_{10}	$PM_{2.5}$	CO_2
Tug Boat	**Main**	0.44	0.21	0.25	1	0.67	0.67	1
	Auxiliary	0.28	0.14	0.16	1	0.44	0.44	1

Time to and from Port of Lo	Tug Boat
Distance (nautical miles) [a]	16.5
Speed (knots) [b]	12
Time (hours)	1.375
Max. Daily Number of Trips to or from Port of Long Beach	2

a. Distance based on route from Port of Long Beach to 0.5 miles offshore of desalination plant.

b. Assumed that the Santa Barbara Channel Vessel Speed Speed Reduction Trial would apply as slowest speed.

Maximum Daily Emissions From Daily Travel To/From Port

		pounds/day					
Vessel	**Engines**	**ROG**	**NO_x**	**CO**	**SO_x**	**PM_{10}**	**$PM_{2.5}$**
Tug Boat	Main	1.42	9.10	7.15	0.01	0.27	0.27
	Auxiliary	0.19	0.75	0.56	0.00	0.03	0.03
Total		1.61	9.85	7.71	0.01	0.30	0.30

Maximum Annual Emissions From Daily Travel To/From Port

Vessel	**Engines**	**MT/day**
		CO_2
Tug Boat	Main	2.57
	Auxiliary	0.15
Total		2.73

HB Desal - Wedgewire & Diffuser Install (Other Days)

South Coast AQMD Air District, Annual

1.0 Project Characteristics

1.1 Land Usage

Land Uses	Size	Metric	Lot Acreage	Floor Surface Area	Population
User Defined Industrial	1.00	User Defined Unit	0.00	0.00	0

1.2 Other Project Characteristics

Urbanization	Urban	**Wind Speed (m/s)**	2.2	**Precipitation Freq (Days)**	31
Climate Zone	8			**Operational Year**	2021
Utility Company	Southern California Edison				
CO2 Intensity (lb/MWhr)	592.74	**CH4 Intensity (lb/MWhr)**	0.029	**N2O Intensity (lb/MWhr)**	0.006

1.3 User Entered Comments & Non-Default Data

Project Characteristics - Adjusted SCE CO2 Intensity based on 33% RPS by 2020

Land Use - Offshore construction

Construction Phase - Last day of wedgewire screen and diffuser installation construction

Off-road Equipment - Crane on barge - assumed to operate 6 hours per day. Underwater rivet busters and diamond saws assumed to be electrically powered.

Trips and VMT - 8 one-way worker trips (4 round trips assuming each worker would drive own vehicle)

Table Name	Column Name	Default Value	New Value
tblConstructionPhase	NumDays	0.00	1.00

tblOffRoadEquipment	OffRoadEquipmentUnitAmount	1.00	0.00
tblOffRoadEquipment	OffRoadEquipmentUnitAmount	2.00	0.00
tblOffRoadEquipment	OffRoadEquipmentUnitAmount	2.00	0.00
tblProjectCharacteristics	CO2IntensityFactor	702.44	592.74
tblTripsAndVMT	HaulingTripNumber	0.00	20.00
tblTripsAndVMT	WorkerTripNumber	0.00	8.00

2.0 Emissions Summary

2.1 Overall Construction

Unmitigated Construction

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Year	tons/yr										MT/yr					
2020	9.0000e-005	2.8200e-003	7.1000e-004	1.0000e-005	2.2000e-004	1.0000e-005	2.2000e-004	6.0000e-005	1.0000e-005	7.0000e-005	0.0000	0.7942	0.7942	5.0000e-005	0.0000	0.7955
Maximum	**9.0000e-005**	**2.8200e-003**	**7.1000e-004**	**1.0000e-005**	**2.2000e-004**	**1.0000e-005**	**2.2000e-004**	**6.0000e-005**	**1.0000e-005**	**7.0000e-005**	**0.0000**	**0.7942**	**0.7942**	**5.0000e-005**	**0.0000**	**0.7955**

Mitigated Construction

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Year	tons/yr										MT/yr					
2020	9.0000e-005	2.8200e-003	7.1000e-004	1.0000e-005	2.2000e-004	1.0000e-005	2.2000e-004	6.0000e-005	1.0000e-005	7.0000e-005	0.0000	0.7942	0.7942	5.0000e-005	0.0000	0.7955
Maximum	**9.0000e-005**	**2.8200e-003**	**7.1000e-004**	**1.0000e-005**	**2.2000e-004**	**1.0000e-005**	**2.2000e-004**	**6.0000e-005**	**1.0000e-005**	**7.0000e-005**	**0.0000**	**0.7942**	**0.7942**	**5.0000e-005**	**0.0000**	**0.7955**

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio-CO2	Total CO2	CH4	N20	CO2e
Percent Reduction	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Quarter	Start Date	End Date	Maximum Unmitigated ROG + NOX (tons/quarter)	Maximum Mitigated ROG + NOX (tons/quarter)
1	**7-16-2020**	**9-30-2020**	0.0020	0.0020
		Highest	0.0020	0.0020

3.0 Construction Detail

Construction Phase

Phase Number	Phase Name	Phase Type	Start Date	End Date	Num Days Week	Num Days	Phase Description
1	Building Construction	Building Construction	7/16/2020	7/16/2020	5	1	Screen/Diffuser Construction

Acres of Grading (Site Preparation Phase): 0

Acres of Grading (Grading Phase): 0

Acres of Paving: 0

Residential Indoor: 0; Residential Outdoor: 0; Non-Residential Indoor: 0; Non-Residential Outdoor: 0; Striped Parking Area: 0

OffRoad Equipment

Phase Name	Offroad Equipment Type	Amount	Usage Hours	Horse Power	Load Factor
Building Construction	Cranes	0	4.00	231	0.29
Building Construction	Forklifts	0	6.00	89	0.20
Building Construction	Tractors/Loaders/Backhoes	0	8.00	97	0.37

Trips and VMT

Phase Name	Offroad Equipment Count	Worker Trip Number	Vendor Trip Number	Hauling Trip Number	Worker Trip Length	Vendor Trip Length	Hauling Trip Length	Worker Vehicle Class	Vendor Vehicle Class	Hauling Vehicle Class

Building Construction	0	8.00	0.00	20.00	14.70	6.90	20.00	LD_Mix	HDT_Mix	HHDT

3.1 Mitigation Measures Construction

3.2 Building Construction - 2020

Unmitigated Construction On-Site

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Category	tons/yr										MT/yr					
Off-Road	0.0000	0.0000	0.0000	0.0000		0.0000	0.0000		0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Total	**0.0000**	**0.0000**	**0.0000**	**0.0000**		**0.0000**	**0.0000**		**0.0000**	**0.0000**	**0.0000**	**0.0000**	**0.0000**	**0.0000**	**0.0000**	**0.0000**

Unmitigated Construction Off-Site

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Category	tons/yr										MT/yr					
Hauling	8.0000e-005	2.8100e-003	5.6000e-004	1.0000e-005	1.7000e-004	1.0000e-005	1.8000e-004	5.0000e-005	1.0000e-005	6.0000e-005	0.0000	0.7546	0.7546	5.0000e-005	0.0000	0.7559
Vendor	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Worker	2.0000e-005	1.0000e-005	1.5000e-004	0.0000	4.0000e-005	0.0000	4.0000e-005	1.0000e-005	0.0000	1.0000e-005	0.0000	0.0395	0.0395	0.0000	0.0000	0.0395
Total	**1.0000e-004**	**2.8200e-003**	**7.1000e-004**	**1.0000e-005**	**2.1000e-004**	**1.0000e-005**	**2.2000e-004**	**6.0000e-005**	**1.0000e-005**	**7.0000e-005**	**0.0000**	**0.7942**	**0.7942**	**5.0000e-005**	**0.0000**	**0.7955**

Mitigated Construction On-Site

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Category	tons/yr										MT/yr					
Off-Road	0.0000	0.0000	0.0000	0.0000		0.0000	0.0000		0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Total	**0.0000**	**0.0000**	**0.0000**	**0.0000**		**0.0000**	**0.0000**		**0.0000**	**0.0000**	**0.0000**	**0.0000**	**0.0000**	**0.0000**	**0.0000**	**0.0000**

Mitigated Construction Off-Site

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Category	tons/yr										MT/yr					
Hauling	8.0000e-005	2.8100e-003	5.6000e-004	1.0000e-005	1.7000e-004	1.0000e-005	1.8000e-004	5.0000e-005	1.0000e-005	6.0000e-005	0.0000	0.7546	0.7546	5.0000e-005	0.0000	0.7559
Vendor	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Worker	2.0000e-005	1.0000e-005	1.5000e-004	0.0000	4.0000e-005	0.0000	4.0000e-005	1.0000e-005	0.0000	1.0000e-005	0.0000	0.0395	0.0395	0.0000	0.0000	0.0395
Total	**1.0000e-004**	**2.8200e-003**	**7.1000e-004**	**1.0000e-005**	**2.1000e-004**	**1.0000e-005**	**2.2000e-004**	**6.0000e-005**	**1.0000e-005**	**7.0000e-005**	**0.0000**	**0.7942**	**0.7942**	**5.0000e-005**	**0.0000**	**0.7955**

HB Desal - Wedgewire & Diffuser Install (Other Days)

South Coast AQMD Air District, Summer

1.0 Project Characteristics

1.1 Land Usage

Land Uses	Size	Metric	Lot Acreage	Floor Surface Area	Population
User Defined Industrial	1.00	User Defined Unit	0.00	0.00	0

1.2 Other Project Characteristics

Urbanization	Urban	**Wind Speed (m/s)**	2.2	**Precipitation Freq (Days)**	31
Climate Zone	8			**Operational Year**	2021
Utility Company	Southern California Edison				
CO2 Intensity (lb/MWhr)	592.74	**CH4 Intensity (lb/MWhr)**	0.029	**N2O Intensity (lb/MWhr)**	0.006

1.3 User Entered Comments & Non-Default Data

Project Characteristics - Adjusted SCE CO2 Intensity based on 33% RPS by 2020

Land Use - Offshore construction

Construction Phase - Last day of wedgewire screen and diffuser installation construction

Off-road Equipment - Crane on barge - assumed to operate 6 hours per day. Underwater rivet busters and diamond saws assumed to be electrically powered.

Trips and VMT - 8 one-way worker trips (4 round trips assuming each worker would drive own vehicle)

Table Name	Column Name	Default Value	New Value
tblConstructionPhase	NumDays	0.00	1.00
tblOffRoadEquipment	OffRoadEquipmentUnitAmount	1.00	0.00

tblOffRoadEquipment	OffRoadEquipmentUnitAmount	2.00	0.00
tblOffRoadEquipment	OffRoadEquipmentUnitAmount	2.00	0.00
tblProjectCharacteristics	CO2IntensityFactor	702.44	592.74
tblTripsAndVMT	HaulingTripNumber	0.00	20.00
tblTripsAndVMT	WorkerTripNumber	0.00	8.00

2.0 Emissions Summary

2.1 Overall Construction (Maximum Daily Emission)

Unmitigated Construction

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Year	lb/day										lb/day					
2020	0.1881	5.4672	1.4100	0.0164	0.4389	0.0182	0.4571	0.1195	0.0174	0.1369	0.0000	1,768.2118	1,768.2118	0.1151	0.0000	1,771.0904
Maximum	**0.1881**	**5.4672**	**1.4100**	**0.0164**	**0.4389**	**0.0182**	**0.4571**	**0.1195**	**0.0174**	**0.1369**	**0.0000**	**1,768.2118**	**1,768.2118**	**0.1151**	**0.0000**	**1,771.0904**

Mitigated Construction

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Year	lb/day										lb/day					
2020	0.1881	5.4672	1.4100	0.0164	0.4389	0.0182	0.4571	0.1195	0.0174	0.1369	0.0000	1,768.2118	1,768.2118	0.1151	0.0000	1,771.0904
Maximum	**0.1881**	**5.4672**	**1.4100**	**0.0164**	**0.4389**	**0.0182**	**0.4571**	**0.1195**	**0.0174**	**0.1369**	**0.0000**	**1,768.2118**	**1,768.2118**	**0.1151**	**0.0000**	**1,771.0904**

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio-CO2	Total CO2	CH4	N20	CO2e
Percent Reduction	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

3.0 Construction Detail

Construction Phase

Phase Number	Phase Name	Phase Type	Start Date	End Date	Num Days Week	Num Days	Phase Description
1	Building Construction	Building Construction	7/16/2020	7/16/2020	5	1	Screen/Diffuser Construction

Acres of Grading (Site Preparation Phase): 0

Acres of Grading (Grading Phase): 0

Acres of Paving: 0

Residential Indoor: 0; Residential Outdoor: 0; Non-Residential Indoor: 0; Non-Residential Outdoor: 0; Striped Parking Area: 0

OffRoad Equipment

Phase Name	Offroad Equipment Type	Amount	Usage Hours	Horse Power	Load Factor
Building Construction	Cranes	0	4.00	231	0.29
Building Construction	Forklifts	0	6.00	89	0.20
Building Construction	Tractors/Loaders/Backhoes	0	8.00	97	0.37

Trips and VMT

Phase Name	Offroad Equipment Count	Worker Trip Number	Vendor Trip Number	Hauling Trip Number	Worker Trip Length	Vendor Trip Length	Hauling Trip Length	Worker Vehicle Class	Vendor Vehicle Class	Hauling Vehicle Class
Building Construction	0	8.00	0.00	20.00	14.70	6.90	20.00	LD_Mix	HDT_Mix	HHDT

3.1 Mitigation Measures Construction

3.2 Building Construction - 2020

Unmitigated Construction On-Site

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Category	lb/day										lb/day					
Off-Road	0.0000	0.0000	0.0000	0.0000		0.0000	0.0000		0.0000	0.0000		0.0000	0.0000	0.0000		0.0000
Total	**0.0000**	**0.0000**	**0.0000**	**0.0000**		**0.0000**	**0.0000**		**0.0000**	**0.0000**		**0.0000**	**0.0000**	**0.0000**		**0.0000**

Unmitigated Construction Off-Site

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Category	lb/day										lb/day					
Hauling	0.1519	5.4428	1.0830	0.0155	0.3495	0.0176	0.3670	0.0958	0.0168	0.1126		1,676.6584	1,676.6584	0.1125		1,679.4712
Vendor	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0000	0.0000	0.0000		0.0000
Worker	0.0362	0.0243	0.3271	9.2000e-004	0.0894	6.8000e-004	0.0901	0.0237	6.2000e-004	0.0243		91.5534	91.5534	2.6300e-003		91.6192
Total	**0.1881**	**5.4672**	**1.4100**	**0.0164**	**0.4389**	**0.0182**	**0.4571**	**0.1195**	**0.0174**	**0.1369**		**1,768.2118**	**1,768.2118**	**0.1151**		**1,771.0904**

Mitigated Construction On-Site

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Category	lb/day										lb/day					
Off-Road	0.0000	0.0000	0.0000	0.0000		0.0000	0.0000		0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0000
Total	**0.0000**	**0.0000**	**0.0000**	**0.0000**		**0.0000**	**0.0000**		**0.0000**	**0.0000**	**0.0000**	**0.0000**	**0.0000**	**0.0000**		**0.0000**

Mitigated Construction Off-Site

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Category	lb/day										lb/day					
Hauling	0.1519	5.4428	1.0830	0.0155	0.3495	0.0176	0.3670	0.0958	0.0168	0.1126		1,676.6584	1,676.6584	0.1125		1,679.4712
Vendor	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0000	0.0000	0.0000		0.0000
Worker	0.0362	0.0243	0.3271	9.2000e-004	0.0894	6.8000e-004	0.0901	0.0237	6.2000e-004	0.0243		91.5534	91.5534	2.6300e-003		91.6192
Total	**0.1881**	**5.4672**	**1.4100**	**0.0164**	**0.4389**	**0.0182**	**0.4571**	**0.1195**	**0.0174**	**0.1369**		**1,768.2118**	**1,768.2118**	**0.1151**		**1,771.0904**

HB Desal - Wedgewire & Diffuser Install (Other Days)

South Coast AQMD Air District, Winter

1.0 Project Characteristics

1.1 Land Usage

Land Uses	Size	Metric	Lot Acreage	Floor Surface Area	Population
User Defined Industrial	1.00	User Defined Unit	0.00	0.00	0

1.2 Other Project Characteristics

Urbanization	Urban	**Wind Speed (m/s)**	2.2	**Precipitation Freq (Days)**	31
Climate Zone	8			**Operational Year**	2021
Utility Company	Southern California Edison				
CO2 Intensity (lb/MWhr)	592.74	**CH4 Intensity (lb/MWhr)**	0.029	**N2O Intensity (lb/MWhr)**	0.006

1.3 User Entered Comments & Non-Default Data

Project Characteristics - Adjusted SCE CO2 Intensity based on 33% RPS by 2020

Land Use - Offshore construction

Construction Phase - Last day of wedgewire screen and diffuser installation construction

Off-road Equipment - Crane on barge - assumed to operate 6 hours per day. Underwater rivet busters and diamond saws assumed to be electrically powered.

Trips and VMT - 8 one-way worker trips (4 round trips assuming each worker would drive own vehicle)

Table Name	Column Name	Default Value	New Value
tblConstructionPhase	NumDays	0.00	1.00
tblOffRoadEquipment	OffRoadEquipmentUnitAmount	1.00	0.00

tblOffRoadEquipment	OffRoadEquipmentUnitAmount	2.00	0.00
tblOffRoadEquipment	OffRoadEquipmentUnitAmount	2.00	0.00
tblProjectCharacteristics	CO2IntensityFactor	702.44	592.74
tblTripsAndVMT	HaulingTripNumber	0.00	20.00
tblTripsAndVMT	WorkerTripNumber	0.00	8.00

2.0 Emissions Summary

2.1 Overall Construction (Maximum Daily Emission)

Unmitigated Construction

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Year	lb/day										lb/day					
2020	0.1958	5.5394	1.4603	0.0161	0.4389	0.0185	0.4574	0.1195	0.0177	0.1372	0.0000	1,731.4253	1,731.4253	0.1199	0.0000	1,734.4216
Maximum	**0.1958**	**5.5394**	**1.4603**	**0.0161**	**0.4389**	**0.0185**	**0.4574**	**0.1195**	**0.0177**	**0.1372**	**0.0000**	**1,731.4253**	**1,731.4253**	**0.1199**	**0.0000**	**1,734.4216**

Mitigated Construction

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Year	lb/day										lb/day					
2020	0.1958	5.5394	1.4603	0.0161	0.4389	0.0185	0.4574	0.1195	0.0177	0.1372	0.0000	1,731.4253	1,731.4253	0.1199	0.0000	1,734.4216
Maximum	**0.1958**	**5.5394**	**1.4603**	**0.0161**	**0.4389**	**0.0185**	**0.4574**	**0.1195**	**0.0177**	**0.1372**	**0.0000**	**1,731.4253**	**1,731.4253**	**0.1199**	**0.0000**	**1,734.4216**

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio-CO2	Total CO2	CH4	N20	CO2e
Percent Reduction	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

3.0 Construction Detail

Construction Phase

Phase Number	Phase Name	Phase Type	Start Date	End Date	Num Days Week	Num Days	Phase Description
1	Building Construction	Building Construction	7/16/2020	7/16/2020	5	1	Screen/Diffuser Construction

Acres of Grading (Site Preparation Phase): 0

Acres of Grading (Grading Phase): 0

Acres of Paving: 0

Residential Indoor: 0; Residential Outdoor: 0; Non-Residential Indoor: 0; Non-Residential Outdoor: 0; Striped Parking Area: 0

OffRoad Equipment

Phase Name	Offroad Equipment Type	Amount	Usage Hours	Horse Power	Load Factor
Building Construction	Cranes	0	4.00	231	0.29
Building Construction	Forklifts	0	6.00	89	0.20
Building Construction	Tractors/Loaders/Backhoes	0	8.00	97	0.37

Trips and VMT

Phase Name	Offroad Equipment Count	Worker Trip Number	Vendor Trip Number	Hauling Trip Number	Worker Trip Length	Vendor Trip Length	Hauling Trip Length	Worker Vehicle Class	Vendor Vehicle Class	Hauling Vehicle Class
Building Construction	0	8.00	0.00	20.00	14.70	6.90	20.00	LD_Mix	HDT_Mix	HHDT

3.1 Mitigation Measures Construction

3.2 Building Construction - 2020

Unmitigated Construction On-Site

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Category	lb/day										lb/day					
Off-Road	0.0000	0.0000	0.0000	0.0000		0.0000	0.0000		0.0000	0.0000		0.0000	0.0000	0.0000		0.0000
Total	**0.0000**	**0.0000**	**0.0000**	**0.0000**		**0.0000**	**0.0000**		**0.0000**	**0.0000**		**0.0000**	**0.0000**	**0.0000**		**0.0000**

Unmitigated Construction Off-Site

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Category	lb/day										lb/day					
Hauling	0.1563	5.5127	1.1658	0.0152	0.3495	0.0178	0.3673	0.0958	0.0170	0.1128		1,645.7961	1,645.7961	0.1174		1,648.7310
Vendor	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0000	0.0000	0.0000		0.0000
Worker	0.0395	0.0266	0.2945	8.6000e-004	0.0894	6.8000e-004	0.0901	0.0237	6.2000e-004	0.0243		85.6292	85.6292	2.4600e-003		85.6906
Total	**0.1958**	**5.5394**	**1.4603**	**0.0161**	**0.4389**	**0.0185**	**0.4574**	**0.1195**	**0.0177**	**0.1372**		**1,731.4253**	**1,731.4253**	**0.1199**		**1,734.4216**

Mitigated Construction On-Site

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e
Category	lb/day										lb/day					
Off-Road	0.0000	0.0000	0.0000	0.0000		0.0000	0.0000		0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0000
Total	**0.0000**	**0.0000**	**0.0000**	**0.0000**		**0.0000**	**0.0000**		**0.0000**	**0.0000**	**0.0000**	**0.0000**	**0.0000**	**0.0000**		**0.0000**

Mitigated Construction Off-Site

	ROG	NOx	CO	SO2	Fugitive PM10	Exhaust PM10	PM10 Total	Fugitive PM2.5	Exhaust PM2.5	PM2.5 Total	Bio- CO2	NBio- CO2	Total CO2	CH4	N2O	CO2e

Category	lb/day										lb/day					
Hauling	0.1563	5.5127	1.1658	0.0152	0.3495	0.0178	0.3673	0.0958	0.0170	0.1128		1,645.7961	1,645.7961	0.1174		1,648.7310
Vendor	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0000	0.0000	0.0000		0.0000
Worker	0.0395	0.0266	0.2945	8.6000e-004	0.0894	6.8000e-004	0.0901	0.0237	6.2000e-004	0.0243		85.6292	85.6292	2.4600e-003		85.6906
Total	**0.1958**	**5.5394**	**1.4603**	**0.0161**	**0.4389**	**0.0185**	**0.4574**	**0.1195**	**0.0177**	**0.1372**		**1,731.4253**	**1,731.4253**	**0.1199**		**1,734.4216**

ATTACHMENT B

Cost Calculations

Date of Preparation: 3/22/2019

Ref. Line #	Comparison of Proposed Intake to Alternative Intake Cost Estimate ($'000s)		
	Intake:	**Proposed Intake (E)**	**Alternative Intake (U2 / D2)**
1	**Construction Period (Months)** (1)	**39**	**72**
2	**Financial Close Pricing Year**	2020	2024
3	**Direct Capital Costs**		
4	Pipeline and Associated Infrastructure (2)	-	26,312
5	Trestle and Associated Infrastructure (2)	-	31,541
6	Intake Screen and Related Costs (2)	22,135	22,135
7	Other Project Costs (Unallocable) (2)	2,178	2,178
8	Indirect, Insurance and Overhead Costs (2)	13,372	45,192
9	**Subtotal**	**37,685**	**127,358**
10	Engineering (15%) (2)	5,653	19,104
11	Contingency (40%) (2)	15,074	50,943
12	**Direct Capital Cost (2018$)**	**58,412**	**197,405**
13	*Direct Capital Cost Escalation (to Year of Financial Close)*	2,957	38,736
14	**Direct Capital Cost ($ in Year of Financial Close)**	**61,369**	**236,140**
15	Development and Construction Costs (3)	9,438	53,367
16	Capitalized Interest During Construction (4)	12,816	151,540
17	Financing Fees and Reserves (5)(6)	8,996	32,811
18	**Total Intake Cost Estimate** (7)	**92,618**	**473,858**
19	**Total Intake Cost Estimate - Rounded**	**93,000**	**474,000**
20	***% Increase over E***		*409.7%*

Note: Direct Capital Costs reflect 12' Pipeline Diameter, see cost detail on the 'Direct Capital Cost Detail' tab

(1) Construction Schedule for U2/D2 assumes new Intake construction commences prior to Plant Construction
(2) For detail of Direct Capital Costs components please refer to the color coded legend on the 'Direct Capital Costs (RWB)' tab starting in tab A78
(3) Costs include Property Taxes, Title Insurance, Construction Management and Permitting and Development Costs
(4) Includes a 6 Month Capitalized Interest Contingency
(5) Reserves include Debt Service, Working Capital and Project O&M
(6) Financing Fees include Conduit, Rating Agency, Underwriting, Equity and Advisory Fees
(7) Proposed Intake (E) Total Intake Cost Estimate is in 2020$ and Alternative Intake (U2/D2) is in 2024$ (both the respective year of Financial Close)

Date of Preparation: 3/22/2019

Ref. Line #	Comparison of Proposed Intake to Alternative Intake Cost Estimate ($'000s) | *ISTAP Presentation Basis*		
	Intake:	**Proposed Intake (E)**	**Alternative Intake (U2 / D2)**
1	**Construction Period (Months)** (1)	**39**	**72**
2	**Financial Close Pricing Year**	2020	2024
3	**Direct Capital Costs** (Including Engineering and Escalation Costs)**:**		
4	Pipeline and Associated Infrastructure (2)	-	36,197
5	Trestle and Associated Infrastructure (2)	-	43,390
6	Intake Screen and Related Costs (2)	26,743	30,450
7	Other Project Costs (Unallocable) (2)	2,632	2,997
8	Indirect, Insurance and Overhead Costs (2)	16,156	62,168
9	Contingency (40%) (2)	15,837	60,939
10	**Direct Capital Cost ($ in Year of Financial Close)**	**61,369**	**236,140**
11	Owner's Project Management and Inspection (3)	1,258	3,194
12	Construction Period Financing Costs - Capitalized Interest During Construction (4)	12,816	151,540
13	Reserves (5)	6,335	24,731
14	Closing Related Costs (6)(7)	10,840	58,253
	Total Intake Cost Estimate (8)	**92,618**	**473,858**
	Total Intake Cost Estimate - Rounded	**93,000**	**474,000**
	% Increase over E		*409.7%*

Engineering Allocation	
2020	2024
0%	21%
0%	25%
59%	17%
6%	2%
35%	35%
100%	100%

Escalation Allocation	
2020	2024
0%	15%
0%	18%
44%	13%
4%	1%
26%	26%
26%	26%
100%	100%

Engineering Allocation	
2020	2024
-	30,259
-	36,273
25,455	25,455
2,505	2,505
15,378	51,970

Note: Direct Capital Costs reflect 12' Pipeline Diameter, see cost detail on the 'Direct Capital Cost Detail' tab

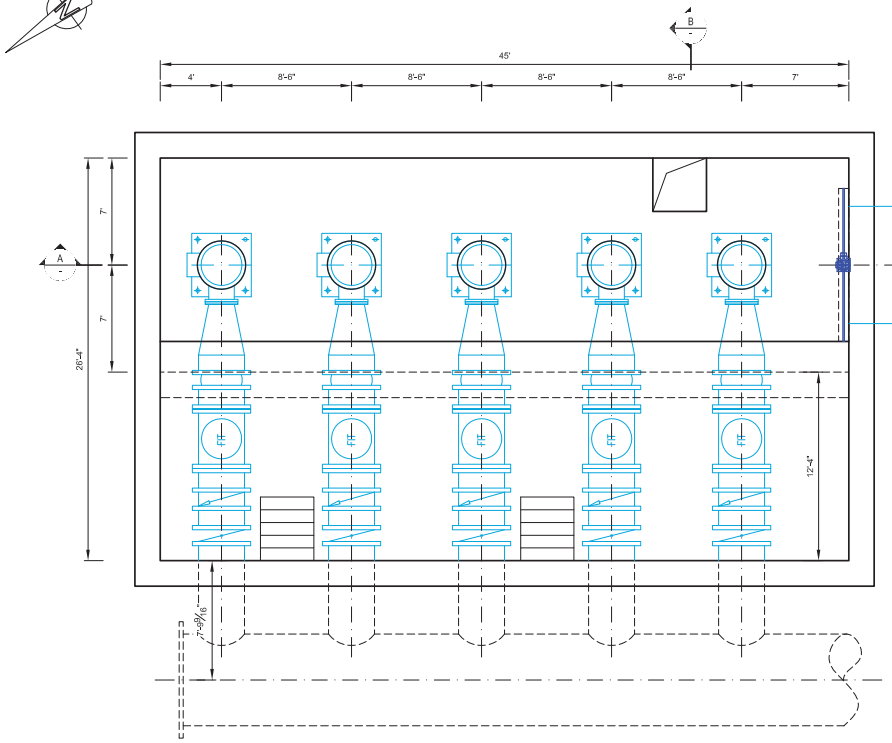
(1) Construction Schedule for U2/D2 assumes new Intake construction commences prior to Plant Construction
(2) For detail of Direct Capital Costs components please refer to the color coded legend on the 'Direct Capital Costs (RWB)' tab starting in tab A78
(3) Costs include Construction Management
(4) Includes a 6 Month Capitalized Interest Contingency
(5) Reserves include Debt Service, Working Capital and Project O&M
(6) Financing Fees include Conduit, Rating Agency, Underwriting, Equity and Advisory Fees
(7) Other Closing Costs include Property Taxes, Title Insurance and Permitting and Development Costs
(8) Proposed Intake (E) Total Intake Cost Estimate is in 2020$ and Alternative Intake (U2/D2) is in 2024$ (both the respective year of Financial Close)

DESCRIPTION	QUANTITY		EQUIPMENT RENTAL				OPERATING EXPENSE						SERVICES TOOLS & SUPPLIES		LABOR EXPENSE				SUBTOTAL		PERMANENT MATERIALS		SUB CONTRACTS		TOTAL DIRECT COST		REMARKS
			CO. OWNED		OTHER		TIRES & TRACKS		EQUIP REPAIR		FUEL-OIL-GREASE				WAGES		ADD-ONS										
			U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	
access roads	2,900	cy													110	319,000				319,000						319,000	Two cofferdams, 4 weeks each 8 crew
sheets for cofferdams	108,000	lb											0.90	97,200						97,200						97,200	Plug @ .90 per pound fob job
frame for cofferdams	30,000	lb											1.50	45,000						45,000						45,000	Plug @ 1.50 per pound fob job
seal concrete	500	cy											200.00	100,000						100,000						100,000	
pipe run on beach	47,250	mh													110	5,197,500				5,197,500						5,197,500	21 crew 50 hours per week, 45 weeks
sheets for beach run	669,900	lb											0.90	602,910						602,910						602,910	buy sheets for 400 feet of trench
frame and struts @ beach	206,000	lb											1.50	309,000						309,000						309,000	W36X230 frame at top with 12" std pipe struts @ 24' c-c
pipe for beach run	6,876	lf																		0	2,180	14,989,680				14,989,680	quote from Ameron
bedding for beach run	14,000	tn												0						0	40	560,000				560,000	1.5 feet deep under pipe
vault at South turn	1	ls												0						0	200,000	200,000				200,000	Plug.
trestle/sheets/pipe	72,560	mh												0	110	8,025,600				8,025,600						8,025,600	4 seasons 19 crew 80 days per season, 12 hours per day
erect/strip trestle equip.	4,000	mh												0	110	440,000				440,000						440,000	
trestle vertical pile	17,780	lf											126.00	2,240,280						2,240,280						2,240,280	24" .500 wall 70' long @ $1.00 per pound
trestle batter pile	5,120	lf											63.00	322,560						322,560						322,560	16" .375 wall 80' long @ $1.00 per pound
dive crew	545	dy																		0			7,000	3,815,000		3,815,000	225 days on land, 320 days on trestle
trestle caps	321,600	lb											1.50	482,400						482,400						482,400	dbl C15X33.5 with cover plates X 50'
trestle rail beams	372,000	lb											1.50	558,000						558,000						558,000	HP 14X89 with 90 lb rail
trestle rail	114,000	lb											1.00	114,000						114,000						114,000	90 lb rail @ 1.00 per pound
trestle walkway	7,600	sf											25.00	190,000						190,000						190,000	
trestle wale beams	206,000	lb											1.50	309,000						309,000						309,000	W 24X117 on both sides of trestle 800'
sheets for trestle	2,296,800	lb											0.90	2,067,120						2,067,120						2,067,120	NZ19-700 plug @ .83 per pound Skyline
trestle template	1	ls											50,000.00	50,000						50,000						50,000	access out front to drive pile
trestle crane gantries	2	ea											1,000,000.00	2,000,000						2,000,000						2,000,000	
trestle pipe gantry	1	ea											1,000,000.00	1,000,000						1,000,000						1,000,000	
trestle material cart	1	ea											750,000.00	750,000						750,000						750,000	
pipe for trestle run	1,900	lf																			1870	3,553,000				3,553,000	Factored down for 12' diameter pipe
bedding for trestle run	4,200	tn																			40	168,000				168,000	
rip rap pipe protection	15,200	tn																			40	608,000				608,000	6' thick rock protection on top of pipe
utilities on trestle	1,900	lf											10.00	19,000						19,000						19,000	
precast intake structure	1	ls																			300000	300,000				300,000	
equip. move in/out beach	1	ls											200,000.00	200,000						200,000						200,000	
equip. move in/out trestle	1	ls											200,000.00	200,000						200,000						200,000	
trestle seasonal moves	1	ls											500,000.00	500,000						500,000						500,000	Cranes stay on trestle, other equipment moves in/out each se
trestle crane down time rent	48	mo			20000	960000														960,000						960,000	3 seasons, 8 mo per season, 2 cranes
beach cranes	106	wk			7665	812490														812,490						812,490	Per Caltrans for Manitowoc 4000 rent & op. cost, nic operator
beach articulated trucks	159	wk			5595	889605														889,605						889,605	
beach welding machines	159	wk			333	52947														52,947						52,947	
beach 185 cfm air comp	106	wk			619	65614														65,614						65,614	
beach 20,000 lb forklift	53	wk			2538	134514														134,514						134,514	
beach yard 50,000 lb fork	53	wk			6018	318954														318,954						318,954	
beach APE 200 vibrohammer	106	wk			8437	894322														894,322						894,322	
beach excavator	53	wk			7744	410432														410,432						410,432	
beach 980 loader	53	wk			5906	313018														313,018						313,018	
trestle cranes	128	wk			7665	981120														981,120						981,120	
trestle pipe gantry op cost	64	wk			1400	89600														89,600						89,600	
trestle material cart op cost	64	wk			1200	76800														76,800						76,800	
trestle vibrohammers	128	wk			8437	1079936														1,079,936						1,079,936	
trestle impact hammer	64	wk			1512	96768														96,768						96,768	
trestle service forklift	64	wk			2538	162432														162,432						162,432	
trestle welding machines	192	wk			333	63936														63,936						63,936	
trestle 185 cfm comp.	192	wk			619	118848														118,848						118,848	
pickups - all	351	wk			653	229203														229,203						229,203	
misc. outside equip. rent	1	ls				675000														675,000						675,000	
crew consumables	127,410	mh											10.00	1,274,100						1,274,100						1,274,100	
Installation of WWS Assembly	1	ls																		17,624,546			0	0		17,624,546	624,546
Airburst																											
Bldg onshore (State Beach)	1	ls																					500,000	500,000		500,000	
Compressors/Air Receivers	1	ls																					1,200,000	1,200,000		1,200,000	
Electrical Service, setup	1	ls																					250,000	250,000		250,000	
Air Piping	10,400	lf																					150	1,560,000		1,560,000	
Allowance for leasing land	1	ls																					1,000,000	1,000,000		1,000,000	
SUBTOTAL	1	ls				8,425,539				0				13,430,570		13,982,100				53,462,755				8,325,000		82,166,435	
Indirects (25% of subtotal)																										20,541,609	
Insurance & enviro (5%)																										4,108,322	
Overhead & profit (25%)																										20,541,609	
SUBTOTAL																										127,357,974	0.55
engineering (15%)																										19,103,696	0.15
Contingency (40%)																										50,942,790	0.40
TOTAL ESTIMATED COST																										197,404,860	

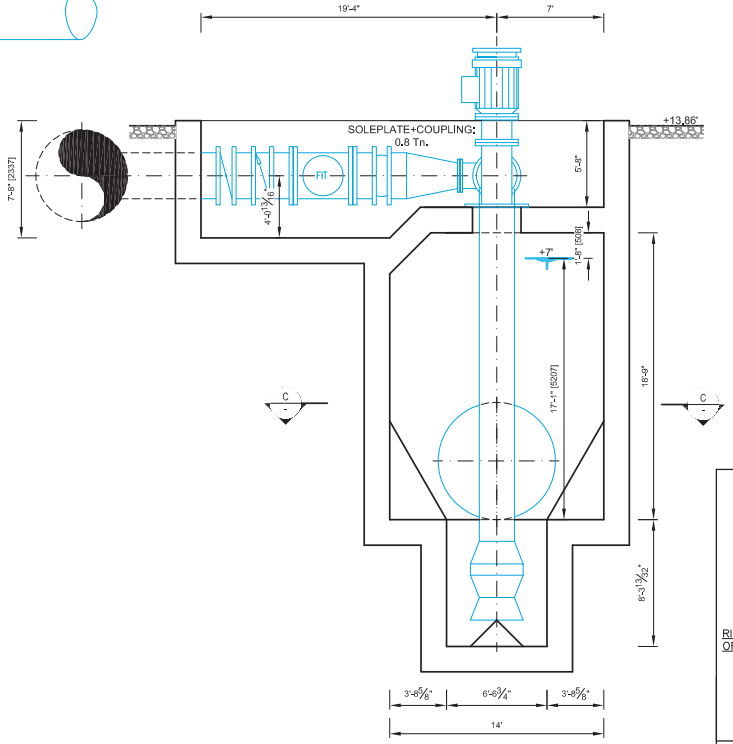
Legend
Pipeline and Associated Infrastructure (2)
Trestle and Associated Infrastructure (3)
Intake Screen and Related Costs (4)
Airburt System
Other Project Costs (Unallocable)
Indirect, Insurance and Overhead Costs (5)
Contingency (40%) (7)

ATTACHMENT C

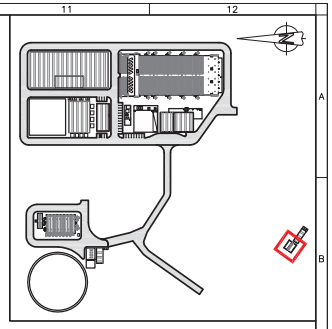
New Intake Pump Station Details



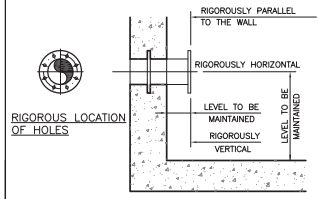
PLAN
SCALE 1/4" = 1'-0"



SECTION B-B
SCALE 1/4" = 1'-0"



FLANGED PUDDLE PIPE LOCATION DETAIL



FRONT VIEW AND SECTION

– HOLES SCREW IN FLANGE WILL NOT MATCH WITH VERTICAL AND HORIZONTAL AXES , THESE WILL BE SYMMETRICAL
– THEY WILL BE PLACED RIGOROUSLY ALIGNED

(*) TO BE DEFINED BY CIVIL WORKS

BUILD "*" GROUP

No.	REVISIONS:	DRN:	DATE:

Notes:

Sheet:

Entity: KSD

Joint venture of: acciona Agua / TETRA TECH

Designed:	M.M.H.	30th SEPTEMBER 2010
Drawn:	A.R-L.	30th SEPTEMBER 2010
Reviewed:	M.M.H.	30th SEPTEMBER 2010
Approved:	J.I.	30th SEPTEMBER 2010
	Initials	Date

Scale: 1/4" = 1'-0"
Graf scale:

Site: HUNTINGTON BEACH SEA WATER DESALINATION PROJECT

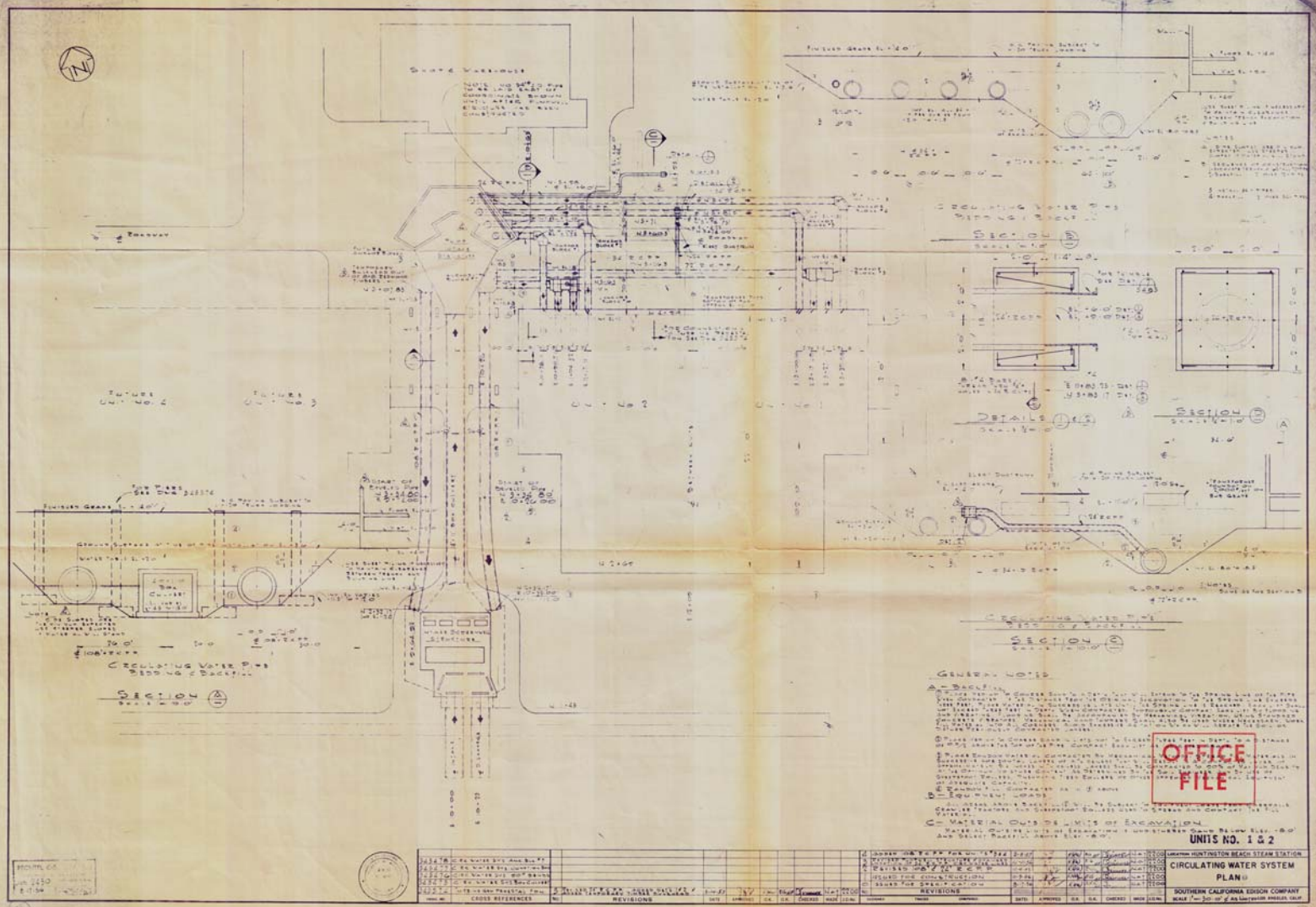
Title of plant: PUMP STATION PLAN AND SECTION

Sheet Size: D
Project Number: 1633124
Overhaul: A
Sheet Number: 1 of 1
Drawing Number: 04 130 000 0001
Type | Zone | Mod. | Array

Cod. 1633124_04_130_000_0001-CAPTACION

ATTACHMENT D

Existing Power Generating Station Intake Infrastructure Details



OFFICE FILE

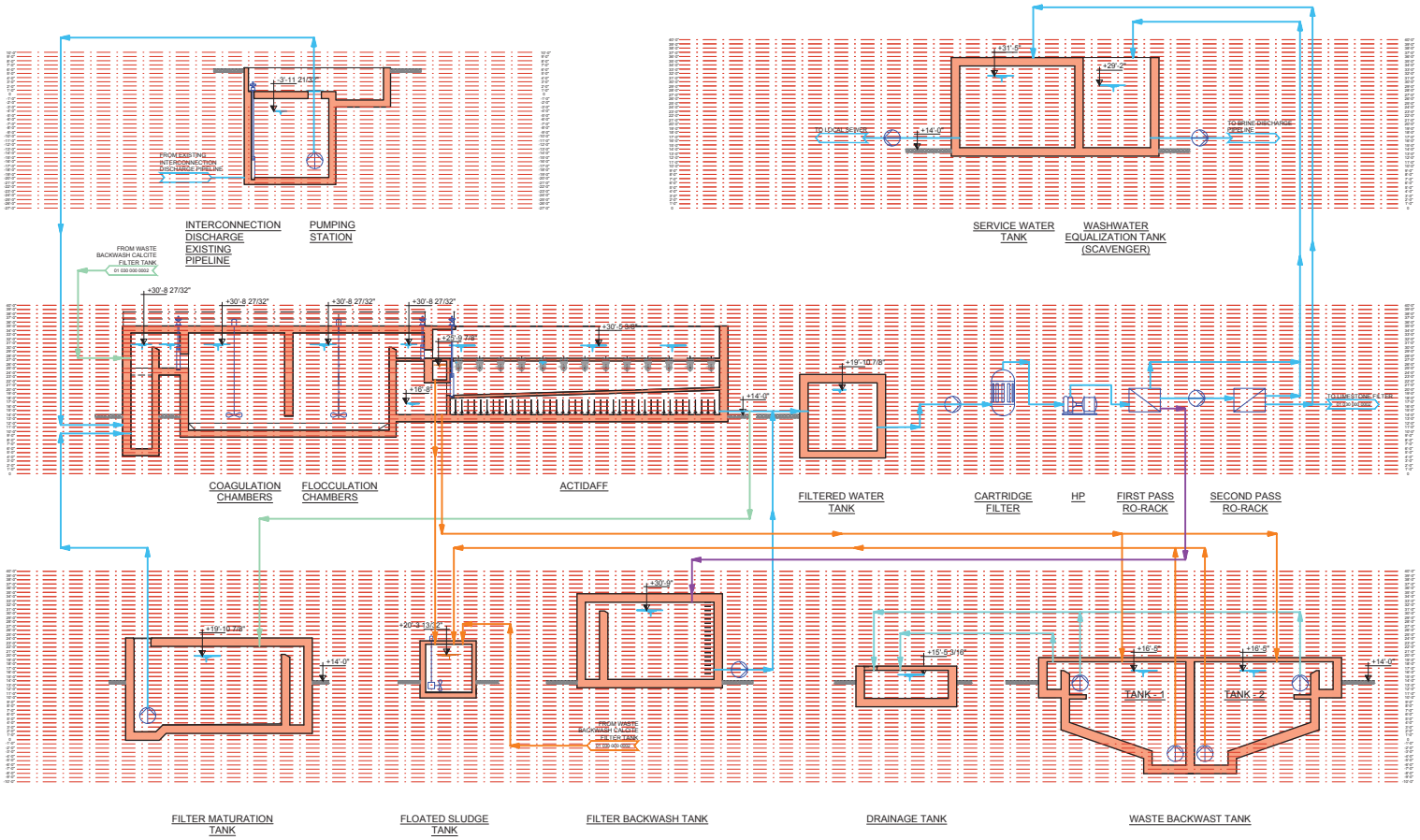
545473-5

ATTACHMENT E

Desalination Plant Hydraulic Profile

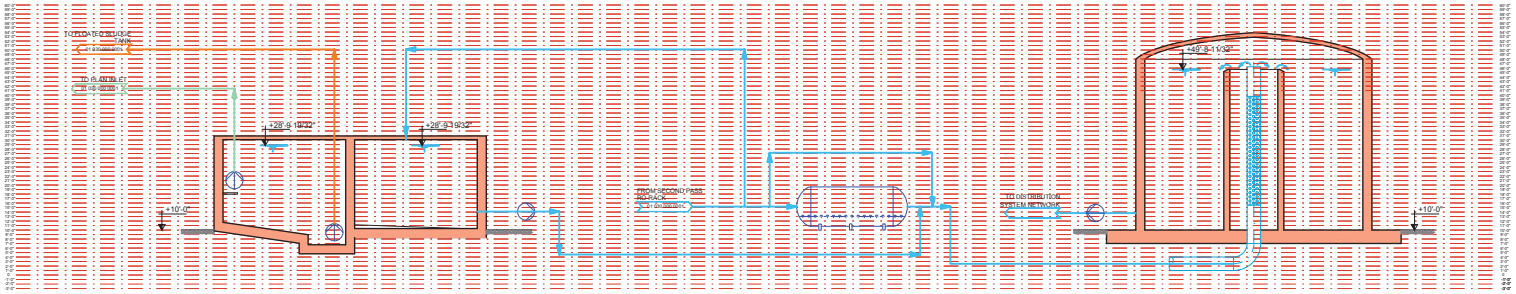
6.8.B PRELIMINARY HYDRAULIC PROFILE

01 030 000 0001 HYDRAULIC PROFILE



No.	REVISIONS:	DRN:	DATE:	Notes:	Sheet:	Entity:	Joint venture of:	Designed: M.M.H. 30th JULY 2016	Site: HUNTINGTON BEACH SEA WATER DESALINATION PROJECT	Sheet Size: D	Project Number: 1633124	Overhaul: A
						KSD	acciona Agua	Drawn: A.T.D. 30th JULY 2016			Sheet Number: 1 of 1	
								Reviewed: M.M.H 30th JULY 2016				
								Approved: J.I. 30th JULY 2016	Title of plant: HYDRAULIC PROFILE LEVES 6'-0" & +14'-0"	Drawing Number: 01 030 000 0001		
								Initials / Date				
								Scale: 3/32" = 1'-0"				
								Graf scale:				

Type | Zone | Mat. | Array



WASTE BACKWASH CALCITE FILTER TANK

BACKWASH CALCITE FILTER TANK

LIMESTONE FILTER

PRODUCT WATER TANK

No.	REVISIONS:	DRN:	DATE:

Notes:

Sheet:

Entity: KSD

Joint venture of: acciona Agua; TETRA TECH

Designed:	M.M.H.	30th JULY 2016
Drawn:	A.T.D.	30th JULY 2016
Reviewed:	M.M.H.	30th JULY 2016
Approved:	J.I.	30th JULY 2016
	Initials	Date

Scale: 3/32" = 1'-0"

Graf scale:

Site: HUNTINGTON BEACH SEA WATER DESALINATION PROJECT

Title of plant: HYDRAULIC PROFILE LEVE 10'-0"

Sheet Size: D

Project Number: 1633124

Overhaul: A

Sheet Number: 1 of 1

Drawing Number: 01 030 000 0002

Type | Zone | Mat. | Array

Cad: 1633124_01_030_000_0001

ATTACHMENT F

NPSH Calculations

PROPOSED OFFSHORE CONDITION ANALYSIS

DESCRIPTION		ID in	ID ft	Area SF	Length ft	Q mgd	Q CFS	V fps	HW Coeff	K Minor Loss	H_L ft	ASSUMPTIONS
Entrance		29.0	2.42	4.59		29	44.87	9.78		0.50	0.743	Entrance losses into pump intake
30" SS Pipe (XS) - Intake Pumps		29.0	2.42	4.59	27.0	29	44.87	9.78	100		0.394	HL detemined using Hazen Williams Equation
Tee, Flanged		29.0	2.42	4.59		29	44.87	9.78		0.20	0.297	Assuming tee in direction from pump, motor, and flow
Sudden Expansion 2.5' to 3.33'	D1 =	24.0	2.00	3.14								Circular section and concrete lined
	D2 =	33.5	2.79	6.12		29	44.87	7.33		0.24	0.198	Dimensions based on the As-Built drawings
40" RCP Pump Conveyance Pipe		33.5	2.79	6.12	35.0	29	44.87	7.33	100		0.253	Conveyance pipe from intake pumps to manifold
Check Valve		33.5	2.79	6.12		29	44.87	7.33		2.0	1.669	Assuming check valve in direction of flow
Ball Valve		33.5	2.79	6.12		29	44.87	7.33		0.05	0.042	Assuming ball valve in direction of flow
Sudden Expansion 3.33' to 14'	D1 =	33.5	2.79	6.12								Circular section and concrete lined
	D2 =	139.0	11.58	105.38		116	179.48	1.70		0.89	0.040	Assumed 14' dia with up to 6" of marine growth
Box Culvert Entrance		139.0	11.58	105.38		116	179.48	1.70		0.50	0.023	Entrance losses into pump intake
14' Box Culvert		139.0	11.58	105.38	350.0	116	179.48	1.70	100		0.032	Box culvert distance between pumpwell to screenwell
Box Culvert Exit		139.0	11.58	105.38		116	179.48	1.70		1.00	0.045	Exit losses from 14' culvert to pipe
14' Existing Concrete Onshore Intake Pipe		139.0	11.58	105.38	650.0	116	179.48	1.70	100		0.060	14' pipe between screenwell and shoreline
14' Existing Offshore Intake Pipe		127.0	10.58	87.97	1500.0	116	179.48	2.04	100		0.214	Existing 14' concrete pipe assuming up to 6" of marine growth located between existing 14' onshore pipe and ~1,500 ft offshore
Sudden Reduction 14' to 12'	D1 =	127.0	10.58	87.97								Circular section and concrete lined
	D2 =	108.6	9.05	64.33		116	179.48	2.79		0.14	0.016	Dimensions based on the As-Built drawings
90 bend		108.6	9.05	64.33		116	179.48	2.79		0.30	0.036	Assuming 90 degree elbow near
12' Proposed Offshore Concrete Intake Pipe		108.6	9.05	64.33	6474.0	116	179.48	2.79	100		1.980	12' concrete pipe assuming up to 6" of marine growth, located from existing 14' pipe to intake location (D2)
Exit		108.6	9.05	64.33		116	179.48	2.79		1.00	0.121	Exit losses from screens

Sub-Total Head Loss		Σ =	6.162
Miscellaneous	20%		1.232
Total Head Loss		**Σ =**	**7.395**

Miscellaneous losses to account for missing fittings and flow obstacles not shown or specified on as-built records

Datum	Value	Description
MSL	2.78	Mean Sea Level
MLLW	0.0	Mean Lower-Low Water
NAVD88	0.18	North American Vertical Datum of 1988
Min Tide	-2.35	Lowest Observed Tide
Min Level	-10	Minimum water level at lowest tide and max flow
Min Head Loss	**7.65**	**Minimum allowable head loss due to min. sea level**

PROPOSED ONSHORE CONDITION ANALYSIS

DESCRIPTION		ID in	ID ft	Area SF	Length ft	Q mgd	Q CFS	V fps	HW Coeff	K Minor Loss	H_L ft	ASSUMPTIONS
Entrance		29.0	2.42	4.59		29	44.87	9.78		0.50	0.743	Entrance losses into pump intake
30" SS Pipe (XS) - Intake Pumps		29.0	2.42	4.59	27.0	29	44.87	9.78	100		0.394	HL detemined using Hazen Williams Equation
Tee, Flanged		29.0	2.42	4.59		29	44.87	9.78		0.20	0.297	Assuming tee in direction from pump, motor, and flow
Sudden Expansion 2.5' to 3.33'	D1 =	24.0	2.00	3.14								Circular section and concrete lined
	D2 =	33.5	2.79	6.12		29	44.87	7.33		0.24	0.198	Dimensions based on the As-Built drawings
40" RCP Pump Conveyance Pipe		33.5	2.79	6.12	35.0	29	44.87	7.33	100		0.253	Conveyance pipe from intake pumps to manifold
Check Valve		33.5	2.79	6.12		29	44.87	7.33		2.0	1.669	Assuming check valve in direction of flow
Ball Valve		33.5	2.79	6.12		29	44.87	7.33		0.05	0.042	Assuming ball valve in direction of flow
Sudden Expansion 3.33' to 14'	D1 =	33.5	2.79	6.12								Circular section and concrete lined
	D2 =	139.0	11.58	105.38		116	179.48	1.70		0.89	0.040	Assumed 14' dia with up to 9" of marine growth
Box Culvert Entrance		139.0	11.58	105.38		116	179.48	1.70		0.50	0.023	Entrance losses into pump intake
14' Box Culvert		139.0	11.58	105.38	350.0	116	179.48	1.70	100		0.032	Box culvert distance between pumpwell to screenwell
Box Culvert Exit		139.0	11.58	105.38		116	179.48	1.70		1.00	0.045	Exit losses from 14' culvert to pipe
14' Existing Concrete Onshore Intake Pipe		139.0	11.58	105.38	650.0	116	179.48	1.70	100		0.060	14' pipe between screenwell and shoreline
90 bend		139.0	11.58	105.38		116	179.48	1.70		0.30	0.014	Assuming 90 degree change in direction
14' Proposed Concrete Onshore Intake Pipe		127.0	10.58	87.97	5917.0	116	179.48	2.04	100		0.844	14' concrete pipe assuming up to 6" of marine growth located between existing 14' onshore pipe and ~1 mile south along shoreline
Sudden Reduction 14' to 12'	D1 =	127.0	10.58	87.97								Circular section and concrete lined
	D2 =	108.6	9.05	64.33		116	179.48	2.79		0.14	0.016	Dimensions based on the As-Built drawings
90 bend		108.6	9.05	64.33		116	179.48	2.79		0.30	0.036	Assuming 90 degree change in direction
12' Proposed Offshore Concrete Intake Pipe		108.6	9.05	64.33	2598.0	116	179.48	2.79	100		0.794	12' concrete pipe assuming up to 6" of marine growth, located from onshore to intake location (D2)
Exit		108.6	9.05	64.33		116	179.48	2.79		1.00	0.121	Exit losses from screens
						Sub-Total Head Loss				Σ =	5.621	Miscellaneous losses to account for missing fittings and flow obstacles not shown or specified on as-built records
						Miscellaneous			20%		1.124	
						Total Head Loss				**Σ =**	**6.745**	

Datum	Value	Description
MSL	2.78	Mean Sea Level
MLLW	0.0	Mean Lower-Low Water
NAVD88	0.18	North American Vertical Datum of 1988
Min Tide	-2.35	Lowest Observed Tide
Min Level	-10	Minimum water level at lowest tide and max flow
Min Head Loss	**7.65**	**Minimum allowable head loss due to min. sea level**

ATTACHMENT G

Acciona Email Correspondence

From: Beneyto Espinosa de los Monteros, Ignacio <ignacio.beneyto.espinosamonteros@acciona.com>
Sent: Wednesday, February 20, 2019 1:04 PM
To: Patrick Crain
Subject: Fwd: Hungtinton Beach IDAM - Sistema de limpieza para T91

Hello Pat,

Please find below email from Quilton. Translation below.

My phone from Spain: +34607256917

Any question please let me know.

Thanks,

Best regards,
to Spanish

Good morning Diego,

According to the conversations held last Friday, I attached the characteristics of the cleaning system for the following conditions of departure:

(3 + 1) Water intakes mod. T91

Unit flow: 6,100 m3 / h

Screen size: A = 1 mm

Maximum water flow rate through the filter: 0.15 m / s

Location distance of the cleaning system to the filters: 740 m (2,430 ft)

Maximum depth at the water intake: 15 m

With all this, the necessary system for cleaning with air would be:

Air pipes to DN150 filters

Air flange in the outlet: DN200

Flange in the outlet: DN1600

Volume of the deposit: 20 m3

Working pressure: 10 Bars

As we indicated in the offer sent in 2017, we would continue with the same philosophy:

Configuration 3 + 1 outlets with cleaning system on the coast, similar to the project (light A = 1mm and maximum speed of water through the filters of 0.15 m / s)

Structural body of the reinforced sockets in superduplex DN32750

Filtering part of the outlets removable by panels, in CuNi 90/10

Air flange DN200 and auxiliary flanges for emergency cleanings

Cleaning system with 1 + 1 tanks and 1 + 1 compressors, with four lines. Each line with a manual valve and an automatic one in DN150. All the set (except the tanks) on stainless steel frame AISI-316 with CE marking of set.

And with the indicated in the offer itself:

Two air receivers 20 m3 (duty / stand by air / receiver to be able to increase frequency of backwash in case of fouling). Due to the size of the receivers, they will be installed out of the skid and final connections on site will be out of supply.

Two compressors (1 + 1) to fill the receivers.

Air lines to the intakes 6 " (DN150) and intake air flanges 8 " (DN200).

EC Declaration of Conformity of the whole Backwash System included.

Greetings,

Web Result with Site Links

Google Translate

--------- Forwarded message ----------
From: "Aritio Garcia, Diego" <diego.aritio.garcia@acciona.com>
Date: Feb 19, 2019 1:59 PM
Subject: Fwd: Hungtinton Beach IDAM - Sistema de limpieza para T91
To: "Beneyto Espinosa de los Monteros, Ignacio" <ignacio.beneyto.espinosamonteros@acciona.com>
Cc:

Begin forwarded message:

From: "QUILTON-Fe Maria Diaz" <m.diaz@quilton.com>
Date: February 18, 2019 at 5:54:20 AM CST
To: "'Aritio Garcia, Diego'" <diego.aritio.garcia@acciona.com>
Subject: Hungtinton Beach IDAM - Sistema de limpieza para T91

Buenos días Diego,

De acuerdo con las conversaciones mantenidas el pasado viernes, adjunto las características del sistema de limpieza para las siguientes condiciones de partida:

- (3+1) Tomas de agua mod. T91
- Caudal unitario: 6.100 m3/h
- Luz de paso: A= 1 mm
- Velocidad máxima de paso del agua a través del filtro: 0,15 m/s
- Distancia de ubicación del sistema de limpieza a los filtros: 740 m (2.430 pies)
- Profundidad máxima a la toma de agua: 15 m

Con todo esto, el sistema necesario para su limpieza con aire sería:

- Tuberías de aire hasta los filtros DN150
- Brida de aire en la toma: DN200
- Brida de agua en la toma: DN1600
- Volumen del depósito: 20 m3
- Presión de trabajo: 10 Bars

Como indicamos en la oferta remitida en 2017, seguiríamos con la misma filosofía:

- Configuración 3+1 tomas con sistema de limpieza en la costa, similar al de proyecto (luz A=1mm y velocidad máxima de paso del agua a través de los filtros de 0,15 m/s)
- Cuerpo estructural de las tomas reforzado en superduplex DN32750
- Parte filtrante de las tomas desmontable por paneles, en CuNi 90/10
- Brida de aire de las tomas DN200 y con brida auxiliar para limpiezas de emergencia
- Sistema de limpieza con 1+1 depósitos y 1+1 compresores, con cuatro líneas. Cada línea con una válvula manual y una automática en DN150. Todo el conjunto (salvo los depósitos) sobre bastidor de inoxidable AISI-316 con marcado CE de conjunto.

Y con lo indicado en la propia oferta:

- Two air receivers 20 m3 (duty/stand by air/receiver to be able to increase frequency of backwash in case of fouling). Due to the size of the receivers, they will be installed out of the skid and final connections on site will be out of supply.
- Two compressors (1+1) to fill the receivers.
- Air lines to the intakes 6" (DN150) and intake air flanges 8" (DN200).
- EC Declaration of Conformity of the whole Backwash System included.

Un saludo,

Fe María Díaz

m.diaz@quilton.com

móvil +34 619 979 450

Central: **BILBAO**

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www.quilton.com

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ATTACHMENT H

GHD Memo

Highlands Construction Solutions, LLC
P.O. Box 785, La Center, Washington 98629
Hcsllc2015@gmail.com
(360)772-2799

6 August 2018

To: **Patrick Crain, Poseidon**
Paul Hermann, GHD

From: Bruce Patterson

Subject: Technical Memorandum, Constructability Consideration for Utilizing D2 Intake

Poseidon is considering moving the intake location for the Huntington Beach Desalination Plant from the existing intake concrete structure to a new location, D2, located approximately 1.2 miles down coast. Intake D2 is situated approximately 1,900 ft from the shoreline.

Poseidon has further advised that the preferred approach to connect to the new location D2 is to extend an offshore pipeline from the existing intake tower. Preliminary pipeline sizing prepared by Poseidon's engineering consultant, GHD, has determined that the pipeline would likely have a diameter of between 10 to 14 ft. due to NPSH requirements for the future onshore intake pump station and taking into account future marine growth.

Poseidon had originally considered an option in which large diameter HDPE piping could be pre-assembled in large strings at the Port of Long Beach and sunk into position with concrete anchors which is a proven method for placement of submarine pipelines that can be mounted on the seabed. Coastal engineers from GHD performed a preliminary evaluation for the option of placing a large diameter pipe, parallel to shore on the seabed, to connect the existing intake structure to the new location D2. Based on the preliminary evaluation by the coastal engineers, the option of placing the pipeline on the seabed has been eliminated.

Poseidon has requested that Highlands Construction Solutions, LLC, (HCSLLC) evaluate other options including placing an offshore pipeline from the existing intake structure to the new location D2 by placing the large diameter in a trench below the seabed. Poseidon also requested that HCSLLC evaluate the feasibility of placing the majority of the pipeline onshore and then extend the pipeline to its new location 1,900 ft offshore by installing a trestle through the surf zone.

This Technical Memorandum evaluates both options from a constructability perspective only. Issues such as permitting requirements or access issues along the proposed alignment are not addressed in this evaluation. For the option determined to be the most feasible, ROM cost estimate and construction schedule is provided.

Background Information:

1. The existing intake piping from the Power Generating Station to the existing intake tower is 14 ft diameter pre-cast concrete piping. Poseidon indicated preliminary sizing of the pipe would

range from 10 to 14 ft without taking into account marine growth. For the purpose of this evaluation, a pipe diameter of 14 ft was considered as this would provide an allowance for marine growth. If further design refinements are performed and it is determined that a smaller diameter pipe can be used, the conclusion reached in this Technical Memorandum would not materially change.

2. For the offshore option, the installation of the large diameter pipe would entail trenching an excavation approximately 20 ft deep by 20 ft wide for a distance of nearly 6,500 ft in a depth of water of approximately 30 ft. Alignment of the pipeline would be parallel to the shore and would be located between 1,500 to 2,000 from the shoreline.
3. For the onshore option, the pipeline would tie-in to the existing 14 ft diameter pipeline and would be placed along the State Beach for a distance of approximately 8,500 ft of which approximately 1,900 ft. would be installed through the surf zone by using a steel trestle. The onshore option would also include two beach structures at the tie-in point to the existing pipeline and the turning point where the pipeline heads offshore. It is assumed that the structures would allow for maintenance of the pipeline.
4. Both of the options presented above would include the installation of wedgewire screens (WWS) at the end of the pipeline (at the D2 location). The details of the installation of the WWS is described in the SEIR (2017). In addition, it is assumed that a concrete intake structure similar to the existing intake structure would be required for both options to connect the pipeline to the WWS.
5. Offshore work involving pile driving is limited to working between July 1 to October 31 due avoid impacts to whale migration. In addition, pile driving is further limited to the work hours starting 30 minutes after sunrise and 30 minutes before sunset.

Feasibility of Options

Option 1 – Offshore Pipeline: There are two approaches for installing an offshore pipeline that requires burial beneath the seabed. The first approach is to utilize floating equipment such as derrick barges to excavate and dump scows to dispose of the dredge spoil and along with a secondary barge to place bedding material and lay the pipe. It is assumed that the scour protection will be required on top of the pipeline using riprap armor which will require that the pipe trench is backfilled. In some jurisdictions, offshore pipe trenches are allowed to fill in naturally from sediment transport that normally is present along the coast line, however, such an approach may not be allowed off of Huntington Beach and is not considered feasible for the purpose of this evaluation.

For a 14 diameter pipeline, the trench depth would be approximately 20 ft deep. As the pipe alignment is located in the surf zone, it is expected that the side slopes of the excavation will be flat – approximately 6:1. For a 20 ft deep excavation with 6:1 side slopes, 103 cy of material must be removed for every linear ft of trench. For the estimated 6,500 feet length of the pipeline, approximately 670,000 cy of material would be excavated. In addition, excavations that are performed near the surf zone are

often subject to filling in via natural sand/sediment transport and it must be assumed that a quantity greater than 670, 000 cy would be excavated to maintain the required trench depth. It is further assumed that the excavated material (when performing an open cut) would need to be loaded on dump scows for offshore disposal. Because of the size of the excavation with the flat side slopes, attempting to side cast the material would result in the excavated material sliding back into the excavation. It is unclear if permits could be obtained for ocean dumping for this quantity of materials.

For the offshore option, the quantity of the excavated material using an open cut approach is not considered viable. If the alignment were outside of the surf zone and located in calmer waters, an open cut approach using floating equipment could be considered. For connecting the existing intake structure to the new D2 location, HCSLLC does not consider the open cut approach a viable option.

An alternative approach for the offshore pipeline installation is to build a trestle and install the 6,500 ft length of large diameter pipe from the trestle. The trestle approach would allow for sheet piles to be placed along the alignment on both sides of the trestle which would significantly decrease the amount of material that would need to be excavated. The sheet pile walls would also prevent material from sluffing into the excavation and would eliminate the need to re-excavate to maintain the trench profile. The trestle with the sheet pile walls could be installed in sections to allow the trestle material and sheet piles to be re-used after the accompanying pipe section is placed. For each section, a connecting trestle from the shore would be required to provide access for personnel, equipment and materials for the pipeline installation. (In addition, each trestle would require a temporary construction haul road from the State Beach parking area to the beginning of the trestle at the shoreline).

Assuming that a beach access trestle is placed approximately every 2,200 feet along the 6,500 ft alignment, three approach trestles would also be required. Each approach trestle would be approximately 2,000 ft in length resulting in a total of 12,500 feet of trestle construction. Trestle construction would require the use of vibratory hammers (with at least 10% of the piles finished with impact hammer to prove that the piles are properly seated). This would restrict the construction operation to work from July 1 to October 31 each year (120 calendar days). The production rate for installing a trestle and sheet piles is assumed to be 10 linear feet of trestle per day. It is further assumed that sheet piles would all be removed for the section being constructed during the same whale season. A trestle that is installed parallel to the shore with sheet pile walls would be subject to a substantial wave loading and therefore sheet piles should be removed before the winter storm season. Because the sheet pile walls and the trestle are parallel to the beach and a large surface area is exposed to wave action there is a significant risk that a storm event could cause major damage to the sheet pile walls and the trestle. Assuming a production rate of 10 linear feet per day, the time required to install the pipe using the trestle/sheet pile approach subject to the whale season limitations is 10 years. While it may be possible to reduce the schedule by having several trestle sections and pipe laying operations running in parallel, the shore impact for laydown and access roads under an expedited schedule would also be a

factor. One could believe that the State Beach would allow one access road running from the State Beach parking to the shore line but it is unlikely that multiple access roads would be approved.

Because of the multi-year construction timeframe required to install the offshore pipeline 1.2 miles down coast, parallel to the shoreline, the offshore option does not appear to be a viable option. Besides the public nuisance aspect (visual, noise and public beach access limitation), there is also a significant safety concern. A trestle that is installed parallel to the shoreline is subject to severe wave loading especially caused by winter storms. If a trestle needs to be installed, it should be installed perpendicular to the shore and the timeframe for having the trestle exposed in the ocean should be reduced. For these reasons, HCSLLC recommends that the offshore trestle option is also eliminated for consideration.

Option 2: Onshore Installation

The onshore option to connect to D2 involves laying new 14-foot diameter pipe approximately 1.2 miles down coast along the beach from the existing seawater intake pipeline and then offshore approximately 1,900 feet to the D2 location where a new intake structure and screens will be installed. Construction would proceed as follows:

- A sheet pile cofferdam will be constructed over and South of the existing 14 feet diameter pipeline to create access to open up the side of the existing pipe and install a stub for the new pipe. After the cofferdam is excavated a concrete seal will be placed to allow dewatering of the cofferdam.
- An opening will be cut in the side of the existing pipe and the new 14 feet diameter stub will be installed and sealed to the existing pipe.
- The end of the stub will be protected, the cofferdam will be backfilled and the South wall of sheets will be removed to allow installation of sheet piles for the alignment downcoast along the beach to start.
- An access manhole for future maintenance will be provided at the tie in to the existing pipe.
- The trench for installation of the new 14 feet diameter pipe will be 24 feet wide and 24 feet deep. It is anticipated that approximately the bottom 14 feet of the trench will be below the ambient water table so this pipe installation will be done “in the wet”.
- Sheet piles will be used to shore the trench to avoid collapse of the trench and the need to open up the trench with wide side slopes. Sheets will be provided to shore 400 feet of trench at a time. An external frame at the top of the sheet piles with horizontal struts at approximate 24 feet intervals will be needed to support the shoring system.
- A work area of approximately 100 feet wide and 1000 feet long will be required for the shoring, excavation, pipe laying, backfill, and shoring removal operations.
- A staging area of approximately five acres on the beach adjacent to the work area will be required to stage pipe, bedding materials, and shoring materials.

- Shoring installation will be performed with a crawler crane and a vibratory pile hammer ahead of the excavation operation.
- A 70 ton class hydraulic excavator will be used to dig the trench. The external frame for the shoring system will be installed along the sheet pile walls. Prior to installation of the struts the excavator will work between the sheets and dig the first 8 feet of excavation keeping above the ambient water table. The excavator will then dig to final grade and the struts will be installed as the excavation proceeds. The excavator will load all-wheel drive articulated trucks which will drive back down the beach to dump the material behind the pipe laying area for backfill. It is anticipated that enough trench for two joints of pipe (32 feet) per day will be excavated.
- A crawler crane will be used to lay the 14 feet diameter pipe. The pipe will be RCP or mortar lined and coated steel pipe with bell and spigot joints. The joints will be fitted with a double "O" ring and a test port between the O rings so that the joint may be pressure tested for leaks after each section has been laid. It is anticipated that the sections of pipe will be 16 feet long. Two sections of pipe will be installed per day.
- Pipe will be bedded with approximately one foot of bedding material placed in the bottom of the trench.
- Pipe will be delivered by truck to the laydown yard and staged. As needed the pipe will be loaded onto an off-road truck and hauled to the pipelaying crew.
- Pipe will be backfilled with beach sand that has been hauled back from the excavation operation and watered down for consolidation.
- As the backfill is placed the struts will be removed from the shoring system and the frames and sheets will be removed and cycled ahead to the sheet pile driving crew.
- The following equipment is anticipated for use on the pipelaying operation and includes:
    - Two crawler cranes of approximately 150-ton capacity. One to drive the sheet piles and a second crane to lay the pipe and pull the sheet piles.
    - Two 30-ton capacity articulated off road trucks to haul sand and bedding.
    - One off road truck to haul the pipe.
    - Two APE 200 or equivalent vibratory pile hammers to drive and pull the sheets.
    - A 70-ton class hydraulic excavator to perform the excavation.
    - A wheel loader to place the bedding and backfill.
    - A large forklift in the laydown area to unload and handle pipe sections.
    - A 20,000-pound capacity forklift to handle and move sheet piles ahead on the beach.
    - Various small equipment including generators, welding machines, and compressors.
- Access manholes will be installed at approximate 800 feet intervals along the pipeline.
- A vault will be required at the Southern limit of the onshore alignment where the pipe turns out to sea. The vault will be constructed within a shored cofferdam similar to that at the existing pipe tie in.

- Construction of the pipeline from the vault to the inshore end of the trestle will be similar to the work described above.
- A marine trestle approximately 1,900 feet in length will be constructed for access to install the offshore leg of the pipeline. The trestle will be an open structure with steel pipe piles, cap beams, and rail beams to carry gantry mounted equipment. The trestle will also have wales on each side to support the top of temporary sheet piles to shore the trench for the pipe installation.
- The trestle will be supported with steel pipe piles at approximate 15 feet spacing longitudinally with pile caps at 30 feet intervals. Rail beams will be fastened to the pile caps and to the intermediate piles between the caps so the rail beams are supported at 15 feet intervals. The rail beams will be approximately 30 feet apart. A walkway will be provided on one side of the trestle for crew access. The elevation of the top of the rail will be approximately +30 feet. Air and electric utilities will be run along the walkway for use in trestle and pipeline construction. Lighting will be provided on the walkway for safe access and to illuminate the trestle to reduce the potential for a hazard to mariners and members of the public.
- Equipment supported by the trestle will include one (perhaps two) 150-ton capacity cranes mounted on gantries, a self-powered flat deck material cart to haul material, and possibly a pipe laying gantry.
- Pile driving for the trestle will be performed with a combination of vibratory and impact pile hammers. It may be feasible to drive the pipe piles to tip elevation with a vibratory hammer but an impact hammer will be required, at a minimum, to “proof” the piles so that pile capacity can be assured.
- As the trestle construction proceeds sheet piles will be installed on each side of the trestle and secured to the wale system. The sheet piles will provide shoring for excavating the pipe trench.
- It is anticipated that the trestle, sheet piles, excavation, pipe laying, and backfill will progress in stages scheduled to maximize the length of pipe that can be installed within the allowable work window each year. As the pipe is laid and backfilled the sheet piles will be pulled and advanced forward for the next phase of excavation.
- Excavation will be performed with a clamshell and the material will be side cast outside the sheet pile shoring.
- Pipe installation will then proceed as the excavation is completed. The pipe will be transported out the trestle using a crane or pipe gantry, lowered into position, and pulled into place to make the joint with the previously laid pipe. A diver will assist with alignment and to test the joint after it has been completed. The pipe will be held in position and bedding will be placed around the pipe with diver assistance.
- Backfill and armor stone will then be placed over the completed portion of the pipeline.
- Sheet pile shoring will then be pulled in stages and moved ahead for use in the next pipe installation stage.

- When the pipeline has reached the end of the trestle, trestle equipment may be used to install the precast intake structure.
- A similar intake structure to the existing intake is contemplated. The seabed will be excavated, possibly with the use of sheet pile shoring. Steel piles will be driven to support the precast elements of the intake structure. The precast segments of the structure will then be lowered onto the steel piles. Tremie concrete will then be placed to seal the structure at the bottom and lock it to the steel piles. After the intake header and screen system has been installed a precast concrete lid will be installed on top of the intake structure.
- After all elements of the intake system are complete the sheet piles and trestle may be removed progressively back to the shore.

The WWS will require an air burst system to keep the debris from clogging the screens. For the WWS option based on using the existing intake structure, the air compressor/receivers could be placed by the forebay of the existing power generating station. Individual air piping could then be routed through the existing 14 ft diameter intake line. For the D2 intake option, it would no longer be possible to place the air compressors/receivers at the power generating site as the total distance that the air piping would need to be routed is nearly 9,000 ft. The air compressors/receivers would need to be located on-shore in the vicinity of the piping where it turns to go offshore. This would reduce the distance air piping distance to approximately 2600 ft, which is manageable. This would require that Poseidon is able to obtain land on the state beach to build an aboveground structure to house the compressor/air receivers. For purpose of this Technical Memo, it is assumed that Poseidon would be able to lease land for the air compressor building.

Preliminary Schedule: The preliminary construction schedule for the installation of alternative intake at the D2 location is shown in Attachment 1. The preliminary schedule includes the timing for the onshore construction as well as the offshore work. The offshore work requires the use of vibratory hammers and limited impact pile driving equipment, and therefore the work has been restricted to comply with the whale migration requirements as described in the 2017 SEIR – July 1 through October 31. The schedule also includes schedule contingency risk for both the offshore and onshore work. Onshore work may be impacted by Least Tern nesting restrictions which would prolong the pipe laying operation. For the offshore work, schedule contingency was added to account for delays caused by sea conditions. Least Tern nesting also impacts offshore work as materials for the offshore work are moved from the onshore staging area to the trestle, and this risk is included in the schedule. The schedule also illustrates the relationship between the onshore desalination plant construction and construction of the new D2 intake system. The commissioning of the desalination plant can't begin until the intake system is installed. The removal of the trestle can, however, extend past the start of plant commissioning.

Budgetary Pricing: A rough order magnitude cost has been prepared for the onshore option and is presented in Attachment 2.

Highlands Construction Solutions, LLC
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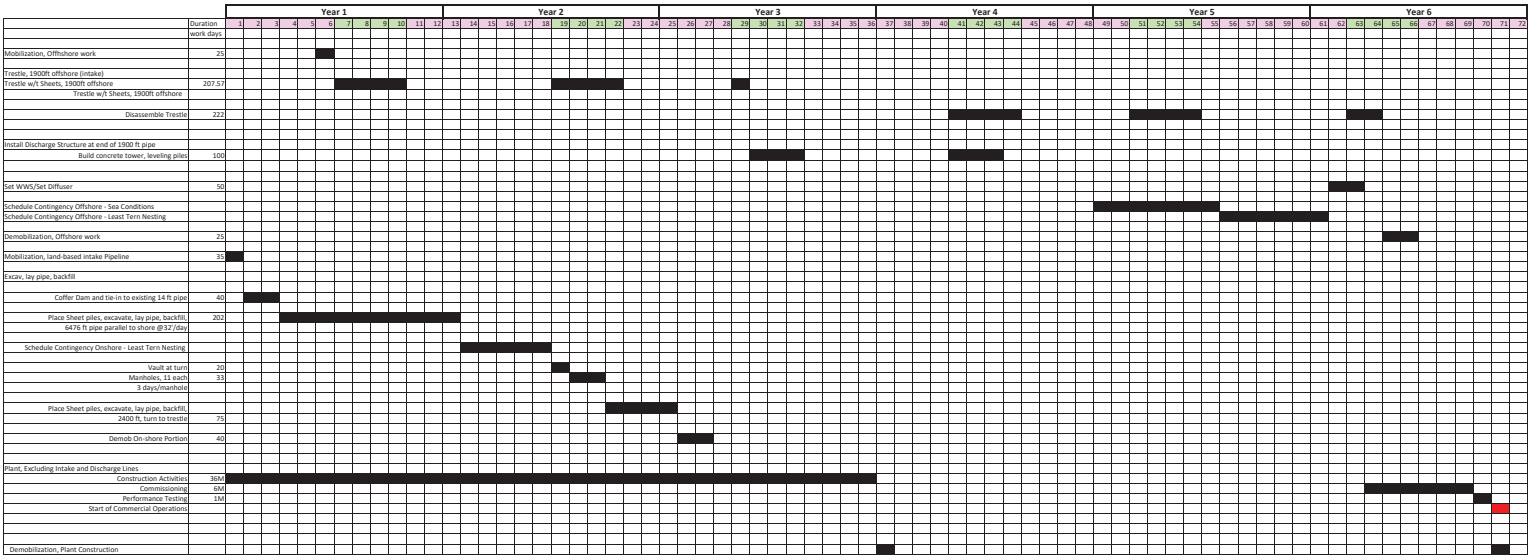
Hcsllc2015@gmail.com
(360)772-2799

Poseidon

Alternate Intake D2



Months highlighted in Green not subject to whale migration restrictions

Revision 2

PROJECT: Huntington Beach Desal
ITEM: Onshore Alignment to D2

ESTIMATOR: BP
DATE: August 6, 2018

DESCRIPTION	QUANTITY		EQUIPMENT RENTAL				OPERATING EXPENSE								LABOR EXPENSE											
			CO. OWNED		OTHER		TIRES & TRACKS		EQUIP REPAIR		FUEL-OIL-GREASE		SERVICES TOOLS & SUPPLIES		WAGES		ADD-ONS		SUBTOTAL		PERMANENT MATERIALS		SUB CONTRACTS		TOTAL DIRECT COST	
			U.P.	TOTAL	U.P	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL	U.P.	TOTAL
c'dams @ tie-in & corner	3,200	mh													110	352,000				352,000						352,000
sheets for cofferdams	108,000	lb											0.90	97,200						97,200						97,200
frame for cofferdams	30,000	lb											1.50	45,000						45,000						45,000
seal concrete	500	cy											200.00	100,000						100,000						100,000
pipe run on beach	47,250	mh													110	5,197,500				5,197,500						5,197,500
sheets for beach run	669,900	lb											0.90	602,910						602,910						602,910
frame and struts @ beach	206,000	lb											1.50	309,000						309,000						309,000
pipe for beach run	6,876	lf																		0	1,000	6,876,000				14,989,680
bedding for beach run	14,000	tn												0						0	40	560,000				560,000
vault at South turn	1	ls												0						0	200,000	200,000				200,000
trestle/sheets/pipe	109,440	mh												0	110	12,038,400				12,038,400						12,038,400
erect/strip trestle equip.	4,000	mh												0	110	440,000				440,000						440,000
trestle vertical pile	17,780	lf											126.00	2,240,280						2,240,280						2,240,280
trestle batter pile	5,120	lf											63.00	322,560						322,560						322,560
dive crew	545	dy																		0			7,000	3,815,000		3,815,000
trestle caps	321,600	lb											1.50	482,400						482,400						482,400
trestle rail beams	372,000	lb											1.50	558,000						558,000						558,000
trestle rail	114,000	lb											1.00	114,000						114,000						114,000
trestle walkway	7,600	sf											25.00	190,000						190,000						190,000
trestle wale beams	206,000	lb											1.50	309,000						309,000						309,000
sheets for trestle	2,296,800	lb											0.90	2,067,120						2,067,120						2,067,120
trestle template	1	ls											50,000.00	50,000						50,000						50,000
trestle crane gantries	2	ea											1,000,000.00	2,000,000						2,000,000						2,000,000
trestle pipe gantry	1	ea											1,000,000.00	1,000,000						1,000,000						1,000,000
trestle material cart	1	ea											750,000.00	750,000						750,000						750,000
pipe for trestle run	1,900	lf																			2180	4,142,000				4,142,000
bedding for trestle run	4,200	tn																			40	168,000				168,000
rip rap pipe protection	15,200	tn																			40	608,000				608,000
utilities on trestle	1,900	lf											10.00	19,000						19,000						19,000
precast intake structure	1	ls																			300000	300,000				300,000
equip. move in/out beach	1	ls											200,000.00	200,000						200,000						200,000
equip. move in/out trestle	1	ls											200,000.00	200,000						200,000						200,000
trestle seasonal moves	1	ls											1,000,000.00	1,000,000						1,000,000						1,000,000
trestle crane down time rent	80	mo			20000	1600000														1,600,000						1,600,000
beach cranes	106	wk			7665	812490														812,490						812,490
beach articulated trucks	159	wk			5595	889605														889,605						889,605
beach welding machines	159	wk			333	52947														52,947						52,947
beach 185 cfm air comp	106	wk			619	65614														65,614						65,614
beach 20,000 lb forklift	53	wk			2538	134514														134,514						134,514
beach yard 50,000 lb fork	53	wk			6018	318954														318,954						318,954
beach APE 200 vibrohammer	106	wk			8437	894322														894,322						894,322
beach excavator	53	wk			7744	410432														410,432						410,432
beach 980 loader	53	wk			5906	313018														313,018						313,018
trestle cranes	176	wk			7665	1349040														1,349,040						1,349,040
trestle pipe gantry op cost	88	wk			1400	123200														123,200						123,200
trestle material cart op cost	88	wk			1200	105600														105,600						105,600
trestle vibrohammers	176	wk			8437	1484912														1,484,912						1,484,912
trestle impact hammer	88	wk			1512	133056														133,056						133,056
trestle service forklift	88	wk			2538	223344														223,344						223,344
trestle welding machines	264	wk			333	87912														87,912						87,912
trestle 185 cfm comp.	264	wk			619	163416														163,416						163,416
pickups - all	438	wk			653	286014														286,014						286,014
misc. outside equip. rent	1	ls				675000														675,000						675,000
mechanic for down time	6,400	mh													110	704,000				704,000						704,000
crew consumables	170,290	mh											10.00	1,702,900						1,702,900						1,702,900
Installation of WWS Assembly	1	ls																					17,000,000	17,000,000		17,000,000

Airburst																										
Bldg onshore (State Beach)	1	ls																					500,000	500,000		500,000
Compressors/Air Receivers	1	ls																					1,200,000	1,200,000		1,200,000
Electrical Service, setup	1	ls																					250,000	250,000		250,000
Air Piping	10,400	lf																					150	1,560,000		1,560,000
Allowance for leasing land	1	ls																					1,000,000	1,000,000		1,000,000
SUBTOTAL	1	ls				3,555,170				0				11,502,360		18,027,900				34,239,540				3,815,000		89,507,340
Indirects (25% of subtotal)																										22,376,835
Insurance & enviro (5%)																										4,475,367
Overhead & profit (25%)																										22,376,835
SUBTOTAL																										138,736,377
engineering (15%)																										20,810,457
contingency (40%)																										55,494,551
TOTAL ESTIMATED COST																										215,041,384

ATTACHMENT I

Conditions Assessment

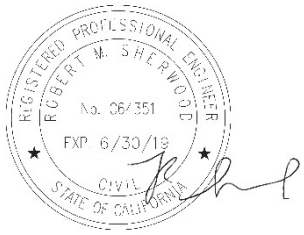
Poseidon Water

AES Huntington Beach Desalination Plant

Intake and Discharge Pipeline Integrity Assessment Inspection and Report

Robert M. Sherwood, P.E.

CA License No. C64351



GHD | 175 Technology Drive, Suite 200, Irvine, CA 92618

11110796 | Report No 01 | March 18, 2019

Executive Summary

The goal of the pipeline inspection and this subsequent Pipeline Integrity Assessment Inspection and Report is to document the condition of the existing AES Huntington Beach reinforced concrete intake and discharge piping and related structures, assess their structural integrity and make recommendations for the intended future use and maintenance of this key infrastructure. This due diligence measure was identified by Poseidon Water during the permitting process as a voluntary lease condition.

From February 12 to 14, 2018, the 14-foot diameter reinforced concrete intake and discharge pipes were inspected by Subsea Global Solutions (SGS), formerly Parker Diving Service, under the supervision of Robert M. Sherwood, PE, a civil/waterfront engineer with 18+ years of marine structures design and inspection experience. Other structures inspected included the submerged portions of the onshore Intake Screenwell Structure, along with the offshore intake and discharge towers. In addition to inspection divers, SGS used a remotely operated underwater vehicle (ROV) that was launched from onshore into the piping, via the opening at the Intake Screenwell Structure, to document the condition of the interior of the pipes and offshore towers. The ROV traversed the entire lengths of both pipes and the video feed from the ROV was reviewed during the investigation to determine the condition of the pipe and whether portions of the pipe are structurally compromised.

Concrete cores were extracted from the onshore and offshore ends of the intake and discharge piping and the sides of the offshore tower structures. The cores were tested by Tourney Consulting Group (TCG) for compressive strength (which relates to the concrete density), the amount of acid-soluble chloride (which relates to the potential intrusion of chlorides into the concrete), and were analysed with a petrographic examination (for assessing the condition, pH and composition of the concrete).

Findings

The existing intake and discharge pipes are 14-foot inside diameter reinforced concrete pipe installed in the early 1960s. The seawater intake pipeline extends offshore approximately 1,840 feet seaward of the beach while the discharge pipeline (outfall) terminates 1,500 feet offshore. Areas of the pipe inner wall surface exposed by SGS inspection divers or at air pockets along the soffit of the pipe showed no signs of corrosion or spalling and were generally observed to be in good condition. No noticeable leaks or infiltration of material were observed along the lengths of the piping. Manholes along both pipelines were inspected and found to be airtight and in good condition.

The precast concrete intake and discharge towers at the offshore ends of the pipes were found to be in generally good condition by divers and the ROV team. The grouted seam between both pipes and towers was found to have some spalling and irregularities up to 4 inches deep in need of patching to maintain the future integrity of the pipeline.

Core samples taken from the ends of the intake and discharge pipelines displayed the following:

- An average concrete compressive strength in the range of 6,000 psi, appropriate for precast concrete piping in the marine environment and suggesting a low water to cement ratio for the original pipe concrete mix which likely used about 750 pounds of cementitious material per cubic yard of concrete.

- The chloride concentration tests indicate that the walls of the pipes are subject to the migration of chlorides into the concrete at varying levels. The chloride penetration concentrations at the depth of the inner most ring of reinforcing steel are critical in our analysis, and levels at this 2-1/2 to 3 inch depth were observed to be on the order of 341 to 1,844 parts per million for the four pipe cores. The correlating range of values of chloride ion per mass of cement in the concrete were measured to be from 0.2% to 1.0%, with 0.4% being an established threshold value for the onset of chloride induced corrosion of the concrete according to internationally recognized standards.
- The petrographic examination of the pipeline core sample similarly showed that the inside of the pipe is subject to micro cracks within approximately 0.5 inch from the surface, and chemical leaching and reduced crystallinity of the cement paste within 2.0 inches of pipe wall.
- Application of pH-indicator found reduced pH levels (below 9) within 0.3 inches of pipe wall.

The reinforcing steel remnants observed in the pipe cores appeared to be in very good condition suggesting that there is adequate cover and density of concrete protecting the steel for the over 50 years that it has been in service. The concrete aggregate appeared to be well graded throughout the core samples taken with a good distribution of coarse and fine particles, indicating good placement and consolidation of concrete during pipe manufacture. These combined tests indicate that the pipe reinforcing steel may be subject to the initiation of corrosion over the 40-year design life of the new desalination plant, though no visible signs of corrosion were observed on walls of the pipeline or in the core samples.

The core samples taken from the walls of the intake and discharge towers displayed the following:

- An average concrete compressive strength of around 5,000 psi, a minimum appropriate value for a precast concrete structure in the marine environment and suggesting a low water to cement ratio for the original pipe concrete mix which likely used about 700 pounds of cementitious material per cubic yard of concrete.
- The chloride concentration tests indicate that the walls of the tower are also subject to the migration of chlorides into the concrete at varying levels. The chloride penetration concentrations at the likely depth of reinforcing steel are critical in our analysis, and levels at this 2-1/2 to 3 inch depth were observed to be 7,217 parts per million for the Intake Tower and 2,278 parts per million for the Discharge Tower. The correlating values of chloride ion per mass of cement in the towers were 4.18% and 1.32%, values that indicate significant potential for chloride induced corrosion of the precast concrete.
- The petrographic examination of the tower core sample similarly showed that the inside of the pipe is subject to micro cracks within approximately 0.75 inch from the surface, and chemical leaching and reduced crystallinity of the cement paste within 2.4 inches of pipe wall.
- Application of pH-indicator found reduced pH levels (below 9) within 0.6 inches of pipe wall.

No reinforcing steel was observed in the concrete cores taken from the intake and discharge towers. The concrete aggregate appeared to be well graded throughout the core samples taken with a good distribution of coarse and fine particles, indicating good placement and consolidation of concrete during manufacture of the precast structure. The test results indicate that the tower concrete is slightly less dense than the pipeline concrete and may be subject to higher levels of chloride intrusion, though no visible signs of corrosion were observed on walls of the offshore towers or in the core samples.

Recommendations

The proposed intake and discharge pipeline modifications will be designed so as not to add significant head loss to the existing flow thru capacity of the pipe and associated structures. Alternatives for the intake/discharge system components and configuration were developed and refined during the concept development and CEQA/EIR permitting process. The top of the existing discharge tower will be sealed up with a new concrete cap, and three duckbill diffusers and a center port will be installed on the top of the tower. With the proposed modifications the head loss is estimated to be 4.99 feet H_2O, which is within the structural design parameters for the existing discharge pipe and within the pressure ranges that the pipe is subject to under current operating conditions.

The submerged intake and discharge piping has held up well for the over 50 years that it has been in service. Per the SGS report, both the diver and ROV inspections of the pipeline indicated that it appeared to be in good condition with no major discrepancies encountered. Robert Sherwood, P.E. was supervising this effort and was present during the entire ROV inspection and noted that there were no significant signs of distress or leaking of the pipelines.

The grouted connections between the 14-foot diameter pipelines and the offshore towers were observed to have some spalling and irregularities in need of patching to maintain the future integrity of the pipeline. While desalination related modifications are being made to the intake and discharge towers, the grouted seams between the pipe and the towers should be re-grouted where voids or areas of distress were observed to prolong the functional life of this structure. A grout material such as 5-Star Products® Cementitious Underwater High-Strength Grout can be placed in situ by divers, and the repair may also require that an interior liner be applied in this location with an underwater cure epoxy-based binder.

Concrete core analysis and visual observations of the condition of the pipe suggest that the pipe is reaching roughly the mid-point of its service life, with the initiation of corrosion of the pipe reinforcing steel likely to occur over the anticipated 40-year design life of the new desalination plant, and that the pipelines should function as intended since corrosion is anticipated to be minimal over this period.

Increased salinity in the discharge pipeline is anticipated to roughly increase from 34.5 parts per thousand (ppt) to a maximum of 70 ppt once the desalination plant starts operation. This inspection report should serve as a baseline from which to estimate the project service life of the existing pipeline with increased salinity in the discharge. This increased salinity should not affect the integrity of the submerged pipeline due to the relatively low oxygen levels, especially as compared to a concrete structure above water in the splash zone.

Future periodic inspections should provide the means to confirm that the service life estimates remain valid throughout the operating period. GHD anticipates that the piping and related structures should be visually inspected every 10 years, with core samples taken in the same locations every 20 years to properly establish the condition of the intake and discharge systems over time. If significant deterioration and spalling of the offshore concrete pipe is observed in future investigations, slip-lining the interior of the pipe with a new large diameter HDPE pipe may need to occur (we estimate approximately 50 years or more from this date).

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1. Background

The goal of the pipeline inspection and this subsequent Pipeline Integrity Assessment Inspection and Report is to document the condition of the existing AES Huntington Beach reinforced concrete intake and discharge piping and related structures, assess their structural integrity and make recommendations for the intended future use and maintenance of this key infrastructure. This due diligence measure was identified by Poseidon Water during the permitting process and undertaken as a voluntary lease condition.

The seawater intake pipeline extends offshore approximately 1,840 feet offshore, while the discharge pipeline (outfall) terminates approximately 1,500 feet offshore, as shown below in Figure 1. The 14-foot inner diameter reinforced concrete intake and discharge pipelines have inlet and outlet structures on the seafloor at their offshore terminus to direct flow and deter fish and mammals from entering the pipes.

Figure 1 HB Desalination Plant with Intake/Discharge Pipelines
(Source: Digitalglobe 2007, City of Huntington Beach 2010)

1.1 Past Pipeline Inspections by AES

Recent past inspections of the AES power plant intake and discharge piping and towers have been performed by Subsea Global Solutions (SGS), formerly Parker Diving Service, on a regular basis with previous inspections occurring on the following dates with a summary of the related findings:

March 2010 – Intake and Discharge Tower Inspection

Intake Tower

- mammal barrier grate cleaned and repaired
- top corner of tower has some damage (note: will be replaced during Desalination Plant retrofit)

Discharge Tower

- grating was found to be off the opening at top of tower

February 2011 – Circulating Water System Inspection

Intake Pipe

- Pipe inspected from onshore Screenwell Structure to 1,000 feet offshore, with no spalling, deterioration or any other structural abnormalities to the concrete observed in the pipeline.

Discharge Pipe

- Pipe inspected from onshore Screenwell Structure to 1,000 feet offshore, with no spalling, deterioration or any other structural abnormalities to the concrete observed in the pipeline.

November 2013 – Intake and Discharge Tower Inspection

Intake Tower

- mammal barrier grate cleaned and repaired
- top corner of tower has some damage (note: will be replaced during Desalination Plant retrofit)

Discharge Tower

- grating was found to be off the opening at top of tower
- chain is wearing groove in top of tower (note: will be replaced during Desalination Plant retrofit)

February 2014 – Mammal Barrier Installation and Outfall Inspection

Intake Tower

- mammal barrier grate cleaned and repaired

Discharge Tower

- grating was found to be off the opening at top of tower

December 2016 – Mammal Barrier Installation

Intake Tower

- mammal barrier grate cleaned and repaired

In general, the intake and discharge tower barriers and grating were in need of constant cleaning, maintenance and repair. Additionally, the tops of the concrete towers have some minor spalls and wear areas from chains and other gear dragging over the structures. Worth noting is that the existing tops of the offshore towers will be replaced with new concrete lids during the desalination plant retrofit.

1.2 Inspection Approach

From February 12 to 14, 2018, the 14-foot diameter reinforced concrete intake and discharge pipes were inspected by SGS under the supervision of Robert M. Sherwood, PE, a civil/waterfront engineer with 18+ years of marine structures design and inspection experience. Other structures inspected included the submerged portions of the onshore Intake Screenwell Structure, along with the offshore intake and discharge towers (see Figure 2). In addition to inspection divers, SGS used a remotely operated underwater vehicle (ROV) that was launched from onshore into the piping, via the opening at the Intake Screenwell Structure, to document the condition of the interior of the pipes and offshore towers. The ROV traversed the entire length of both pipes and the video feed from the ROV was reviewed during the investigation to determine the condition of the pipe and whether the pipe and offshore towers are structurally compromised.

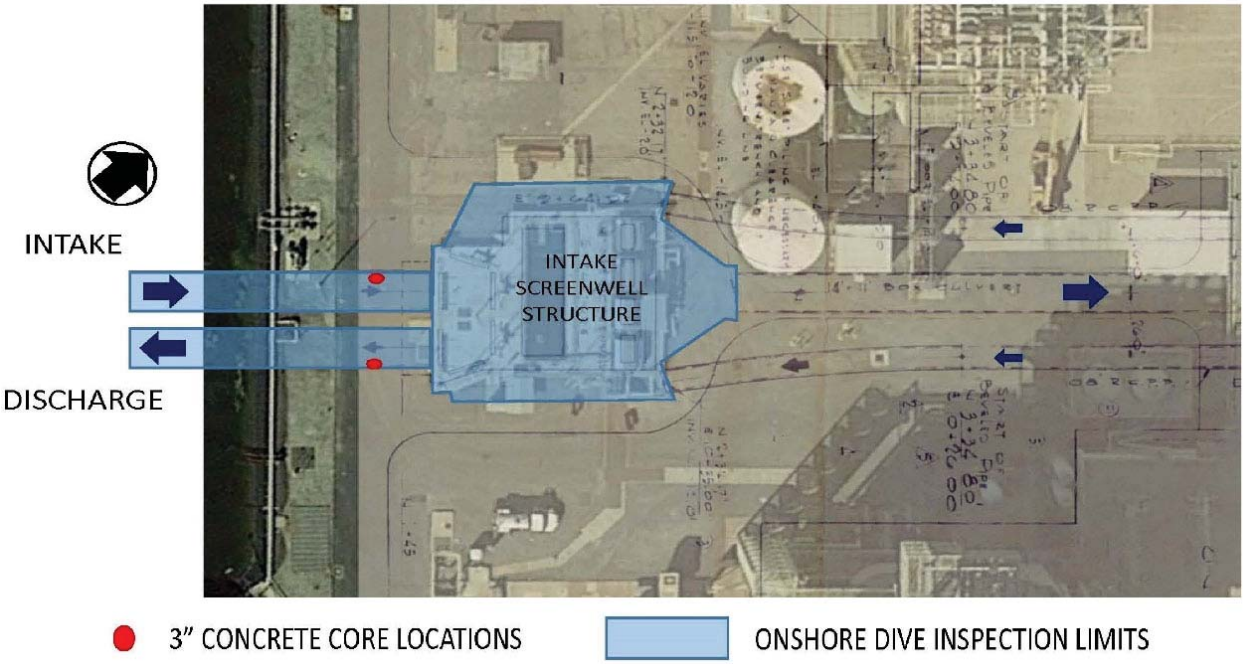

Figure 2 Onshore Dive Inspection and Core Locations

A total of six (6) concrete cores were extracted from the onshore and offshore ends of the intake and discharge piping and the sides of the offshore tower structures (see Figure 3). The dimensions of the cores were approximately 3 inches in diameter by 11 to 14 inches long. The cores were tested by Tourney Consulting Group (TCG) for compressive strength (which relates to the concrete density), the amount of acid-soluble chloride (which relates to the potential intrusion of chlorides into the concrete), and were analysed with a petrographic examination (for assessing the condition, pH and composition of the concrete).

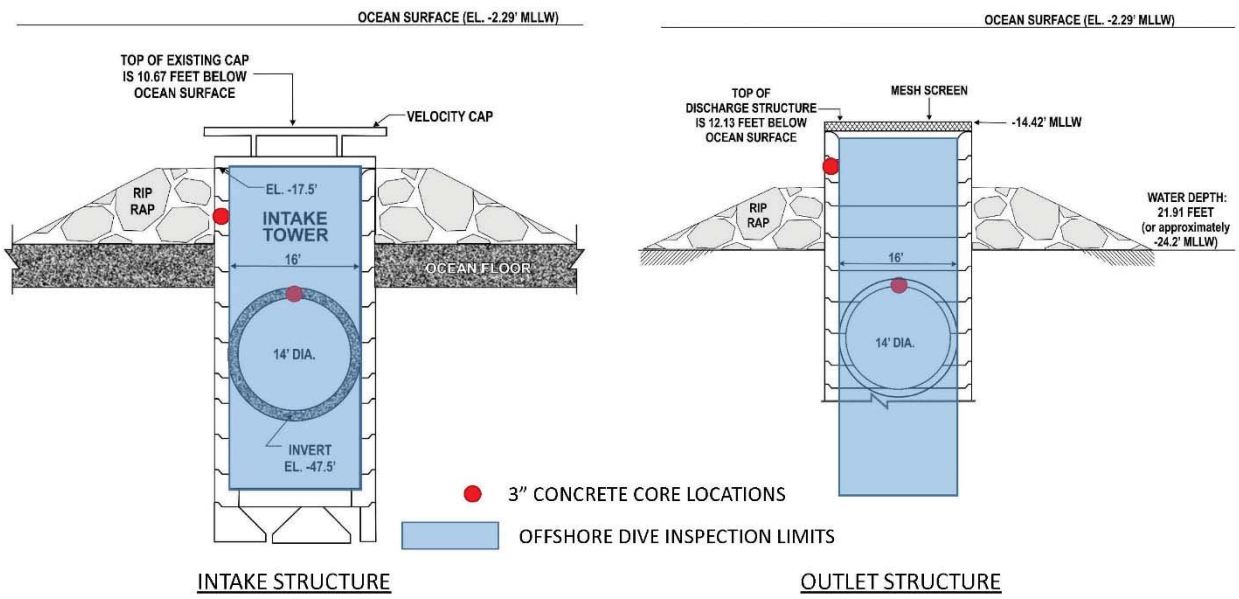

Figure 3 Offshore Intake and Discharge Tower Inspection and Core Locations

1.3 Inspection Methodology

Dive inspection followed the American Society of Civil Engineers (ASCE) *Underwater Investigations Standard Practice Manual* [9] to categorize the conditions observed. Per this manual, inspections may focus on three levels of effort summarily described as follows:

Level I – Visual or tactile inspection of underwater components without removal of marine growth.

Level II – Partial marine growth removal of a statistically representative sample.

Level III – Non-destructive testing or partially destructive testing of a representative sample.

All three levels of underwater inspection were incorporated during the SGS dive inspection of the Intake Screenwell Structure, Pump Intake Structure, pipelines and offshore intake and discharge towers (structures). Level III testing for this assessment includes extracting concrete cores for analysis.

2. Dive and ROV Observations

2.1 Existing Physical Conditions

The existing intake and discharge pipes, and related structures such as the intake and discharge towers, were installed in the early 1960s. The orientations of the pipes and structures, along with approximate manhole locations, are shown below in Figure 4.

Figure 4 Intake/Discharge Pipeline, Manhole and Tower (Structure) Locations

2.1.1 Intake and Discharge Piping

Areas of the pipe inner wall surface exposed by SGS inspection divers or at air pockets along the soffit of the pipe showed no signs of corrosion or spalling and were generally observed to be in good condition.[2] No noticeable leaks or infiltration of material were observed along the lengths of either the intake or discharge piping. Manholes along both pipelines were inspected and found to be airtight and in good condition.

2.1.2 Intake and Discharge Towers (Structures)

The precast concrete intake and discharge towers at the offshore ends of the pipes were found to be in generally good condition by divers and the ROV team.[2] The grouted seam between both pipes and towers was found to have spalling and irregularities up to 4 inches deep in need of patching.

3. Concrete Test Results

The concrete cores were labelled, packed, and shipped by GHD to TCG to be tested for compressive strength (which relates to the concrete density), the amount of acid-soluble chloride (which relates to the potential intrusion of chlorides into the concrete), and a petrographic examination (for assessing the condition, pH and composition of the concrete).[3]

Testing was in accordance with the following standards:

- ASTM C42: Standard Test Method for Obtaining and Testing Drilled Cores (compressive strength)
- ASTM C1152: Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete
- ASTM C856: Petrographic Examination of Hardened Concrete

3.1 Test Results for Intake and Discharge Pipelines

Concrete core samples were taken from the onshore and offshore ends of the intake and discharge pipelines (four total) and displayed the following:

- An average concrete compressive strength in the range of 6,000 psi, appropriate for precast concrete piping in the marine environment and suggesting a low water to cement ratio for the original pipe concrete mix which likely used about 750 pounds of cementitious material per cubic yard of concrete.
- The chloride concentration tests indicate that the walls of the pipes are subject to the migration of chlorides into the concrete at varying levels. The chloride penetration concentrations at the depth of the inner most ring of reinforcing steel are critical in our analysis, and levels at this 2-1/2 to 3 inch depth were observed to be on the order of 341 to 1,844 parts per million for the four pipe cores. The correlating range of values of chloride ion per mass of cement in the concrete were measured to be from 0.2% to 1.0%, with 0.4% being an established threshold value for the onset of chloride induced corrosion of the concrete according to internationally recognized standards.[8]
- The petrographic examination of the pipeline core sample similarly showed that the inside of the pipe is subject to micro cracks within approximately 0.5 inch from the surface, and chemical leaching and reduced crystallinity of the cement paste within 2.0 inches of pipe wall.
- Application of pH-indicator found reduced pH levels (below 9) within 0.3 inches of pipe wall.

The reinforcing steel remnants observed in the pipe cores appeared to be in very good condition (see Figure 5) suggesting that there is adequate cover and density of concrete protecting the steel for the over 50 years that it has been in service. The concrete aggregate appeared to be well graded throughout the core samples taken with a good distribution of coarse and fine particles, indicating good placement and consolidation of concrete during pipe manufacture. These combined tests indicate that the pipe reinforcing steel may be subject to the initiation of corrosion over the 40-year design life of the new desalination plant, though no visible signs of corrosion were observed on the walls of the pipeline or in the core samples.

Figure 5 Freshly Extracted Concrete Pipe Cores with Reinforcing Steel

3.2 Test Results for Intake and Discharge Towers

A total of two (2) concrete core samples were taken from the walls of the intake and discharge towers and displayed the following:

- An average concrete compressive strength of around 5,000 psi, a minimum appropriate value for a precast concrete structure in the marine environment and suggesting a low water to cement ratio for the original pipe concrete mix which likely used about 700 pounds of cementitious material per cubic yard of concrete.
- The chloride concentration tests indicate that the walls of the tower are also subject to the migration of chlorides into the concrete at varying levels. The chloride penetration concentrations at the likely depth of reinforcing steel are critical in our analysis, and levels at this 2-1/2 to 3 inch depth were observed to be 7,217 parts per million for the Intake Tower and 2,278 parts per million for the Discharge Tower. The correlating values of chloride ion per mass of cement in the towers were 4.18% and 1.32%, values that indicate significant potential for chloride induced corrosion of the precast concrete.
- The petrographic examination of the tower core sample similarly showed that the inside of the pipe is subject to micro cracks within approximately 0.75 inch from the surface, and chemical leaching and reduced crystallinity of the cement paste within 2.4 inches of pipe wall.
- Application of pH-indicator found reduced pH levels (below 9) within 0.6 inches of pipe wall.

No reinforcing steel was observed in the concrete cores taken from the offshore intake and discharge towers. The concrete aggregate appeared to be well graded throughout the core samples taken with a good distribution of coarse and fine particles,[3] indicating good placement and consolidation of concrete during manufacture. The test results indicate that the tower concrete is slightly less dense than the pipeline concrete and may be subject to higher levels of chloride intrusion, though no visible signs of corrosion were observed on walls of the offshore towers or in the core samples.

4. Discussion

4.1 Proposed Versus Original Design Flows and Head Losses

Per the findings of the May 2017 SEIR [1] (Page 2-20):

> *… Poseidon has provided two engineering studies of projected outfall pipeline back-pressure, in terms of head loss, resulting from adding a multiport diffuser (Alden 2017a, Alden 2017b). In addition, in a review of the design capacity of the existing pipeline under the following baseline conditions, GHD (2017a) reported the following.*
>
> - ***Maximum design flow, original conditions.*** *The discharge facilities were designed to carry discharge cooling water from four power generating units at 514 MGD. Under these original conditions and design flow conditions, the head loss ranged from 5.46 to 5.83 feet of water elevation head (feet H_2O).*
> - ***Maximum flow, existing conditions.*** *Currently, generating Units 1 and 2 are in operation and Units 3 and 4 are operating in a synchronized condenser (a device that is not connected to anything but spins freely) mode (using little or no cooling water), for a total of up to 387 MGD. Under these baseline flow conditions, the head loss ranges from 4.88 to 5.12 feet H_2O.*
> - ***Effect on system pressure****. Reduced flow (from 514 to 387 MGD) in the system from original design to existing conditions would reduce system pressure by 10 to 13 percent. When the HBGS ceases OTC use, the reduction in the volume of water discharged from the HB Desalination Plant would further reduce the system pressure compared with the design pressure of the discharge system.*
>
> *While the average annual flow during stand-alone operation of the HB Desalination Plant is expected to be approximately 106.7 MGD for the intake and 56.7 MGD directed out of the discharge, the engineering study of back-pressure with the proposed modifications conducted by Alden (2017b) also considered the proposed diffuser under an operational scenario of 127 MGD through only the three duckbill diffusers (with the center port capped). With the proposed modifications, Alden (2017b) estimated the head loss to be 4.99 feet H_2O, or within the structural design parameters for the existing discharge pipe and within the pressure ranges that the pipe is subject to under current operating conditions.*

In other words, the head losses, and related back-pressure, in the proposed design are similar to the existing condition and should not adversely impact the performance and integrity of the submerged piping and appurtenances such as the airtight manholes. Verification of this approach will be provided during the detailed final design phase of the project.

4.2 Chloride Ion Penetration in Reinforced Concrete

The American Concrete Institute (ACI) Committee Report 222R, Protection of Metals in Concrete against Corrosion,[8] states:

> *Chloride ions are common in nature and very small amounts are normal in concrete-making materials. Chloride ions may also be intentionally added into the concrete, most often as a*

constituent of accelerating admixtures. Dissolved chloride ions may also penetrate hardened concrete in structures exposed to marine environments or to deicing salts.

The rate of corrosion of steel reinforcement embedded in concrete is influenced by environmental factors. Both oxygen and moisture must be present if electrochemical corrosion is to occur. Reinforced concrete with significant gradients in chloride-ion content is vulnerable to macrocell corrosion, especially when subjected to cycles of wetting and drying. This condition often occurs in highway bridges and parking structures exposed to deicing salts and in structures in marine environments. Other factors that affect the rate and level of corrosion are heterogeneity in the concrete and the reinforcing steel, pH of the concrete pore water, carbonation of the portland cement paste, cracks in the concrete, stray currents, and galvanic effects due to contact between dissimilar metals. Design features and construction practices also play an important role in the corrosion of embedded steel. Mixture proportions of the concrete, thickness of concrete cover over the reinforcing steel, crack-control measures, and implementation of measures designed specifically for corrosion protection are some of the factors that help control the onset and rate of corrosion.

Increased salinity in the discharge pipeline is anticipated to roughly increase from 34.5 parts per thousand (ppt) to a maximum of 70 ppt once the desalination plant starts operation.

Per the May 2017 SEIR [1] (Page 2-21):

> *Introduction of concentrated brine to the discharge pipeline may also increase the rate of salt and related deterioration mechanisms in the concrete such as chloride-induced rebar corrosion and salt scaling. Since the outfall is submerged and not subjected to wet/dry conditions, the effects would not be as pronounced as when compared to similar salinity exposure in tidal (wet/dry) areas. In addition, the discharge pipeline was designed for service in seawater, which would have included provisions to mitigate chloride-induced rebar corrosion. Recent assessment of the existing Encina Power Station outfall in Carlsbad, which was built at a similar time to the HBGS and which operates in a similar environment, has shown these concrete pipelines to exhibit higher durability than similarly-designed structures that were built slightly later (GHD 2017b).*

Provisions to mitigate chloride-induced rebar corrosion in marine concrete mentioned in the SEIR description would typically include the following:

- Using a high-strength (dense) concrete mix with a compressive strength of 5,000 psi or greater
- A concrete mix with a water to cement ratio of 0.40 or less
- A concrete mix with a durable type of Portland cement appropriate for the marine environment (Type II or V)
- Substituting the cement in the concrete mix with up to 25% pozzolanic materials such as silica fume and/or Class F fly ash
- Addition of a corrosion inhibiting admixture
- A minimum concrete cover of 2 to 2-1/2 inches over reinforcing steel

From our observations and core sample test results, it appears that the concrete used for the pipeline at AES Huntington Beach is appropriate for use in the marine environment (likely utilizing most if not all of these provisions), especially having held up well for the over 50 years that it has been in continual service.

5. Summary and Recommendations

The submerged intake and discharge piping has held up well for the over 50 years that it has been in service. Per the SGS report, both the diver and ROV inspections of the pipeline indicated that it appeared to be in good condition with no major discrepancies encountered. Robert Sherwood, P.E. was supervising this effort and was present during the entire ROV inspection and noted that there were no significant signs of distress or leaking of the pipelines. Concrete core analysis suggests that the pipe is reaching roughly the mid-point of its service life, with the initiation of corrosion of the pipe reinforcing steel likely to occur over the 40-year design life of the new desalination plant, and that the pipelines should function as intended since corrosion is anticipated to be minimal over this period.

The proposed intake and discharge pipeline modifications will be designed so as not to add significant head loss to the existing flow thru capacity of the pipe and associated structures. Alternatives for the intake/discharge system components and configuration were developed and refined during the concept development and CEQA/EIR permitting process. The top of the existing discharge tower will be sealed up with a new concrete cap, and three duckbill diffusers and a center port will be installed on the top of the tower. With the proposed modifications the head loss is estimated to be 4.99 feet H_2O,[6] within the structural design parameters for the existing discharge pipe and within the pressure ranges that the pipe is subject to under current operating conditions.

The grouted connections between the 14-foot diameter pipelines and the offshore towers were observed to have some spalling and irregularities in need of patching to maintain the future integrity of the pipeline. While desalination related modifications are being made to the intake and discharge towers, the grouted seams between the pipe and the towers should be re-grouted where voids or areas of distress were observed to prolong the functional life of this structure. A grout material such as 5-Star Products® Cementitious Underwater High-Strength Grout can be placed in situ by divers, and the repair may also require that an interior liner be applied in this location with an underwater cure epoxy-based binder.

Increased salinity in the discharge pipeline is anticipated to roughly increase from 34.5 parts per thousand (ppt) to a maximum of 70 ppt once the desalination plant starts operation. This inspection report should serve as a baseline from which to estimate the project service life of the existing pipeline with increased salinity in the discharge. This increased salinity should not adversely affect the integrity of the submerged pipeline over the next 40 years. This is due to the relatively low oxygen levels, especially as compared to a concrete structure above water in the splash zone. Future periodic inspections should provide the means to confirm that the service life estimates remain valid throughout the operating period.

GHD anticipates that the piping and related structures should be visually inspected every 10 years, with core samples taken in the same locations every 20 years to properly establish the condition of the intake and discharge systems over time. If significant deterioration and spalling of the offshore concrete pipe is observed in future investigations, slip-lining the interior of the pipe with a new large diameter high-density polyethylene (HDPE) pipe may need to occur (we estimate approximately 50 years or more from this date).

6. References

1. Draft Supplemental Environmental Impact Report for the Seawater Desalination Project at Huntington Beach: Outfall/Intake Modifications & General Lease Industrial Use (PRC1980.1) Amendment (Lease Modification Project); State Clearinghouse No. 2001051092; California State Lands Commission EIR Number: 794; PRC1980.1.
2. Subsea Global Solutions (SGS) 2018. AES Huntington Beach Intake/Discharge Piping and Structure Condition Assessment, February 12 to 14, 2018.
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4. Alden (Alden Research Laboratory, Inc.) 2016. Huntington Beach Desalination Plant Intake/Discharge Feasibility Assessment. Report Prepared for Poseidon Water. March 14, 2016. (Included as Appendix H of Poseidon's Application to the CSLC).
5. Alden (Alden Research Laboratory, Inc.) 2017a. Huntington Beach Desalination Facility Discharge Diffuser Design. February 23. Report prepared for Poseidon Resources (Surfside) LLC. (Included as a supplement to Poseidon's Application to the CSLC).
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7. American Concrete Institute, Building Code Requirements for Structural Concrete (ACI 318)
8. American Concrete Institute, Protection of Metals in Concrete Against Corrosion (ACI 222R)
9. American Society of Civil Engineers (ASCE) 2001. Underwater Investigations Standard Practice Manual No. 101.

Huntington Beach Intake/Discharge Piping

Concrete Durability Assessment

Huntington Beach, California

Prepared For:

GHD, Inc.
175 Technology Drive, Suite 200
Irvine, CA 92618

Attn: Robert Sherwood, PE

April 19, 2018

Prepared By:

TOURNEY CONSULTING GROUP, LLC

Mark Dixon, E.I.T.
Engineer II

Paul Tourney, P.E.
President

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Introduction

Tourney Consulting Group (TCG) conducted laboratory testing on concrete specimens extracted from the existing concrete intake/discharge pipes and towers in Huntington Beach, California. The objective of this durability assessment was to determine the current condition of the reinforced concrete pipes and towers, identify possible causes of degradation, and develop general recommendations for extending the service life. It is understood that the pipes are around 40 years old and will be used for a desalination plant and an additional 50 years of service life is desired.

TCG did not conduct a site visit or any visual observations of the structures. All concrete specimens were provided to TCG and tested at our laboratory. TCG analyzed the test results to conclude any findings and recommendations. No previous data of the structure was provided, such as as-built drawings, mix designs, previous assessments or test results, etc. Understanding of the structure is based on discussions with GHD.

Concrete Samples

Concrete coring and sampling was completed by GHD and cores were shipped to TCG's laboratory for testing. A total of six cores were extracted from the intake/discharge pipes and towers, ranging from about 11-14 inches in length and 2.75 inches inches in diameter. It is understood the towers and pipes were constructed with two different concrete mixtures.

Laboratory Testing

Laboratory Testing Overview

Laboratory testing on the concrete cores consisted of compressive strength, acid-soluble chloride profile, and petrographic examination. A summary of the tests performed on each sample is presented in Table 1.

Table 1 – Testing Summary

Core #	Core Location	Concrete	Test		
			ASTM C856 Petrographic	ASTM C42 Compressive Strength	ASTM C1152 Chloride Profile
1	Offshore Intake Tower	Mix A	√	√	√
2	Offshore Discharge Tower	Mix A	-	√	√
3	Onshore Intake Pipe	Mix B	√	-	√
4	Onshore Discharge Pipe	Mix B	-	-	√
5	Offshore Intake Pipe	Mix B	-	√	√
6	Offshore Discharge Pipe	Mix B	-	√	√

Compressive Strength

Compressive strength testing was conducted according to ASTM C42. The results are summarized in Table 2. The detailed laboratory report is presented in the Appendix. Note that segments of the cores for strength testing were taken starting at about 4 inches below the exposed surface due to limited amount of samples and therefore represent the body of the core. Near surface sections (0-4 in.) were utilized for chloride and petrographic analyses.

Table 2 – Compressive Strength

Core #	Strength (psi)	Fracture Type
1	4,710	4
2	5,050	1
5	5,440	2
6	6,630	2

Chloride Profiling

Chloride profiling was conducted in accordance with ASTM C1152. Seven depth increments were measured for chloride to establish the profile. Seven ½-inch disks per specimen were cut at predetermined depths and crushed into a powder. Majority of measurements were taken from 0-3.5 inches however, some measurements were taken at deeper depths. The powders were then measured for chloride concentration according the ASTM test method. The results were plotted in a graph as shown in Figure 1. Concrete cover depth of the top layer of steel rebar is unknown. It is assumed the cover depth is 1.5 inches according to ACI 318 for precast concrete exposed to weather with No. 6 through No. 11 bars. A typical chloride threshold for corrosion initiation is considered 500 ppm for black steel reinforcement. The chloride content at the depth of reinforcement for each sample indicates that the chloride threshold has been exceeded.

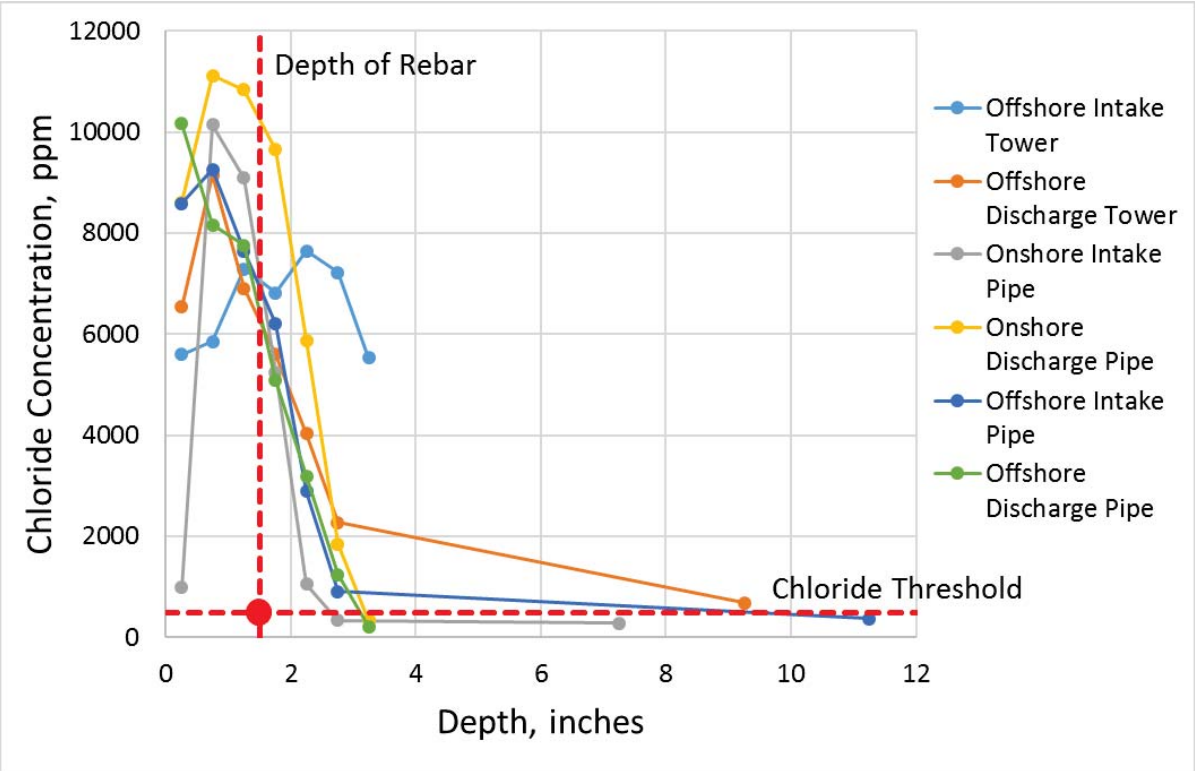


Figure 1: Measured Chloride Profiles

Petrographic Examination

The petrographic analysis was completed by Braun Intertec on two concrete cores in accordance with ASTM C856 "Standard Practice for Petrographic Examination of Hardened Concrete". The examination was used to determine the concrete quality and also to observe the presence, if any, of deterioration/distress and potential causes such as carbonation, sulfate attack, alkali-silica reaction (ASR), delayed ettringite formation (DEF), or corrosion. The detailed report is presented in the Appendix. A brief summary of findings is as follows:

<u>Surface and near surface concrete</u>

- Minor to moderate erosion of surface paste was found at the sample surface. Depth of lost concrete is unknown but likely shallow.
- Minor to moderate surface parallel hairline cracking and microcracking was found in the near surface region of core #1 to a depth of 0.75 inches. Minor surface perpendicular hairline cracking to a depth of 0.75 inches with one to a depth of 5.5 inches.
- Severe surface parallel hairline cracking and microcracking was found in the near surface region of core #2 in the outer 0.4-0.6 inches. Minor surface perpendicular hairline cracking to a depth of 0.4 inches.
- This moderate to severe near surface cracking is likely related to seawater attack, sulfate attack, and salt crystallization distress from long-term exposure of seawater.

- Leaching and reduced crystallinity of the cement paste to a depth of 2.0 to 2.4 inches was found. The absence of portlandite suggest the concrete has been leached of calcium and possibly sulfates, which is likely due to long-term movement of seawater.
- Most cracks, microcracks, and voids in the cement paste of the near surface concrete are lined or partially filled with secondary deposits such as ettringite, gypsum, calcium carbonate, and possibly brucite and other undefined salts. The deposits are consistent with materials produced by the interaction of the leached calcium and sulfates with seawater.

Body of Concrete (beyond depth of moderate to severe microcracking)

- The remaining body of concrete is in considerably better condition than the near surface region.
- No large cracks were present and very limited evidence of potentially deleterious reactions.
- Traces of alkali-silica reaction (ASR) is noted based on small amounts of ASR gel seen in the voids of some aggregate particles; however, there is little apparent cracking or evidence of distress due to this.
- Minor microcracking is noted throughout the body of the core; however, these microcracks do not exhibit a pattern indicative of significant distress.

Conclusions and Recommendations

Based on the limited amount of information and samples provided, the following conclusions were made from the laboratory test results with regards to durability:

- Body of the concrete appears to be in fair to good condition.
- Even though at the near surface there is leaching and secondary deposits of ettringite, brucite, gypsum, and other undefined salts partially filling the cracks, they do not appear to be causing cracking and may actually help reduce permeability in those voids. This is not predicted to be an issue.
- Moderate to severe hairline and microcracking has occurred to a depth of about 0.75 inches, which could potentially contribute to the acceleration of chloride ingress; however, chloride concentrations have already exceeded the chloride threshold for corrosion at the level of rebar.
- Some minor amounts of corrosion product have been observed on top layer of steel in the samples that were cored through rebar. However, the corrosion rates are likely very low due to the lack of oxygen from being submerged underwater.
- Very difficult to predict the corrosion rate and the rate of seawater and sulfate attack from the limited testing at only a single point in time.
- Currently, we believe the concrete is sufficient with regards to durability; however, recommend that the concrete be monitored and evaluated over time.

Liability and Limitations

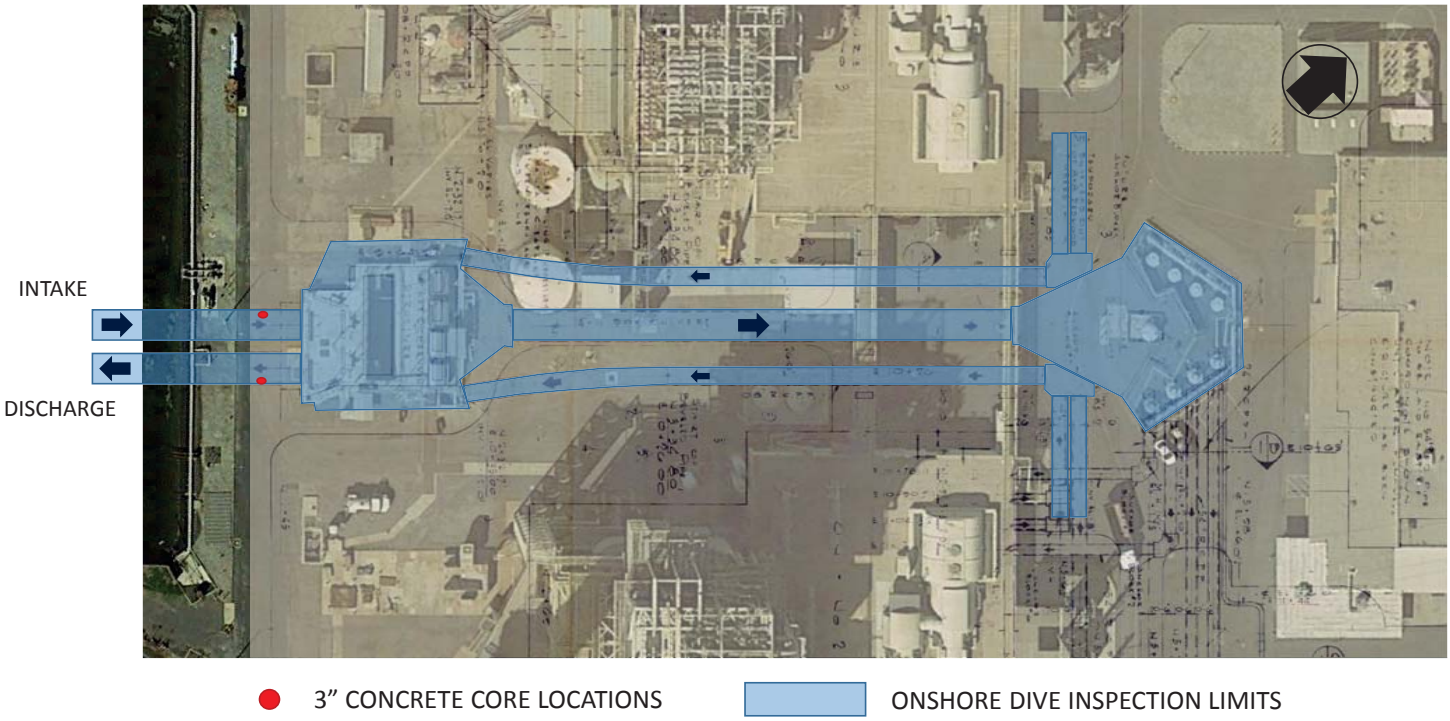
This report contains professional opinions and judgments based on laboratory testing and documents that have been provided to us. This report is believed to be accurate within the limitations of the information obtained. TCG reserves the right to modify our recommendations should more data or information become available.

APPENDIX

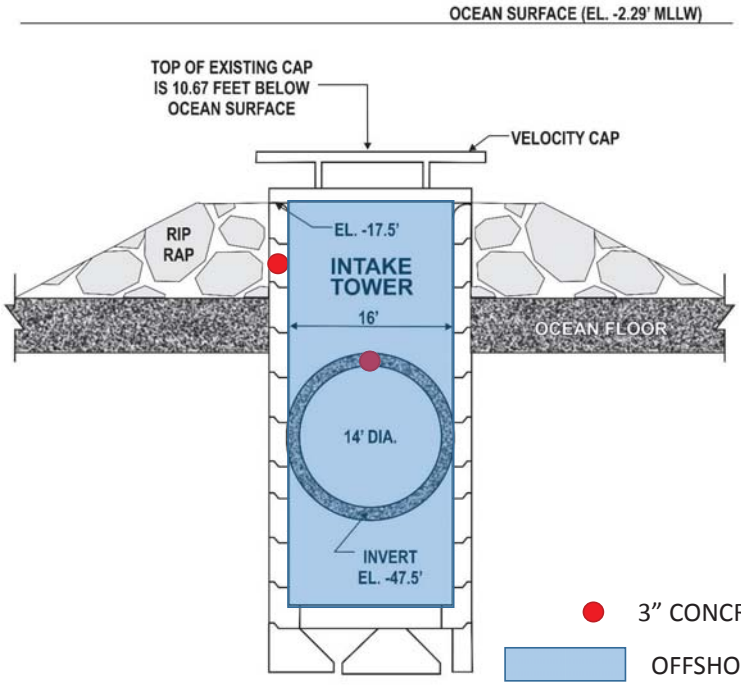
A. **Core Locations**
B. **Compressive Strength Test Report**
C. **Chloride Test Report**
D. **Petrographic Report**

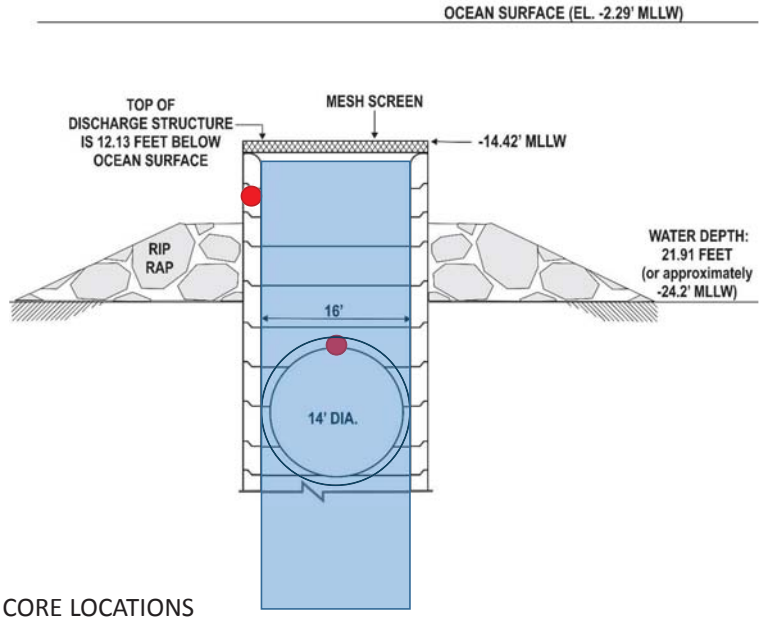
Appendix A – Core Locations

AES HB – Dive Inspection Scope for Future Desalinization Lines



AES HB – Dive Inspection Scope for Future Desalinization Lines





3" CONCRETE CORE LOCATIONS

OFFSHORE DIVE INSPECTION LIMITS

INTAKE STRUCTURE

OUTLET STRUCTURE

Appendix B – Compressive Strength

Tourney Consulting Group LLC
3401 Midlink Drive
Kalamazoo, MI 49048

Phone: 269.384.9980
Fax: 269.384.9981
www.tourneyconsulting.com

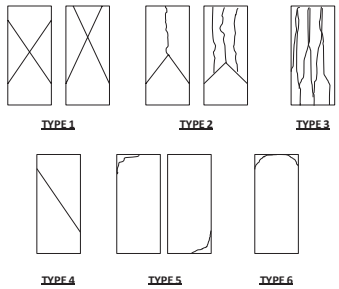
ASTM C42
Compression Testing of Drilled Concrete Cores

Client: GHD
175 Technology Drive Suite 200
Irvine, CA 92618 USA
Attention: Robert Sherwood
Phone: 949.648.5262

TCG Project Number: 18018
Project Description: Huntington Beach Desalination Plant
Sample Description: 2-3/4" Diam cores from Intake and Discharge Pipes and Towers
TCG Technician: Joe Duke
Required Strength: Unknown

Date Cored: 2/12, 13, 14/18
Date Received: 03/02/18
Date Reported: 03/22/18

Fracture Type



Core ID	Test Date	Age In Days	Avg. (2) Diam. in.	Avg.(2) Length Before Capping in.	Avg. (5) Length After Capping in.	L/D	L/D Factor	Max Load Pounds-Force	Type Fracture	Strength psi
1	03/22/18		2.719	4.521	4.665	1.716	**0.977**	28000	4	**4,710**
2	03/22/18		2.719	5.534	5.686	2.091	**1.000**	29300	1	**5,050**
5	03/22/18		2.717	5.540	5.675	2.089	**1.000**	31500	2	**5,440**
6	03/22/18		2.716	4.851	4.984	1.835	**1.000**	38400	2	**6,630**

Note: All cores were cut square and smooth. The cores were cut to length, capped with sulfur caps and conditioned in Ziploc bags @ 73°F for 5 days. The cores were broke with sulfur caps.

Core ID	Date Placed	Location	Tested		Core Cuts		Nom Max. Size of Agg. In.	Lbs./Ft^3
			Moist	Dry	One End	Both Ends		
1	Unknown	Offshore Intake Tower 2/13/18	XX			XX	3/4 to 1"	**101.6**
2	Unknown	Offshore Discharge Tower 2/13/18	XX			XX	3/4 to 1"	**103.5**
5	Unknown	Intake Pipe Offshore 2/14/18	XX			XX	3/4 to 1"	**111.1**
6	Unknown	Discharge Pipe Offshore 2/12/18	XX			XX	3/4 to 1"	**108.6**

Note: The coarse aggregate appears very porous.

Reviewed By:

Larry Wachowski
Laboratory Manager

Appendix C – Chloride Report

Tourney Consulting Group LLC
3401 Midlink Drive
Kalamazoo MI 49048

TCG TOURNEY CONSULTING GROUP

Phone (269)-384-9980
Fax (269)-384-9981
www.tourneyconsulting.com

ASTM C 1152 Acid-Soluble Chloride Ion Contents (PPM)

Client: GHD
175 Technology Drive Suite 200
Irvine, CA 92618 USA
Robert Sherwood
Attention: 949.648.5262

Date Received: 03/02/18
Date Tested: 03/22/18
Date Reported: 04/06/18

TCG Project No. 18018
Job Description: **Huntington Beach Desalination Plant**

Sample Description: Six (6) 2-3/4" diameter cores were received for testing from our client. The cores were cut at the 1/2" depth increments listed below. All samples were dried and crushed to ensure all material passed a #20 Sieve.
The Chloride data is reported by mass of sample.

TCG Technician: MW

	Depth Increments in Inches									
	0 to 1/2"		1/2" to 1"		1" to 1-1/2"		1-1/2" to 2"		2" to 2-1/2"	
Sample ID	Chloride Content ppm	Chloride content (% by mass of sample)	Chloride Content ppm	Chloride content (% by mass of sample)	Chloride Content ppm	Chloride content (% by mass of sample)	Chloride Content ppm	Chloride content (% by mass of sample)	Chloride Content ppm	Chloride content (% by mass of sample)
#1 Offshore Intake Tower	5597	0.560%	5854	0.585%	7282	0.728%	6820	0.682%	7651	0.765%
#2 Offshore Discharge Tower	6537	0.654%	9144	0.914%	6903	0.690%	5599	0.560%	4026	0.403%
#3 Onshore Intake Pipe	993	0.099%	10140	1.014%	9088	0.909%	5253	0.525%	1051	0.105%
#4 Onshore Discharge Pipe	8603	0.860%	11119	1.112%	10831	1.083%	9644	0.964%	5872	0.587%
#5 Offshore Intake Pipe	8588	0.859%	9259	0.926%	7648	0.765%	6198	0.620%	2885	0.289%
#6 Offshore Discharge Pipe	10162	1.016%	8164	0.816%	7754	0.775%	5086	0.509%	3192	0.319%

	Depth Increments in Inches									
	2-1/2" to 3		3" to 3-1/2"		7" to 7-1/2"		9" to 9-1/2"		11" to 11-1/2"	
Sample ID	Chloride Content ppm	Chloride content (% by mass of sample)	Chloride Content ppm	Chloride content (% by mass of sample)	Chloride Content ppm	Chloride content (% by mass of sample)	Chloride Content ppm	Chloride content (% by mass of sample)	Chloride Content ppm	Chloride content (% by mass of sample)
#1 Offshore Intake Tower	7217	0.722%	5536	0.554%	-	-	-	-	-	-
#2 Offshore Discharge Tower	2278	0.228%	-	-	-	-	688	0.069%	-	-
#3 Onshore Intake Pipe	341	0.034%	-	-	280	0.028%	-	-	-	-
#4 Onshore Discharge Pipe	1844	0.184%	342	0.034%	-	-	-	-	-	-
#5 Offshore Intake Pipe	908	0.091%	-	-	-	-	-	-	373	0.037%
#6 Offshore Discharge Pipe	1249	0.125%	220	0.022%	-	-	-	-	-	-

Reviewed By;

Larry Wachowski
Laboratory Manager

Appendix D – Petrographic Report

Braun Intertec Corporation
11001 Hampshire Avenue S
Minneapolis, MN 55438

Phone: 952.995.2000
Fax: 952.995.2020
Web: braunintertec.com

April 6, 2017 Project B1802592

Mr. Mark Dixon
Tourney Consulting Group LLC
3401 Midlink Drive
Kalamazoo, MI 49048

Re: Petrographic Examination of Core Samples from Concrete Tower and Pipe Structures of a Desalination Plant in California; TCG Project No. 18018

Dear Mr. Dixon:

We are pleased to present the results of petrographic analysis of cores taken from concrete structures at a desalination plant in California.

Background

Two core samples, respectively labeled as *Core #1* and *Core #3,* were received from Mr. Mark Dixon of Tourney Consulting Group LLC (TCG), Kalamazoo, Michigan. Cores #1 and #3 were reportedly cut vertically from the following specific locations in the plant:

- Core #1 – Offshore intake tower
- Core #3 – Onshore – HB Desal @ intake pipe

The whole length of Core #1 and the outer approximate half of Core #3 were longitudinally saw-cut in half and one of the resulting halves of each sample were retained by TCG for additional, chemical analysis. Figures 1, 2, and 3 document the as-received appearance and condition of the two concrete samples.

Mr. Dixon further stated that the concrete pipes have been submerged in seawater for about 40 years. No additional information was provided by TCG regarding the structures, exposure conditions of the concrete, or desalination plant. Furthermore, no mix design information was available, although it is TCG's understanding that the tower and pipe were made from two different concrete mixtures

Purpose

Petrographic examination (ASTM C856) was requested for each core samples to: 1) document the general composition and condition of concrete, 2) describe any deleterious reaction that are causing or contributing to distress in the concrete, and 3) determine the extent and depth of damage, if and where present.

Methodology

Petrographic examination of the provided concrete samples was conducted in accordance with ASTM C856, "Standard Practice for Petrographic Examination of Hardened Concrete." Each sample was initially examined and photographed in as-received condition. Clear epoxy was applied to outer portions of each core and allowed to set and harden to preserve some visibly distressed concrete prior to sample preparation. A slice of concrete was saw-cut longitudinally from each core and the resulting surfaces of each slice were ground and lapped to a flat, smooth, semi-polished finish. The resulting polished cross sections were examined visually and with the aid of a stereomicroscope at magnifications up to 45X. Saw-cut and freshly broken surfaces of the cores were also subjected to similar examination.

One of the polished surfaces of the cross section from each core was treated with pH-indicator (phenolphthalein) solution to evaluate the depth and pattern of carbonation. The solution imparts a deep magenta stain to high-pH, non-carbonated cement paste, but does not stain, or only imparts a light pink coloration, to reduced-pH paste.

Thin sections were also produced from each of the core samples to study additional microscopical features of the concrete. To make these thin sections, small rectangular blocks, or "billets," were saw-cut from the outer portions of each core and one of the resulting surfaces of each billet was polished, cleaned, and impregnated with clear epoxy. Excess material was struck from the surfaces and epoxy was allowed to set. Each billet was then re-polished and adhered to a glass microscope slide with clear epoxy. After the epoxy set, the thickness of the mounted specimen was reduced to approximately 20 to 30 µm (0.0008 to 0.0012 in.) and a coverslip was applied using clear epoxy. The resulting "thin sections" were examined with a petrographic (polarized-light) microscope at magnification of up to 400X.

The microscopical observations were performed in the Braun Intertec office in Spring Green, Wisconsin.

Findings

Petrographic examination reveals evidence of significant alteration and degradation of the concrete at and near the exposed outer ends of each of the provided cores that is likely related to exposure of the concrete to seawater and seawater attack. The surface and near surface concrete exhibits numerous hairline cracks and microcracks, alteration and chemical leaching of the cement paste, and presence of secondary deposits in most cracks and microcracks. Figures 3 through 12 provide photographic documentation of the appearance and condition of the concrete, as well as the cracking and alteration of the cement paste in the near-surface regions.

Tables 1 and 2 summarize some of the salient observations in the altered and degraded near-surface concrete in each core.

Table 1. Summary of Observed Damage and Cracking in Concrete Cores from Desalination Plant

Core ID and Location	Distress at Surface	Distress in Near-Surface Region
#1 - Offshore Intake Tower	Moderate erosion of surface paste reveals aggregate in low to moderate relief. Depth of lost concrete unknown. Some attached shelly accumulations and biological matter with associated flaking and pitting of concrete.	Minor to moderate surface parallel, hairline cracking and microcracking in outer 19 mm (0.75 in.). Minor surface-perpendicular hairline cracking in outer 9 mm (0.75 in.); one hairline crack extends to depth of 14 cm (5.5 in.).
#2 –-Onshore HB Desal @ Intake Pipe	Minor to moderate erosion of surface paste reveals aggregate in low relief. Depth of lost concrete unknown, but likely shallow. Minor attached shelly accumulations and biological matter with associated flaking and pitting of concrete.	Severe surface parallel, hairline cracking and microcracking in outer approximate 10 to 15 mm (0.4 to 0.6 in.). Minor surface-perpendicular hairline cracking in outer approximate 10 mm (0.4 in.).

Table 2. Summary of Observed Alteration and pH Change in Concrete Cores from Desalination Plant

Core ID and Location	Depth of Reduced pH[1]	Depth of Carbonation[2]	Depth of Leached Cement Paste[2,3]
#1 - Offshore Intake Tower	Non-uniform; 8 to 15 mm (0.3 to 0.6 in.).	Non-uniform; 2 to 2.5 mm (0.08 to 0.10 in.)	Estimated 5 to 6 cm (2.0 to 2.4 in.), with non-uniform reduction of portlandite at greater depths.
#2 –-Onshore HB Desal @ Intake Pipe	Non-uniform; 5 to 8 mm (0.2 to 0.3 in.).	Non-uniform; 0.5 to 1.5 mm (0.02 to 0.06 in.); spotty carbonation along microcracks to 5 mm (0.2 in.)	Estimated 5 cm (2.0 in.), with non-uniform reduction of portlandite reduction of portlandite at greater depths.

1 Based on pH-indicator (phenolphthalein) staining method.

2 Based on microscopical observation of carbonation products in petrographic thin sections.

3 Based on lack of portlandite (calcium hydroxide) crystal in cement paste, as seen in thin sections.

The severe cracking and defoliating pattern seen in the near-surface region of concrete in Core #3 (Figures 6, 8, and 11), and to a lesser extent in Core #1 (Figures 5, 8, and 10), is likely related to, and caused by, seawater attack, sulfate distress, and salt crystallization distress from long-term exposure of the concrete to seawater. Biological activity and growth of plants and calcareous shelly organisms, and related shallow leaching of calcium, have also likely contributed to damage and degradation of the existing outer surface reportedly exposed to the seawater.

Degradation of the concrete at and near the exposed outer surface of the concrete appears to be manifested in surface-parallel cracking, scaling, and exfoliating of the surface concrete to a fairly limited and defined depths (see Table 1). Besides the very apparent cracking and scaling of the surface and near-surface concrete, the concrete at and near the exposed outer surfaces exhibit fairly deep alteration in the form of leaching and reduced crystallinity of the cement paste. Application of pH-indicator (phenolphthalein) solution to cut , broken, and polished sections of each core found reduced pH (lower than an estimated 9) in near-surface concrete to depths of up to 15 mm (0.8 in.) and 8 mm (0.4 in.), respectively in Cores #1 and #3. However, microscopical examination of thin sections made from the outer portions of each core reveal shallower carbonation of the cement paste than the zone of reduced pH indicated by the pH stain tests. Further microscopical study reveals that the cement paste in the near-surface concrete is lacking portlandite crystals (a common and expected hydration product of portland cement). Such absence of portlandite, as well as other feature of the cement paste, suggest that the concrete has been leached of calcium and possibly sulfate and other constituents. The depth of reduced-portlandite, leached paste is not well defined in the examined cores, but appears to extend to depths of approximately 5 to 6 cm (2.0 to 2.4 in.), beyond which remnant portlandite becomes more common. Leaching of the cement paste was likely promoted by the long-term movement of mildly aggressive water / seawater over the concrete surfaces for reportedly 40 years, which has dissolve, removed, and relocated calcium and sulfates in the concrete and contributed to the growth of secondary deposits in cracks and voids.

Microscopical examination revealed that most cracks and microcracks, as well as many voids in the cement paste of the near-surface concrete are lined or partially filled with secondary deposits (Figure 12). Such deposits include ettringite (calcium sulfo-aluminate hydrate), gypsum (calcium sulfate dehydrate), calcium carbonate (in cracks and voids in first few millimeters), and possibly brucite (magnesium hydroxide), and other, undefined salts. Microscopical methods confirmed the presence of ettringite, gypsum, and the carbonate deposits, but could not adequately determine the presence of other compounds. Soluble salts from the seawater may also have been present that could not be fully identified, or were dissolved from water used to extract and prepare the core samples. X-ray diffraction analysis of the deposits is recommended to further study these deposits. Nonetheless, the deposits appear to be consistent with materials often produced by interaction of cement hydrates with seawater, or a mildly aggressive treated water.

Beyond the depth of moderate to severe surface-parallel cracking and intense leaching in the near-surface region of concrete, the remaining body of concrete in each core appears to be in considerably better condition with no large cracks and very limited evidence of potentially deleterious reaction. Traces of what may represent alkali-silica reaction (ASR) are noted in the concrete, based on the presence of a meager amount of suspected ASR gel seen in voids along the periphery of some aggregate particles. However, the gel appears to be present and accommodated within large and abundant voids in the highly porous manufactured aggregate, and little apparently-related cracking or evidence of distress is noted. Furthermore, voids in the cement paste exhibit secondary sulfate deposits without apparent, associated cracking. Minor microcracking is noted in the cement paste throughout the body of each core; however, these microcracks are far less common than in the near-surface region, are more random in orientation, and do not exhibit a pattern indicative of significant distress in the body of concrete.

The only segment of reinforcement exhibited in the provided samples is a segment of smooth-sided, 7/8-in. diameter steel reinforcing bar present in Core #3. The bar exhibit only traces of brown, ferruginous corrosion products and is judged to be in good condition. The bar was received separated from the concrete core, but imprints of the bar surfaces on the concrete suggest good consolidation of the surrounding concrete.

Additional Findings

Concrete comprising the two provided core samples are somewhat similar in overall composition, but differ sufficiently to indicate two different lightweight concrete mixtures may be represented. The following additional findings and descriptions of the two concrete samples are based on the results of the petrographic examinations, details of which are provided in the attached petrographic data report.

Core #1

1. Concrete represented by Core #1 is composed of manufactured lightweight coarse and fine aggregate dispersed in a non-air-entrained paste of portland cement. The lightweight concrete appears well consolidated.

2. Air content in the body of the concrete sample is estimated in the range of 2 to 3 percent, by volume of concrete. Even through the hardened cement paste exhibits some small, spherical voids (less than 1 mm), consistent with entrained air, such small voids are scarce and are of volume low, and the concrete is not believed to be intentionally air entrained.

3. Properties of the cement paste in the body of the core are judged to be consistent with the moderately low water-cement ratio (w/c). The light to medium gray cement paste in the body of the core is moderately hard to hard, exhibits low apparent absorbency to drops of water, and paste-aggregate bond is tight. Some residual (partially hydrated) grains of portland cement

clinker are seen microscopically in the cement paste, confirming its presence in the concrete. However, due to the reported age of the concrete, continued hydration of the cement, and alteration (i.e., leaching) of the concrete, no further numeric estimation of the w/c of the concrete is given.

4. Aggregate used in the lightweight concrete appears to be composed mostly of expanded lightweight coarse and fine aggregate, likely expanded clay. A small amount of siliceous sand is also noted in the fine aggregate. Overall, the aggregate appears to be gap-graded with low volume of intermediate-sized particles between 6 and 13 mm (No. 8 and 1/2-in.), and the manufactured sand appears somewhat coarsely graded.

5. The concrete exhibits apparently normal volume of mineral constituents. However, petrographic examination does not provide confirmation of the actual mass proportions of mineral constituents of the concrete; additional chemical analysis of the concrete would be needed to further investigate these issues.

Core #3

1. Concrete represented by Core #3 is also composed of manufactured lightweight coarse and fine aggregate dispersed in a non-air-entrained paste of portland cement. The lightweight concrete appears well consolidated.

2. Air content in the body of the concrete sample is estimated in the range of 1 to 2 percent, by volume of concrete. Again, even though the hardened cement paste exhibits some small, spherical voids (less than 1 mm), the concrete is not believed to be intentionally air entrained.

3. Properties of the cement paste in the body of the core are judged to be consistent with the moderately low water-cement ratio (w/c). The mottled, light to medium gray cement paste in the body of the core is moderately hard to hard, exhibits low apparent absorbency to drops of water, and paste-aggregate bond is tight. Numerous residual (partially hydrated) grains of portland cement clinker are seen microscopically in the cement paste, confirming its presence in the concrete. Again, due to the reported age of the concrete, continued hydration of the cement, and alteration (i.e., leaching) of the concrete, no numeric estimation of the w/c is given.

4. Aggregate used in the lightweight concrete appears to be composed mostly of expanded lightweight coarse and fine aggregate, probably expanded clay, similar to aggregate seen in Core #1, but gradation is comparatively different. Only trace amounts of siliceous sand are noted in the fine aggregate. Overall, the aggregate appears to be gap-graded with low volume of intermediate-sized particles, between 6 and 13 mm (No. 4 and 1/2-in.), and overall low volume of coarse aggregate. The fine aggregate also appears somewhat coarsely graded.

Additional Discussion

Each of the provided core samples exhibits evidence of moderate to severe damage to the surface and near-surface concrete, mainly in the form of surface parallel cracking, that is attributed in large part to seawater attack. However, the depth of such damage is limited and the remaining body of the concrete appears to be in fair to good condition. Secondary deposits are common to abundant in cracks and voids in the near-surface concrete that likely formed in response to intake of constituents of the mildly aggressive seawater, and long-term leaching and movement of soluble constituents in the concrete. Due to the reported exposure of the concrete to seawater, and evidence of related form of sulfate attack and chemical leaching, additional analysis of the concrete is warranted to determine a profile of sulfates and chloride contents, both of which may influence continued serviceability of the structures and repair options. It is our understanding that the portions of the cores retained by TCG were to be analyzed for chloride content. It may be prudent to subject the same powered specimen to sulfate content analysis.

General Remarks

Findings of this evaluation are based solely on examination and analysis of the provided samples and may not necessarily represent the materials and condition of materials elsewhere in the same structure.

In performing its services, Braun Intertec used that degree of care and skill ordinarily exercised under similar circumstances by reputable members of its profession currently practicing in the same locality. No warranty, express or implied, is made.

Samples will be retained for 30 days from the date of this report unless additional instructions for longer sample retention are provided.

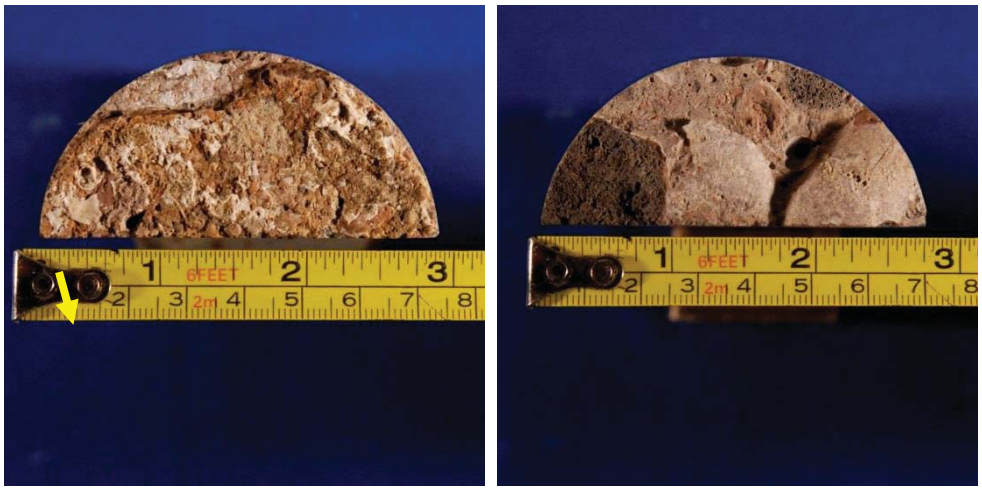
If you have any questions or concerns, please do not hesitate to contact Ron Sturm at 612.280.6505.

Sincerely,

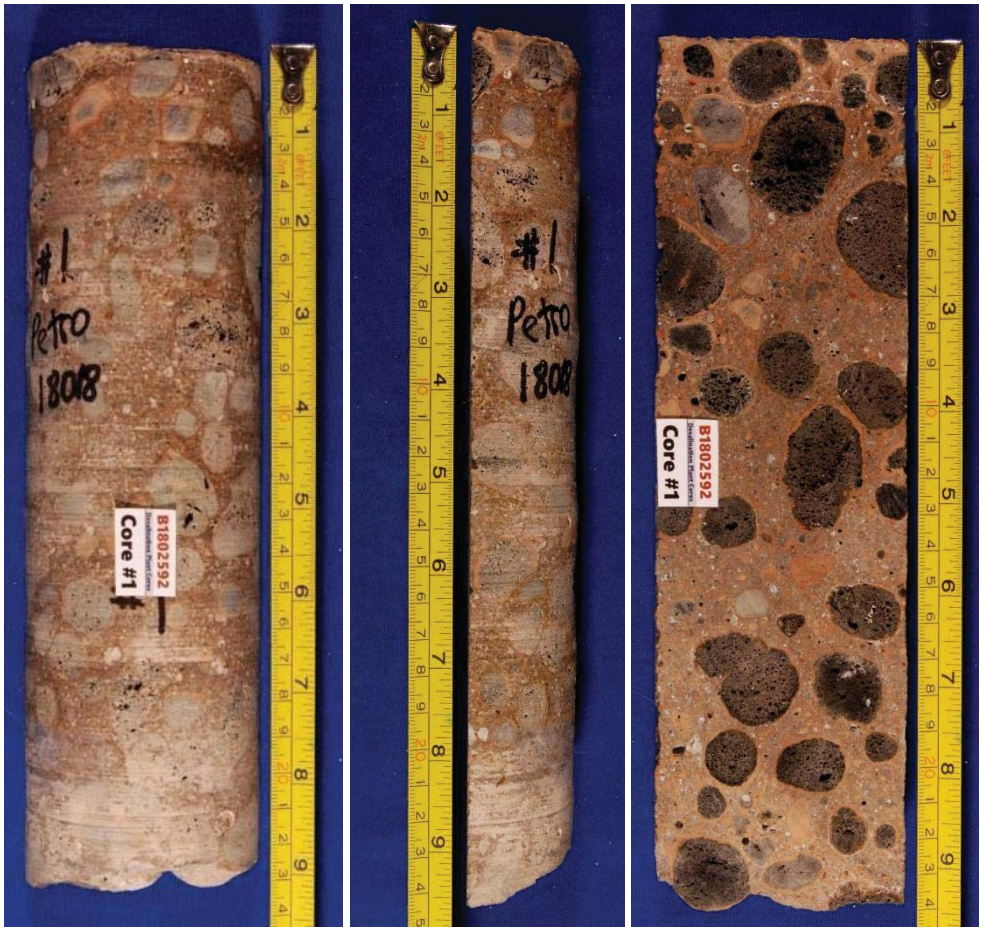
BRAUN INTERTEC CORPORATION

Ronald D. Sturm
Senior Petrographer

Alfred J. Gardiner
Principal - Manager Concrete Consulting



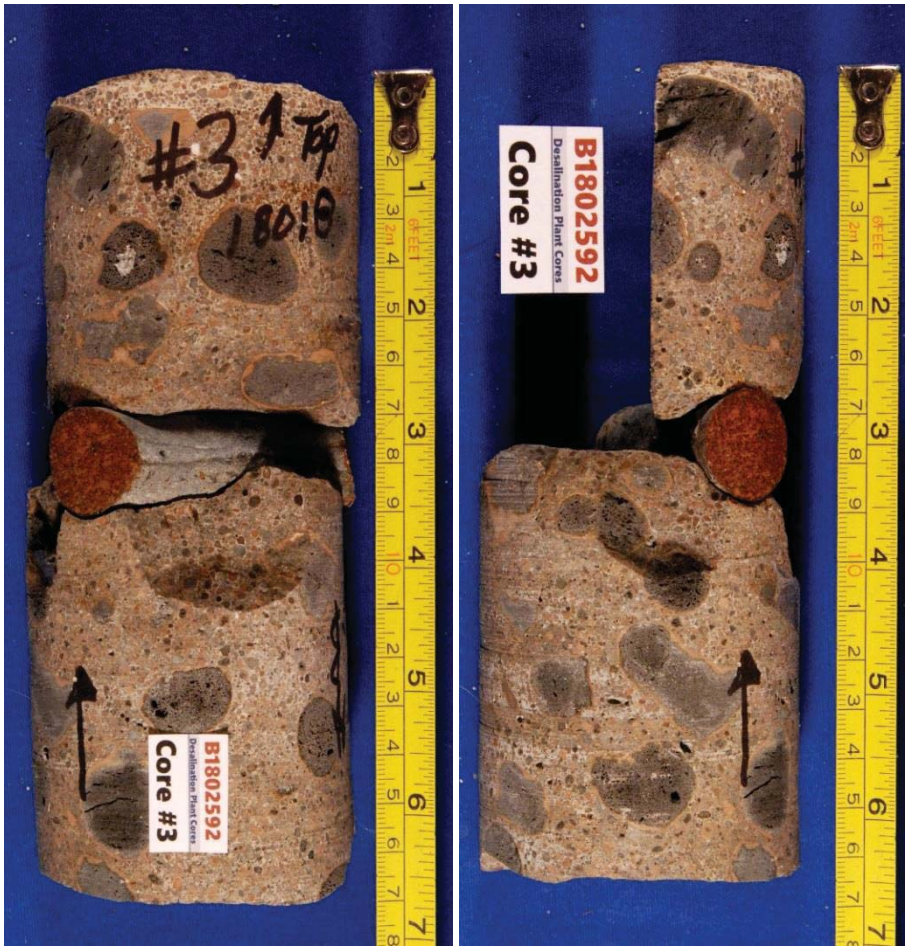
1a. Exposed outer end (left) and inner end (right).



1b. Sides of core; exposed outer end up.

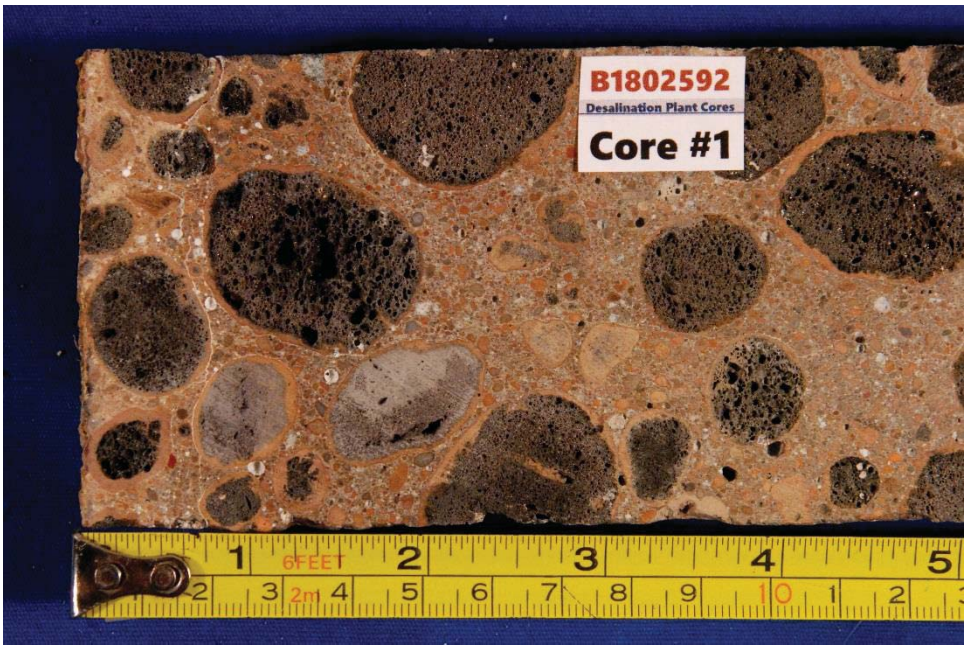
Figure 1. Core #1, as received for petrographic examination. Note the core was received longitudinally saw-cut in half. Also note the deteriorated condition of the exposed, outer end surface.

2a. Exposed outer end (left) and inner end (right).

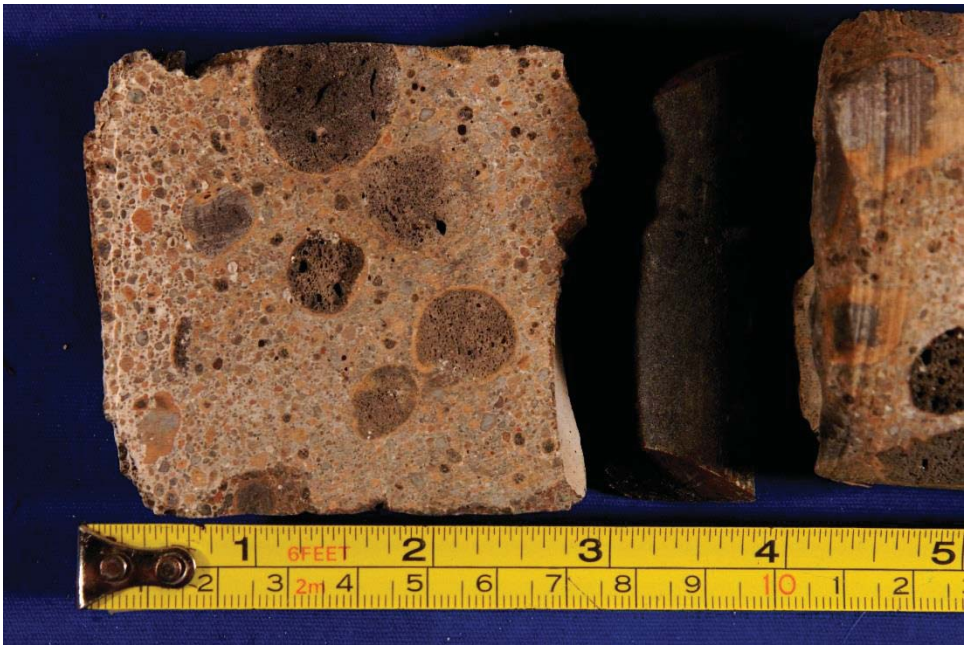


2b. Side views of reassembled core; exposed outer end up.

Figure 2. Core #3, as received for petrographic examination. Note the outer portion of the core was longitudinally saw-cut in half. Also note the presence of a smooth reinforcing bar and the deteriorated condition of the exposed, outer end surface.



3a. Outer portion of Core #1 cut cross section.



3b. Outer portion of Core #3 cut cross section.

Figure 3. Closer views of the saw-cut outer cross sections of the two cores, as received for examination. Note the discolored and cracked appearance of the outer portion of concrete in each.

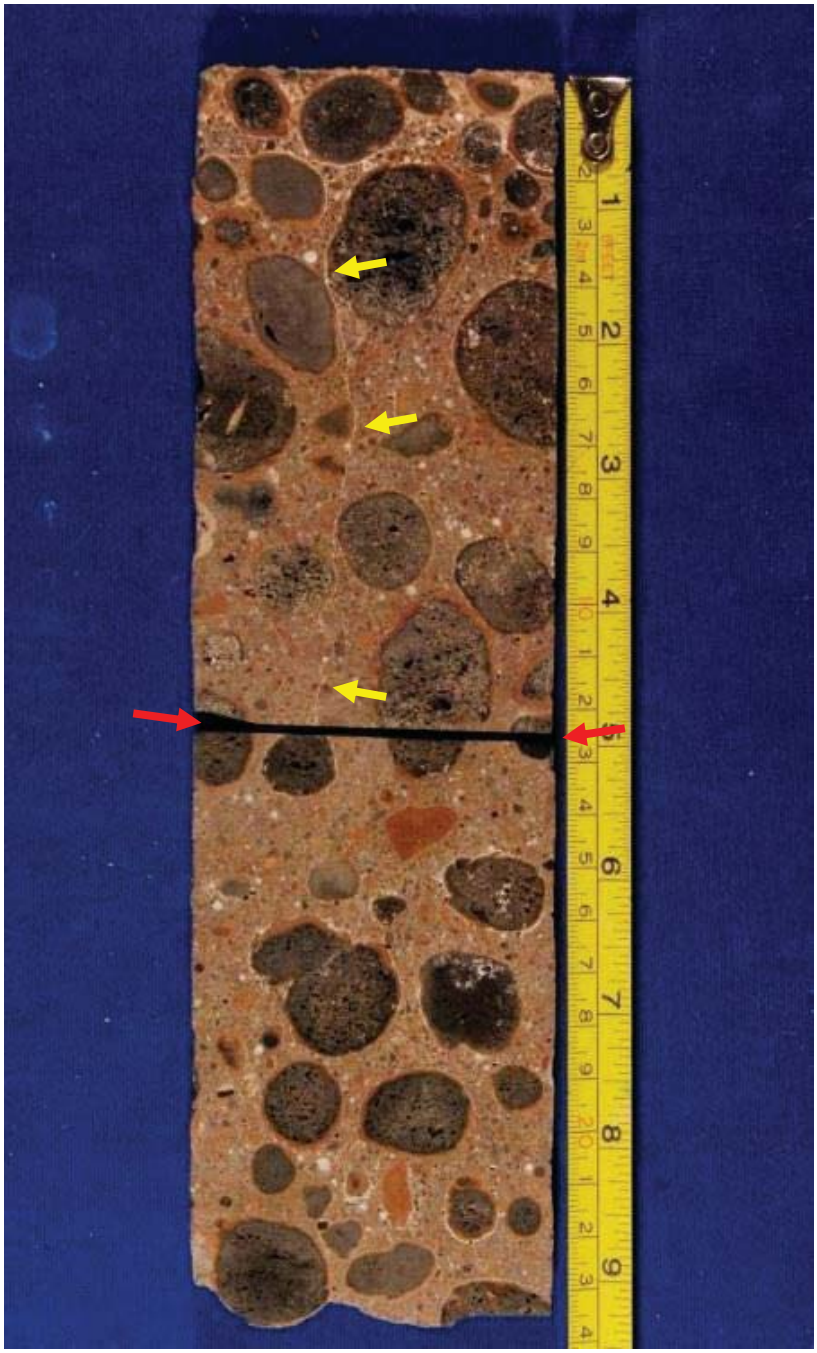
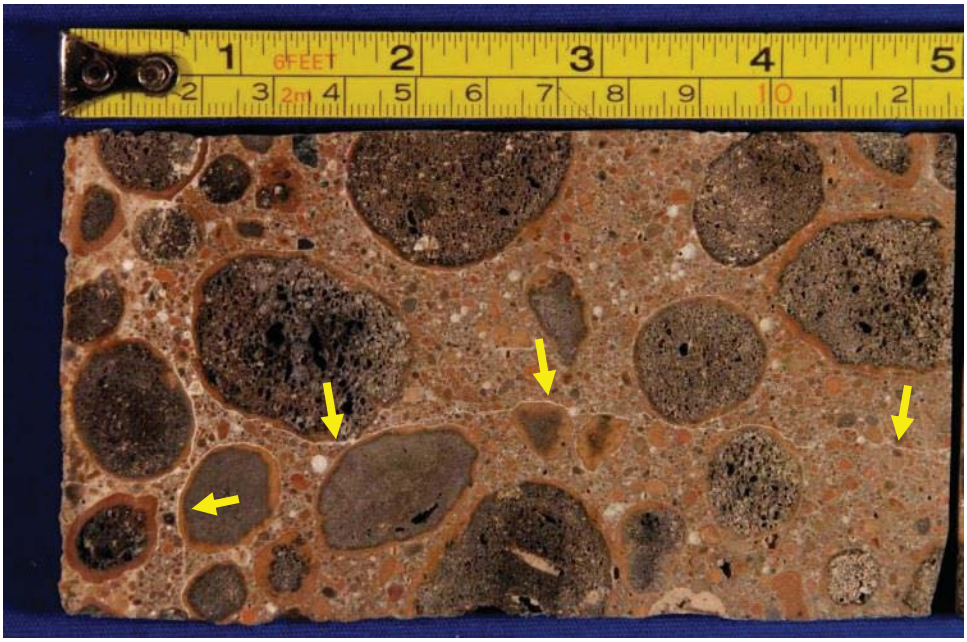
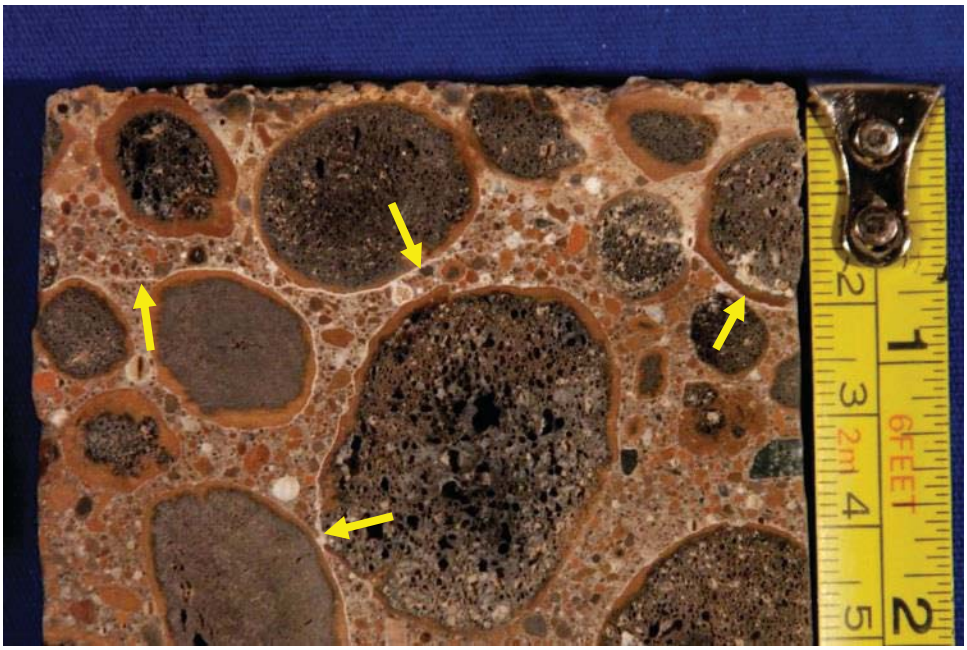


Figure 4. **Polished, longitudinal cross section of Core #1 (exposed outer surface facing up) shows the general appearance and condition of the concrete sample. Note the discoloration and distress in the concrete at and near the top end (further documented in Figure 5). Also note the longitudinal hairline crack, marked with yellow arrows, and use of manufactured lightweight aggregate. Red arrows mark a transverse saw cut made during sample preparation.**



5a. Outer 12.7 cm (5 in.) of core; outer end to left.



5b. Outer 5.1 cm (2 in.) of core; outer end up.

Figure 5. **Closer views of the polished outer portion of Core #1 further document the distress, cracking, and discoloration of the concrete at and near the top surface. Yellow arrows mark examples of hairline cracks filled with white crystalline secondary deposits.**

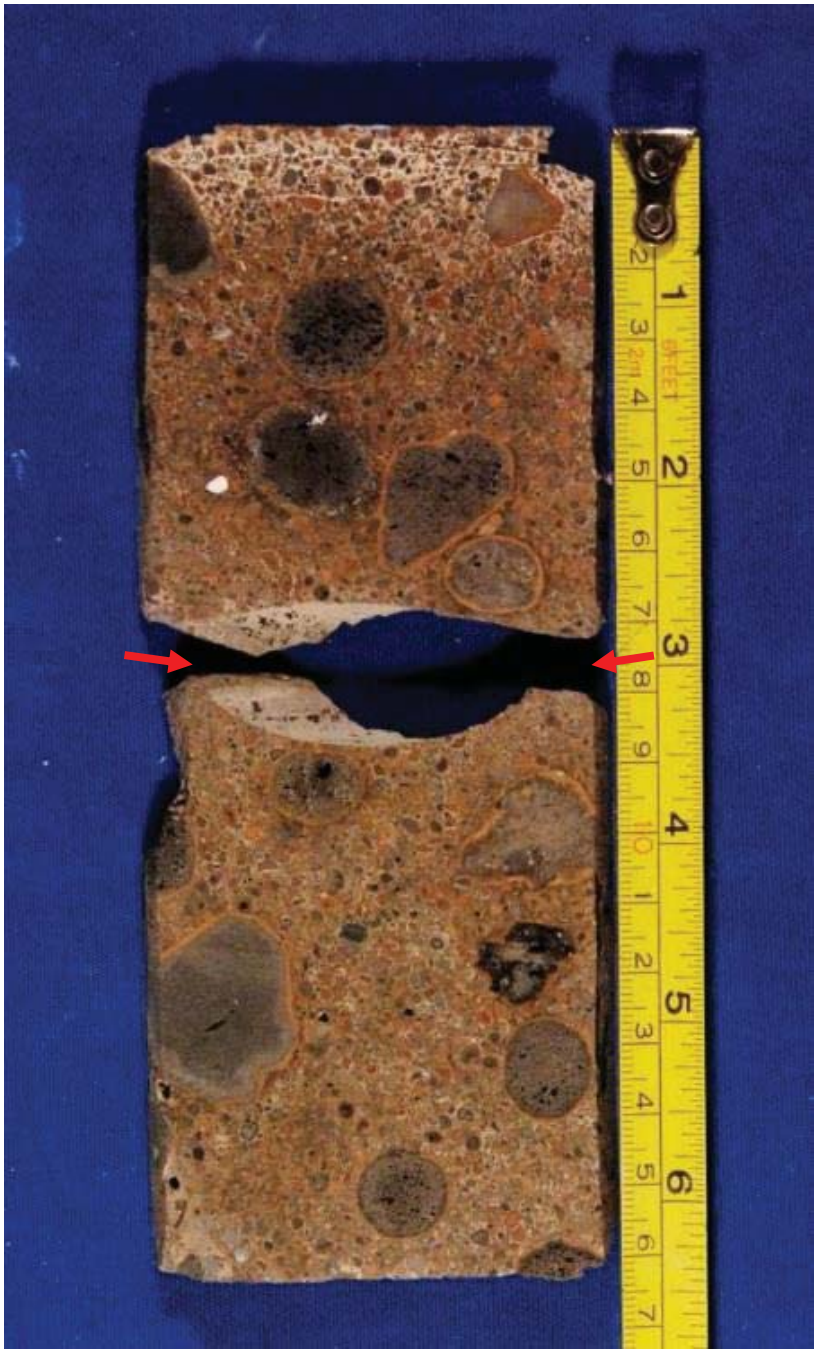
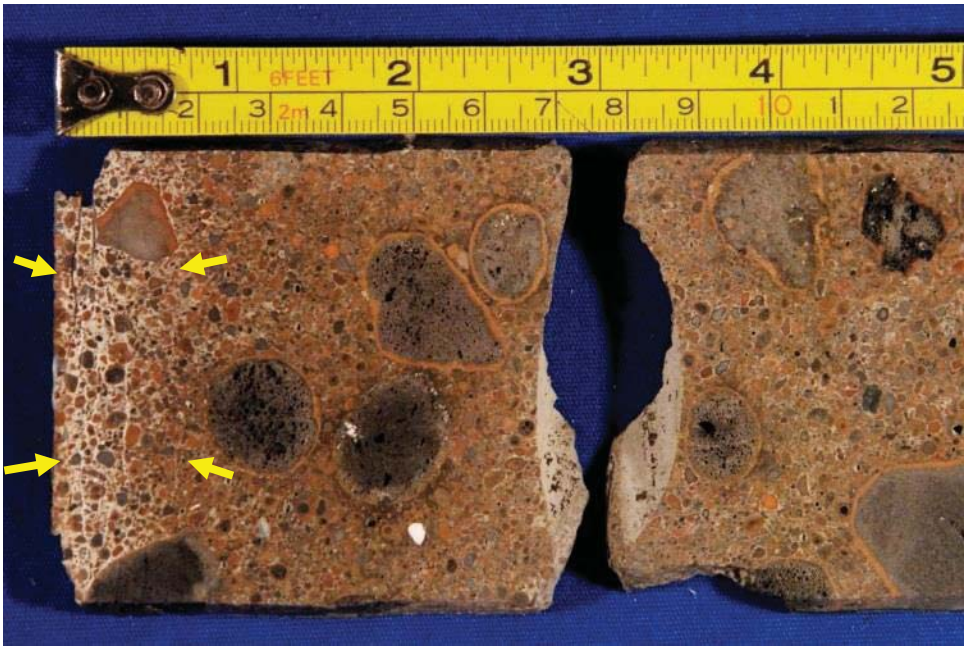
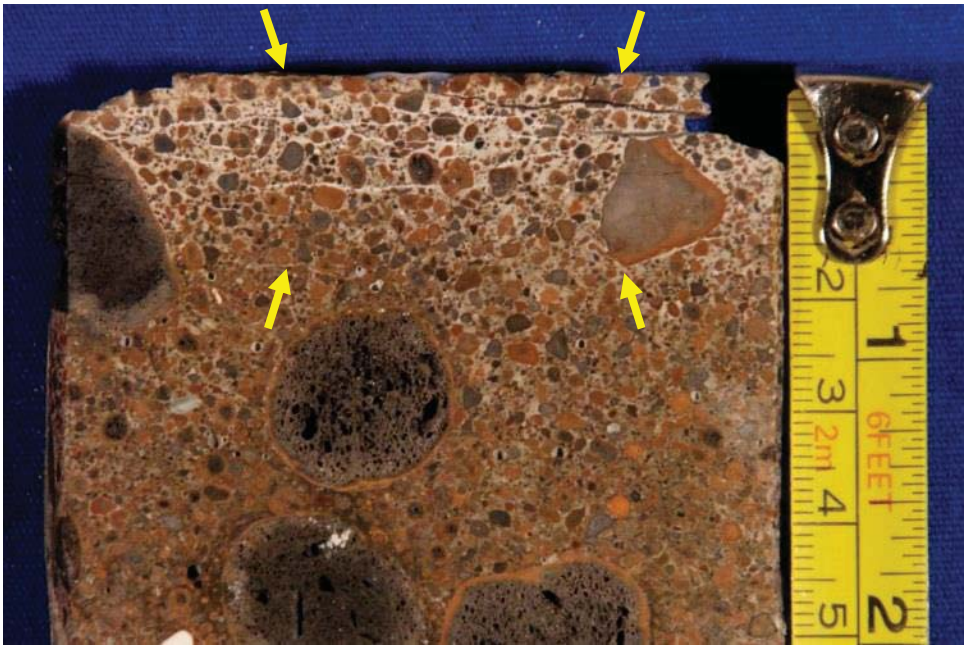


Figure 6. **Reassembled, polished, longitudinal cross section of Core #3 (outer end facing up) shows the general appearance and condition of the concrete sample. Note the discoloration and distress in the concrete at and near the outer end (further documented in Figure 7). Red arrows mark the location of a smooth, 22-mm (7/8-in.) diameter steel reinforcing bar (bar not shown). Also note the use of manufactured lightweight aggregate in the concrete, and low volume of coarse aggregate.**

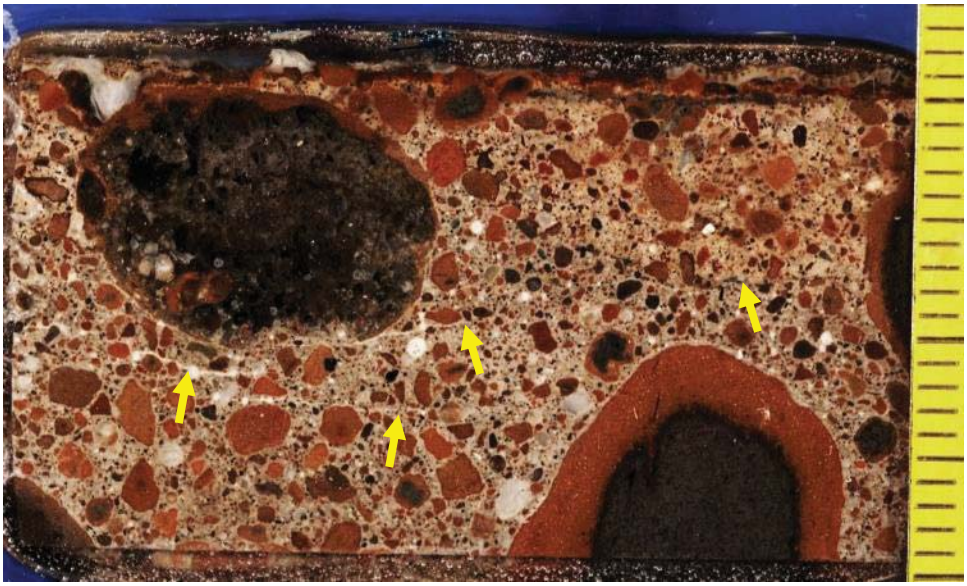


7a. Outer 12.7 cm (5 in.) of core; top end to left.

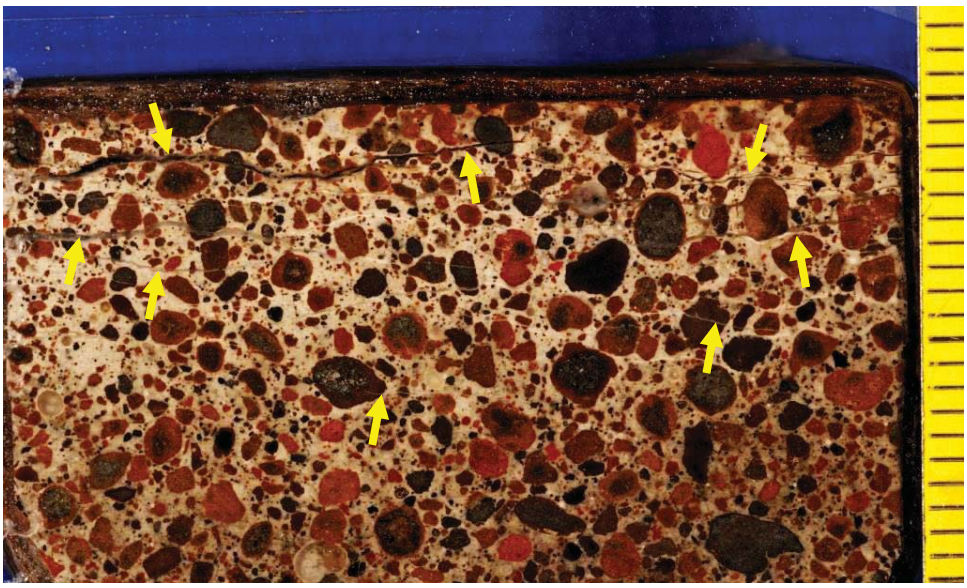


7b. Outer 5.1 cm (2 in.) of core; top end up.

Figure 7. Closer views of the polished outer portion of Core #3 further document the distress, cracking, and discoloration of the concrete at and near the outer or top surface (marked between yellow arrows).

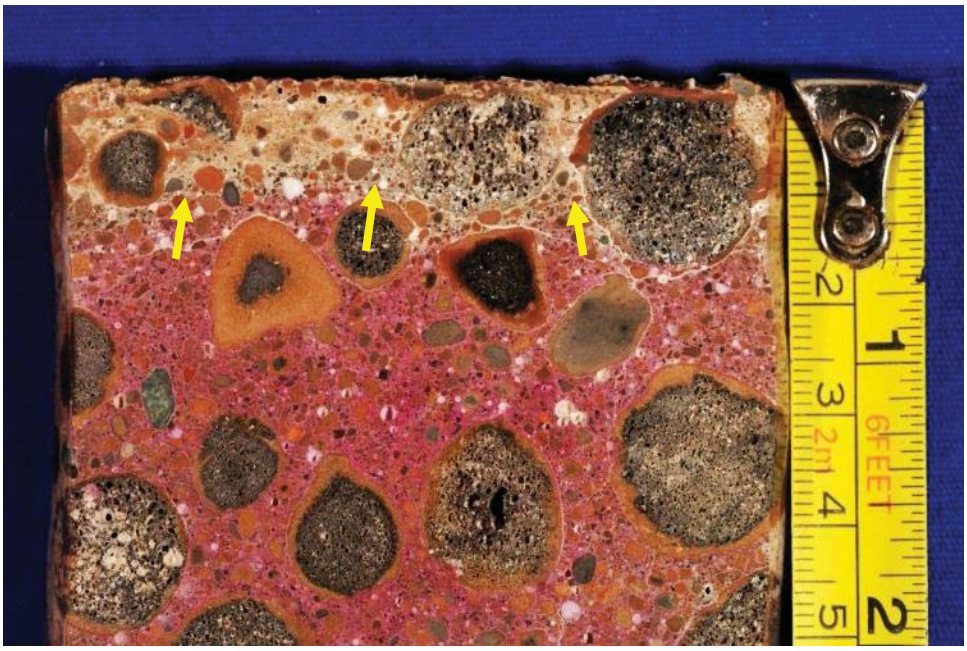


8b. Core #1; outer surface facing up.

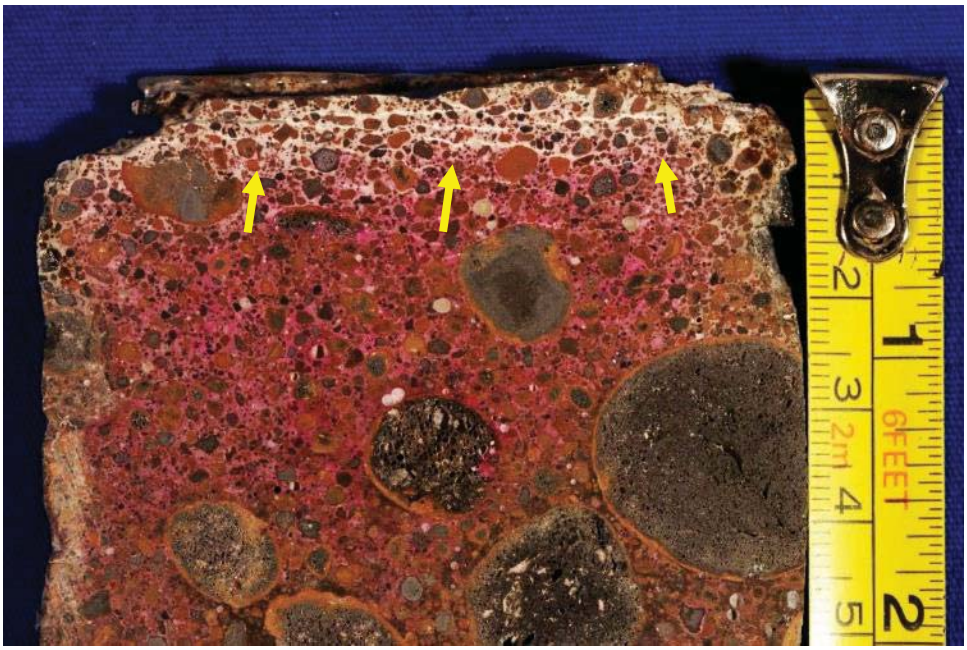


8b. Core #3; outer surface facing up.

Figure 8. **Close-up views of the polished thin section billets from Cores #1 and #3, as seen through attached glass microscope slides, show numerous surface-parallel microcracks and discoloration of the cement paste at and near the outer/top surfaces. Also note that manufactured lightweight coarse and fine aggregate were used in the concrete. The scales are marked in millimeter increments.**

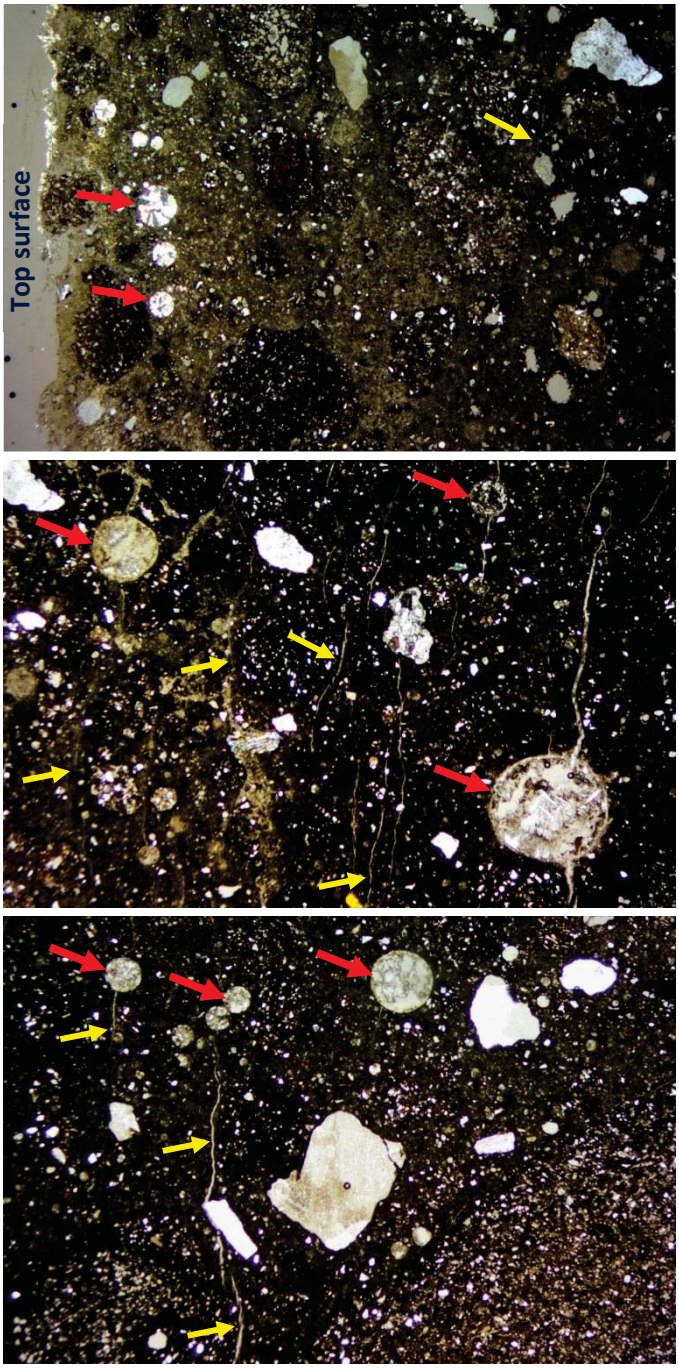


9a. Core #1.



9b. Core #3.

Figure 9. **Stained, vertical cross sections of Cores #1 and #3 show the depth and pattern of pH reduction (marked by arrows). Each cross sections was treated with pH-indicator (phenolphthalein) solution that imparts a deep magenta stain to high-pH, non-carbonated cement paste, but does not stain reduced-pH paste (lower than 9). Further study reveals the lowering of pH is mostly due to leaching and little carbonation of paste is apparent from the stain.**

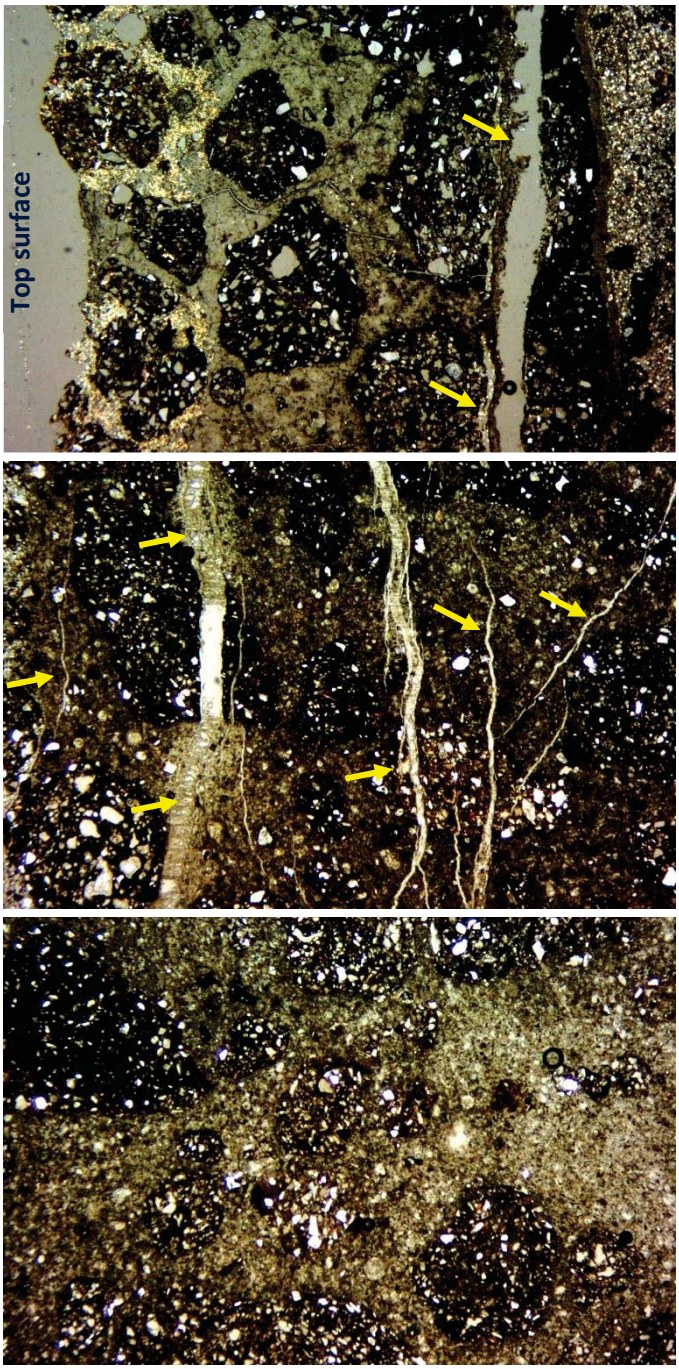


10a. Outer 6 mm.

10b. 6 to 12 mm from outer (top) end.

10c. 12 to 18 mm from outer (top) end.

Figure 10. **Thin-section photomicrographs of outer portions of Core #1, at the three indicted depths, show extensive surface-parallel microcracking and void-filling deposits in the concrete. Yellow arrows mark examples of microcracks. Red arrows mark voids filled with secondary deposits. Each image is shown in partially-crossed polarized light. 20X magnification. Length of each field, left to right, is 6 mm (0.24 in.).**

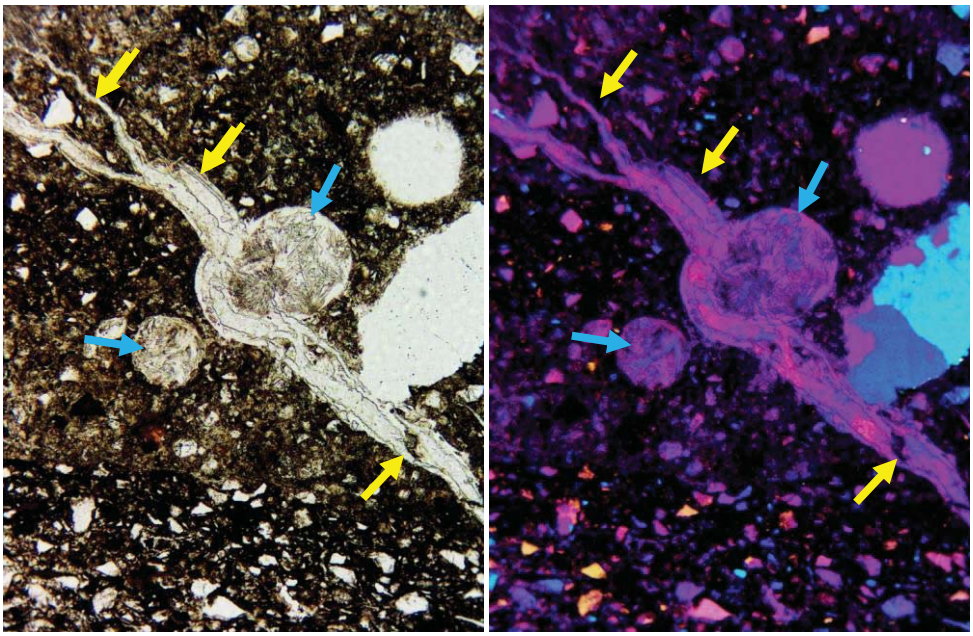


11a. Outer 3 mm.

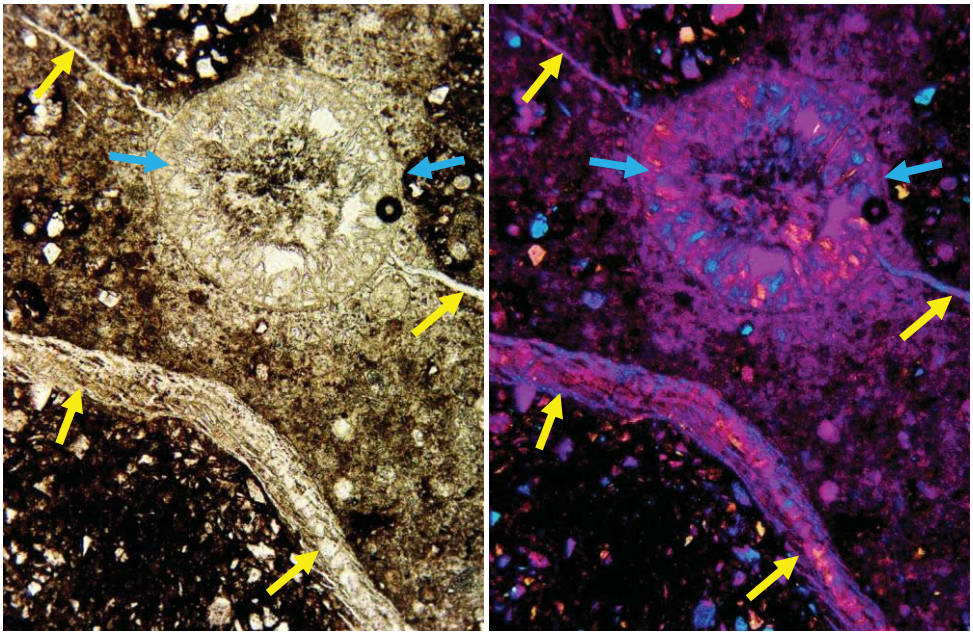
11b. 3 to 6 mm from outer (top) end.

11c. 13 to 16 mm from outer (top) end.

Figure 11. **Thin-section photomicrographs of outer portions of Core #3, at the three indicted depths, show extensive surface-parallel microcracking and void-filling deposits in the concrete. Yellow arrows mark examples of microcracks lined or filled with secondary deposits. Each image is shown in transmitted, partially-crossed polarized light. Note the scarcity of cracks in the lower image. 40X magnification. Length of each field, left to right, is 3 mm (0.12 in.).**



12a. Core #1.



12b. Core #3

Figure 12. Thin-section photomicrographs show secondary sulfate compounds lining and partially filling cracks (yellow arrows) and voids (blue arrows) in the two concrete samples. Each image pair is shown in transmitted, plane-polarized light (left) and cross-polarized light with 1st-order wave plate (right). Length of each field, top to bottom, is approximately 1.2 mm (0.05 in.).

Petrographic Examination of Hardened Concrete - ASTM: C 856

Client:
Tourney Consulting Group
Kalamazoo, MI

Project Description:
Petrographic Examination of Concrete Cores from Desalination Plant
TCG Project No.: 18018

Sample Information Sample: Core #1

Sample Location: Offshore Intake Tower.

Reported Distress Observed: Surface deterioration; condition assessment.

General Observations

Sample Dimensions:

Analysis was performed on a portion of a partial-depth, drilled, cylindrical concrete core with a diameter of 6.9 cm (2.7 in.) and length of 22.8 to 23.9 cm (9.0 to 9.4 in.). Weight of the sample is 0.62 kg (1.37 lb.). Sample was received longitudinally saw-cut in half; half retained by TCG.

Outer (Top) End:

Surface is a moderately deteriorated and exhibits erosion of most of the concrete surface, exposing aggregate in low to moderate relief. The surface is also partially covered with a light brown stain or discoloration, and attached pieces of biological growth, including light-colored shelly material and darker plant matter. The concrete surface beneath the biological matter appears fairly flat to rough with shell and biological material extending into fine recesses of the concrete. Portions of the surface concrete can be lightly scratched to a shallow depth of less than 1 mm (0.04 in).

Inner End:

Rough and uneven fracture surface of concrete that passes through aggregate.

General Physical Conditions:

The body of concrete was received intact and does not exhibit visible major cracks. Minor to moderate surface parallel hairline cracking and microcracking is present in the outer 19 mm (0.75 in.). Minor surface-perpendicular hairline cracking is seen in the outer 9 mm (0.75 in.). One hairline crack extends to depth of 14 cm (5.5 in.). All of the above-described cracks and microcracks extends mostly through aggregate and is mostly filled with white, finely crystalline secondary deposits. A significant portion of the outer rim of the core is chipped. Microcracking is sparse to common in the remainder of the core, away from the cracked outer region. The concrete visibly appears to be well consolidated with only sparse entrapped air voids up to 4 mm (0.16 in.) across.

Reinforcement:
None present in core.

Aggregate

Coarse: Manufactured lightweight aggregate, probably expanded clay. Aggregate pieces are mostly rounded to sub-rounded, equant, and exhibit semi-rough to semi-smooth outer surfaces free of apparent coatings. Aggregate gradation is uneven, with a low volume of intermediate-sized, 6 to 13 mm (0.25 to 0.50 in.) particles and an observed top size of 25 mm (1 in.). Coarse aggregate appears somewhat non-uniformly dispersed in the concrete.

Fine: Mostly manufactured lightweight aggregate, probably expanded clay, with minor amounts of siliceous sand. The manufactured aggregate grains are mostly rounded to sub-rounded, equant, and exhibit semi-rough to semi-smooth outer surfaces free of apparent coatings. Aggregate gradation is uneven and somewhat rich in 2 to 5 mm (0.08 to 0.20 in.) particles. Most of fine aggregate appears uniformly dispersed in the concrete. Natural sand is composed primarily of rock quartz, quartzite, and feldspar. Rock grains are mostly sub-angular to less commonly sub-rounded and angular, equant to slightly elongate, and exhibit mostly smooth to semi-rough outer surfaces free of apparent coatings.

Paste

Depth of Carbonation	Patchy and non-uniform to a depth of 2 to 2.5 mm (0.08 to 0.10 in.). Minor carbonation seen on faces of a few near-surface microcracks.
Air Content	Estimate 2 to 3 percent, by volume of concrete and attributed to entrapped air. The concrete is not air entrained, based on the scarcity of small, spherical voids (less than 1 mm) in the hardened cement paste.
Paste Color	Light to medium gray throughout body of core. Non-uniformly light brown to light tan-gray in outer 6 to 13 mm (0.24 to 0.5 in.) of the core. Cement paste exposed along the outer end surface of the core exhibits a beige to light brown discoloration that does not extend significantly into the sub-surface concrete
Paste Hardness	Moderately hard to hard in the body of the core. Cement paste exposed along portions of the outer surface, particularly brown discolored paste, is locally slightly softer, but only to a shallow depth of less than 1 mm (0.04 in.). Cement paste in outer 6 to 13 mm (0.24 to 0.5 in.) ranges from moderately hard to moderately soft.

Paste Luster	Subvitreous to vitreous in body of core. Subvitreous to dull along outer end surface and in most of outer 6 to 13 mm (0.24 to 0.5 in.) of concrete.
Paste Absorbency	Moderately low.
Paste-Aggregate Bond	Tight; surfaces of freshly broken concrete pass mostly through aggregate.
Residual (Unhydrated) Cement	Unhydrated portland cement clinker particles are scarce to common in the cement paste throughout the core. “Ghost relics” of suspected in-situ hydrated clinker are common to abundant.
Supplementary Cementitious Materials	None observed.
Calcium Hydroxide	Small portlandite crystals are observed in minor to moderate amounts in most of the inner portion of the core, excluding the outer approximate 5 to 6 cm (2.0 to 2.4 in.) where portlandite crystals are scarce to apparently absent.
Estimated Water-Cement Ratio (w/c)	Moderately low, based on properties of the concrete and cement paste. However, due to the reported age of the structure, continued hydration of the cement paste, and localized alteration of the concrete, the estimated w/c is judged to be speculative and a numeric estimated value is not given.
Microcracking	Microcracks are common in the cement paste throughout most of the body of the core, Microcracks are common to abundant in the outer approximate 19 mm (0.75 in.) of the core, as previously described. . Microcracks along the outer face of the core extend mainly perpendicular to the surface, but a few sub-surface, surface parallel microcracks are noted immediately beneath some of the attached biological material.
Secondary Deposits	White to off-white, finely crystalline secondary deposits line or partially fill most cracks, microcracks, air voids in the hardened paste, in the outer approximate 19 mm (0.75 in.) of the core. Most of the deposits appear to be composed of ettringite ($3CaO•Al_2O_3•3CaSO_4•32H_2O$), but other compounds may include gypsum ($CaSO_4•2H_2O$), calcium carbonate (in microcracks at and near outer surface), and possibly brucite ($Mg(OH)_2$), and other salts. Possible traces of alkali-silica reaction gel are seen in pores along the periphery of several pieces of manufactured lightweight aggregate.

Miscellaneous	Portions of the outer few millimeters of the cement paste are intensely leached and exhibits an amorphous appearance when viewed in thin section.

Examination By:
Ronald D. Sturm
Senior Petrographer

Petrographic Examination of Hardened Concrete - ASTM: C 856

Client:
Tourney Consulting Group
Kalamazoo, MI

Project Description:
Petrographic Examination of Concrete Cores from Desalination Plant
TCG Project No.: 18018

Sample Information Sample: Core #3

Sample Location: Onshore-HB Desal Intake Pipe.

Reported Distress Observed: Surface deterioration; condition assessment.

General Observations

Sample Dimensions:

Analysis was performed on a portion of a partial-depth, drilled, cylindrical concrete core with a diameter of 6.9 cm (2.7 in.). The core was received transversely broken at a depth of 55 to 73 mm (22 to 3.3 in.) and at the boundary with an embedded steel reinforcing bar (Figure 2). Reassembled length of the core is approximately 16.1 to 17.0 cm (6.3 to 6.7 in.), including steel bar. Weight of the sample is 0.90 kg (2.0 lb.) with the steel bar; 0.71 kg (1.56 lb.) without the bar. Sample was received with the outer segment longitudinally saw-cut in half; half retained by TCG.

Outer (Top) End:

Surface is a moderately deteriorated and exhibits erosion of most of the concrete surface, exposing aggregate in low to moderate relief. The surface is also partially covered with a light brown stain or discoloration, and attached pieces of biological growth, including light-colored shelly material and darker plant matter. The concrete surface beneath the biological matter appears fairly flat to rough with shell and biological material extending into fine recesses of the concrete. Portions of the surface concrete can be lightly scratched to a shallow depth of less than 1 mm (0.04 in).

Inner End:

Rough and uneven fracture surface of concrete that passes through aggregate.

General Physical Conditions:

The core was received broken transversely into two segments, separated at a depth of approximately 55 to 73 mm (2.2 to 3.3 in.) from the outer (top) end along a fracture extending along an embedded steel reinforcing bar. The sample was received with the steel reinforcing bar separated from the core. Moderate to severe surface parallel hairline cracking and microcracking is present in the outer 10 to 15 mm (0.4 to 0.6 in.). Minor surface-perpendicular hairline cracking is also seen in the outer 10 mm

(0.4 in.) of the core. A significant portion of the outer rim of the core is chipped. Microcracking is sparse to common in the remainder of the core, away from the cracked outer region. The concrete visibly appears to be well consolidated with only sparse entrapped air voids up to 3 mm (0.12 in.) across.

Reinforcement:
A segment of 7/8-in. diameter, smooth sided, steel reinforcing bar is embedded in the core with 66 to 68 mm (2.6 to 2.7 in.) of cover from the outer (top) end. Examination of exposed surfaces of the bar reveal only trace to minor corrosion of steel surface.

Aggregate

Coarse: Manufactured lightweight aggregate, probably expanded clay. Aggregate pieces are mostly rounded to sub-rounded, equant, and exhibit semi-rough to semi-smooth outer surfaces free of apparent coatings. Aggregate gradation is uneven, with a low volume of intermediate-sized, 6 to 13 mm (0.25 to 0.50 in.), particles and an observed top size of 25 mm (1 in.). Overall volume of coarse aggregate appears low. Coarse aggregate appears somewhat non-uniformly dispersed in the concrete.

Fine: Mostly manufactured lightweight aggregate, probably expanded clay, with minor amounts of siliceous sand. The manufactured aggregate grains are mostly rounded to sub-rounded, equant, and exhibit semi-rough to semi-smooth outer surfaces free of apparent coatings. Aggregate gradation is uneven and somewhat rich in 2 to 5 mm (0.08 to 0.20 in.) particles. Most of fine aggregate appears uniformly dispersed in the concrete. Natural sand is composed primarily of rock quartz, quartzite, and feldspar. Rock grains are mostly sub-angular to less commonly sub-rounded and angular, equant to slightly elongate, and exhibit mostly smooth to semi-rough outer surfaces free of apparent coatings.

Paste

Depth of Carbonation	Patchy and non-uniform to a depth of 1 to 1.5 mm (0.02 to 0.06 in.). Minor carbonation seen on faces of a few near-surface microcracks in outer 5 mm (0.2 in.).
Air Content	Estimate 1 to 2 percent, by volume of concrete and attributed to entrapped air. The concrete is not air entrained, based on the scarcity of small, spherical voids (less than 1 mm) in the hardened cement paste.
Paste Color	Light to medium gray throughout body of core. Non-uniformly light brown to tan-gray in outer 6 to 10 mm (0.24 to 0.4 in.) of the core. Cement paste exposed along the outer end surface of the core exhibits a beige to brown discoloration that does not extend significantly into the sub-surface concrete

Paste Hardness	Moderately hard to hard in the body of the core. Cement paste exposed along portions of the outer surface, particularly brown discolored paste, is locally slightly softer, but only to a shallow depth of less than 1 mm (0.04 in.). Cement paste in outer 6 to 10 mm (0.24 to 0.4 in.) ranges from moderately hard to moderately soft.
Paste Luster	Subvitreous to vitreous in body of core. Subvitreous to dull along outer end surface and in most of outer 10 mm (0.4 in.) of concrete.
Paste Absorbency	Moderately low.
Paste-Aggregate Bond	Tight; surfaces of freshly broken concrete pass mostly through aggregate.
Residual (Unhydrated) Cement	Unhydrated portland cement clinker particles are common in the cement paste throughout the core. “Ghost relics” of suspected in-situ hydrated clinker are common to abundant.
Supplementary Cementitious Materials	None observed.
Calcium Hydroxide	Small portlandite crystals are observed in minor to moderate amounts in most of the inner portion of the core, excluding the outer approximate 5 cm (2.0 in.) where portlandite crystals are scarce to apparently absent.
Estimated Water-Cement Ratio (w/c)	Moderately low, based on properties of the concrete and cement paste. However, due to the reported age of the structure, continued hydration of the cement paste, and localized alteration of the concrete, the estimated w/c is judged to be speculative and a numeric estimated value is not given.
Microcracking	Microcracks are common in the cement paste throughout most of the body of the core, Microcracks are common to abundant in the outer approximate 10 mm (0.4 in.) of the core, as previously described. . Microcracks along the outer face of the core extend mainly perpendicular to the surface, but a few sub-surface, surface parallel microcracks are noted immediately beneath some of the attached biological material.

Secondary Deposits	White to off-white, finely crystalline secondary deposits line or partially fill most cracks, microcracks, air voids in the hardened paste, in the outer approximate 10 mm (0.4 in.) of the core. Most of the deposits appear to be composed of ettringite, but other compounds may include gypsum, calcium carbonate (in microcracks at and near outer surface), and possibly brucite and other salts. Possible traces of alkali-silica reaction gel are seen in pores along the periphery of several pieces of manufactured lightweight aggregate.

Examination By:
Ronald D. Sturm
Senior Petrographer

SUBSEA GLOBAL
SOLUTIONS

AES Huntington Beach

Intake / Discharge
Piping and Structure Condition Assessment

February 12 to 14, 2018
Huntington Beach, CA, USA

Client:

Poseidon Water
5780 Fleet Street, Suite 140
Carlsbad, CA 92008

Job Number: 3066

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AES Huntington Beach

Intake / Discharge Piping and Structure Condition Assessment

SGS Project Manager: Michelle Campagna

SGS Dive Supervisors: Eugene Van, Jimmy Nguyen

JOB SUMMARY

Subsea Global Solutions (SGS) US West Coast LLC attended the AES Huntington Beach Facility to provide commercial diving services required, as contracted by Poseidon Water, to perform a condition assessment of the intake and discharge pipelines and structures of the facility's circulating water system.

SGS provided both onshore and offshore services to assist with the condition assessment, consisting of three separate teams: SGS Onshore Dive Team, SGS ROV Inspection Team, and SGS Offshore Dive Team.

SGS teams performed internal condition inspections of four. pipelines including: onshore intake pipeline, onshore cross-over discharge to intake pipeline, offshore discharge pipeline and the offshore intake pipeline. Internal visual inspection of pipelines where conducted utilizing either in-water commercial divers or a remotely operated vehicle (ROV), both equipped with HD video for inspection documentation. Pipeline inspections focused on the general condition of the pipeline and whether portions of the pipe have been structurally compromised, with attention to the interior wall of the pipe, joints, and manhole locations (if applicable).

SGS teams performed further condition inspections of the three structures including: onshore intake screen-well structure, offshore intake velocity-cap / structure and offshore discharge structure. Visual inspections of the structures focused on the overall general condition of the structure and on whether portions of the structure have been structurally compromised. During the inspection of the offshore intake and discharge structures, the dive team also performed an inspection of the rip-rap protection surrounding the structures, measuring and mapping the rip-rap to determine the size, condition, elevation, and slope.

In addition to the condition inspections noted above, SGS teams also extracted six (6) ea. 3" diameter by approximately 14" long concrete cores at the following locations: one (1) ea. core from the onshore terminus of the offshore intake pipeline, one (1) ea. core from the onshore terminus of the offshore discharge pipeline, two (2) ea. cores from the offshore intake structure, and two (2) ea. cores from the offshore discharge structure. After extraction of the cores, the core drilled holes were immediately filled and patched with Splash Zone A-788 Two-Part Epoxy Repair Compound. The extracted cores were labeled and transferred to GHD for laboratory testing.

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SCOPE OF WORK & SUMMARY OF OBSERVATIONS

SGS Onshore Dive Team:

Onshore Intake Pipeline Internal Inspection

The SGS onshore dive team performed a penetration dive, internal visual condition inspection of the onshore intake pipeline. The pipeline was accessed via a manhole, consisting of two (2) manhole covers, located adjacent to the pump-well structure. The full extent of the 14' by 11' reinforced concrete square pipeline was inspected with video from the pump-well structure to the intake screen-well structure. The pipeline's concrete appeared to be in generally good condition, with no major discrepancies encountered. No spalling, rust bleed, exposed rebar, or other anomalies were observed. Hard and soft marine growth was observed throughout the entire pipeline, with growth ranging between 4 to 8" in depth on the ceiling and on the East and West walls. The floor of the pipeline overall exhibited less growth with more sporadic hard growth. Hard and soft growth on the floor measured on average at a depth of 2" with sporadic hard growth measuring up to 6" in depth. Soft silty sedimentation was also observed throughout the onshore intake pipeline. Sedimentation depths increased towards the South end of the pipeline with depths over 3'. At the South end of the onshore intake pipeline, the dive team also inspected four (4) traveling screens. All screens were observed to be in good condition with minimal marine growth.

In addition to the concrete onshore intake pipeline, the dive team also inspected the intake pump housings and impellers. The pump housings and impellers appeared to be in good condition with no discrepancies encountered. Hard and soft marine fouling on the pump housings ranged on average in depth of 4 to 6". The impellers were observed to have minimal to no marine growth, and appeared to be in good condition.

Onshore Cross-Over Discharge / Intake Internal Pipeline Inspection

The SGS onshore dive team performed a penetration dive, internal visual condition inspection of the onshore cross-over pipeline that connects the onshore terminus of the offshore discharge and intake pipelines. The reinforced concrete square pipeline was accessed via the intake screen-well structure. The inspection was conducted beginning at the discharge pipeline moving West through the crossover pipeline to the intake pipeline. The full extent of the crossover pipeline, approximately 100 LF, was inspected with video. The crossover pipeline concrete appeared to be in generally good condition, with no major discrepancies encountered. Pieces of concrete were observed on the floor of the pipeline at the West end where the crossover pipeline terminates. Inspection of the condition of the concrete pipeline in the vicinity of the concrete pieces found no indication of any structural damage. Marine growth on the pipeline was less than observed in other areas of the circulating water system, due to the presence of a current that flows through the crossover pipeline from the intake pipeline to the offshore discharge pipeline. Hard marine fouling approximately 1 to 1-1/2" in thickness was observed on the ceiling and both the North and South walls of the pipeline. Minimal to no marine growth was observed on the pipeline floor, with soft marine fouling and sparse tubeworm growth primarily observed. A thin layer of sediment was observed on the floor with no major areas of accumulation. No pipeline joints were observed during the inspection.

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Offshore Discharge & Intake Pipelines Onshore Terminus Core Samples

The SGS onshore dive team extracted two (2) ea. core samples, one (1) ea. from the offshore discharge pipeline onshore terminus and one (1) ea. from the offshore intake pipeline onshore terminus.

The core sample for the onshore terminus of the offshore discharge pipeline was located approximately 50' from the intake screen-well forebay grizzly-bars, approximately half-way up the inside West wall of the pipeline, 8" from a pipe joint. During core drilling operations, the 3" diameter core sample broke into three (3) sections with the first section 3" long, second section 7" long, and the third section 3" long. Only the first and second sections of the core sample were successfully extracted, for a total core sample length of 10". The dive team was unable to extract the third section of the core sample. As directed by GHD, the dive team left the third section of the core sample in-place. After extraction of the core sample, the dive team immediately filled and patched the core drill hole with Splash Zone A-788 Two-Part Epoxy Repair Compound.

The core sample for the onshore terminus of the offshore intake pipeline was located at approximately 50' from the intake screen-well forebay grizzly-bars, approximately half-way up the inside of the West wall of the pipeline. During the core drilling operations, the 3" diameter core sample broke into three (3) sections with the first section 3" long, the second section 9" long, and third section 3" long. The dive team was able to successfully extract all three sections of the core sample, for a total core sample length of 14". After extraction of the core sample, the dive team immediately filled and patched the core drill hole with Splash Zone A-788 Two-Part Epoxy Repair Compound.

SGS ROV Inspection Team:

ROV Offshore Discharge and Intake Pipeline Internal Inspection

SGS in partnership with Hibbard Inshore, LLC (Hibbard) performed an internal inspection utilizing a Remotely Operated Vehicle (ROV) of the offshore discharge and intake pipelines. The offshore discharge and intake pipelines were both accessed via the intake screen-well forebay. The inspection was performed by the Seabotix LBV 600 ROV and both video and sonar were recorded to the VisualSoft data acquisition software with written annotation. The video and the video record of the sonar is synchronized with the inspection notes, selecting a single note from the inspection will play the synchronized video.

In general, both the offshore discharge and intake pipelines appeared to be in good condition, with no major discrepancies encountered. The inspection focused on the interior wall of the pipe, joints, and manhole locations. A grading system in accordance with ASCE Underwater Investigations Standard Practice Manual to categorize conditions was utilized. Further details of the ROV inspection are included as an attachment.

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SGS Offshore Dive Team:

Offshore Intake Structure Inspection & Core Samples

The SGS offshore dive team performed a general condition inspection of the inside and outside of the offshore intake structure. The inside of the intake structure appeared to be in generally good condition, with no major discrepancies noted. Hard and soft growth was observed covering the intake structure with and average depth of 1 to 2".

In addition to the internal inspection of the intake structure, the SGS offshore dive team also conducted a condition inspection of the outside of the intake structure, including a survey of the protective rip-rap surrounding the structure. Overall the intake structure appeared to be in good condition, with two areas of concrete spalling noted on the top concrete curb of the velocity cap. One area of concrete spalling was observed adjacent to the Northwest corner on the West side, measuring from 8 to 12" in width. The other area of concrete spalling was observed at the Northeast corner, measuring 2 to 3" in depth that penetrated all the way through the top concrete curb on the velocity cap. No discrepancies were observed on the top of the velocity cap or intake structure walls. Heavy hard marine growth, in the form of muscles, was observed covering the entire velocity cap top curb as well as portions of the concrete intake structure walls.

Four (4) mammal barriers were also inspected. At the time of the inspection the concrete ledges where the mammal barriers are connected to the intake structure, were heavily fouled with hard marine growth, in the form of muscles. Three (3) of the four (4) gratings the East, South, and North gratings, had experience failure of their fastening system, either by possible concrete failure, swell vibrations, and/or corrosion of the gratings themselves. The North grating had become completely detached and was found sitting against the North wall. All gratings were cleaned and re-attached to the intake structure with 316 SS redhead fasteners. In addition, the manhole opening on the intake structure was also inspected. At the time of the inspection the manhole cover was missing. The manhole cover was replaced with 316 SS and secured into position with 316 SS redhead fasteners.

The protective rip-rap surrounding the intake structure consisted of large rock/boulders on average ranging in size from 5 to 10' in dimmension. The distance from the top of the intake structure to the top of the rip-rap was relatively consistent, with lower levels of rip-rap observed on the onshore Northwest / North side of the intake structure. Rip-rap top elevations were at minimum approximately 3' below the bottom concrete ledge of the mammal barriers. The rip-rap extended out from the structure anywhere from 27' to 36'. Beyond the rip-rap, the sea floor consisted of a sandy sediment. For further details on the intake structures protective rip-rap dimensions and elevations please refer to Figure 4.

The SGS offshore dive team also extracted two (2) ea. 3" diameter concrete core samples from the offshore intake structure, one (1) ea. core sample from the offshore terminus of the intake pipeline and one (1) ea. core sample from the intake structure. Both concrete core samples were taken from the inside of the intake pipeline system, which was accessed via the velocity cap area of the intake structure.

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The core sample for the offshore terminus of the intake pipeline was located approximately 12' from the offshore end of the pipe, on the East wall, located at approximately the 02:30 o'clock position. During the core drilling operations, the 3" core sample broke into three (3) sections. The dive team was able to successfully extract all three (3) sections of the core sample, for a total concrete core sample length of 14". After extraction of the core sample, the dive team immediately filled and patched the core drill hole with Splash Zone A-788 Two-Part Epoxy Repair Compound.

The core sample for the intake structure was located on the North wall 5' from the top of the structure. The dive team was able to successfully extract a 3" diameter by 14" long concrete core sample. After extraction of the core sample, the dive team immediately filled and patched the core drill hole with Splash Zone A-788 Two-Part Epoxy Repair Compound.

SGS Offshore Dive Team:

Offshore Discharge Structure Inspection & Core Samples

The SGS offshore dive team performed a general condition inspection of the inside and outside of the discharge structure. The inside of offshore discharge structure appeared to be in generally good condition. Spalling was observed consistently at the joint where the offshore terminus of the discharge pipeline connects to the discharge structure. Concrete erosion and spalling was observed on average of 4" deep at the joint, with the largest amount of concrete spalling located at the 10:30 – 11:00 o'clock position at a depth up to 3'. Hard and soft marine growth was observed covering the discharge structure with an average depth of 1 to 2".

The SGS offshore dive team also penetrated the discharge pipeline up to the second joint. The inside of the pipeline and the two (2) joints observed all appeared to be in good condition, with no discrepancies noted. The pipeline, like the discharge structure, was fully covered in hard and soft marine growth with an average depth of 1 to 2".

In addition to the internal inspection of the discharge structure, the SGS offshore dive team also conducted a condition inspection of the outside of the discharge structure, including a survey of the protective rip-rap surrounding the structure. Overall the discharge structure appeared to be in good condition with minor concrete spalling observed on the Southwest corner and West wall of the discharge structure. The grating for the discharge structure was no longer on top of the structure in its original position, and currently sits on the onshore side of the structure leaning up against the North wall. The grating appeared to be intact with no major discrepancies found. Two large concrete clump weights were also observed in the vicinity of the discharge structure, one sitting on top of the grating and the other sitting to the West of the discharge structure. Heavy hard and soft marine fouling was observed covering the discharge structure.

The protective rip-rap surrounding the discharge structure consisted of large rock/boulders on average ranging in size from 4 to 8'. The distance from the top of the discharge structure to the top of the rip-rap was relatively consistent, with an average of approximately 4.5' of discharge structure walls being exposed. The rip-rap extended out from the structure anywhere from 19' to 32'. No measurements of the rip-rap were taken on the North wall due to the obstruction from the discharge grating. Beyond the rip-rap, the sea floor consisted of a sandy sediment. For further details on the discharge structures protective rip-rap dimensions and elevations please refer to Figure 5.

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The SGS offshore dive team also extracted two (2) ea. 3” diameter concrete core samples from the offshore discharge structure, one (1) ea. core sample from the offshore terminus of the discharge pipeline and one (1) ea. core sample from the discharge structure. Both concrete core samples were taken from the inside of the discharge pipeline system, which was accessed via the top of the discharge structure.

The core sample for the offshore terminus of the discharge pipeline was located approximately 12’ from the offshore end of the pipe, on the East wall, located at approximately the 02:30 o’clock position. The dive team successfully extracted a fully intact 3” diameter by 14” long concrete core sample. After extraction of the core sample, the dive team immediately filled and patched the core drill hole with Splash Zone A-788 Two-Part Epoxy Repair Compound.

The core sample for the discharge structure was located on the North wall 3’-2” from the top of the structure. During the core drilling operations due to the presence of reinforced bar, the 3” core sample broke into two (2) sections with the first section 10” long and the second section 4” long. Both sections of the core sample were successfully extracted, for a total core sample length of approximately 14”. After extraction of the core sample, the dive team immediately filled and patched the core drill hole with Splash Zone A-788 Two-Part Epoxy Repair Compound.

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Figure 1: General Schematic of AES HB Circulating Water System
(Showing internal penetration tunnel inspections and concrete core samples)

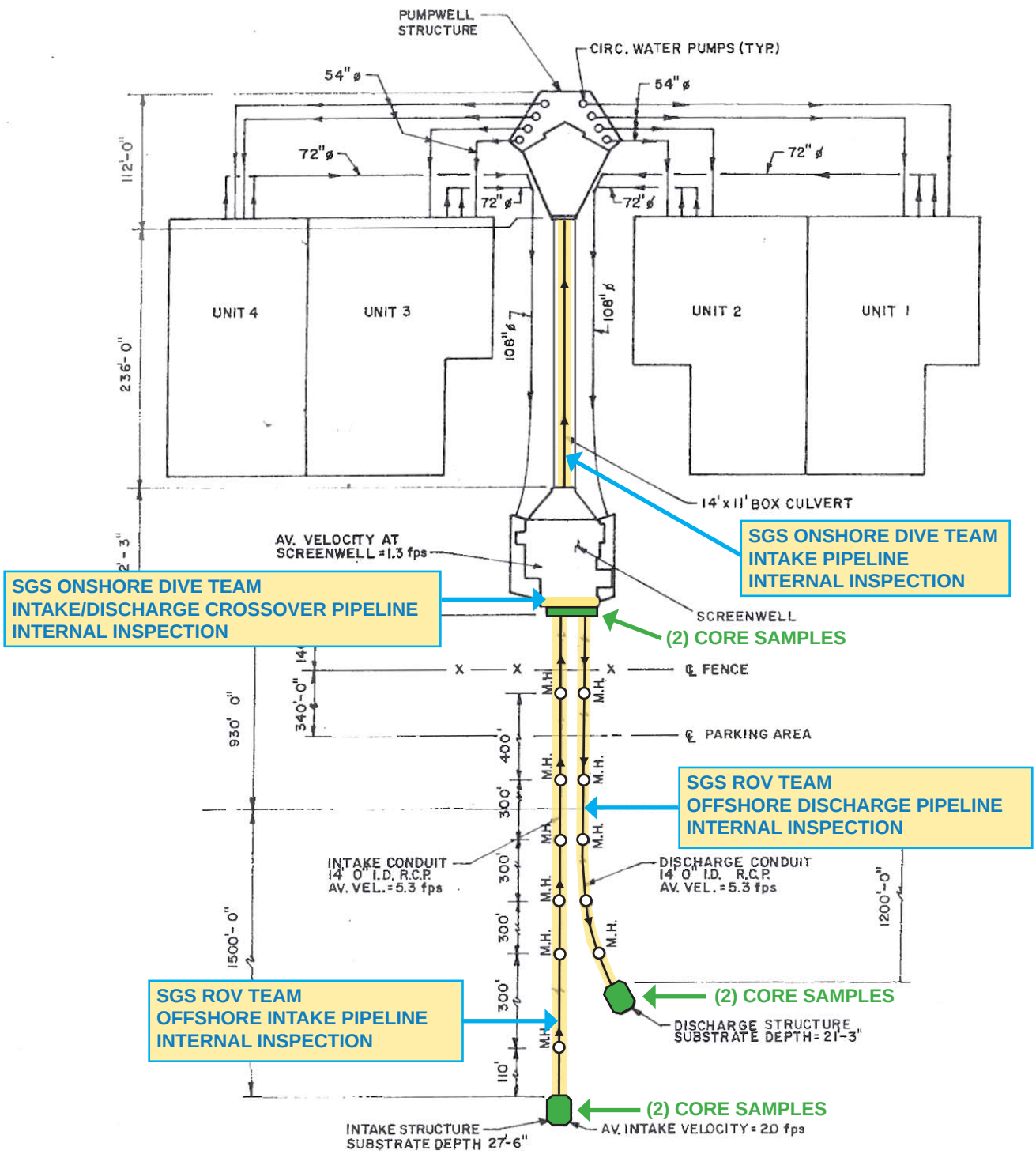

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Figure 2: General Schematic of Onshore Screenwell
(Showing location of crossover pipe)

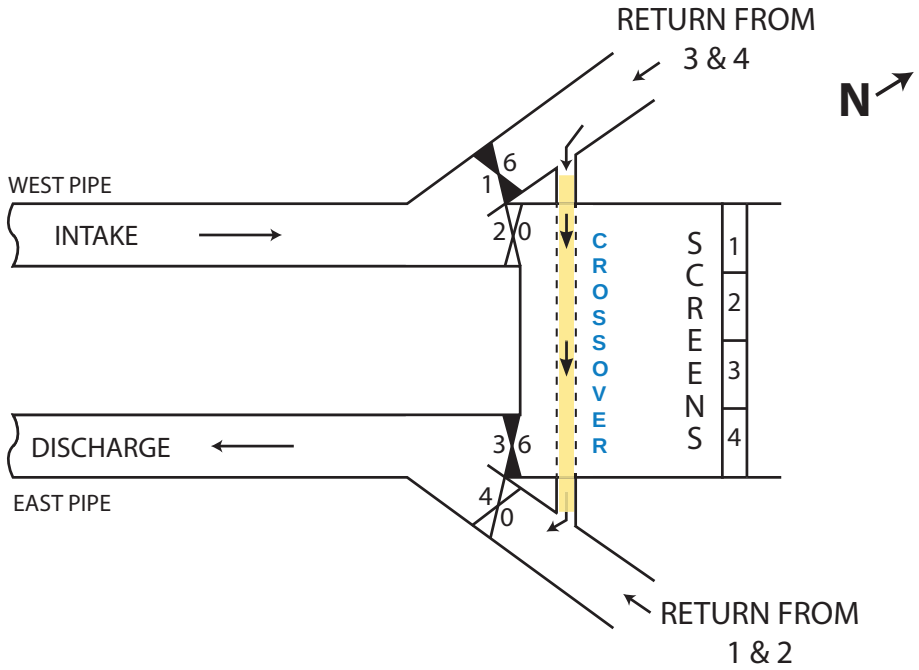

Figure 3: SGS Offshore Dive Team Core Sample Locations
Illustration Not to Scale (NTS)

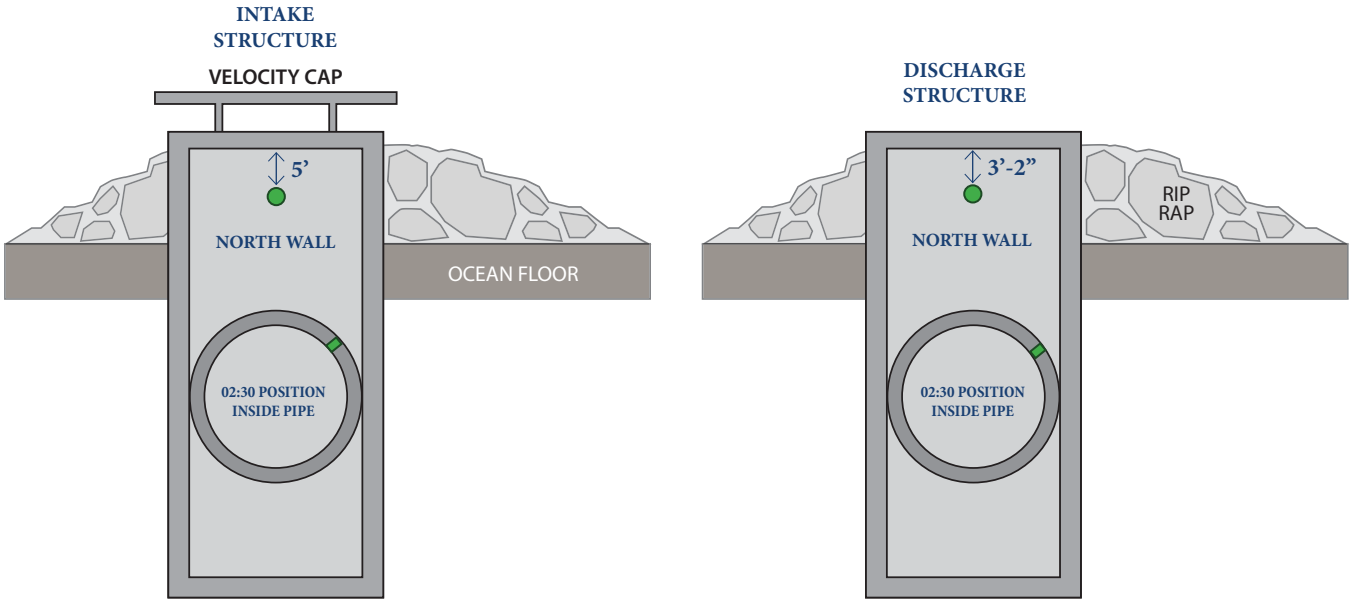

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Figure 4: Offshore Intake Structure Rip-Rap Measurements and Elevations Tide +3 (09:20)

Illustration Not to Scale (NTS)

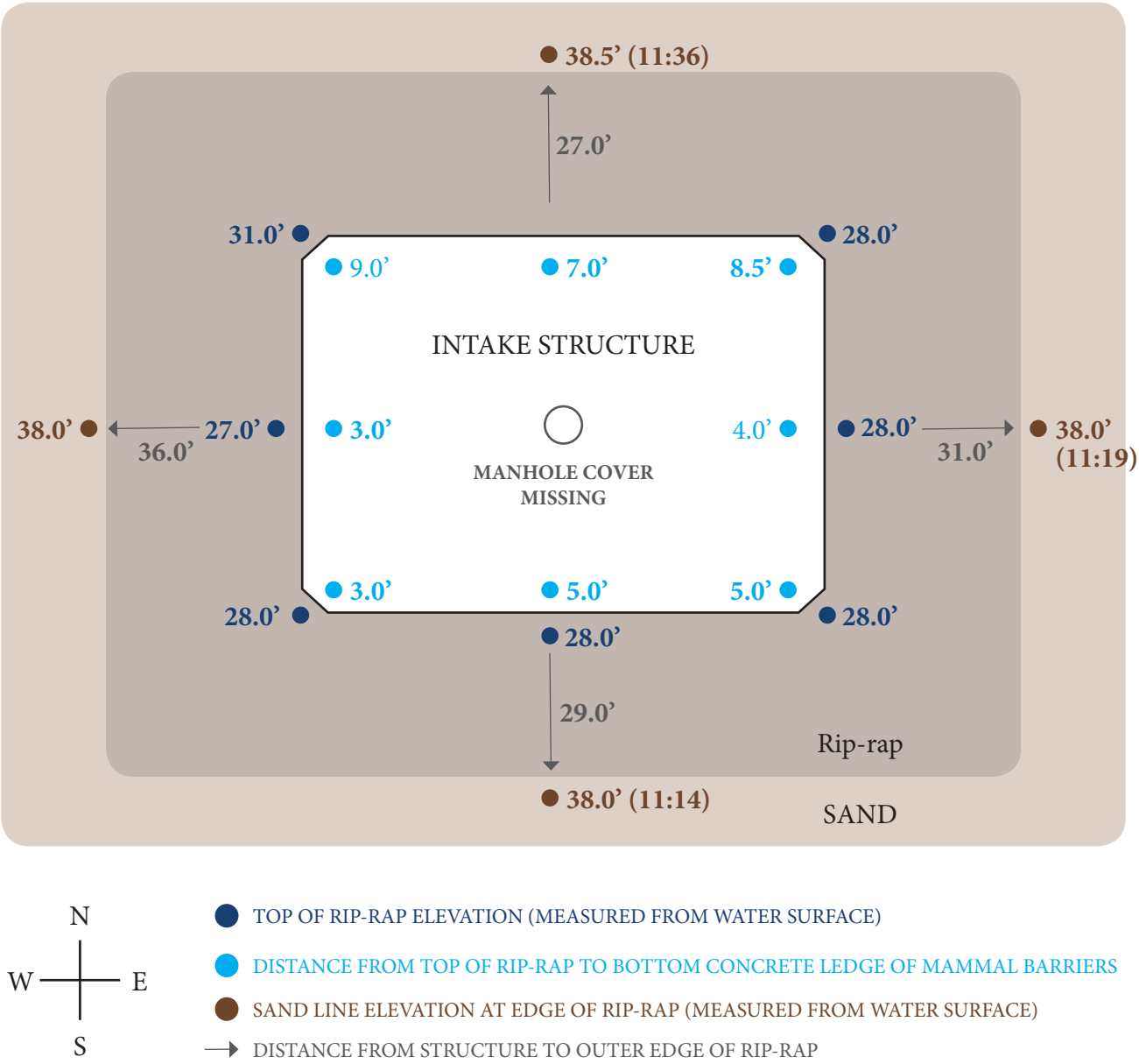

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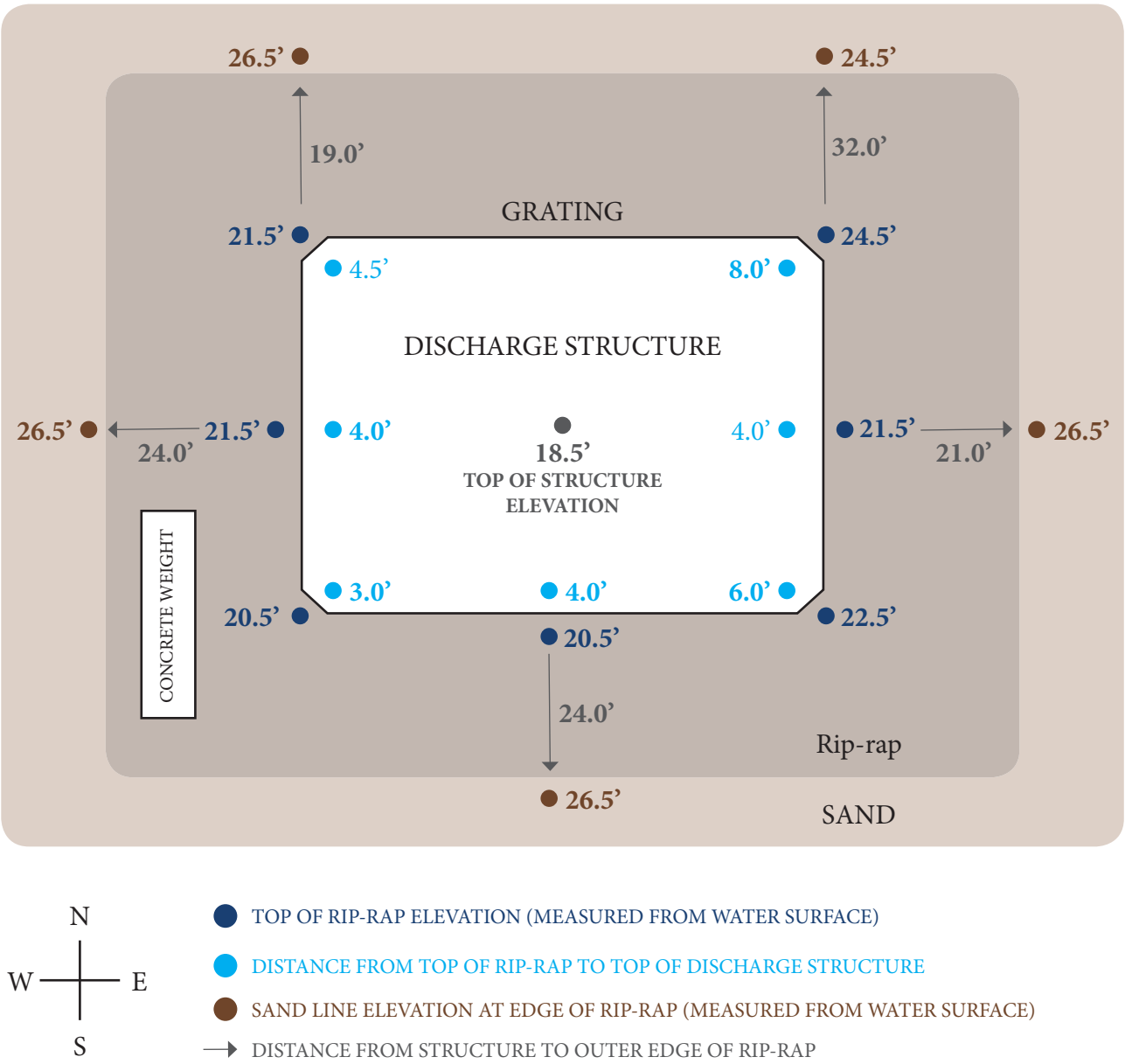
Figure 5: Offshore Discharge Structure Rip-Rap Measurements and Elevations Tide +1.5 (11:44)

Illustration Not to Scale (NTS)



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PROJECT TIMELINE

February 12, 2018

Onshore Dive Team:

- Mobilization of dive team and trailer to AES HB Facility.
- All crew members attend AES HB Safety Orientation.
- SGS dive team sets up dive station and core drilling equipment.
- Dive team to begin with core drilling of discharge pipeline onshore terminus.
- AES HB Lockout Tagout procedures performed.
- Diver enters forebay and discharge pipeline to layout and clean core location, 50' from grizzly bars into the discharge pipeline.
- Installed core drill on discharge pipeline onshore terminus, west side of pipe, 8" away from pipe joint.
- Diver extracted core from discharge pipeline terminus.
- The core broke into (3) sections, first section 3", second section 6", and third section 3".
- The diver was unable to extract the third section from core drilled hole.
- GHD engineer gave approval to leave the third section in core hole.
- Diver patched core hole with splash zone epoxy.

Offshore Dive Team:

- Mobilization of dive team and dive vessel to AES HB Facility.
- AES HB Lockout Tagout procedures performed.
- Setup anchoring system and mooring lines at offshore discharge structure.
- Located and marked discharge structure with identification buoy.
- Entered discharge structure to layout and clean in core location on the pipe and wall of the discharge structure.
- Rock drilled and installed red-heads at both core locations.
- Installed core drill on discharge pipe and core drilled 3" x 14" core, located 12' from end of pipe at 02:30 position.
- Diver extracted core #1 from discharge pipe, fully intact.
- Diver patched core drilled hole with splash zone epoxy.
- Dive team was unable to perform video survey of external discharge structure due to poor visibility conditions.

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February 13, 2018

Onshore Dive Team:

- AES HB Lockout Tagout procedures performed.
- Diver entered forebay and intake pipeline to layout and clean core location at the terminus of the intake pipeline.
- Rock drilled and installed red-heads to secure core drill base in location.
- Core drilled and extracted 3" x 14" sample from intake pipeline, onshore terminus.
- Diver patched core drilled hole with splash zone epoxy.
- Video inspection performed of the approximately 100 LF crossover pipeline from the discharge pipeline to the intake pipeline.

Offshore Dive Team:

- Mobilization of dive team and dive vessel to AES HB Facility.
- Secured mooring lines to discharge structure.
- AES HB Lockout Tagout procedures performed.
- Rock drilled and install red-head in discharge structure wall for installation of core drill base.
- Core drilled hole in discharge structure and extracted 3" x 14" sample from discharge structure.
- Removed in core sample in (3) sections. Location is on North wall 3'-2" from top of structure.
- Diver patched core drilled hole with splash zone epoxy.
- Video inspection performed of discharge structure, surrounding rip-rap, and grating.
- The grating was located inshore, resting on the north wall.
- First joint of discharge pipe found to have 4" up to 3' depth of erosion(better wording) observed at 1030-1100 o'clock positions.

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February 14, 2018

Onshore Dive Team:

SGS dive team conducted pipe penetration for internal inspection of the onshore intake pipeline.

- AES HB Lockout Tagout procedures performed.
- Dive crew removed and reinstalled (2) manhole covers. Installed (5) new bolts and nuts (provided by AES Contract Administrator)
- Divers inspected a total of 400 LF of the intake onshore pipeline.
- Divers inspected (4) traveling screens at the south end of the offshore intake pipe.
- The intake pipe was reported to have no significant visual discrepancies.
- Hard and soft marine fouling was noted on all sides of the pipe, between 2” to 8” in thickness.

Offshore Dive Team:

- Mobilization of dive team and dive vessel to AES HB Facility.
- Secured mooring lines to intake structure.
- AES HB Lockout Tagout procedures performed.
- Rock drilled and installed red-heads in intake structure pipeline, 12’ from end of pipe at the 230 o’clock position.
- Core drilled 3” x 14” hole in intake structure pipeline. Core extracted in (3) sections.
- Diver patched core drilled hole with splash zone epoxy.
- Rock drilled and installed red-heads for second core sample in intake structure, north wall, 5’ from the top of the north structure wall.
- Core drilled 3” x 14” hole in intake structure north wall. Core extracted intact.
- Diver patched core drilled hole with splash zone epoxy.
- Video inspection performed of the intake structure condition, including surrounding rip-rap size and elevations.
- North grating found to be missing, on bottom at the toe of the north wall. Red heads had pulled out of concrete, no fiberglass grating present.
- East grating red heads found to be present and secure, no fiberglass grate present.

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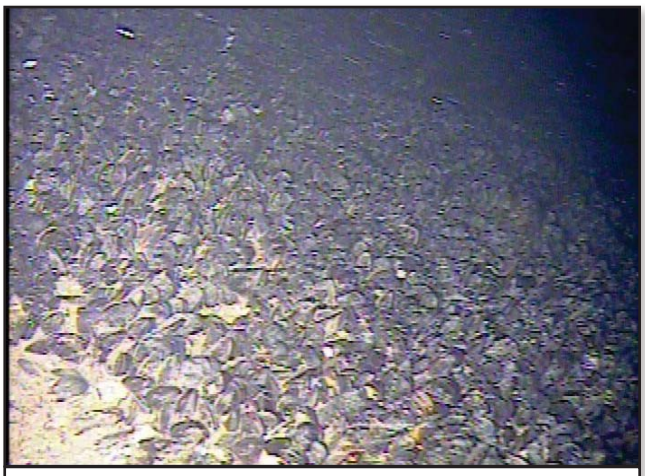
01



Inshore 2/12/18:
Discharge pipeline onshore terminus core location prior to cleaning

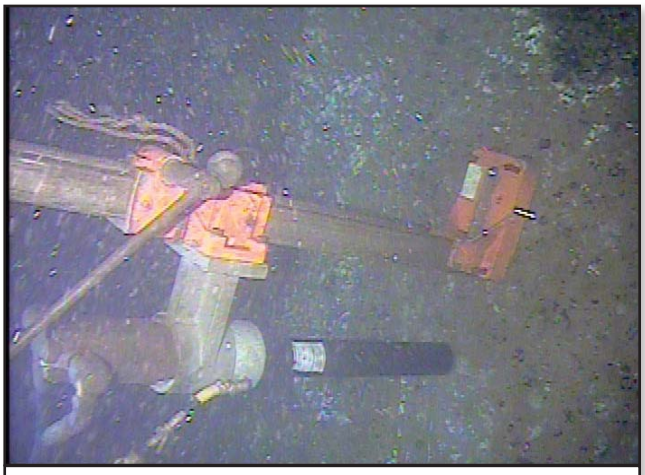
02

Inshore 2/12/18:
Discharge pipeline onshore terminus core location after cleaning, 50’ from grizzly bars.

03

Inshore 2/12/18:
Red heads and core drill installed into location

04

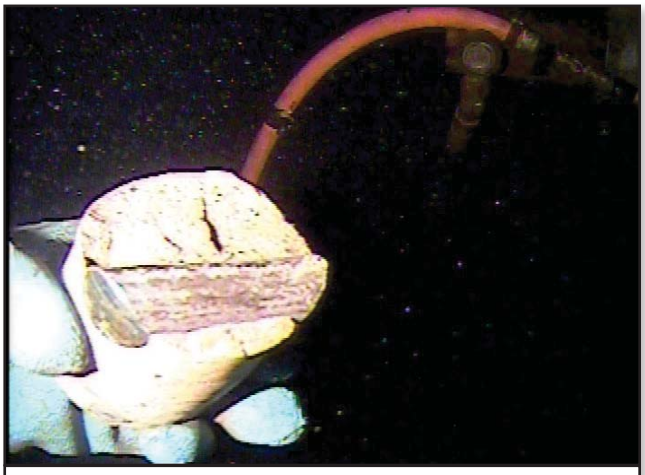


Inshore 2/12/18:
Begin core drilling discharge pipeline onshore terminus core

05

Inshore 2/12/18:
Discharge pipeline onshore terminus core completed.

06



Inshore 2/12/18
Core extracted, broke into (3) sections.
Last section unable to remove.

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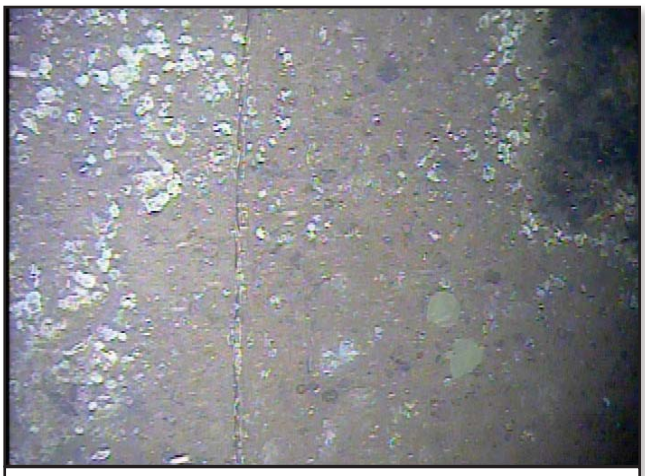
07

Inshore 2/12/18:
Discharge pipeline onshore terminus core drill after splash zone.

08



Inshore 2/12/18:
Splash zone cover of core drill location.

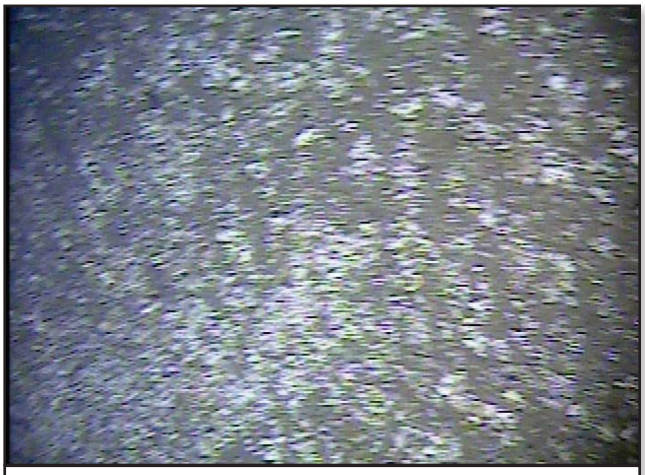
09

Inshore 2/12/18:
Extracted core from discharge pipeline onshore terminus.

10

Inshore 2/12/18:
Extracted core from discharge pipeline onshore terminus.

11



Inshore 2/13/18:
Intake pipeline onshore terminus core location

12

Inshore 2/13/18:
Intake pipeline onshore terminus core sample.

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13

Inshore 2/13/18:
Intake pipeline onshore terminus core sample.

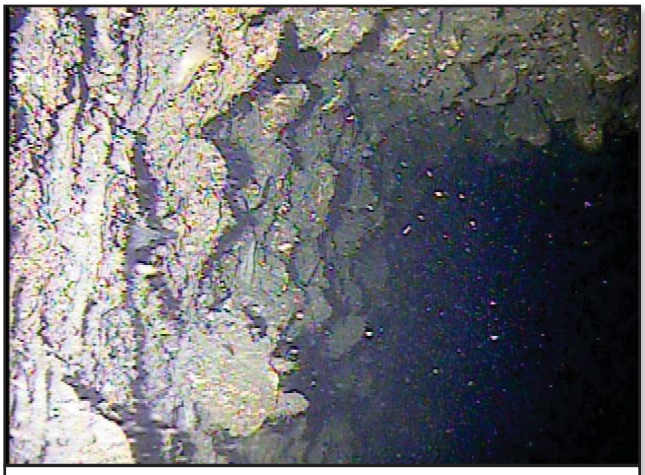
14

Inshore 2/13/18:
Core extracted, broke into (3) sections.

15

Inshore 2/14/18:
Intake pipe marine growth
2-8” in depth

16

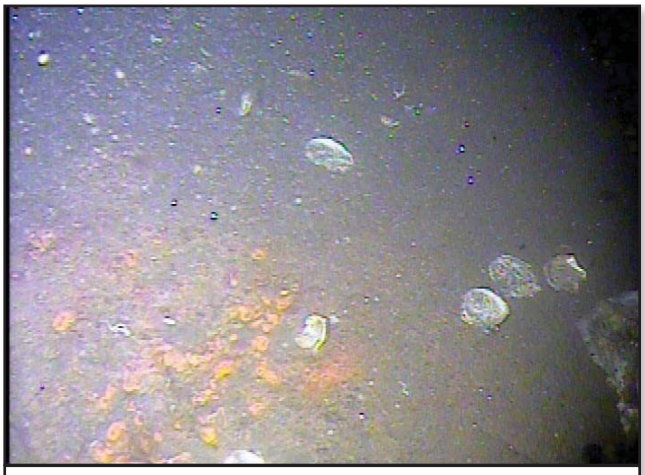


Inshore 2/14/18:
Intake pipe marine growth
2-8” in depth

17

Inshore 2/14/18:
Intake pipe marine growth

18



Inshore 2/14/18:
Intake pipe floor marine growth 2-6” in depth

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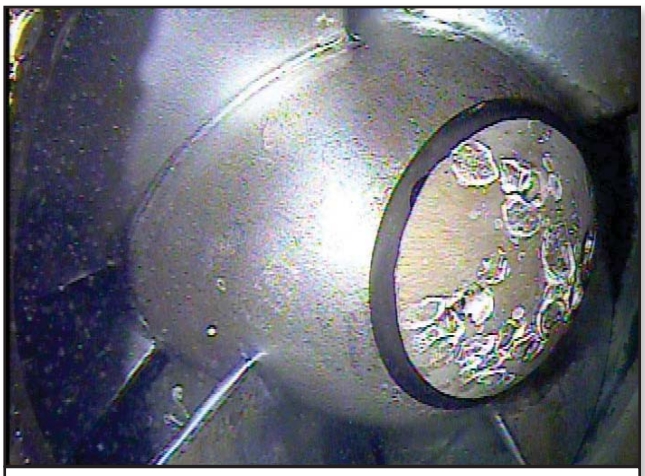
19

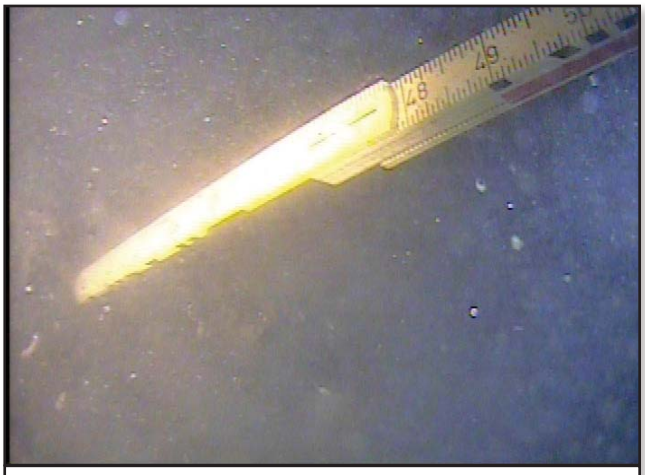
Inshore 2/14/18:
Intake pump housing, marine growth
4-6” in depth

20



Inshore 2/14/18:
Intake pump impeller, good condition

21



Inshore 2/14/18:
South end of intake pipe 42+” of mud / sediment

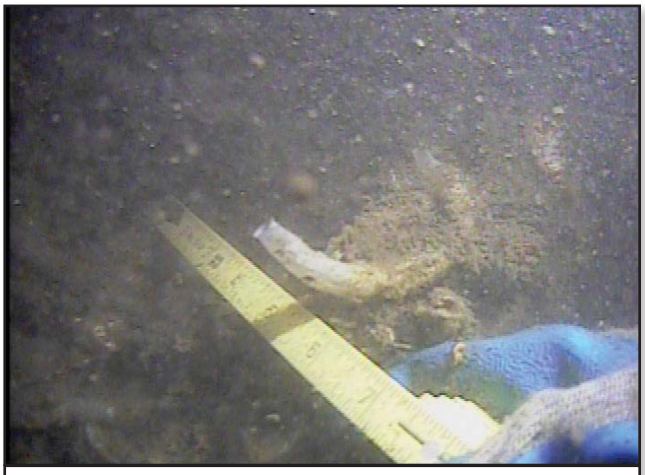
22

Inshore 2/14/18:
Intake travel screen

23

Inshore 2/14/18:
Intake travel screen, above water line

24



Inshore 2/14/18:
Diver measuring depth of marine growth
consistent 2-8”

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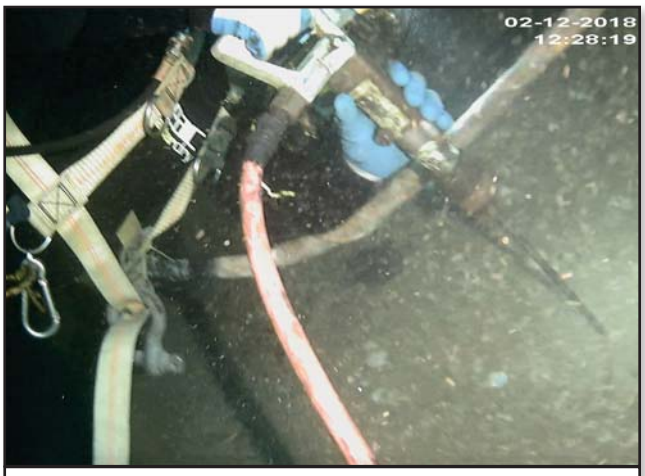


Offshore 2/12/18:
In process of rock drilling in discharge pipe for red head.

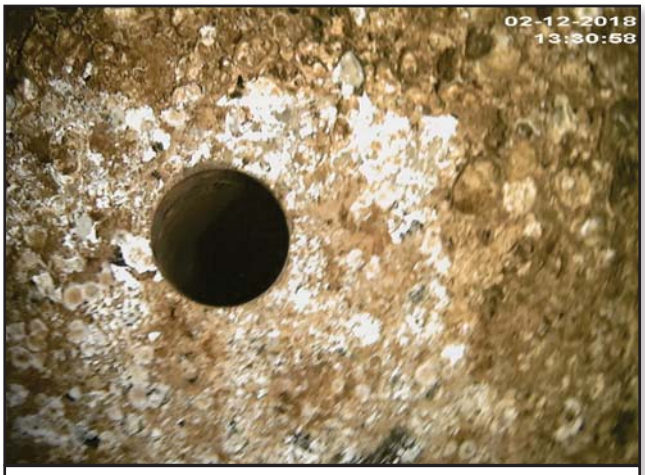
26

Offshore 2/12/18:
Red head installed in discharge pipe 15' in at 230 o'clock for core drill base.

27

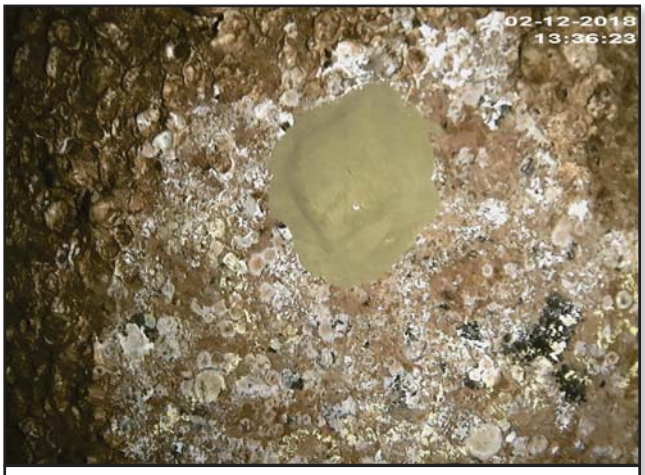
Offshore 2/12/18:
In process of core drilling in discharge pipe after drill mounted.

28



Offshore 2/12/18:
Discharge pipe core hole after sample extracted.

29



Offshore 2/12/18:
Discharge pipe core hole after splash zone epoxy applied.

30

Offshore 2/12/18:
Discharge pipe 3" x 14" core sample after extraction

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Offshore 2/13/18:
In process of core drilling discharge structure.

32

Offshore 2/13/18:
In process of core drilling discharge structure.

33

Offshore 2/13/18:
Core sample hole after splash zone epoxy applied in discharge structure.

34

Offshore 2/13/18:
3” x 14” core sample extracted from discharge structure.

35

Offshore 2/13/18:
Top perimeter of discharge structure.

36

Offshore 2/13/18:
Top perimeter of discharge structure.

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37

Offshore 2/13/18:
Typical rip-rap rocks surrounding discharge structure.

38

Offshore 2/13/18:
Typical rip-rap rocks surrounding discharge structure.

39

Offshore 2/13/18:
Typical rip-rap rocks surrounding discharge structure.

40

Offshore 2/14/18:
Core hole #3 on intake pipe after sample has been extracted..

41

Offshore 2/14/18: Core hole #3 on intake pipe after splash zone applied.

42

Offshore 2/14/18:
Core hole #4 on intake structure after sample has been extracted.

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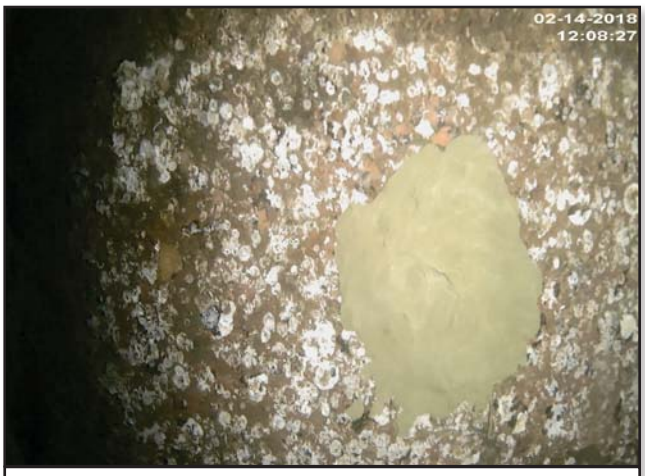

Offshore 2/14/18:
Core hole #4 on intake pipe after splash zone applied.

44

Offshore 2/14/18:
Missing manhole cover on intake velocity cap.

45

Offshore 2/14/18:
Typical hard growth on intake velocity cap.

46

Offshore 2/14/18:
Typical minor hard growth present on all grating bars.

47

Offshore 2/14/18:
West grating overhead fasteners worn/eroded through stainless steel grating.

48

Offshore 2/14/18:
Intake north grating on bottom red heads present after pulled out of concrete.

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ATTACHMENT

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Condition Assessment

Of

Offshore Intake/Discharge Piping

At

AES Huntington Beach

For

Subsea Global Solutions

Version 1
March 12, 2018
Job 518501

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Executive Summary

Hibbard Inshore provided a Remotely Operated Vehicle (ROV) inspection of the 14 foot ID intake and discharge tunnels at the AES Huntington Beach Facility. The inspection took place on February 13th and February 14th, 2018 using a Seabotix LBV 600 ROV. The video, sonar and inspection notes were recorded with the VisualSoft software program. The video and video record of the sonar is synchronized with the inspection notes, selecting a single note from the inspection will play the synchronized video.

The purpose of the inspection was to determine debris levels and general conditions of the pipeline.

In general, there were a few features of note:
1) Air bubbles at the crown of the tunnel in the discharge area.
2) Marine growth on the walls of the tunnel made observation of the concrete difficult, due to this factor the profiling sonar was used extensively to check the ovality of the tunnel
3) Sonar profiles showed on average of 3” of marine growth on walls of tunnel.
4) Sediment depths varied from 6” to 2’ in different areas of tunnel, exact measurements are listed in sonar snapshots with distances.
5) Sonar cross sections did not show any areas of ovality in tunnel.
6) Intake trash racks where observed to be free and clear of debris or obstruction.
7) Discharge grating was not inspected due to being removed at time of inspection to allow diver access.
8) Area around concrete core location was observed and noted in video.
9) No flaws or areas with defects where observed during investigation of pipe.

2 Inspection Method

The ROV inspection was performed with a Seabotix LBV600 vehicle. During the inspection, video and sonar were recorded to the VisualSoft data acquisition software with written annotations. Written notes recorded observations, ROV location, Sonar plot numbers and other pertinent data. The written notes or inspection notes are attached in the appendix. Also, annotated sonar plots from the standard inspection method are attached in the appendix. The inspection notes link all features by time and date. References to the inspection notes are made in the description of features so that the supporting data can be reviewed. Any portion of the video from the ROV has the date and time overlaid on the top of the video. The time can be compared to the inspection notes to locate the picture relative to the structure. A description of the ROV, sonar and sonar interpretation is included later in the report.

The depth on the overlay is the pressure depth of the ROV. When this compared to the elevation of the water surface, the elevation of the ROV can be calculated. The other information on the screen is instrument data to aid in the piloting of the vehicle. The number on the bottom row that precedes the letters “FF” is the pressure depth in feet of fresh water. The number precedes “HD” is the magnetic compass heading. The number that follows the text “CA” is the camera angle. Zero degrees is horizontal and positive numbers are upward and negative are downward camera angles. At the top is the umbilical payout distance in feet.

On February 13th, the ROV was launched into the wet well at the AES Huntington Beach facility and was able to access both the intake and discharge pipes from that location. The ROV was lowered into the wet well by the tether and piloted into position for each penetration. Once the ROV was just inside the 14’ intake and discharge, the counter was set to zero and the ROV traveled along the crown of the tunnel. The ROV then inspected each visible construction joint while traveling downstream in the intake. The maximum penetration distance reached on Feb 13th was 1707ft from the intake tunnel launch point.

On February 14th, The ROV was launched into the wet well and quickly traveled downstream to resume inspecting at the 1700ft distance. The remainder of the tunnel intake was inspected with the final distance measurement from the wet well to the crib being 2484ft. The ROV was then recovered to the wet well and piloted into the discharge tunnel. The ROV traveled upstream in the discharge pipe and inspected each visible construction joint. The maximum penetration distance from the wet well to the discharge crib was 2169ft.

3 Inspection Observations

Figure 1 – Topside Surface control unit for ROV and video recording system

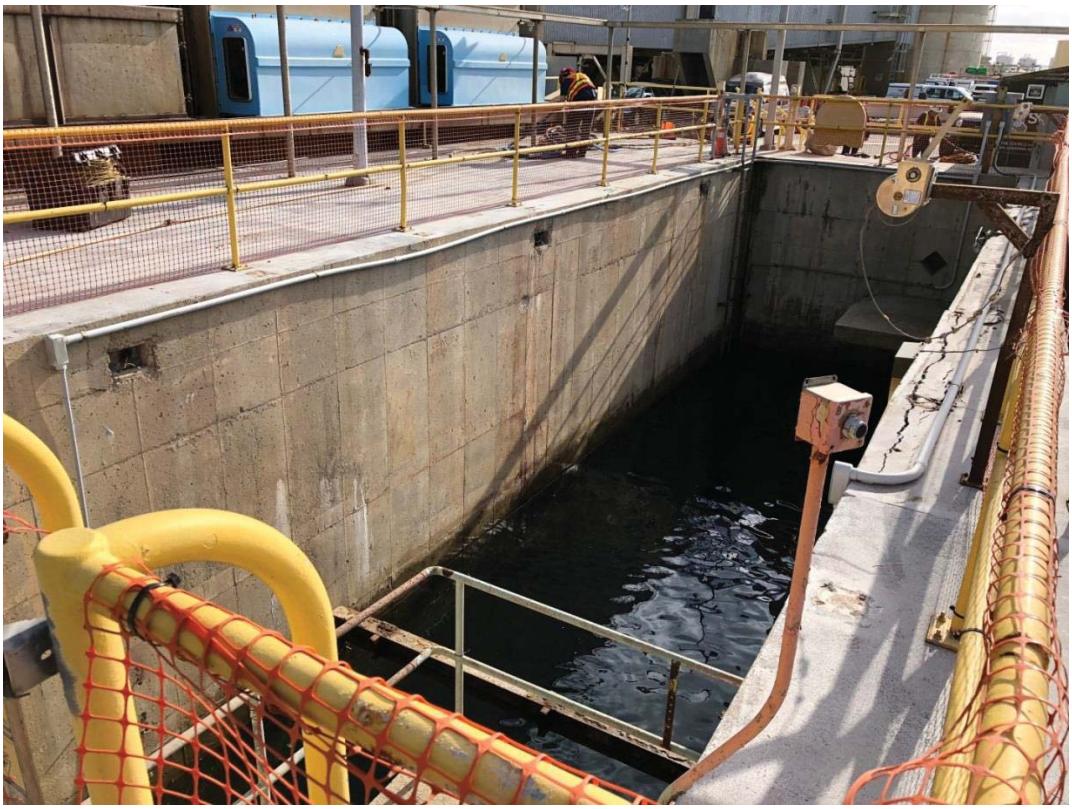

Figure 2 –ROV launch location in wet well

Figure 3 – Screen grab of crown 380ft downstream of wet well in intake.

Figure 4 – Screen grab of tunnel crown near outfall crib 1901ft

3.1 Manhole locations

The discharge and intake tunnels both had manhole shafts at the crown of the tunnel. The shafts where measured by parking the ROV on the crown of the pipe so that profiling sonar could scan the walls of the tunnel as well as the shaft.

3.1.1 Intake Manhole Locations and images

Comment	Tether Dist. (Ft.)
8ft from manhole lid to crown of tunnel, light marine growth	700ft
6.5ft from manhole lid to crown of tunnel, 3ft wide.	1100ft
vertical riser section 6.5ft, 3-3.5ft wide	1400ft
manhole height 6.5ft, 3 ft wide	2000ft
crown of tunnel to manhole 7ft, 3 ft diameter of manhole	2350ft

Figure 5 - Manhole at 700ft (intake), light marine growth

Figure 6- Manhole at 1100ft (intake), relatively little marine growth on lid

Figure 7- Manhole at 1400ft (intake)

Figure 8- Manhole at 1700ft (intake)

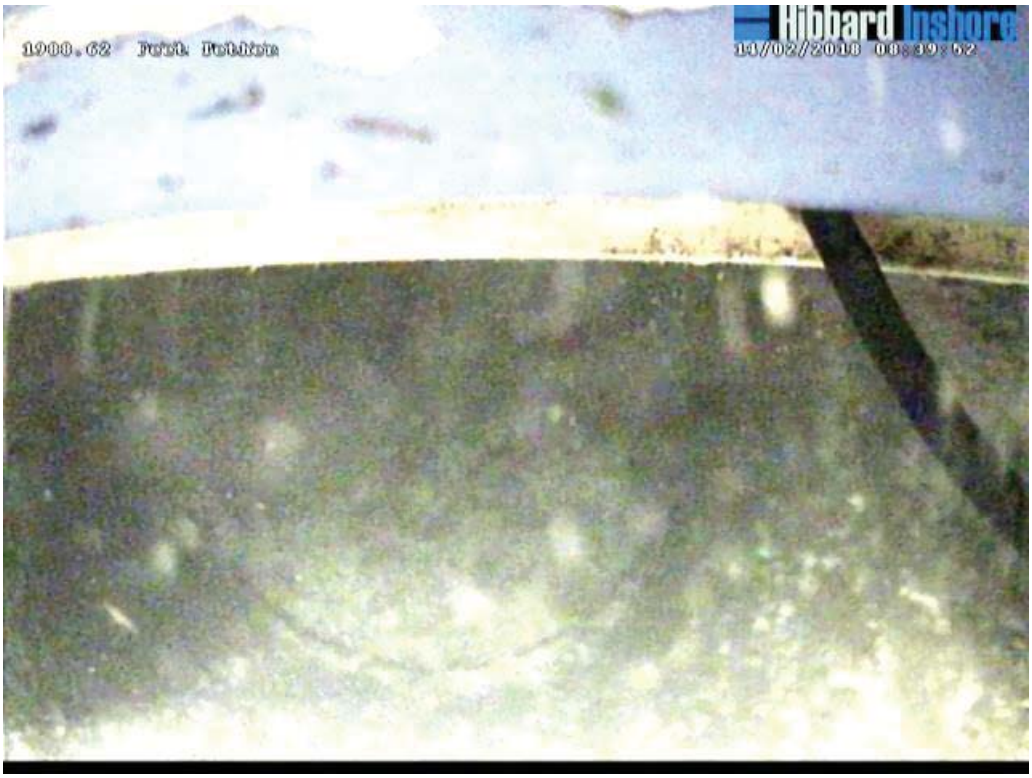

Figure 9- Manhole at 2000ft (intake)

Figure 10- Manhole at 2300ft (intake)

3.1.2 Discharge Manhole locations and images

Comment	Tether Dist. (Ft.)
with cross section of sonar it appears that the surface of the water against the air pocket in man hole is 3ft to 4ft deep, manhole seems to be partially filled with air	710
at manhole, cant see lid, 6.5ft stem length, 3ft wide	1105
1400ft continuing to manhole, manhole appears in good condition, 6.5ft length of stem, 3ft wide	1402
manhole at 1727ft roughly, lid of manhole appears clean, 7ft length of stem 3 ft. wide	1721
1990ft at manhole, 5.4ft stem so 1.6ft air	1990

Figure 11- Manhole at 700ft (Discharge), filled with air

Figure 12- Manhole at 1100ft (Discharge), could not see lid due to conditions in tunnel

Figure 13- Manhole at 1400ft (Discharge)

Figure 14- Manhole at 1700ft (Discharge), lid appears to be clean of marine growth

Figure 15- Manhole at 2000ft (Discharge), air and floating debris in manhole

3.2 Sonar Cross Sections

Sonar cross sections were taken on the return run starting at the interface between the cast iron pipe and the PCCP pipe. The cross sections were taken every 100 feet from 3,163 ft. to 450 ft.

3.2.1 Sonar Interpretation – Tritech Micron

The sonar heads used were two Micron sonar heads. One sonar head was a profiling head used to take cross sections. The other sonar was a sector head used for navigation as well as taking measurements and detecting features.

The sonar plots were recorded as snapshots. The display represents a vertical cross section at the back of the ROV. Up on the page is up and the right side is to the right of the ROV. There is a scale shown on the sonar image that shows the number of feet each range ring is for the ROV. Also, relative and absolute range information between two cursor-selected points is listed on the lower left part of the display. A vertical color bar on the right side shows the range of colors used to display the intensity of the returning sound wave. Red indicates the strongest return, yellow is moderate, and black indicates no return.

Sonar contains two common events that make interpreting the display more complicated. The first event is an echo. The sound wave can bounce off of multiple surfaces before returning, thus creating mirror images of features. This creates the illusion that there are more structures underwater than actually exist. These artifacts are easily distinguished and in many cases reveal additional information during analysis. One of the most common echoes is off of a water to air interface such as an air bubble in a tunnel or the free surface in a shaft. Shadows are the second event. The sound wave from the sonar head cannot see the backside of an object or around corners. These are the two most common events on a sonar plot that make the interpretation more complicated.

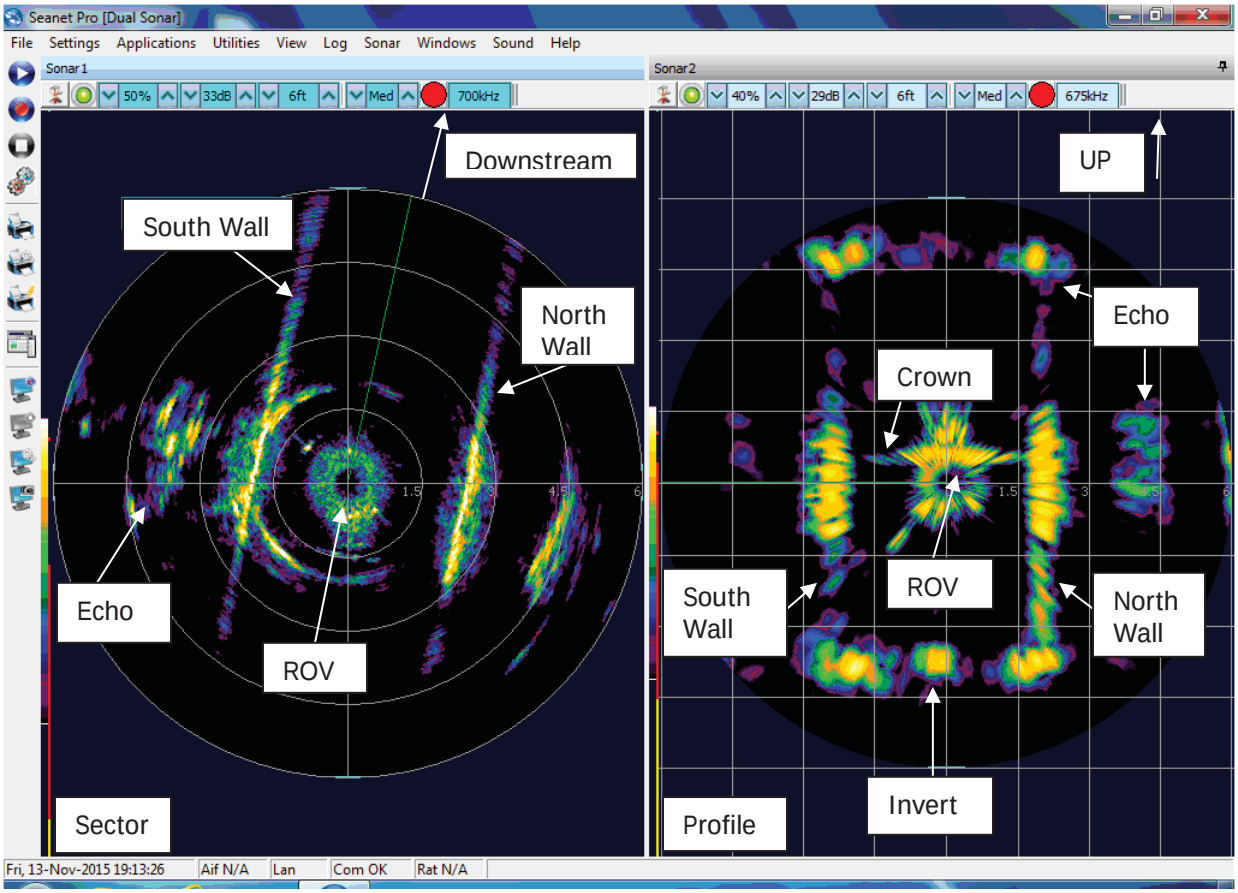

Figure 16 - Example Micron Sonar Image with a rectangular cross section tunnel

3.2.2 Cross Sections

3.2.2.1 Intake

Cross section at 50 feet. The cross section measured 12.1’ tall with no sediment or scaling evident.

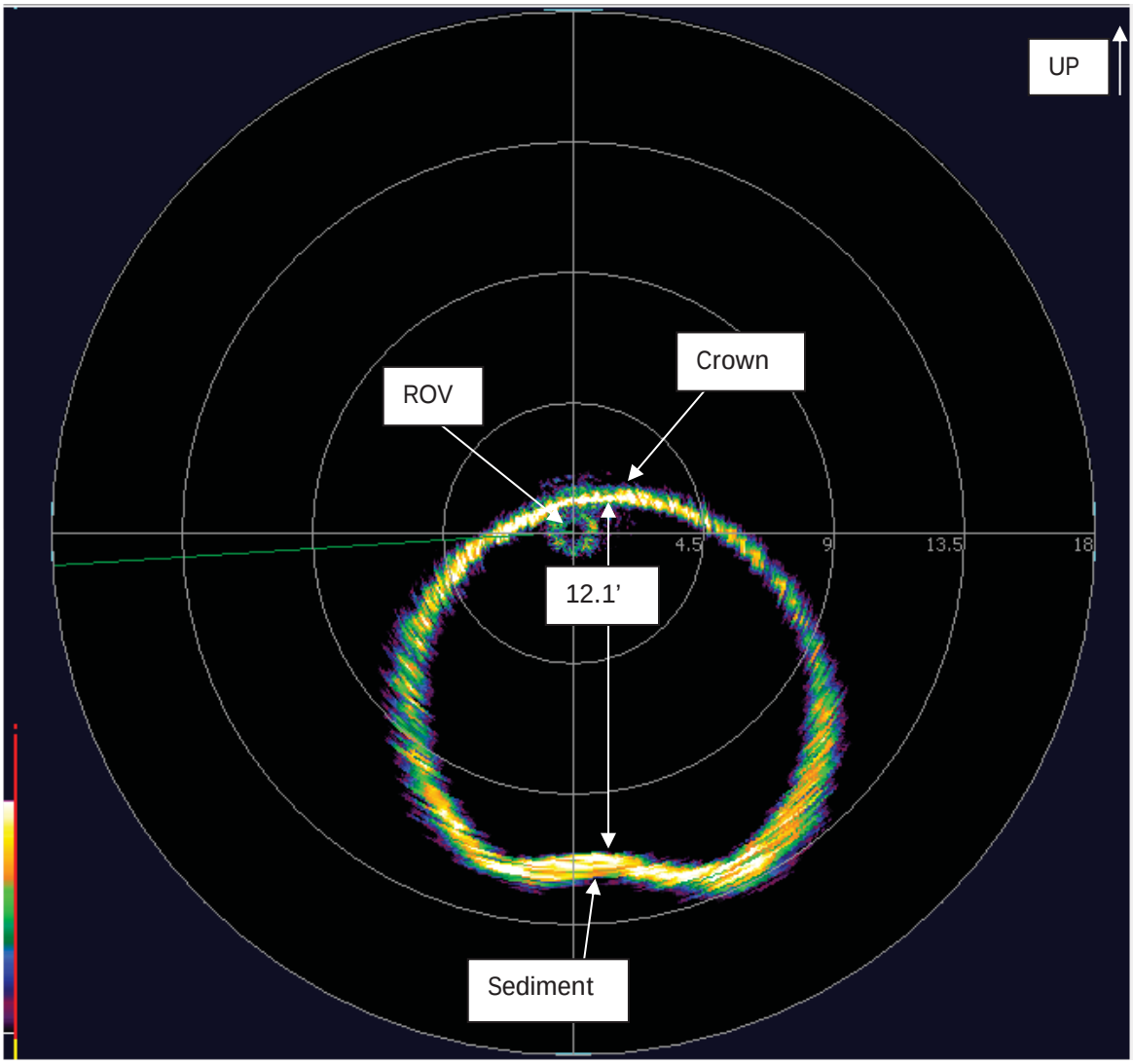

Figure 17 - Cross Section 1 - 50 feet.

Cross section at 100 ft. 12.04” from invert to crown.

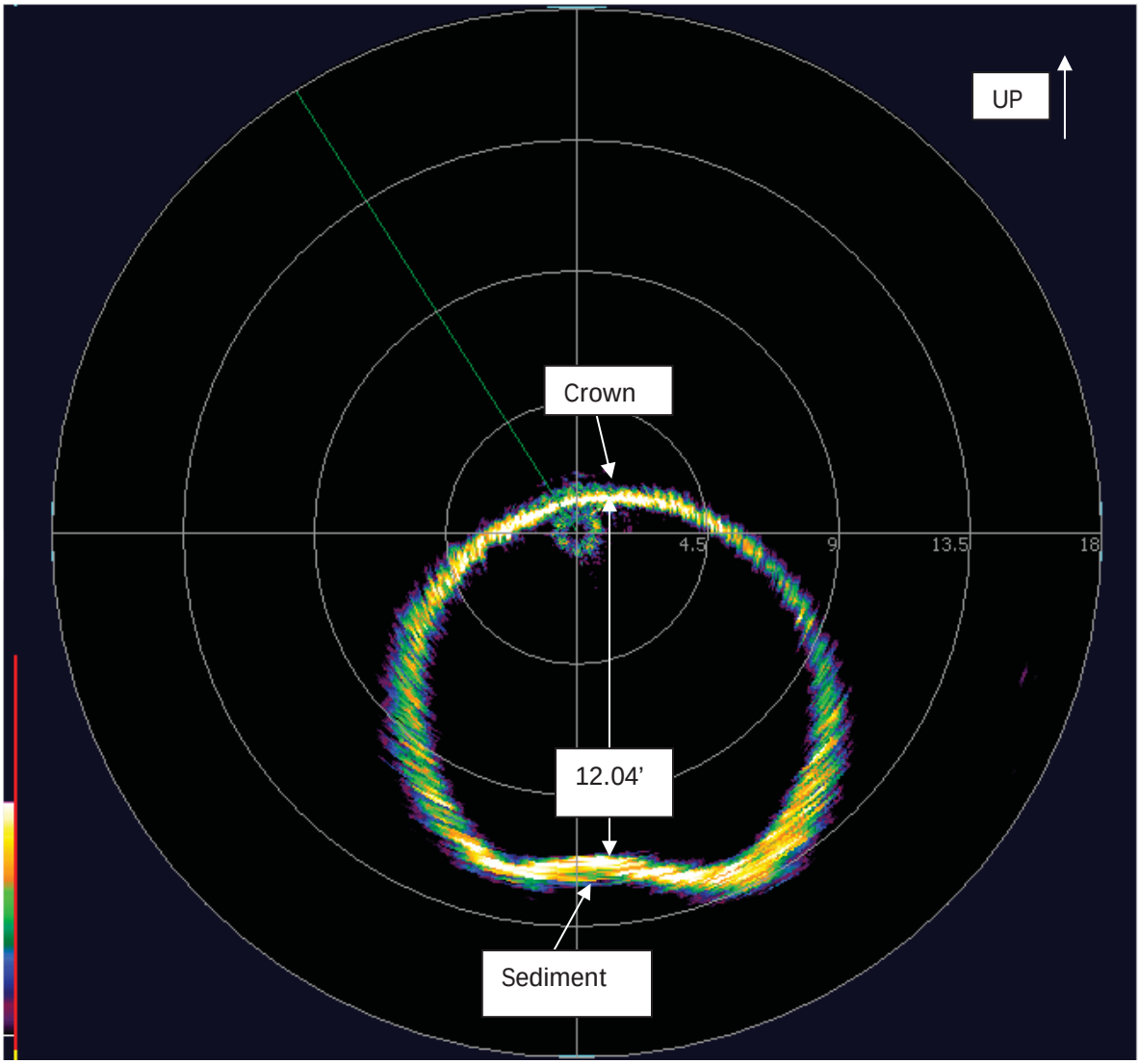

Figure 18 - Cross Section 1 - 100 feet.

Cross section at 150 ft 12.7’ from invert to crown.

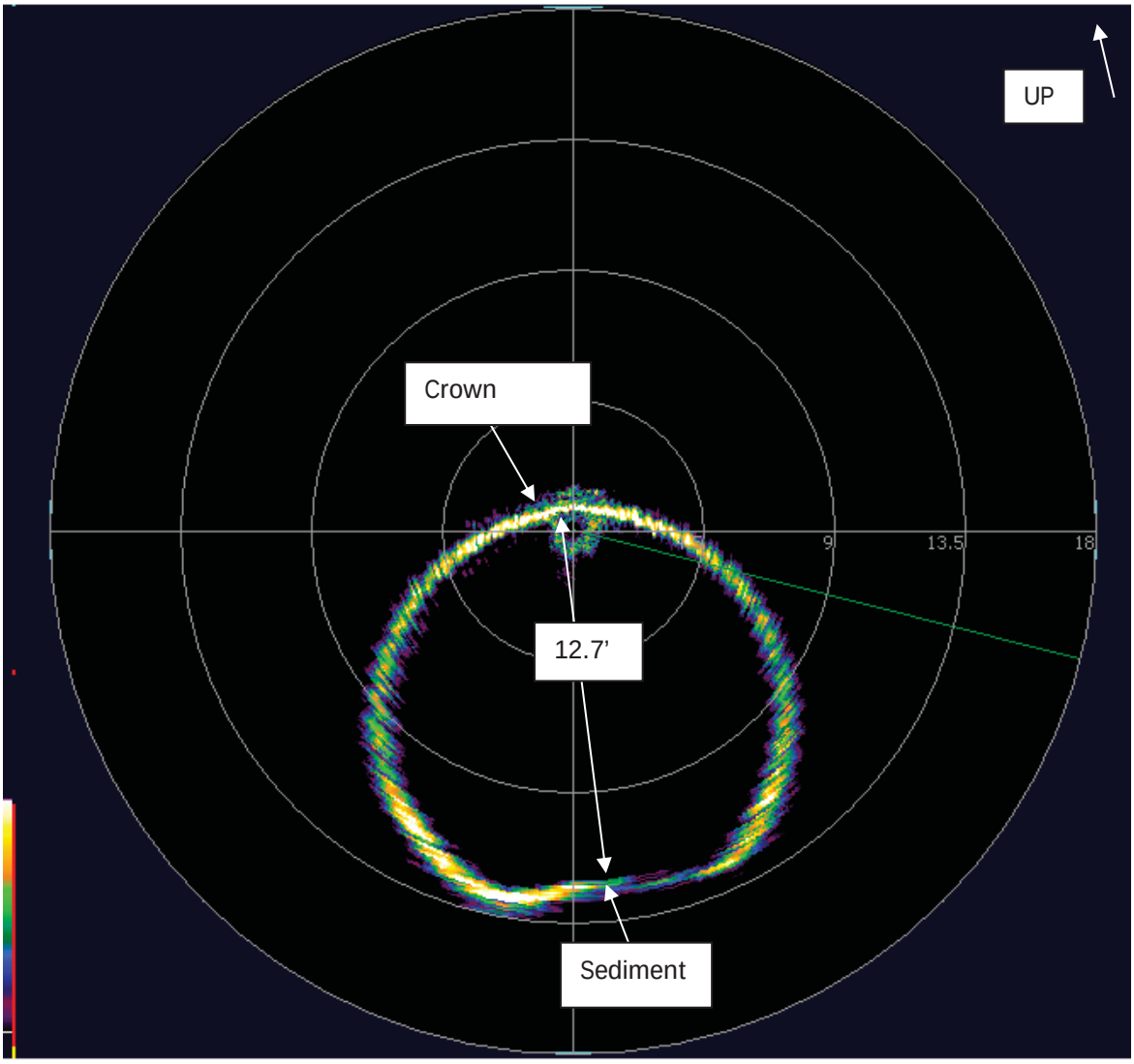

Figure 19 - Cross Section 2 - 150 feet.

Cross Section at 200 ft. 12.2’ from invert to crown.

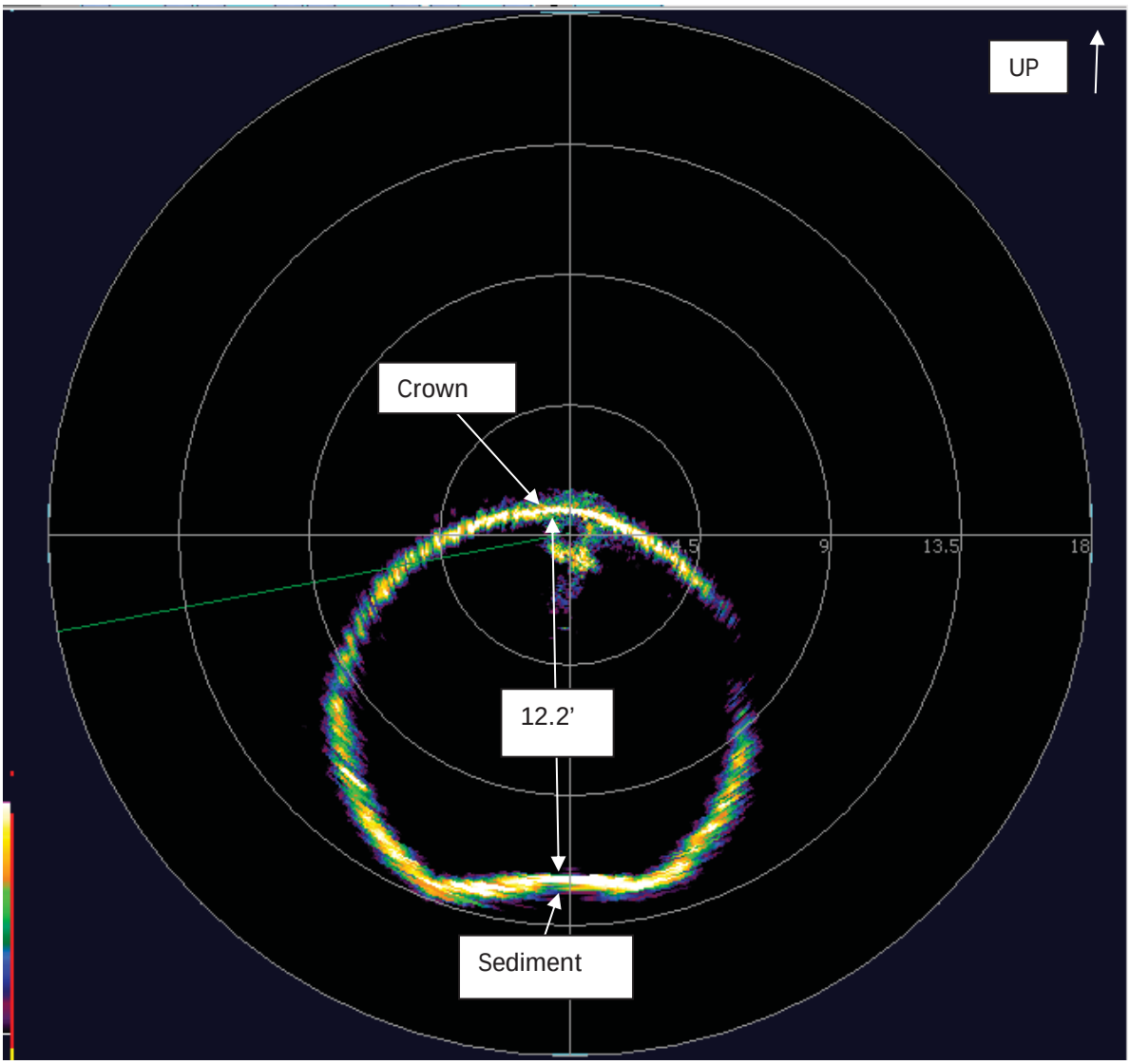

Figure 20 - Cross Section 3 - 200 feet.

Cross section at 250 ft. 12.6’ from invert to crown. No sediment

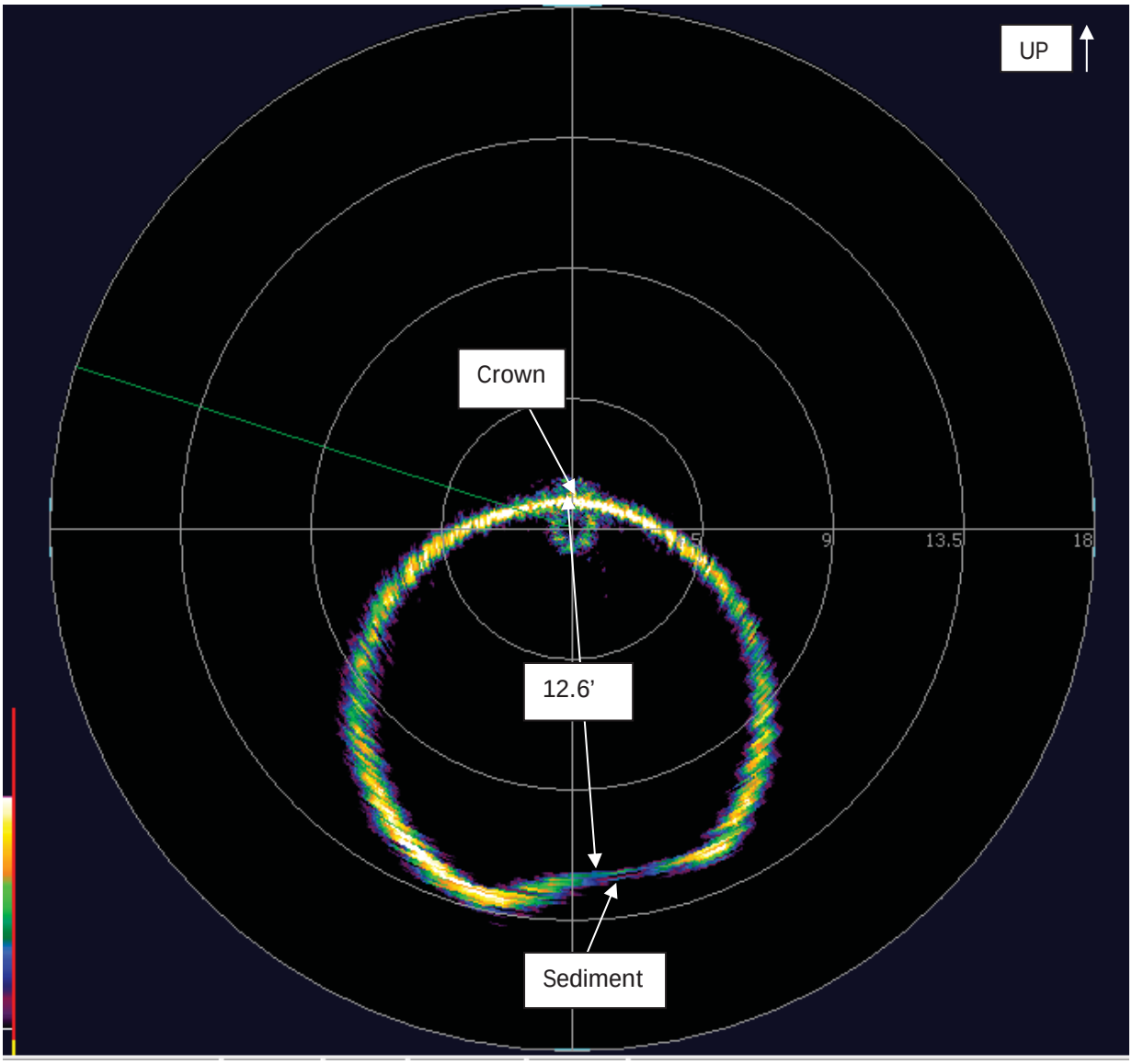

Figure 21 - Cross Section 4 - 250 feet.

Cross Section at 300 ft. 12.4’ from invert to crown.

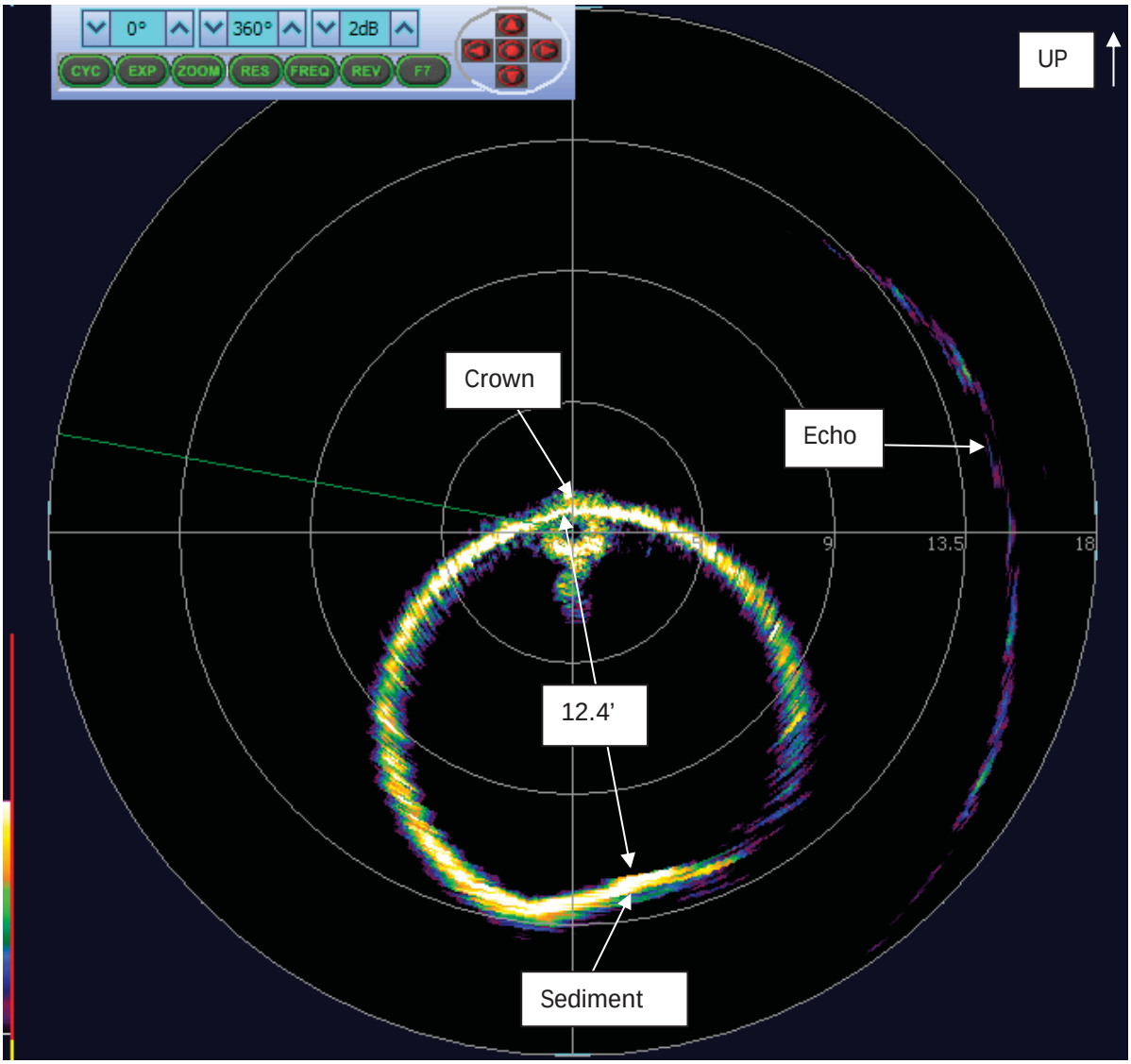

Figure 22 - Cross Section 5 - 300 feet.

Cross section at 350 ft. 12.3’ from invert to crown.

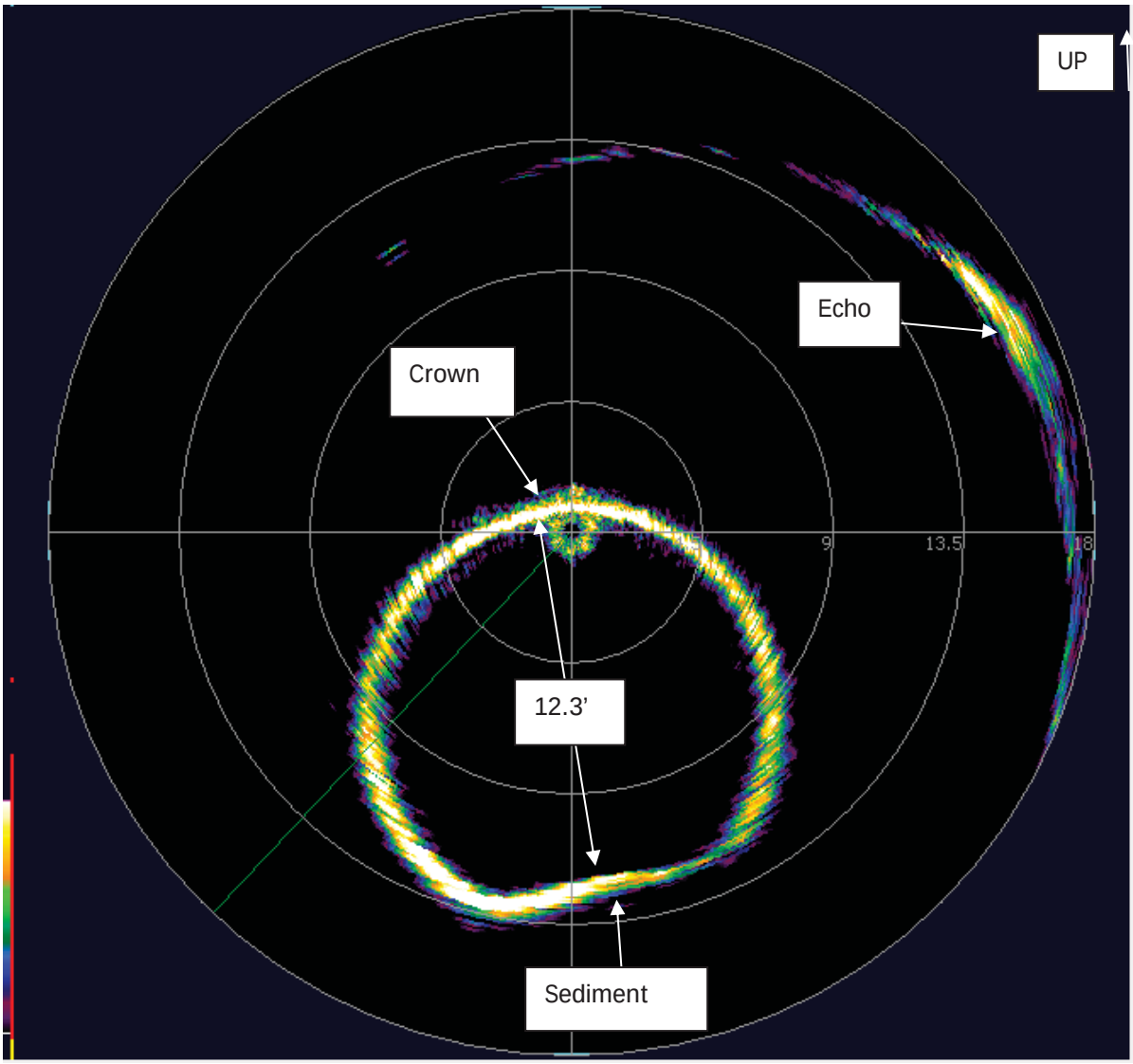

Figure 23 - Cross Section 6 - 350 feet.

Cross section at 400 ft. Measured height and width of tunnel from this scan moving forward. 13.2’ wide by 12’ from invert to crown.

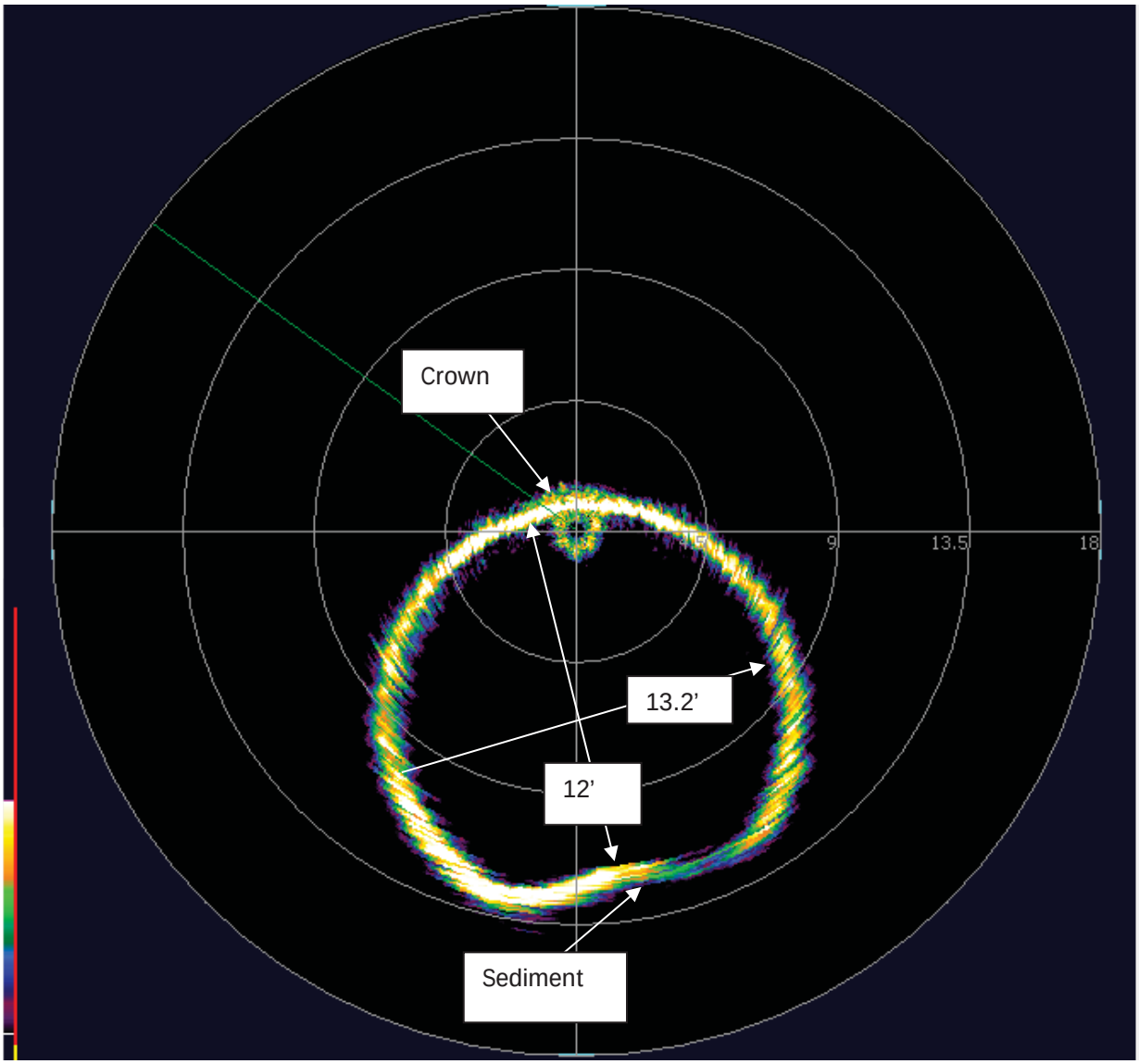

Figure 24 - Cross Section 7 - 400 feet.

Cross section at 450 ft. 13.1’ width by 12’ from invert to crown.

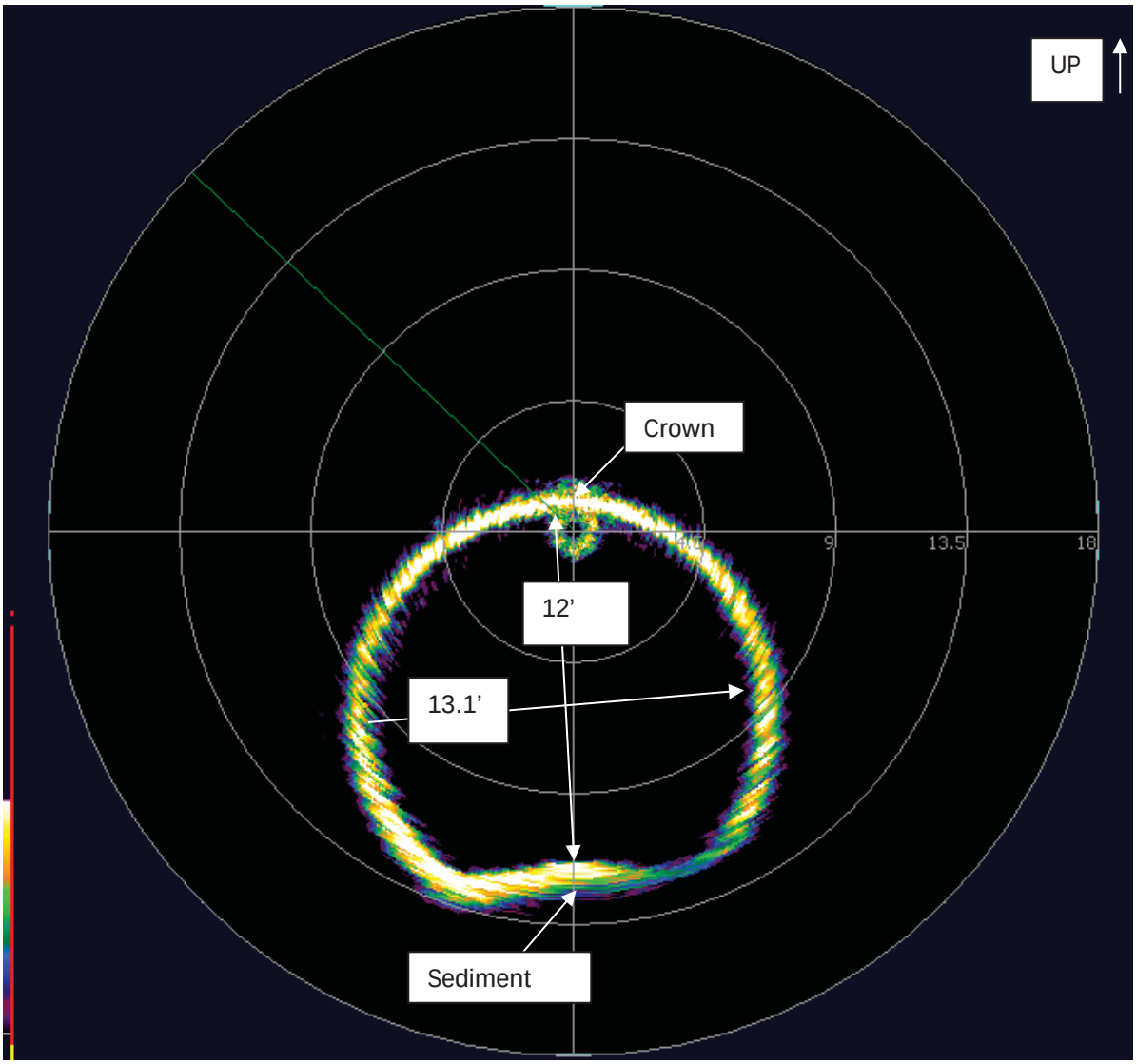

Figure 25 - Cross Section 8 - 450 feet.

Cross section at 500 ft. 13.3’ wide by 12.2’ from invert to crown.

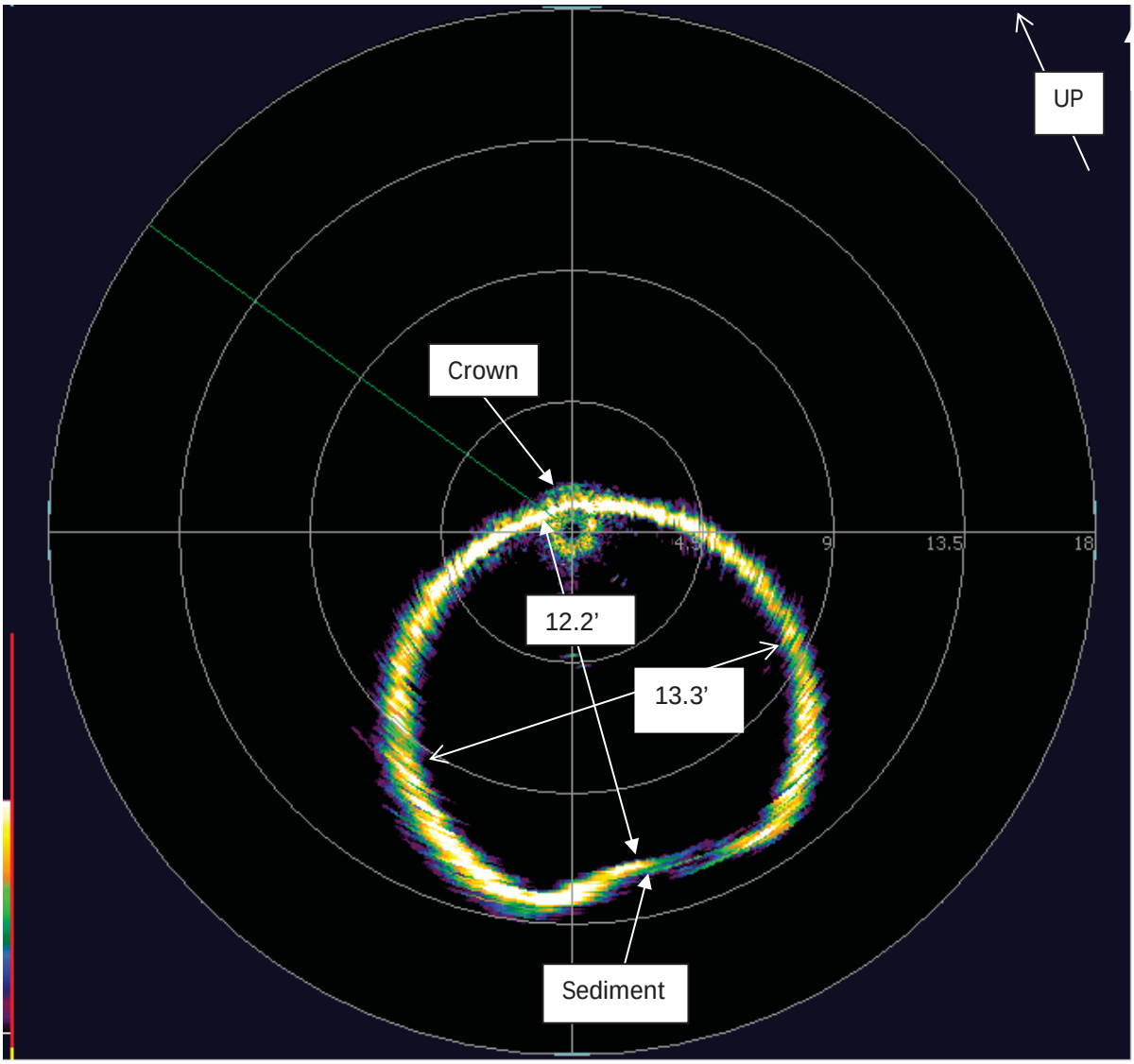

Figure 26 - Cross Section 9 - 500 feet.

Cross Section at 550 ft. 13.3’ wide by 12.3’ from invert to crown. Light amount of sediment

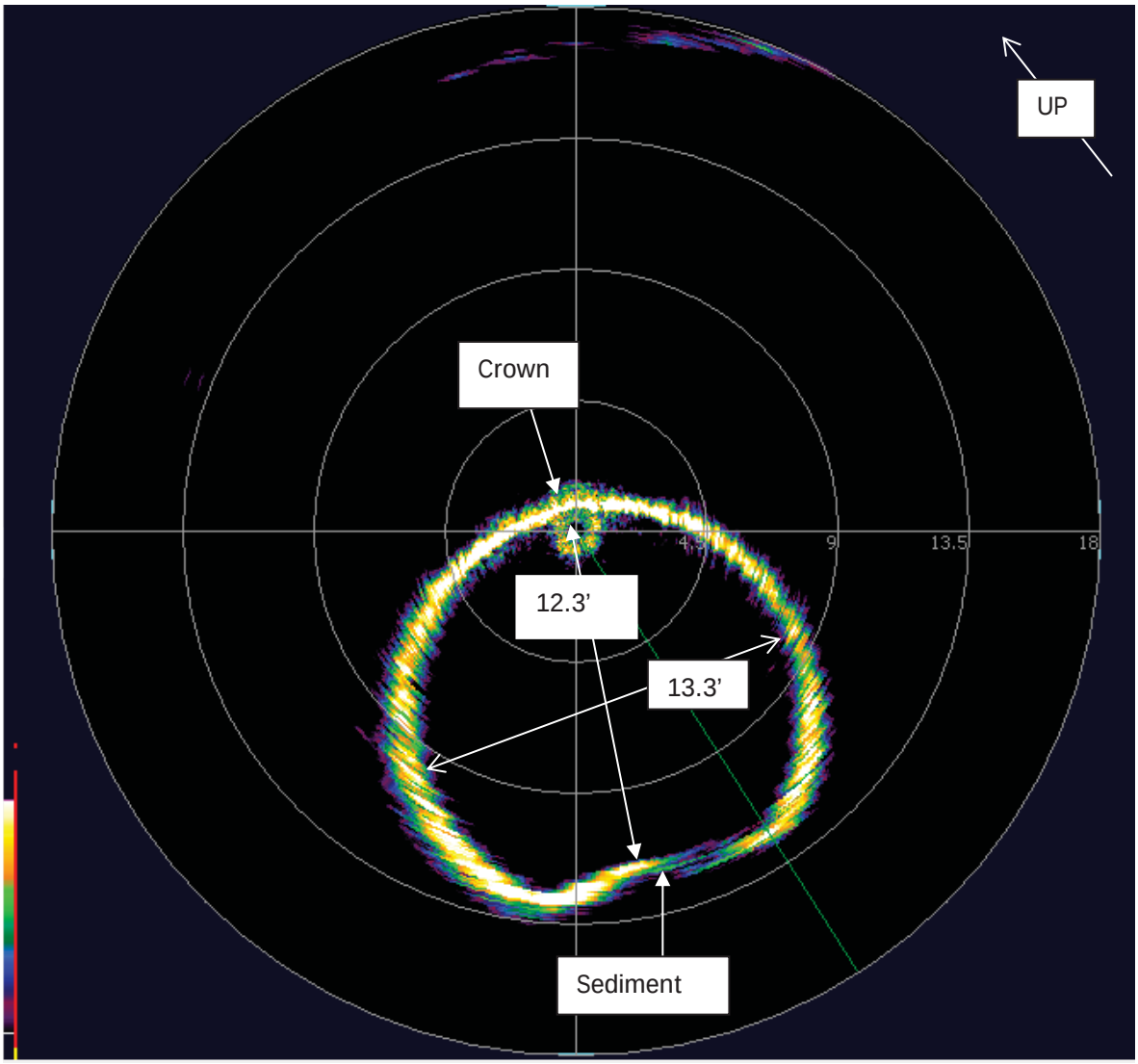

Figure 27 - Cross Section 10 - 550 feet.

Cross section at 600 ft. 13.2’ wide by 12.8’ from invert to crown.

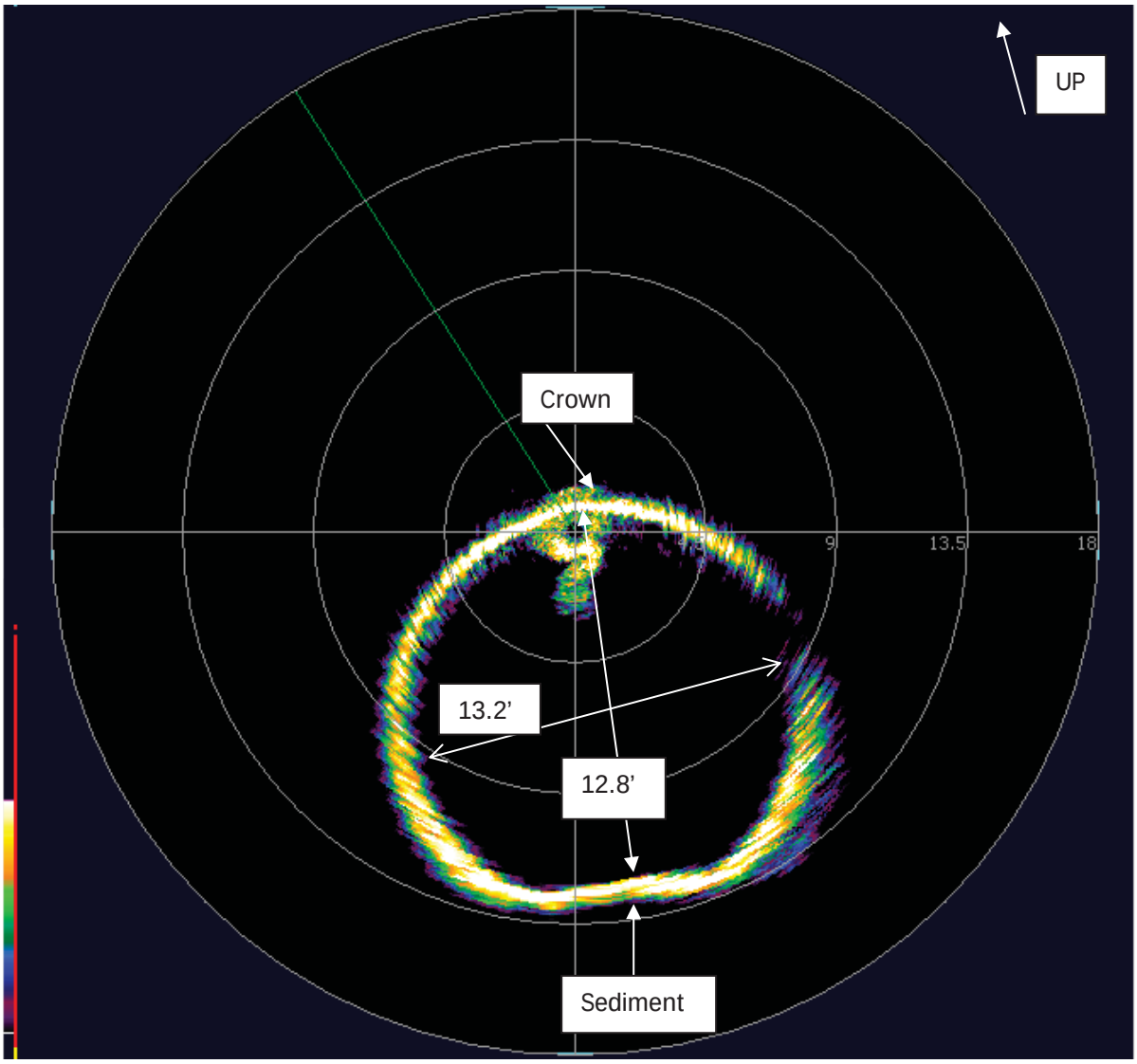

Figure 28 - Cross Section 11 - 600 feet.

Cross section at 650 ft. 13.3’ wide by 30” from invert to crown.

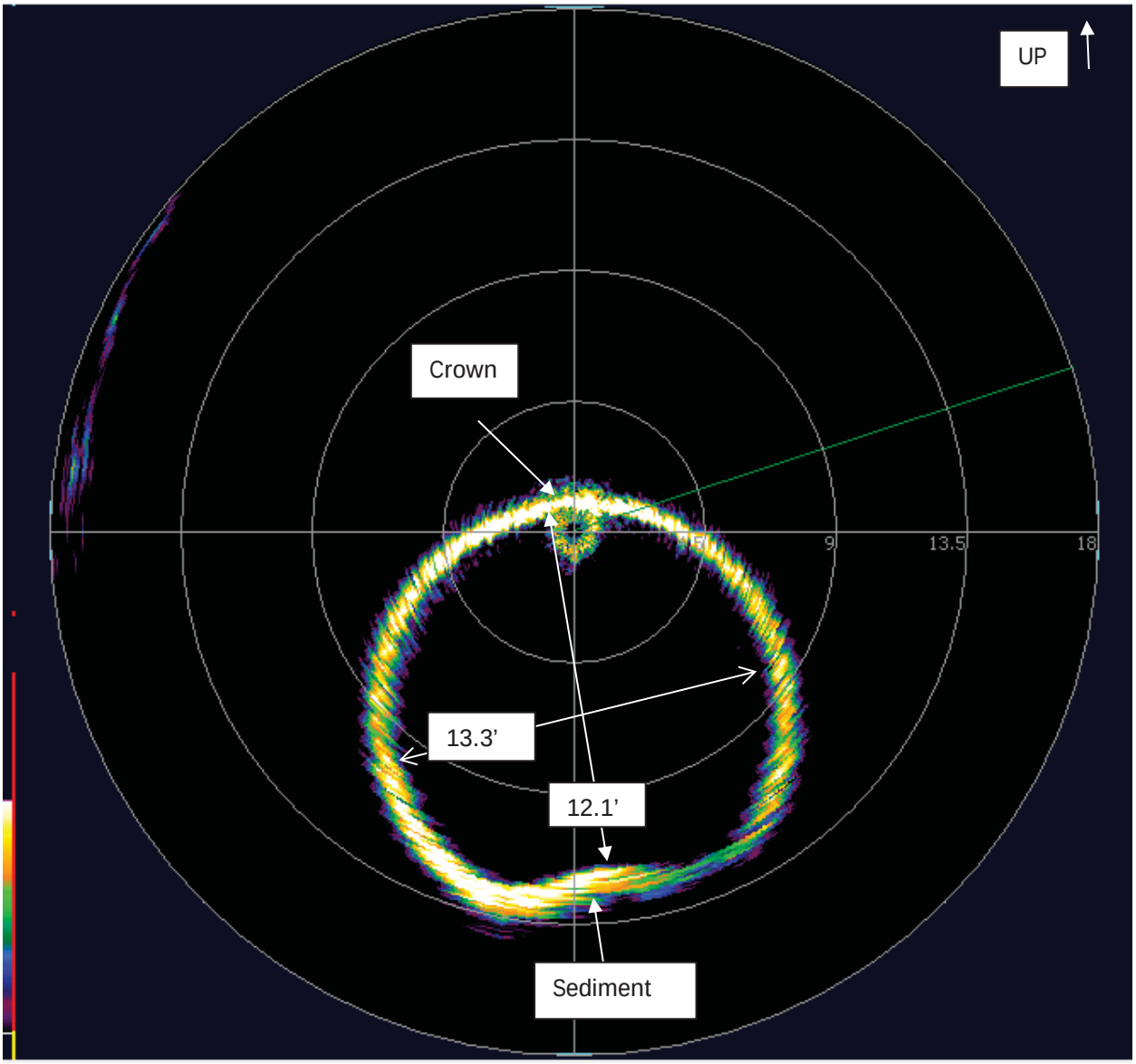

Figure 29 - Cross Section 12 - 650 feet.

Cross section at 700 ft. 13.2’ wide by 12.1’ from invert to crown. First location of manhole

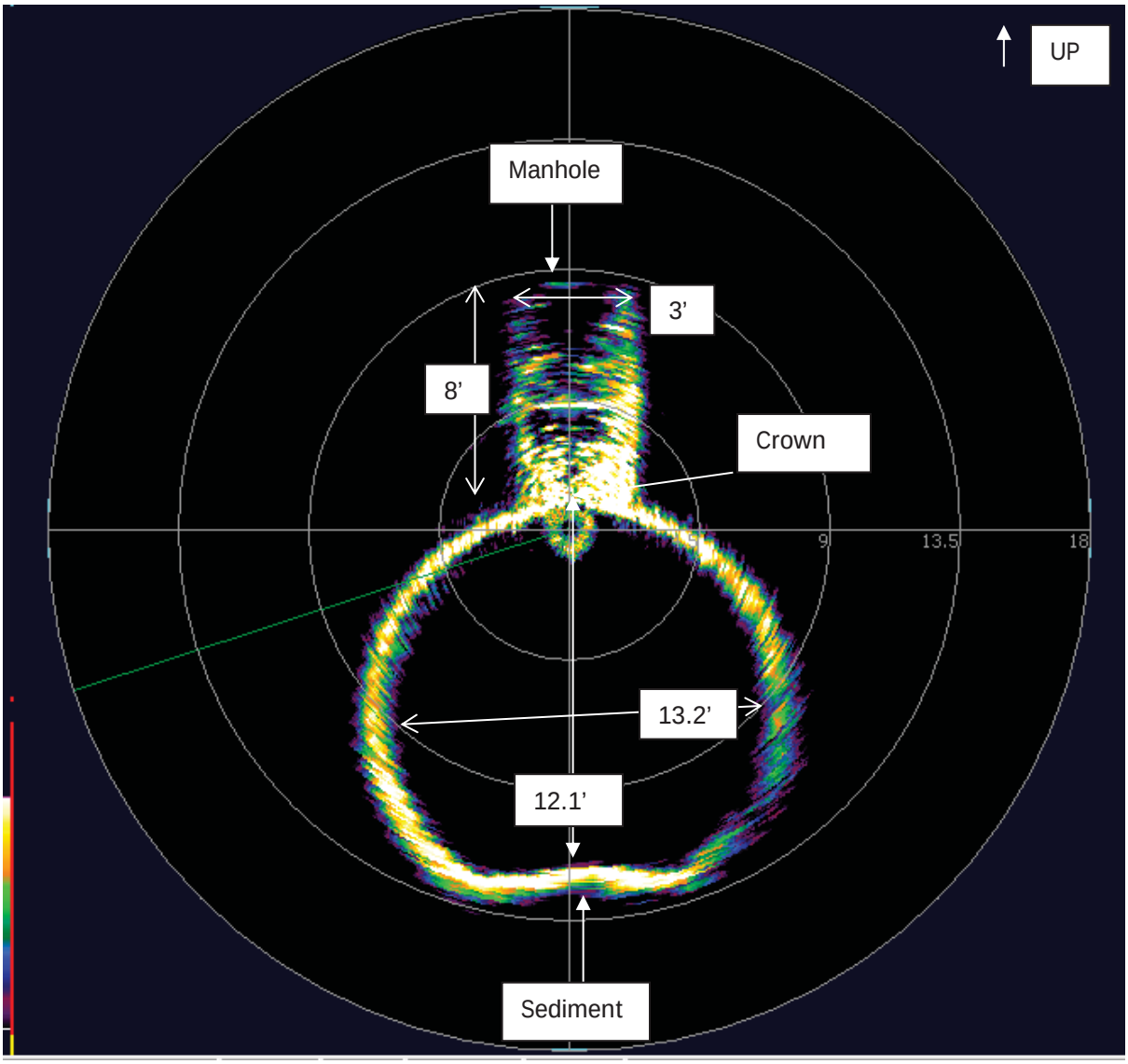

Figure 30 - Cross Section 13 - 700 feet.

Cross section at 750 ft. 13.1’ wide by 12.4’ from invert to crown.

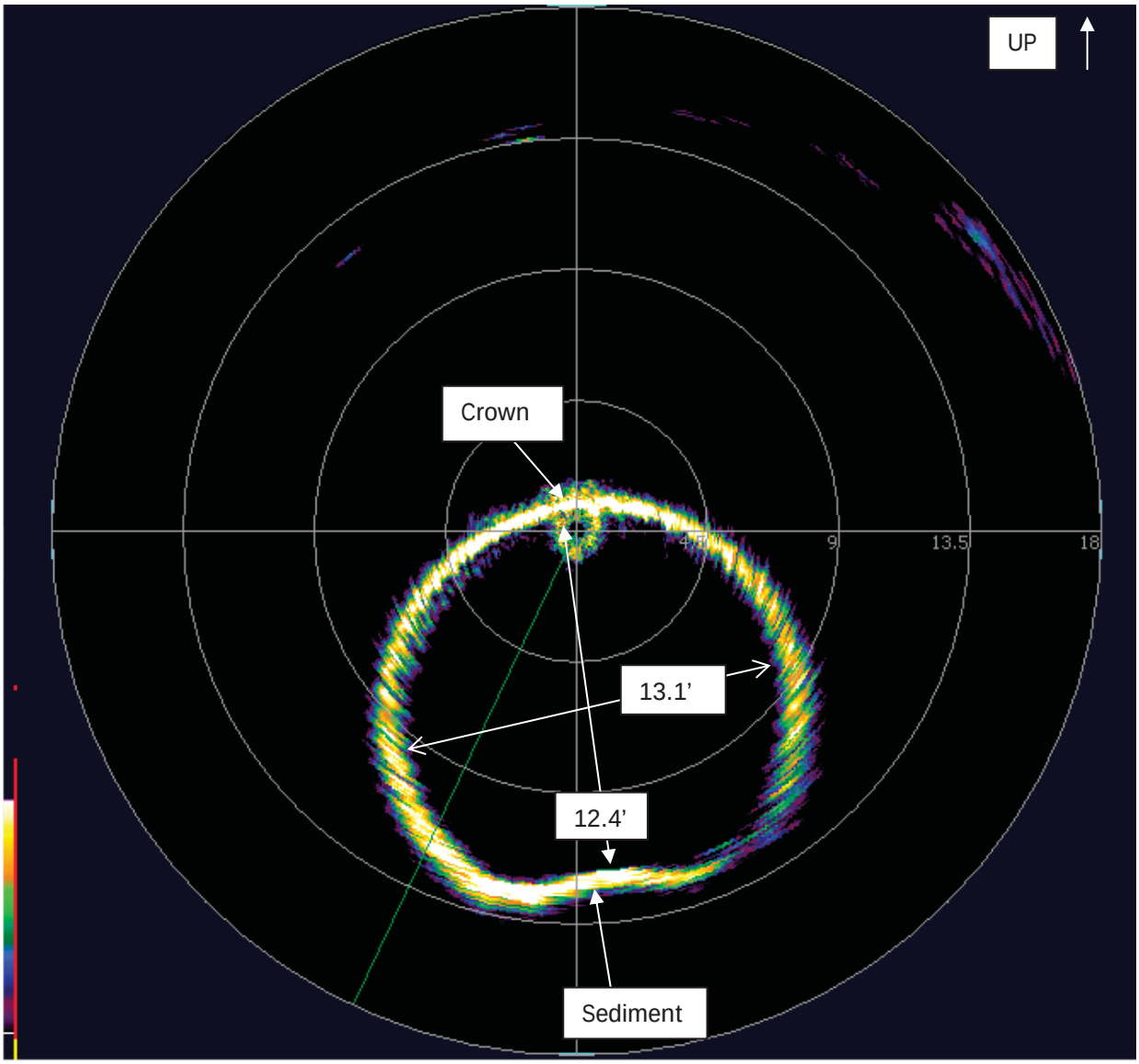

Figure 31 - Cross Section 14 - 750 feet.

Cross section at 800 ft. 13 wide by 12.2’ from invert to crown.

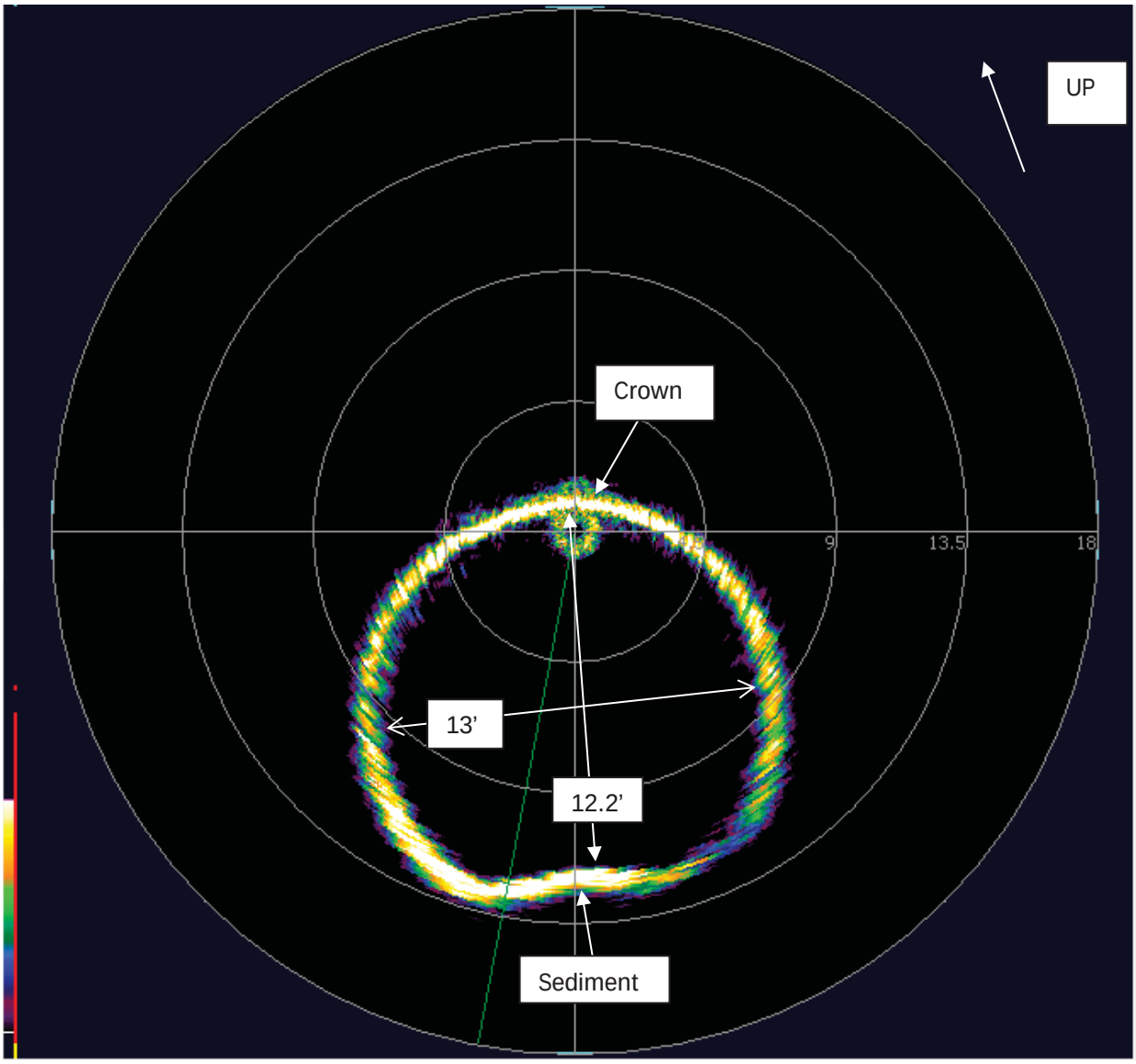

Figure 32 - Cross Section 15 - 800 feet.

Cross section at 850 ft.

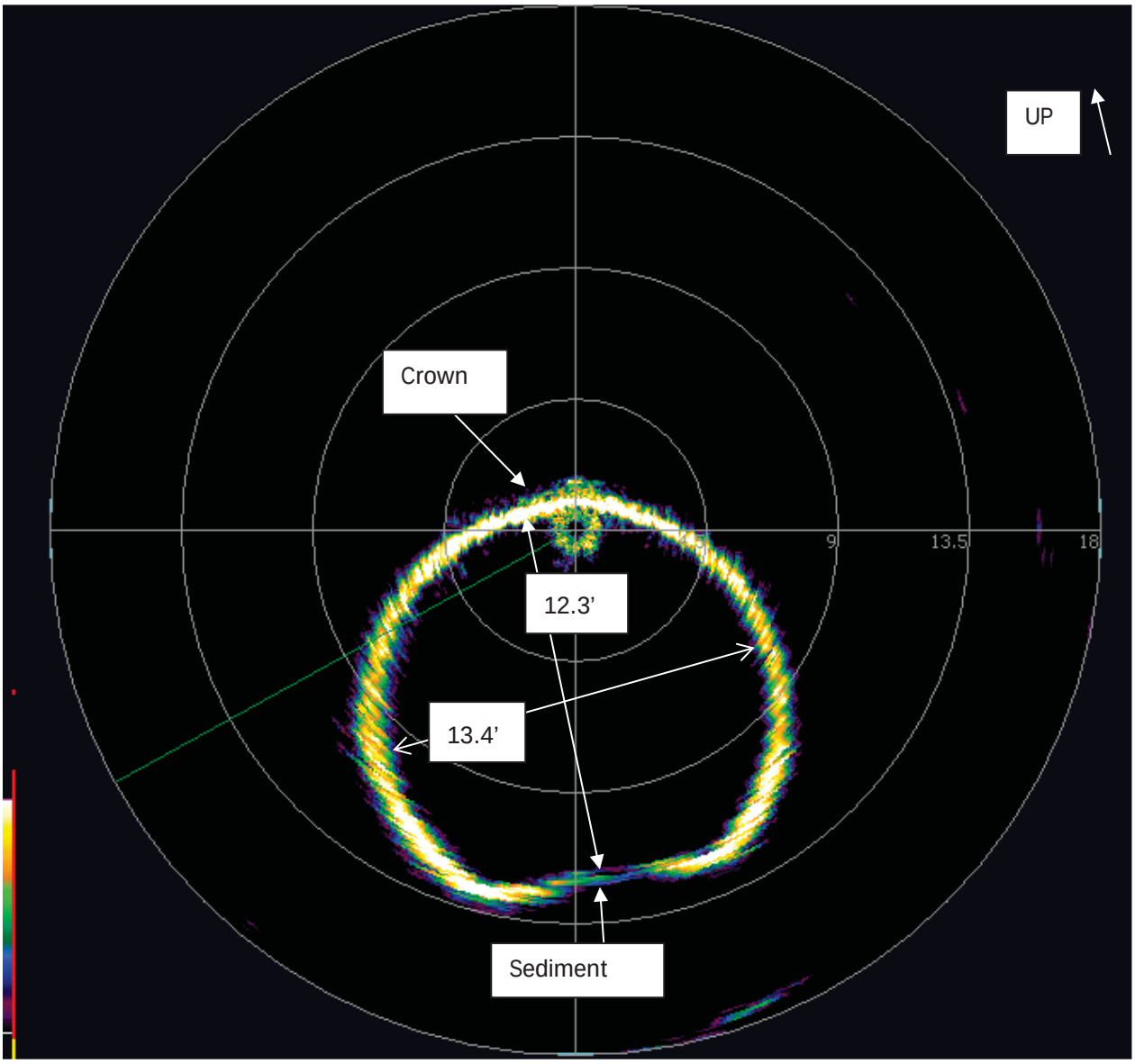

Figure 33 - Cross Section 16 - 850 feet.

Cross section at 900 ft.

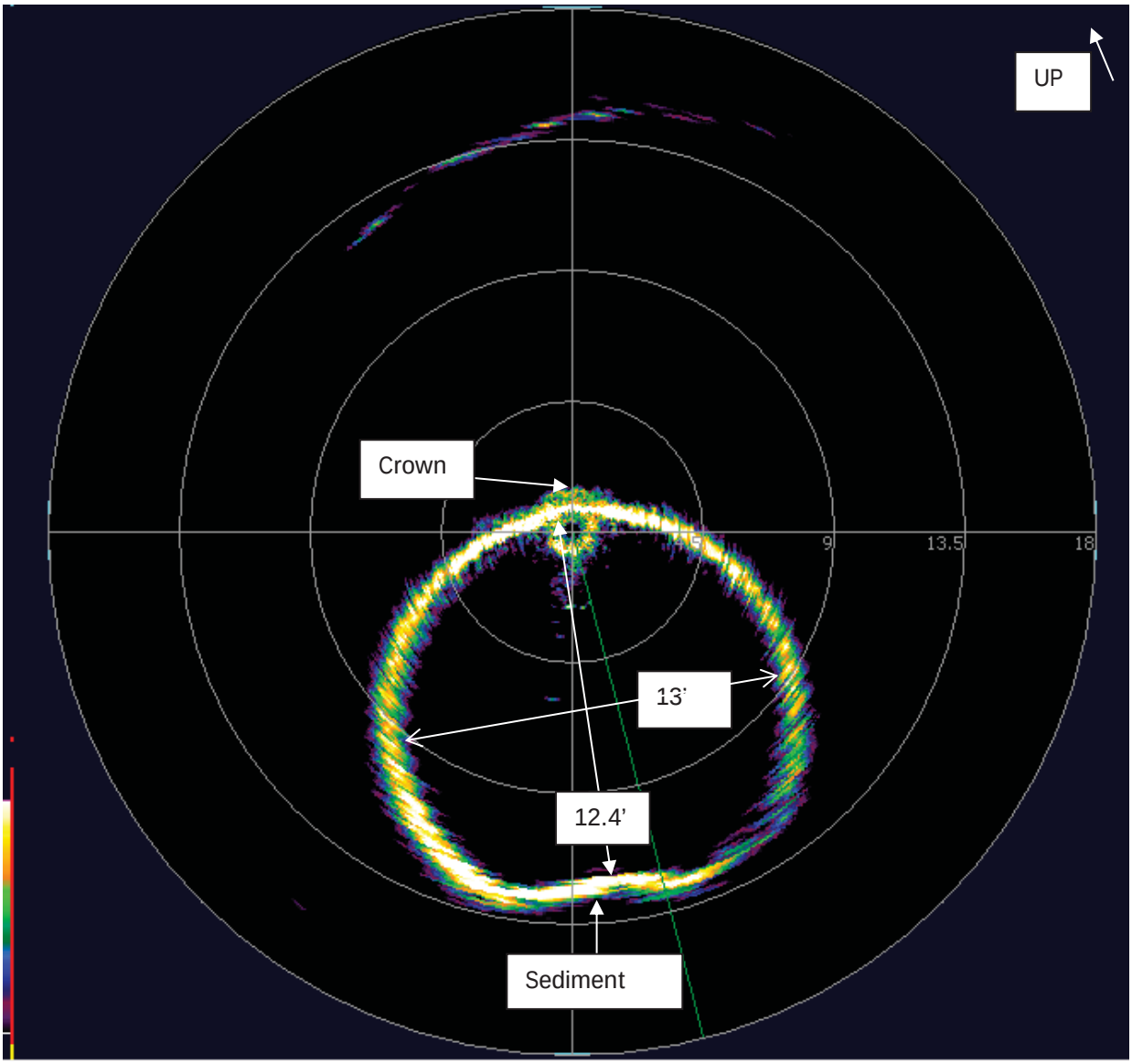

Figure 34 - Cross Section 17 - 900 feet.

Cross section at 950 ft.

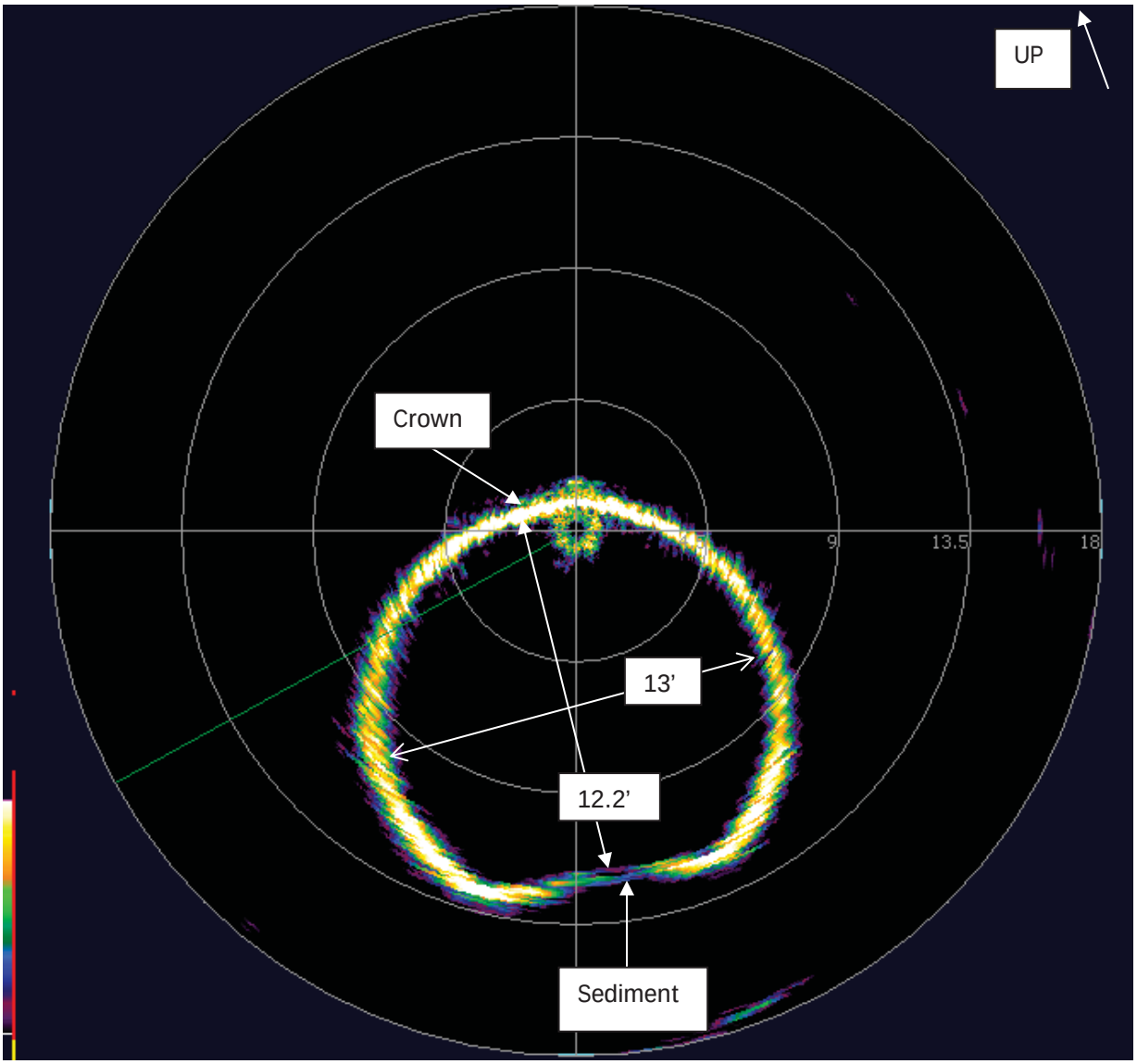

Figure 35 - Cross Section 18 – 950 feet.

Cross section at 1000 ft.

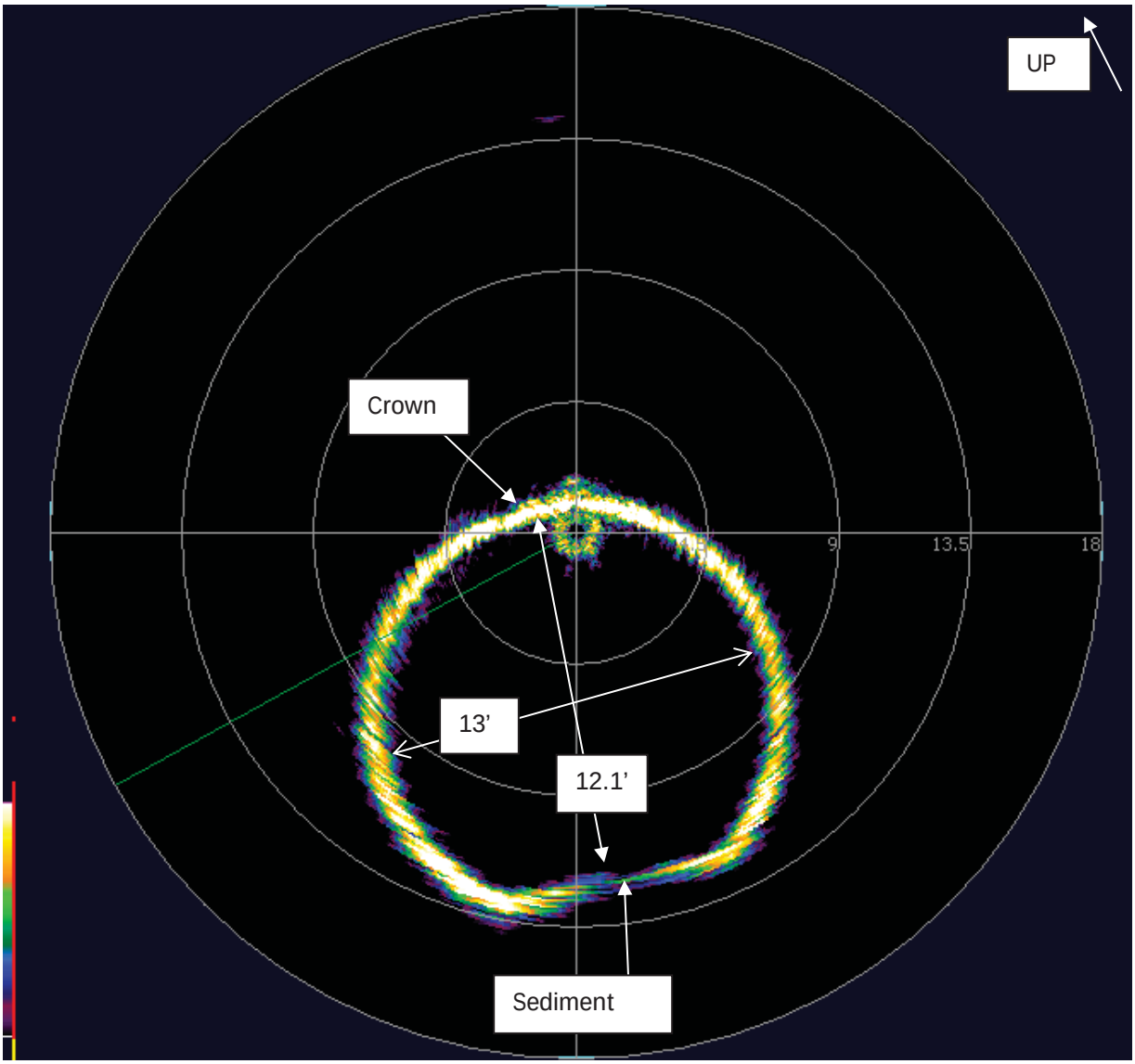

Figure 36 - Cross Section 19 - 1000 feet.

Cross section at 1050ft.

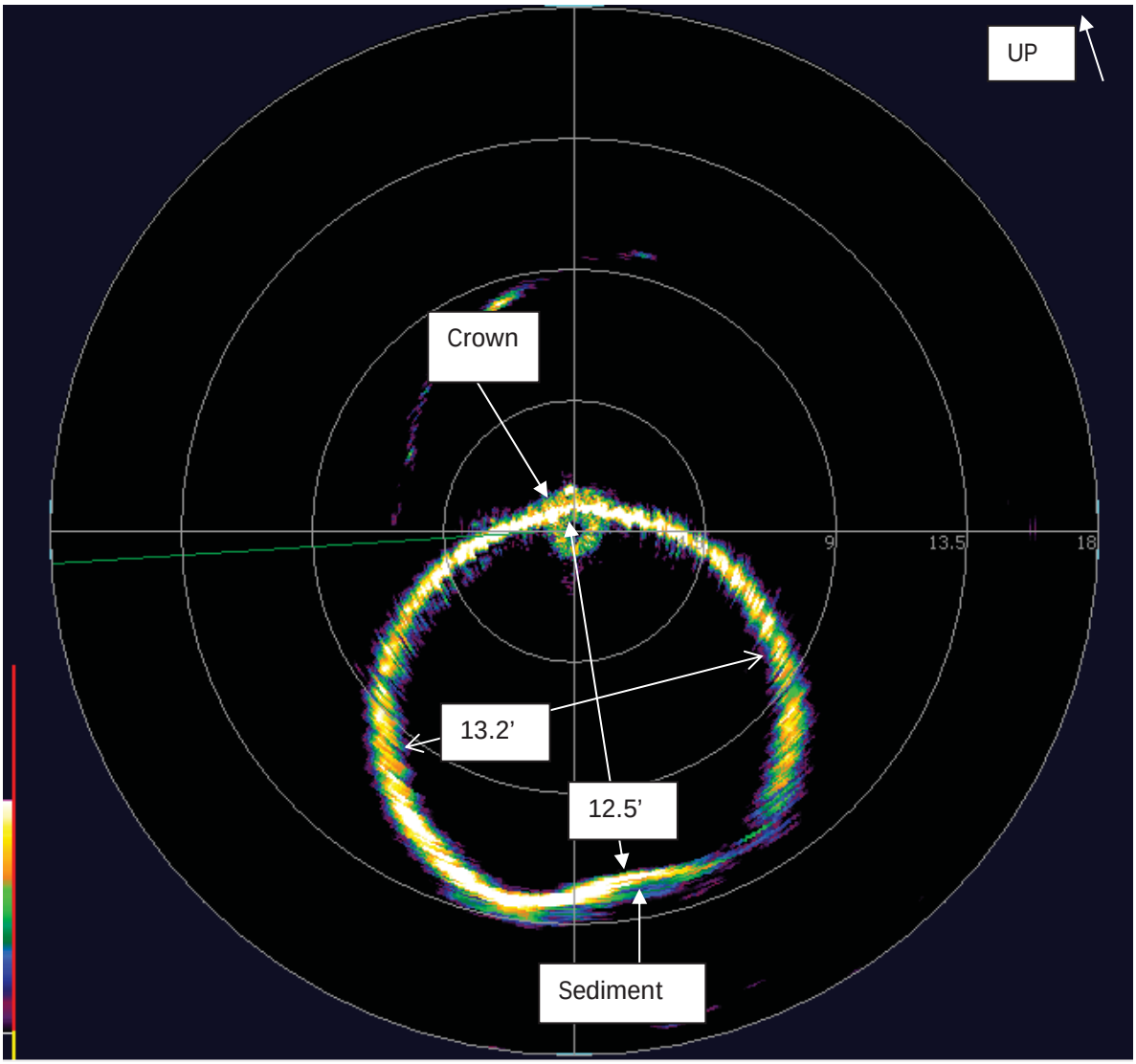

Figure 37 - Cross Section 20 - 1050 feet.

Cross section at 2300 ft.

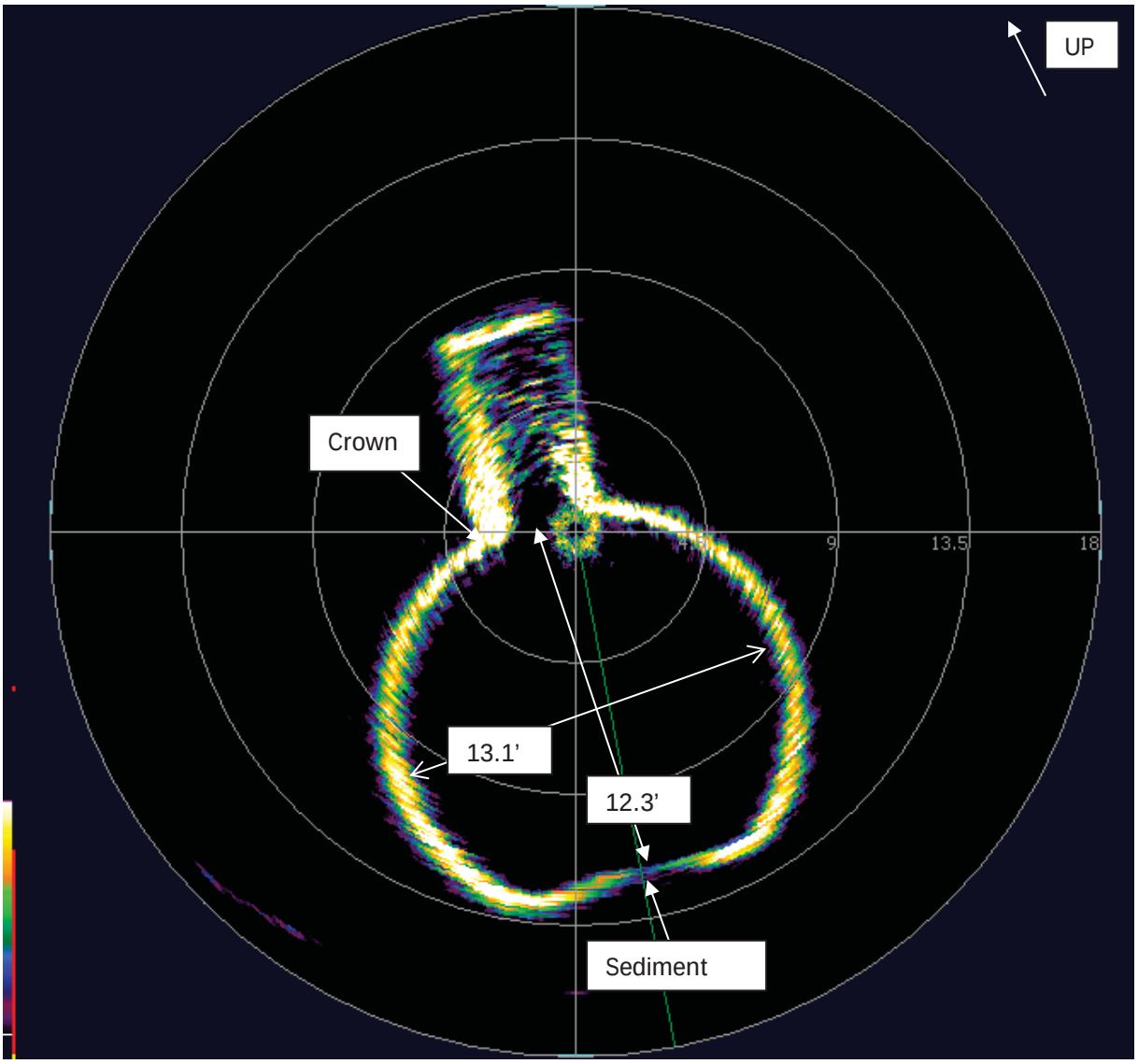

Figure 38 - Cross Section 21 - 1100 feet.

Cross section at 1150 ft.

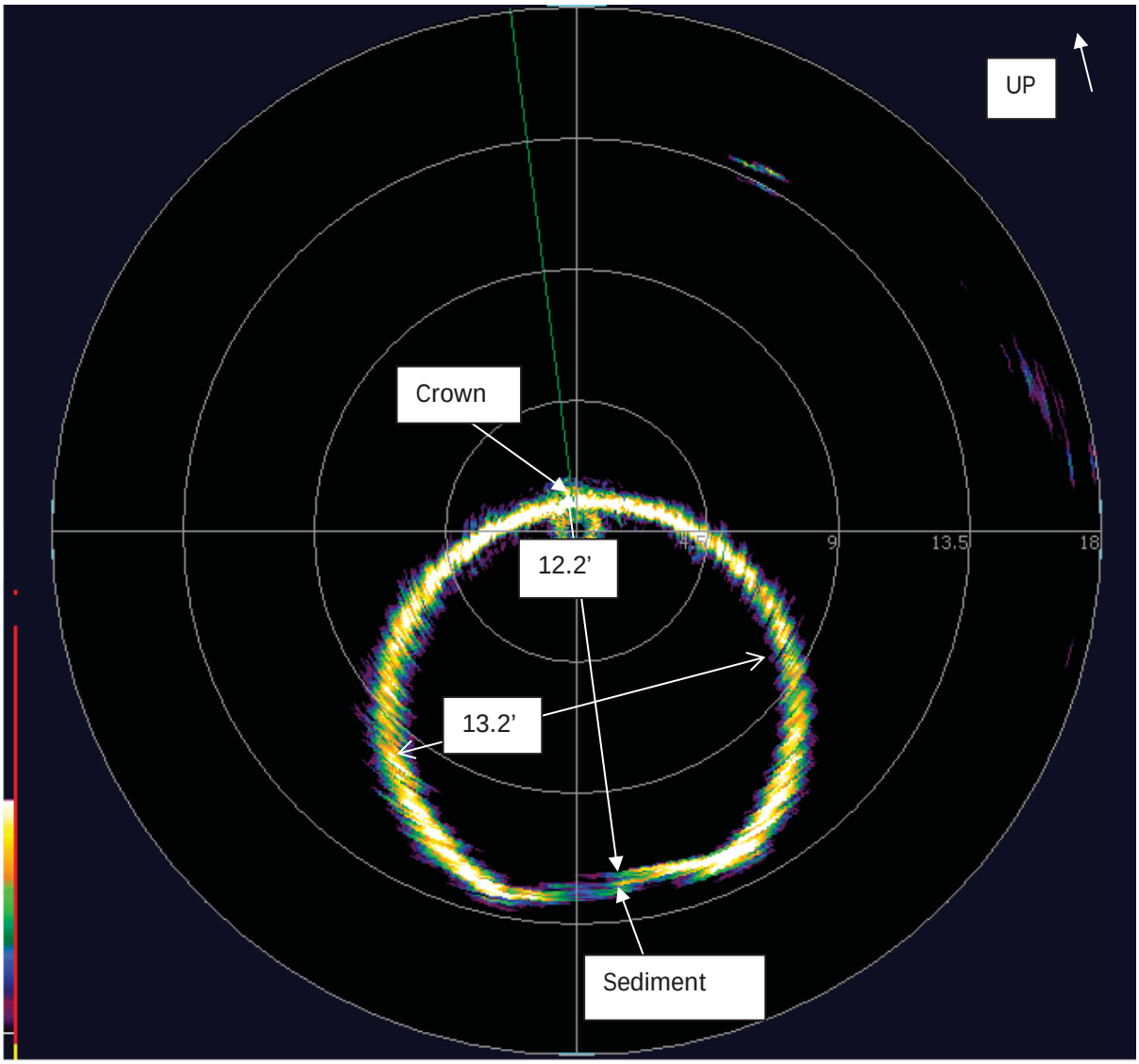

Figure 39 - Cross Section 22 - 1150 feet.

Cross section at 1200 ft.

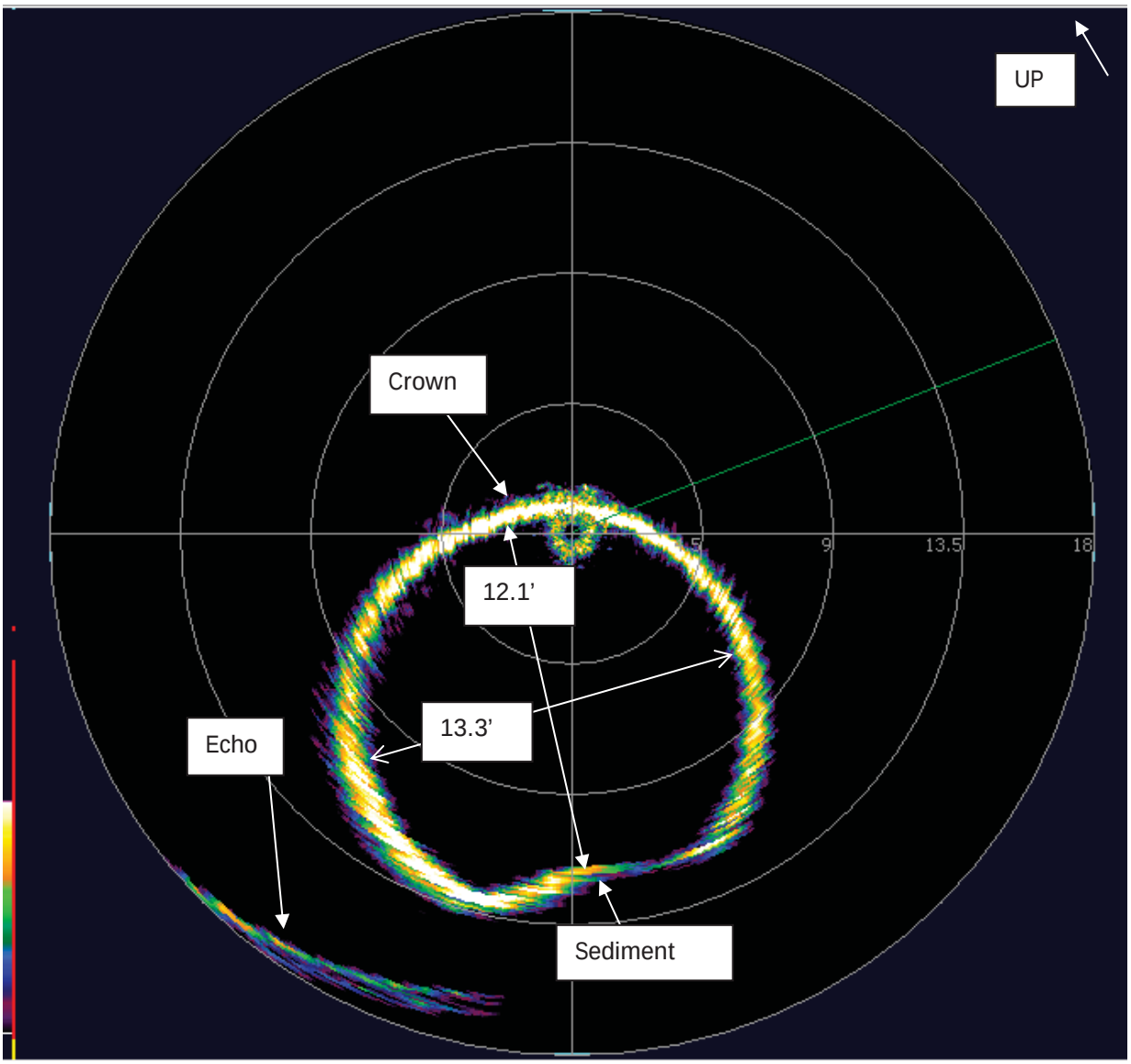

Figure 40 - Cross Section 23 -1200 feet.

Cross section at 1250 ft.

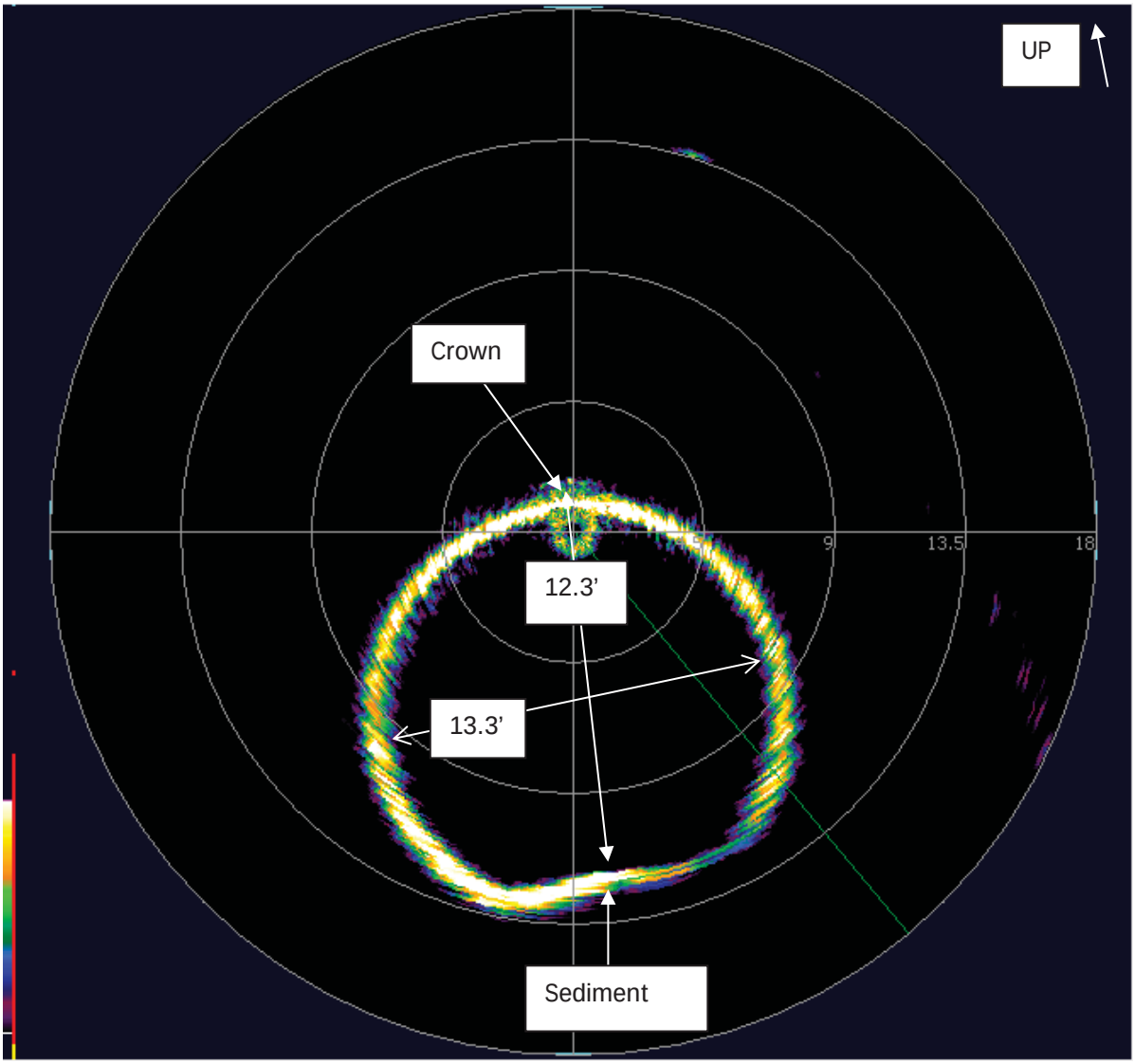

Figure 41 - Cross Section 24 - 1250 feet.

Cross section at 1300 ft.

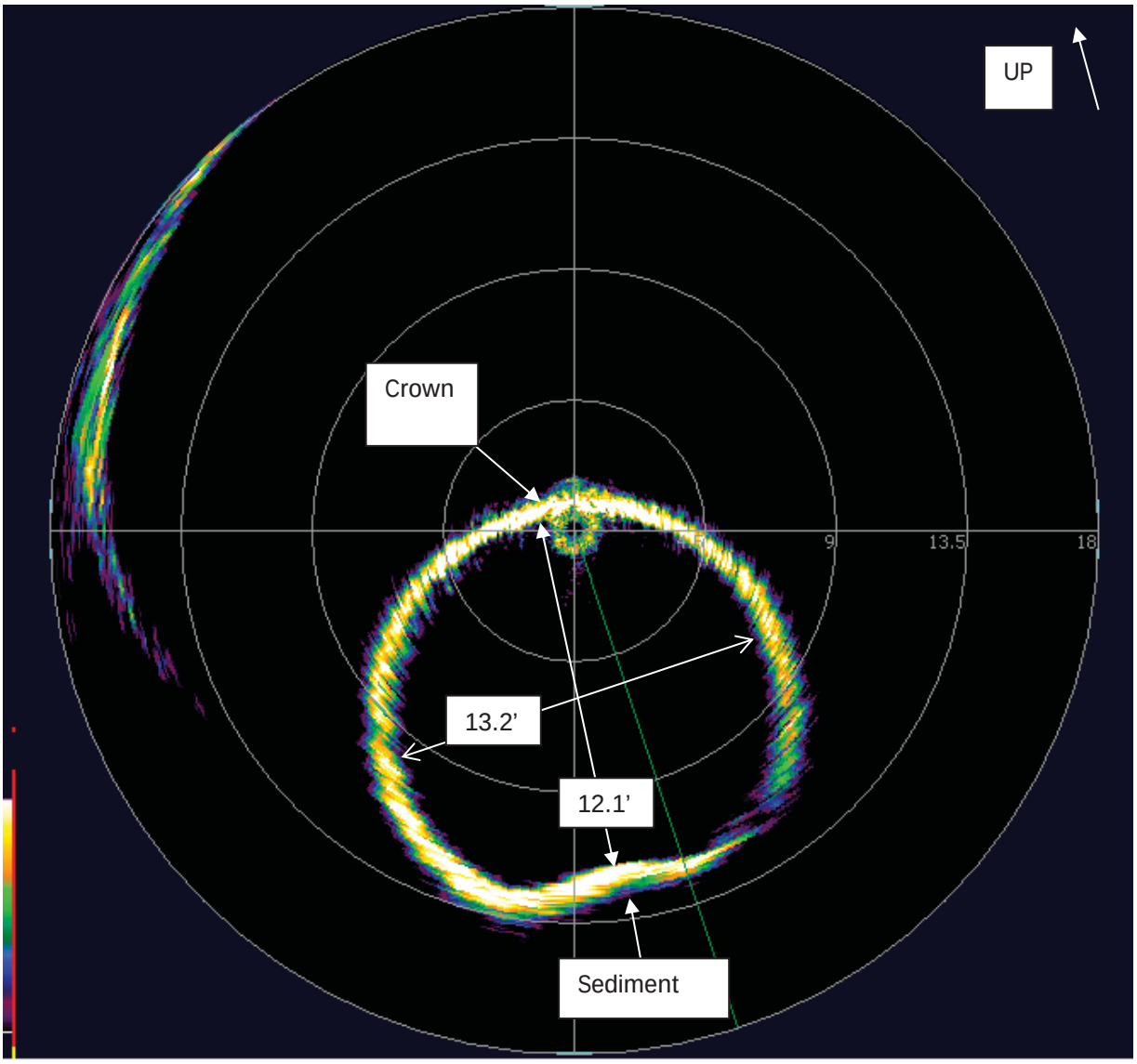

Figure 42 - Cross Section 25 - 1300 feet.

Cross section at 1350ft.

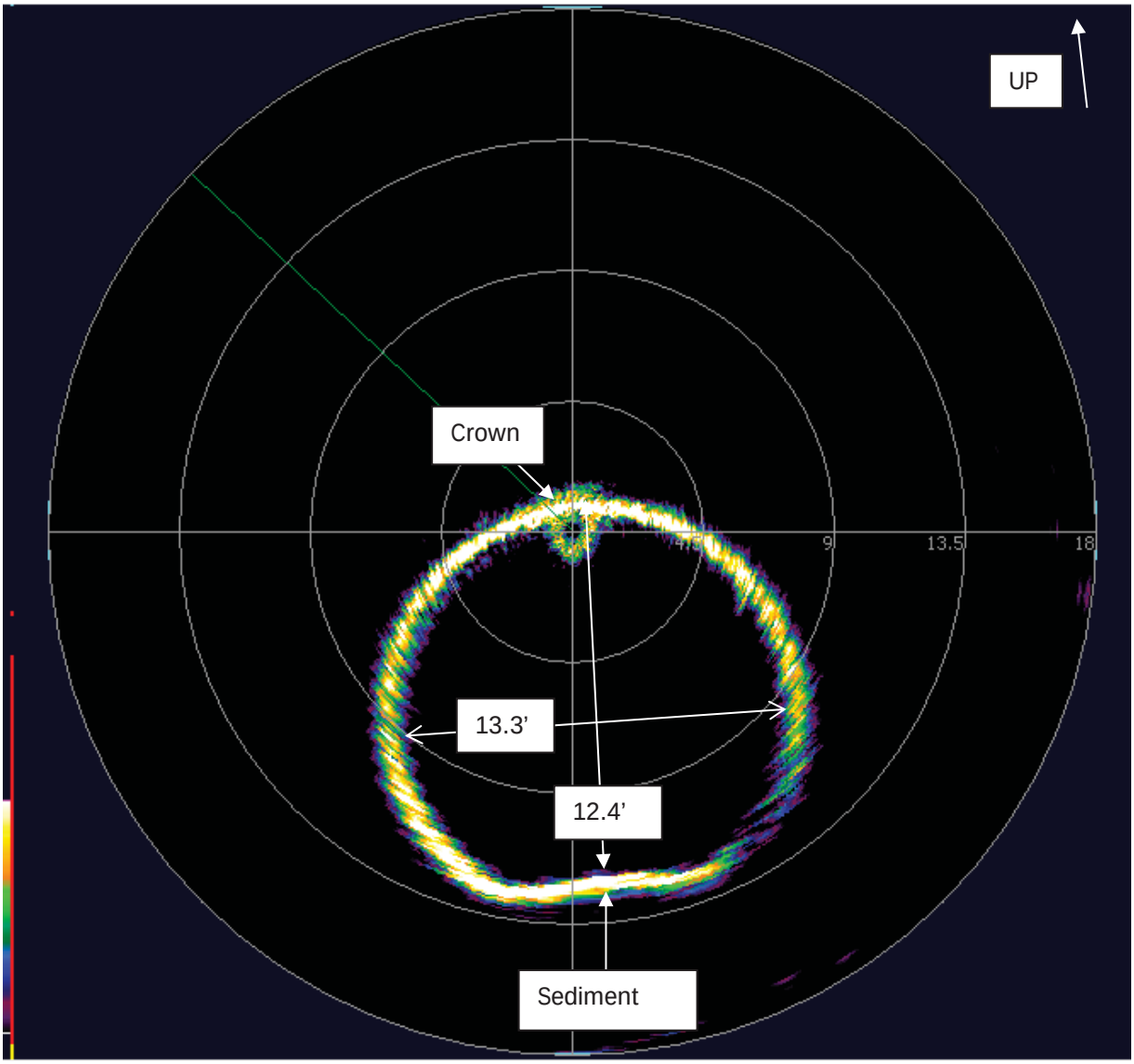

Figure 43 - Cross Section 26 - 1350 feet

Cross section at 1400 ft.

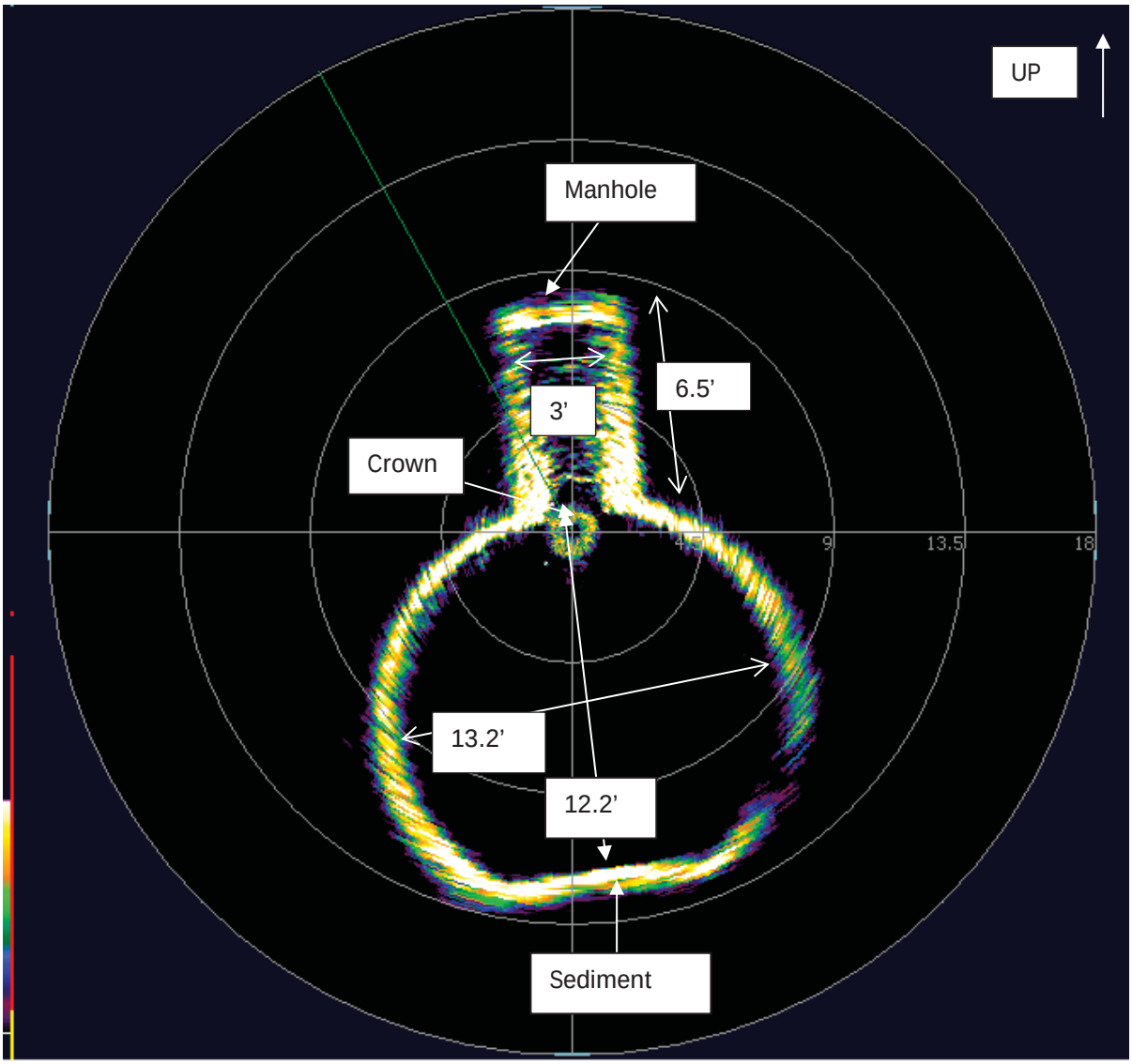

Figure 44 - Cross Section 27 - 1400 feet.

Cross section at 1450 ft. 13’ wide by 12.2’ from invert to crown.

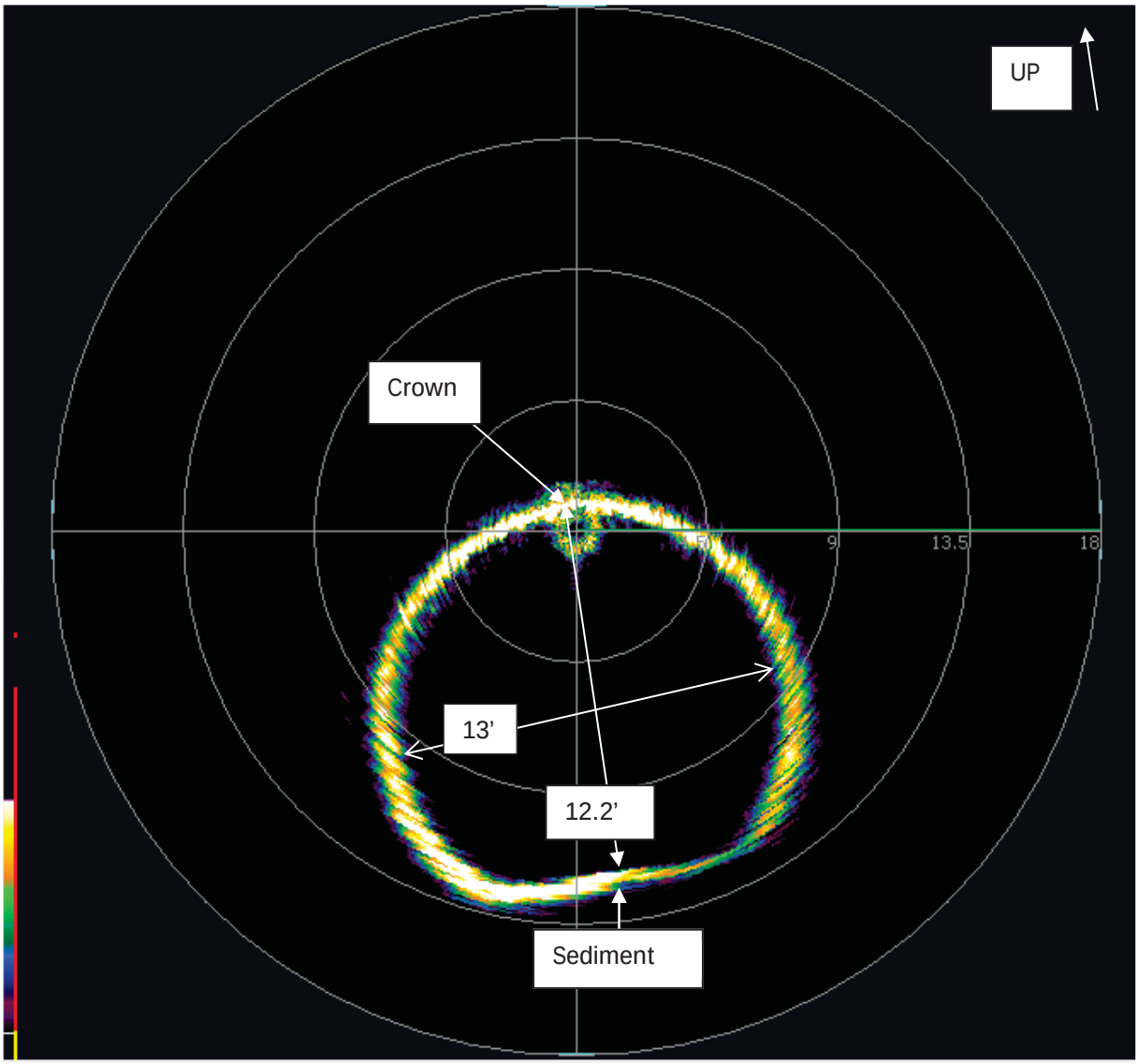

Figure 45 - Cross Section 29 - 1450 feet.

Cross section at 1500 ft.

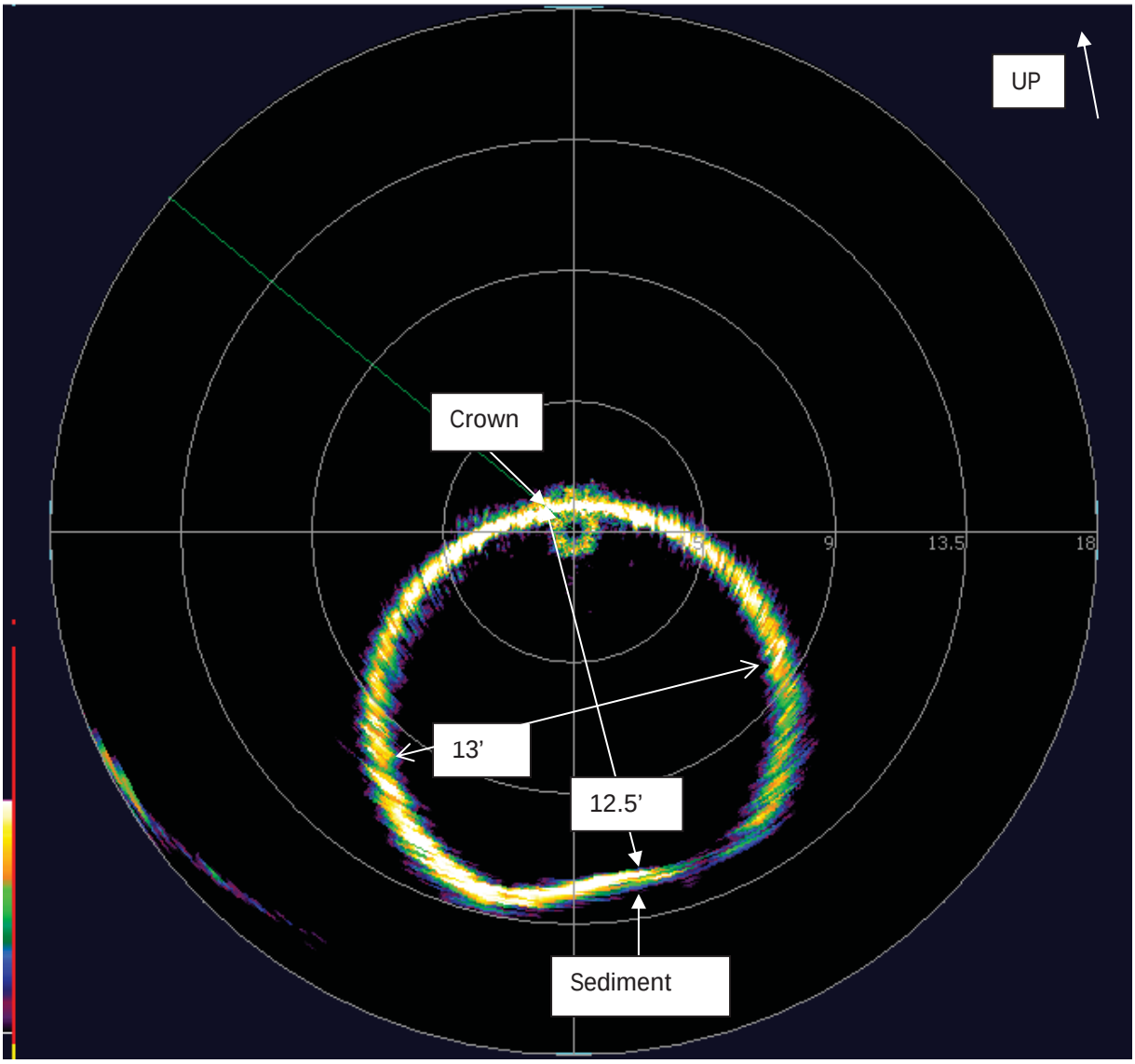

Figure 46 - Cross Section 30 - 1500 feet.

Cross section at 1550 ft.

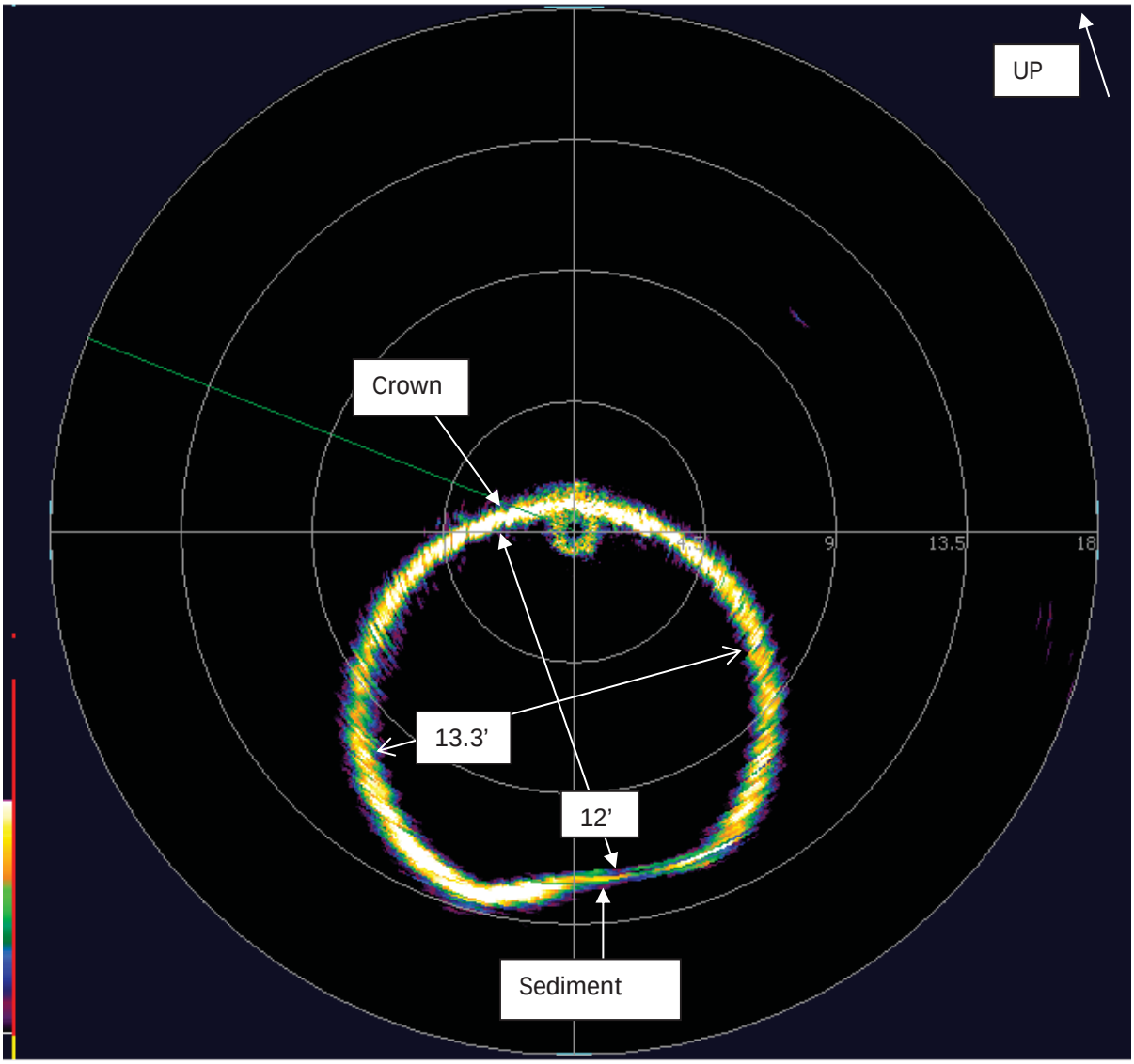

Figure 47 - Cross Section 31 - 1550 feet.

Cross section at 1600 ft.

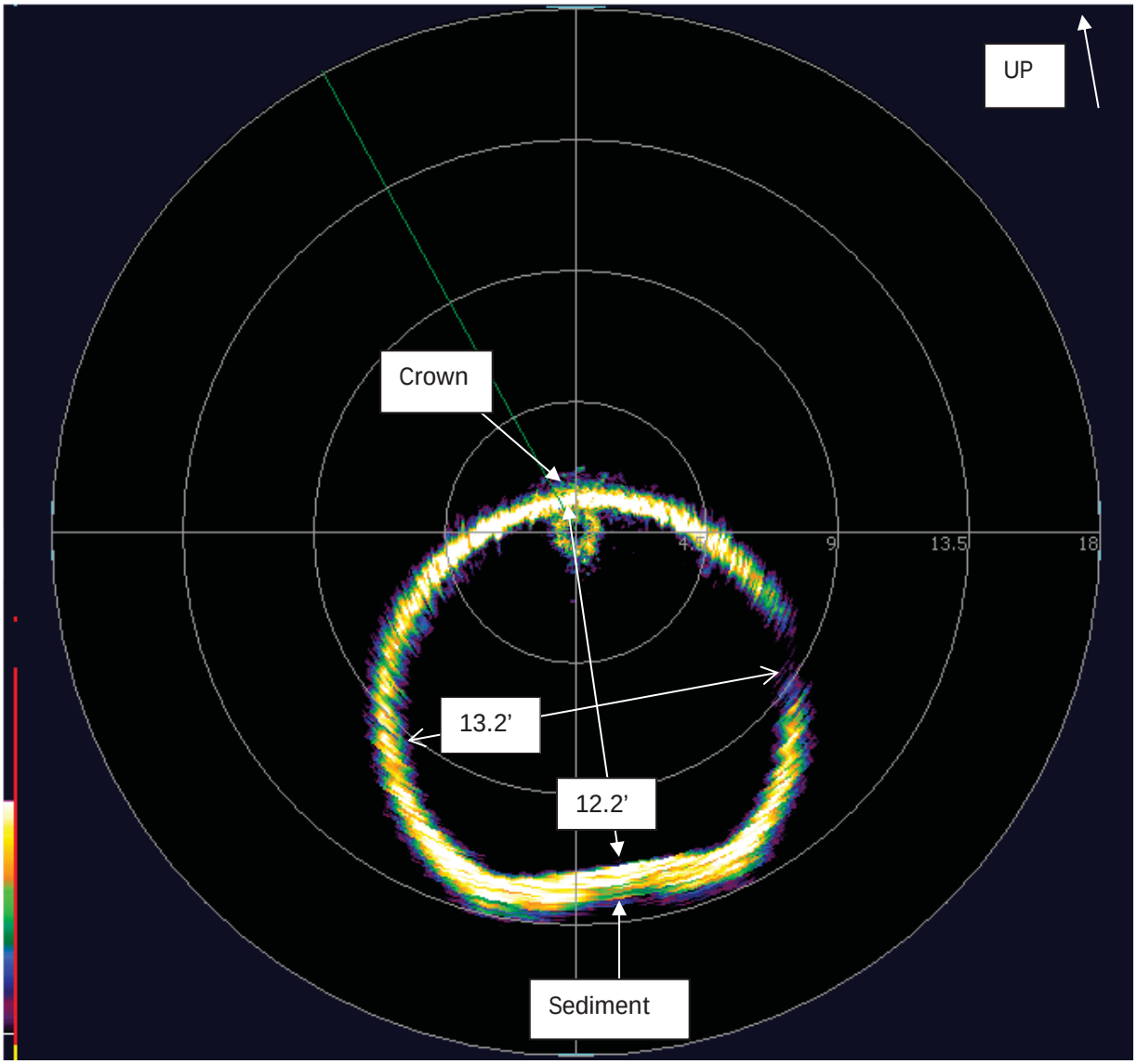

Figure 48 - Cross Section 32 - 1600 feet.

Cross section at 1650 ft.

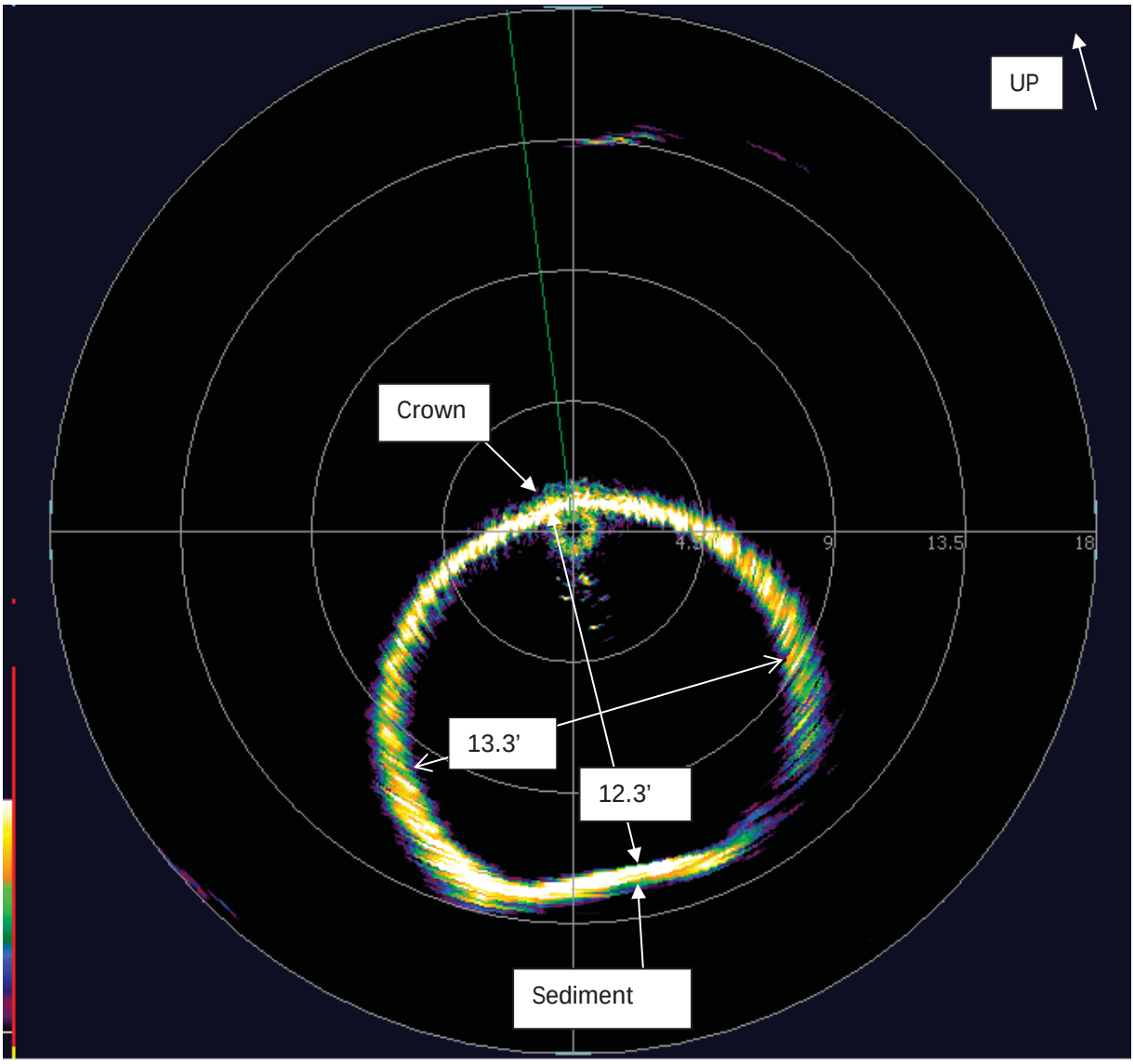

Figure 49 - Cross Section 33 - 1650 feet.

Cross section at 1700 ft. 13.5’ wide by 12.1’ from invert to crown.

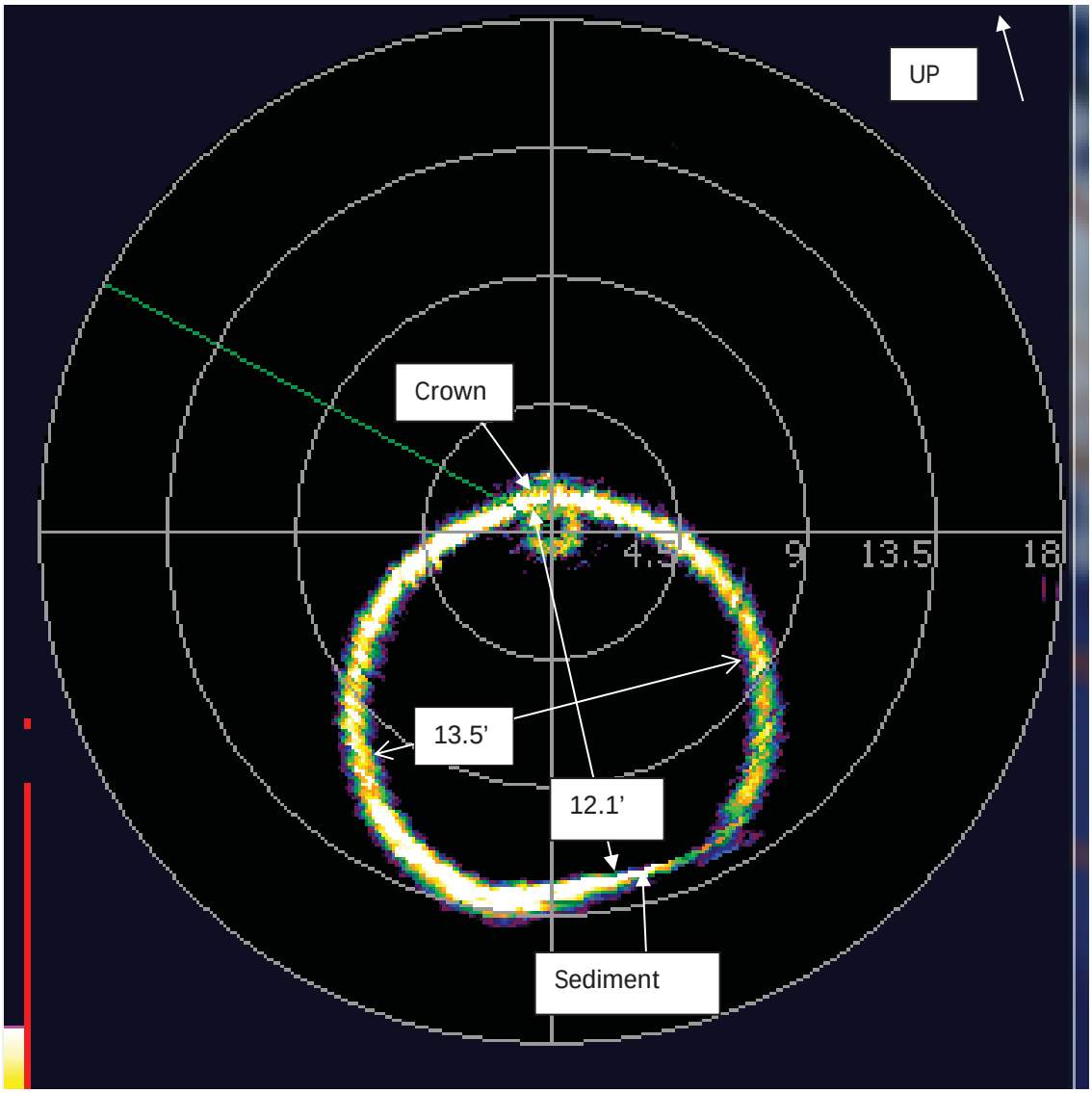

Figure 50 - Cross Section 34 - 1700 feet.

Cross section at 1750ft

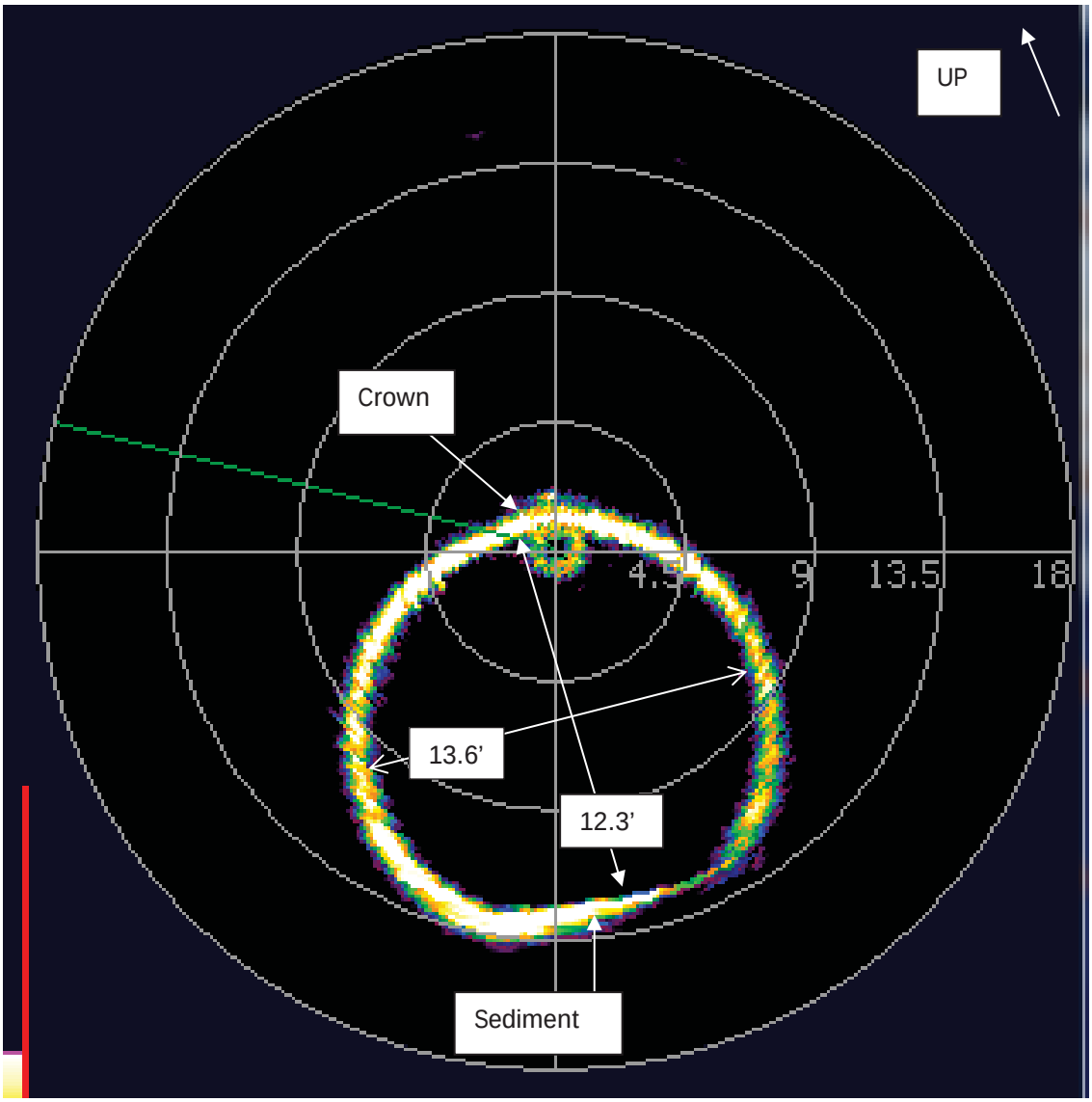

Figure 51 - Cross Section 35 - 1750 feet.

Cross section at 1800 ft.

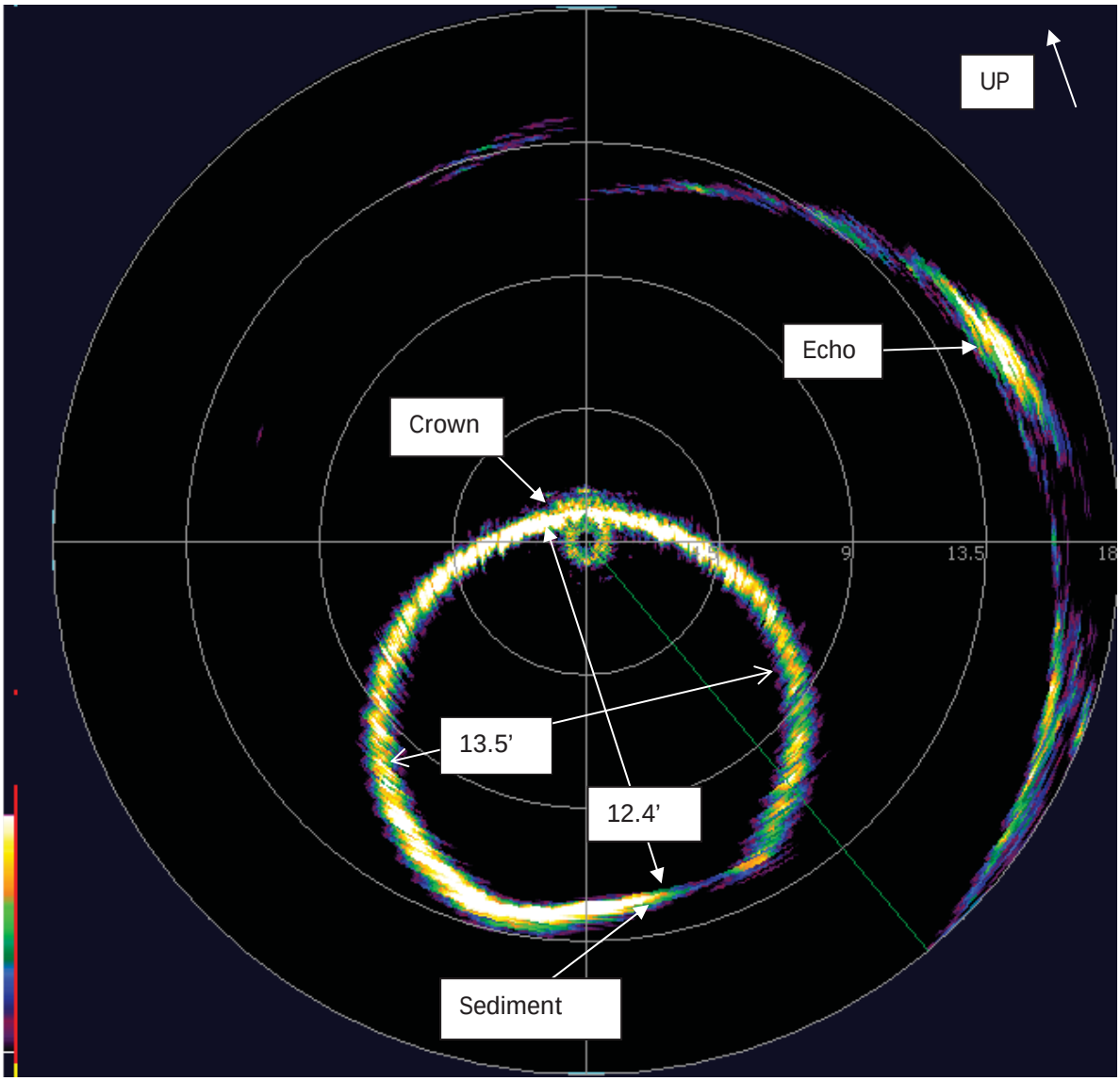

Figure 52 - Cross Section 36 - 1800 feet.

Cross section at 1850ft.

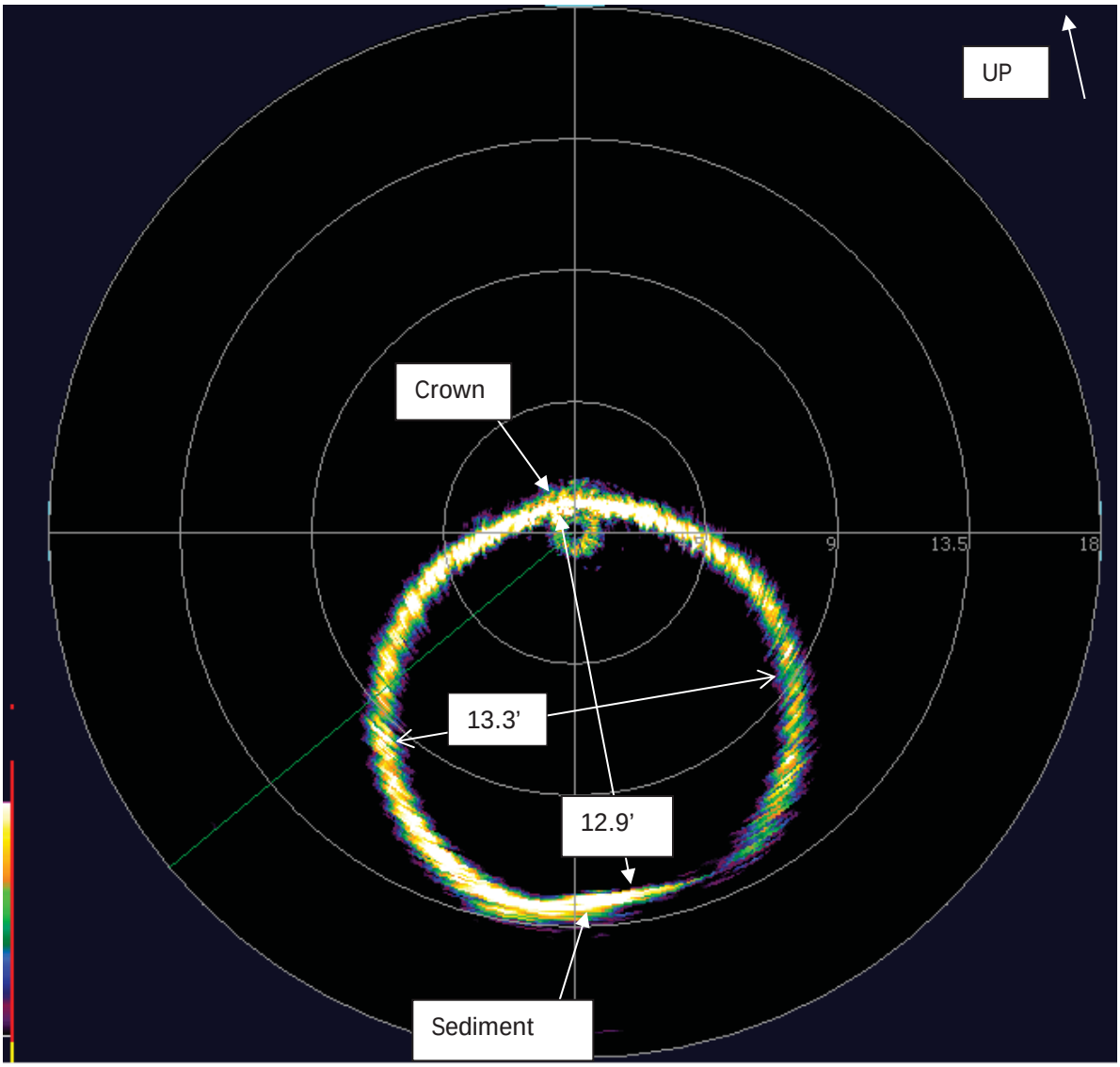

Figure 53 - Cross Section 37 - 1850 feet.

Cross section at 1900 ft.

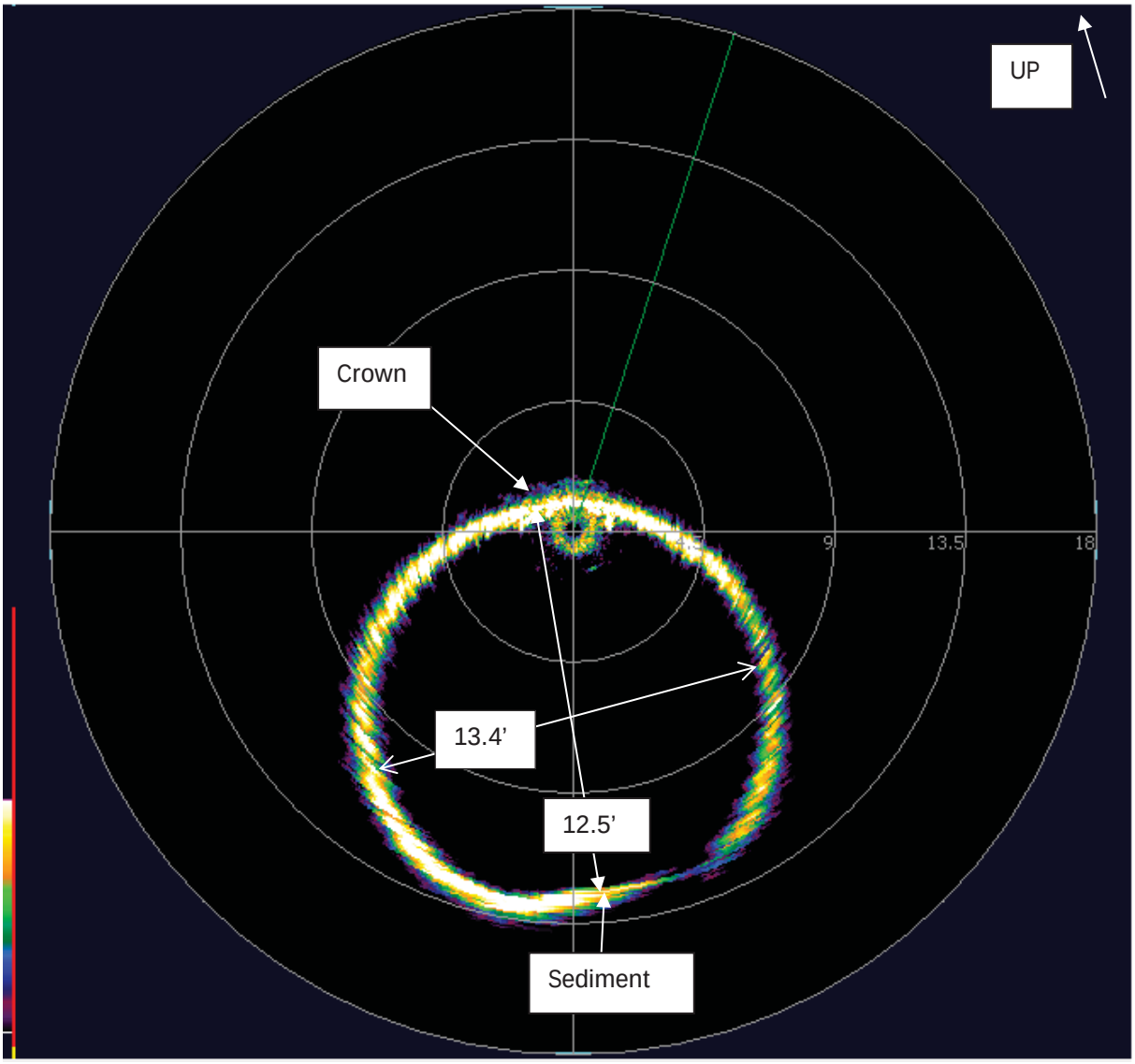

Figure 54 - Cross Section 38 - 1900 feet.

Cross section at 1983ft

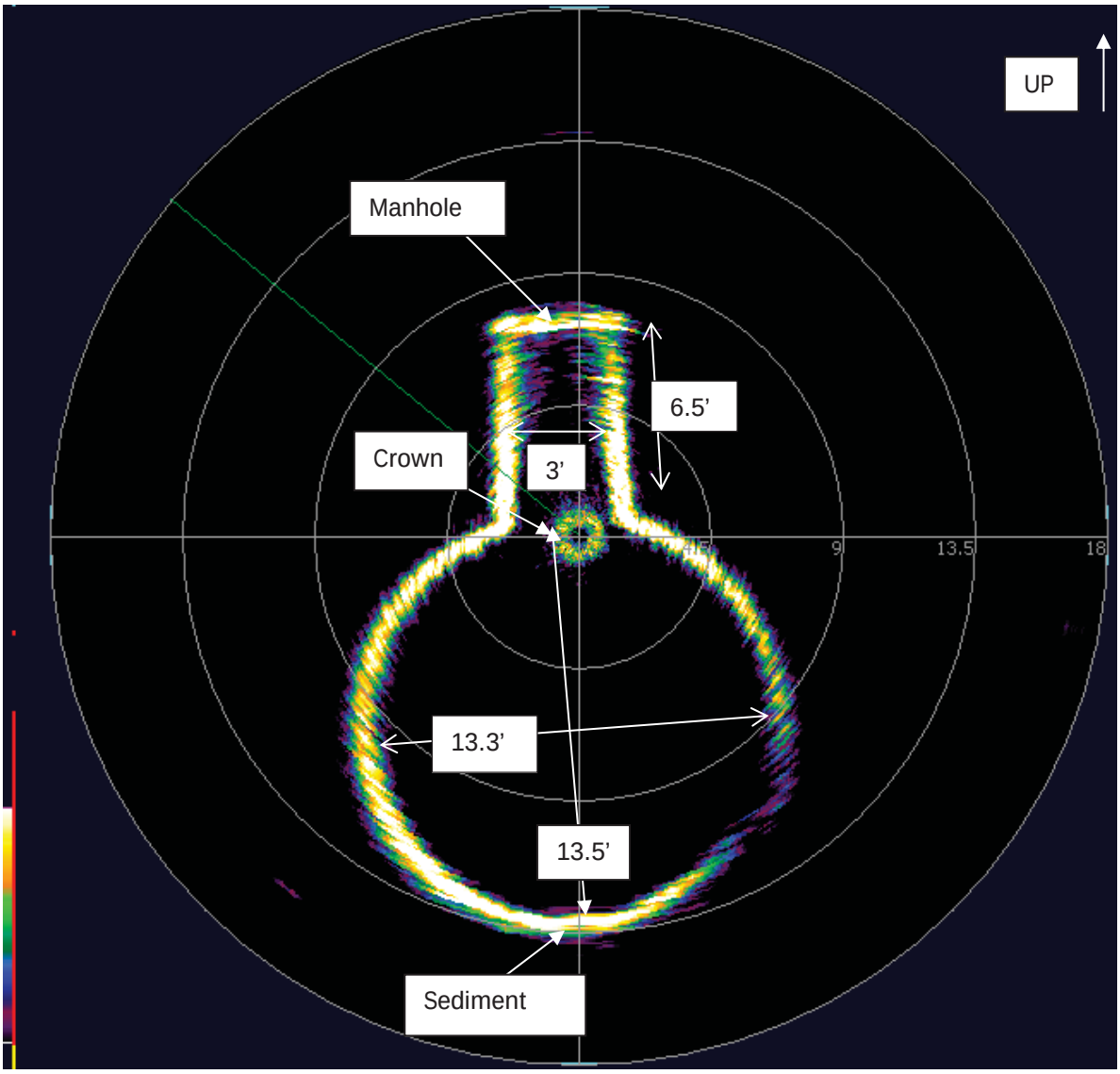

Figure 55 - Cross Section 39 - 2000 feet.

Cross section at 2050 ft.

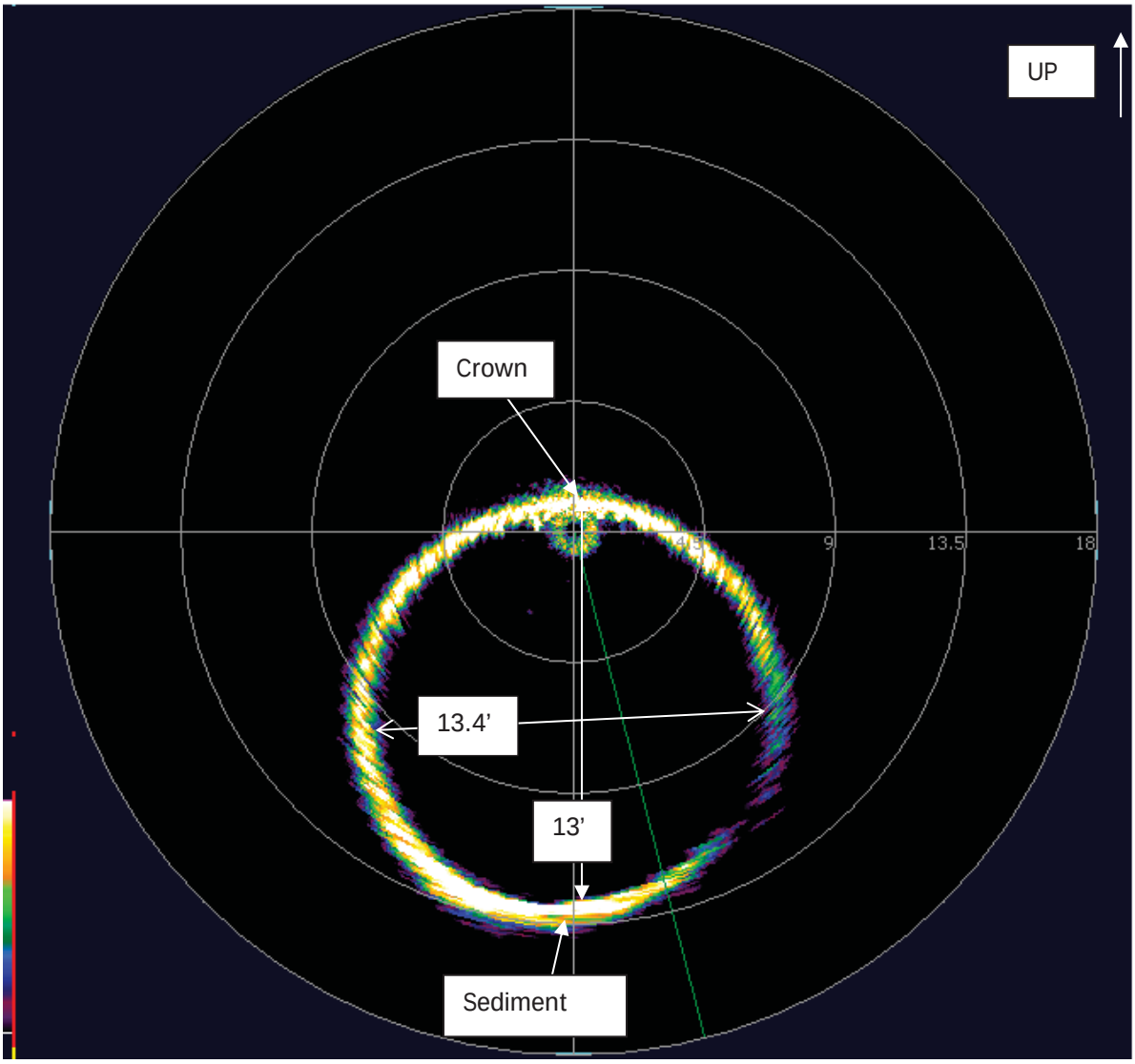

Figure 56 - Cross Section 39 - 2050 feet.

Cross section at 2100 ft.

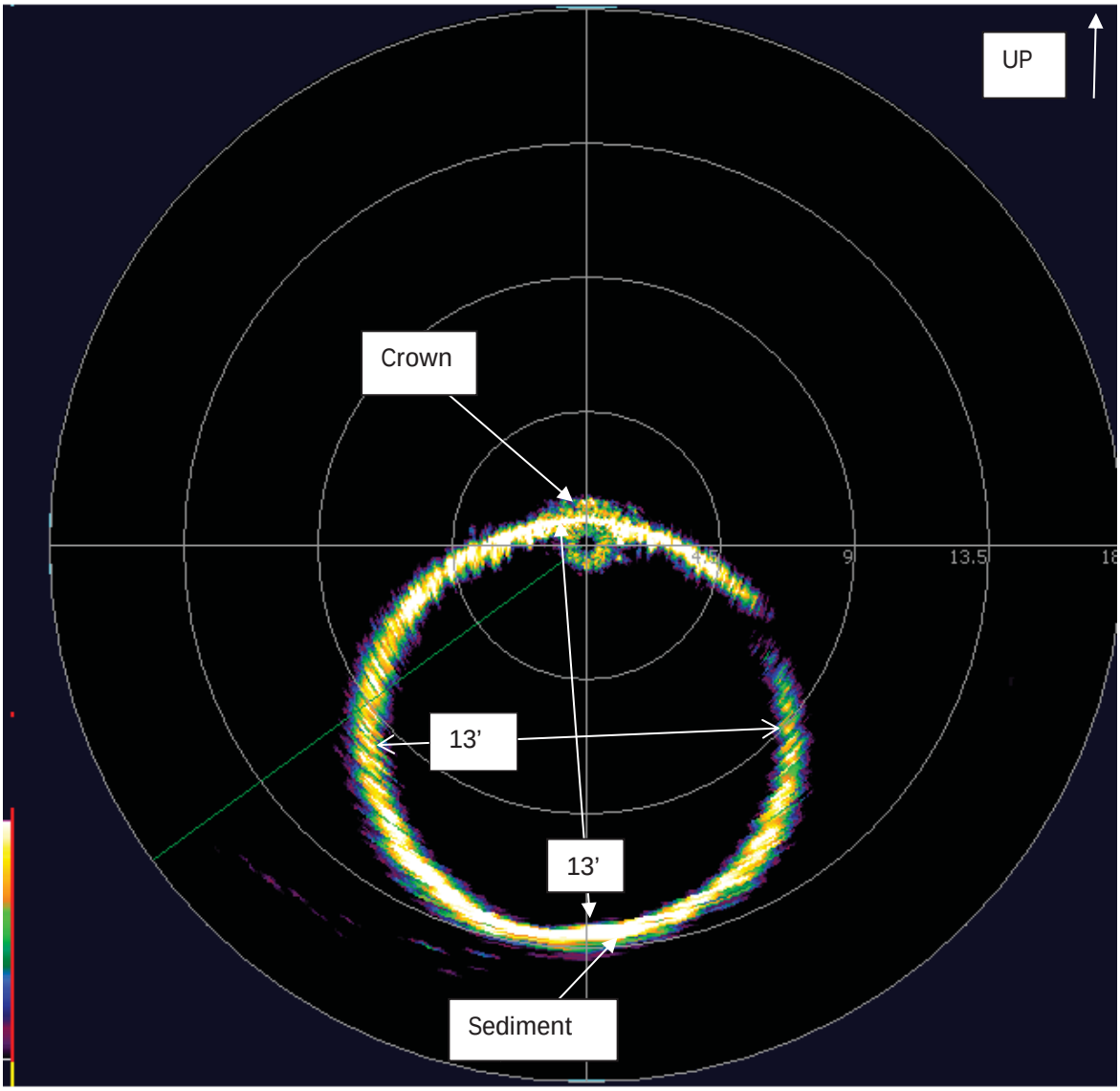

Figure 57 - Cross Section 41 - 2100 feet.

Cross section at 2150 ft.

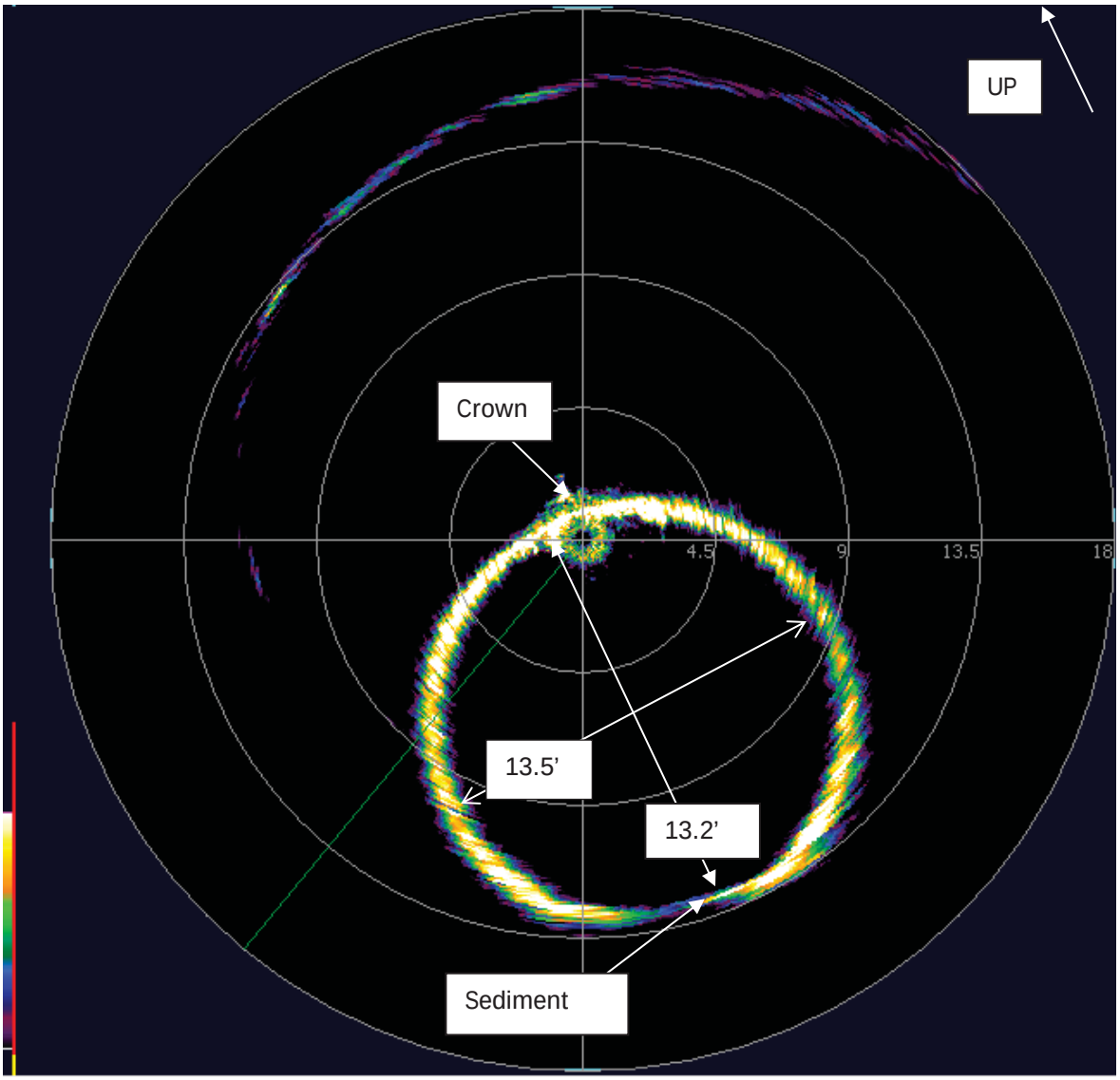

Figure 58 - Cross Section 42 - 2150 feet.

Cross section at 2200 ft.

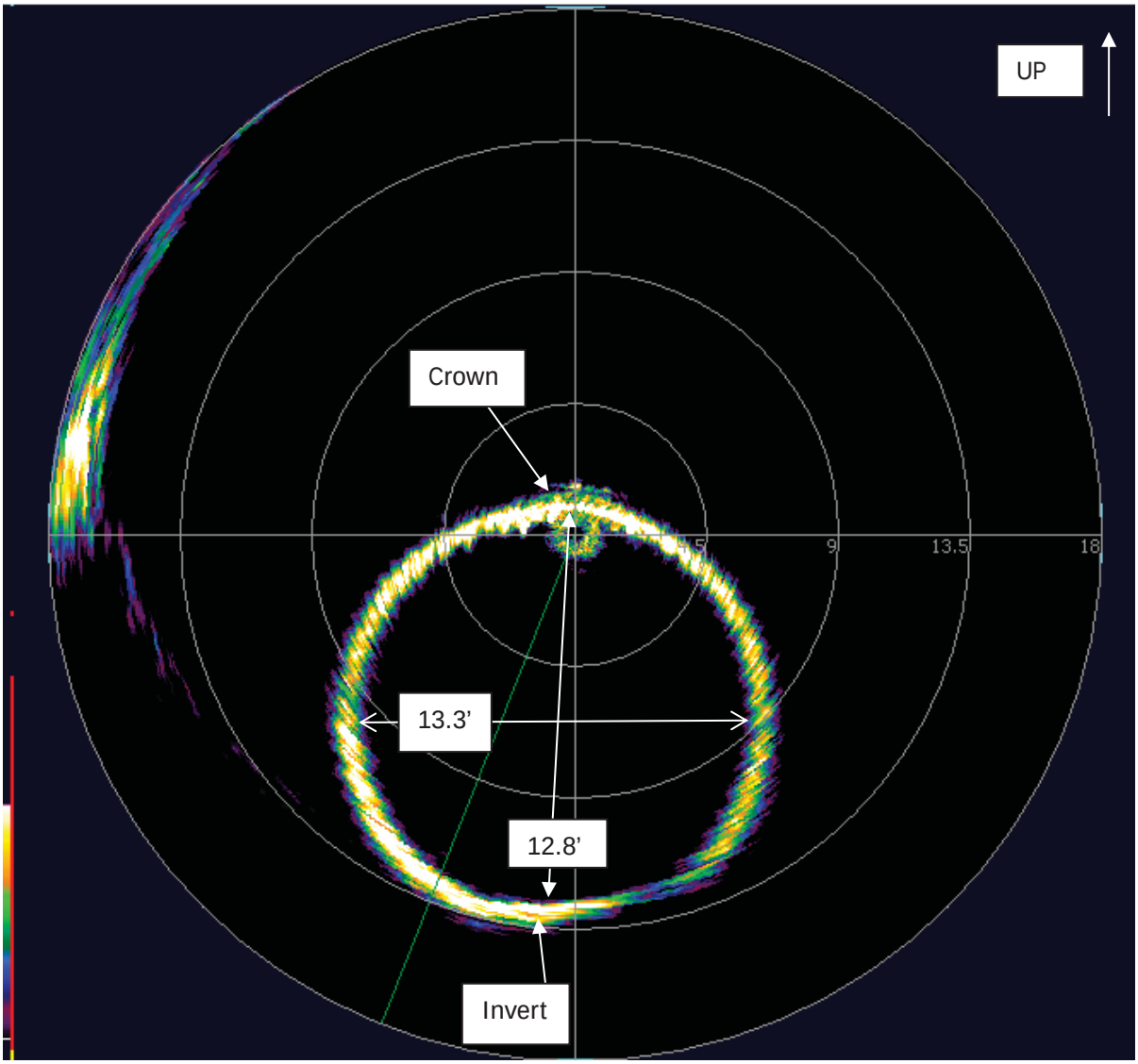

Figure 59 - Cross Section 43 - 2200 feet.

Cross section at 2250 ft.

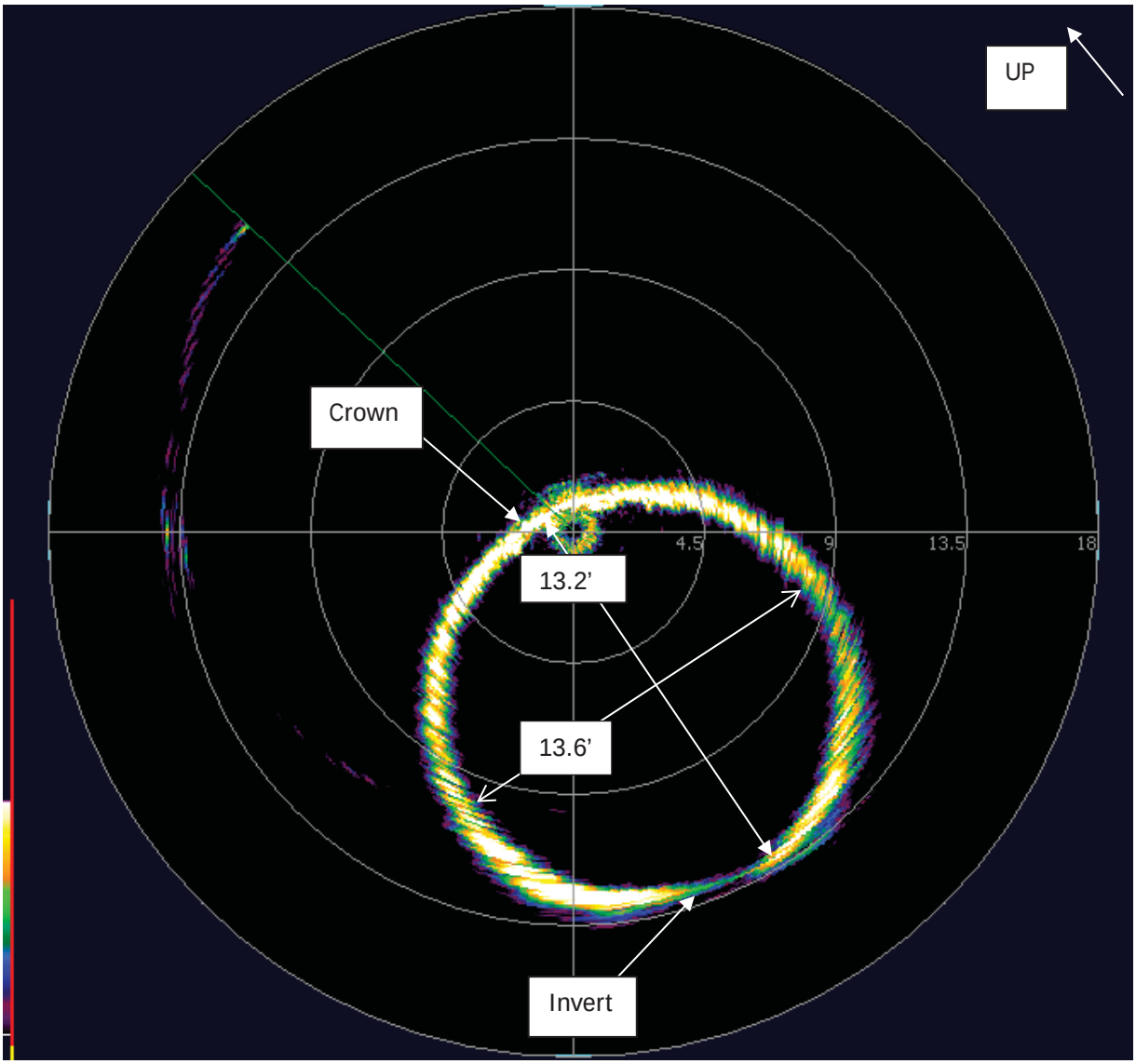

Figure 60 - Cross Section 44 - 2250 feet.

Cross section at 3900 ft.

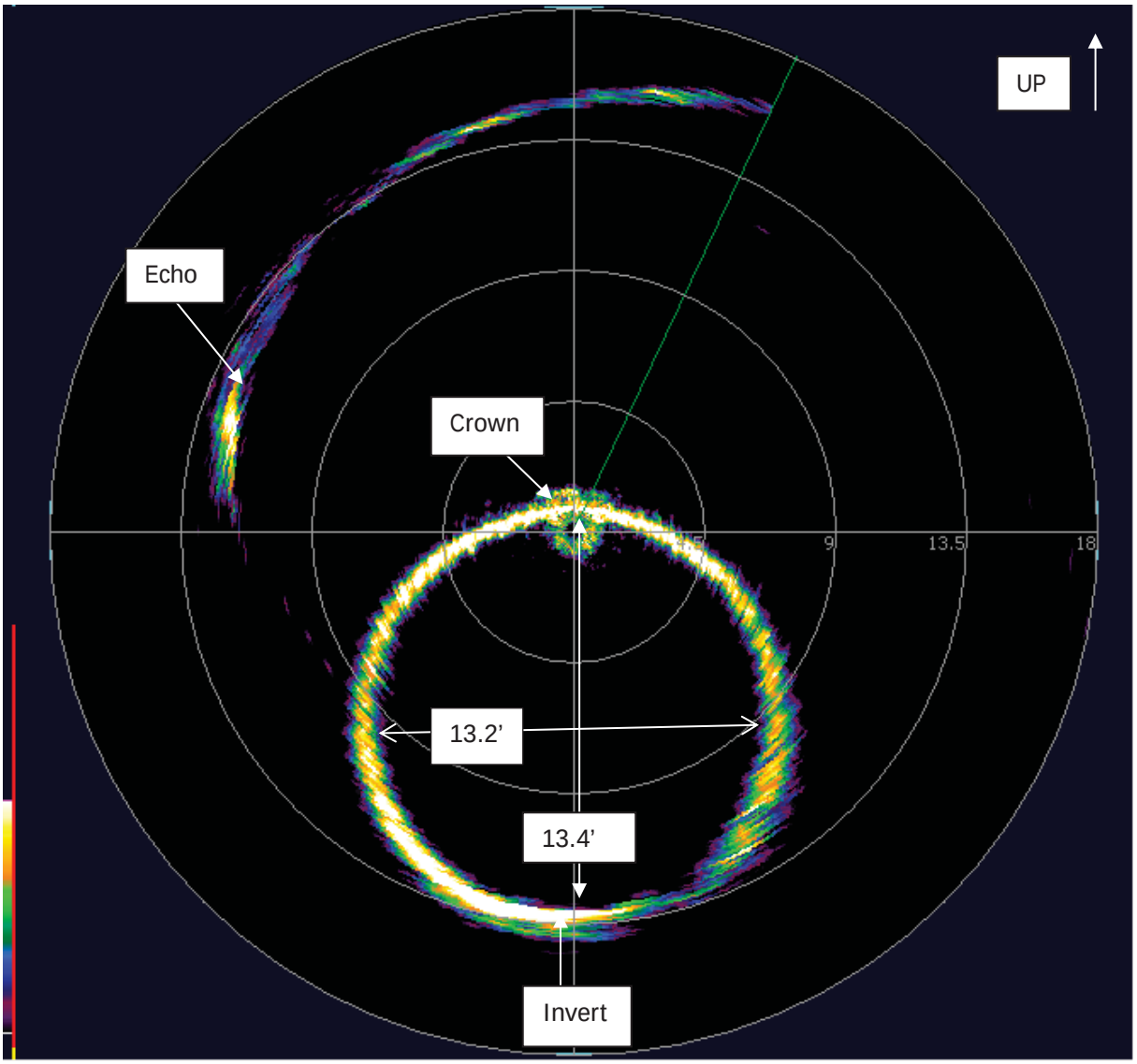

Figure 61 - Cross Section 45 - 2300 feet.

Cross section at 2350 ft.

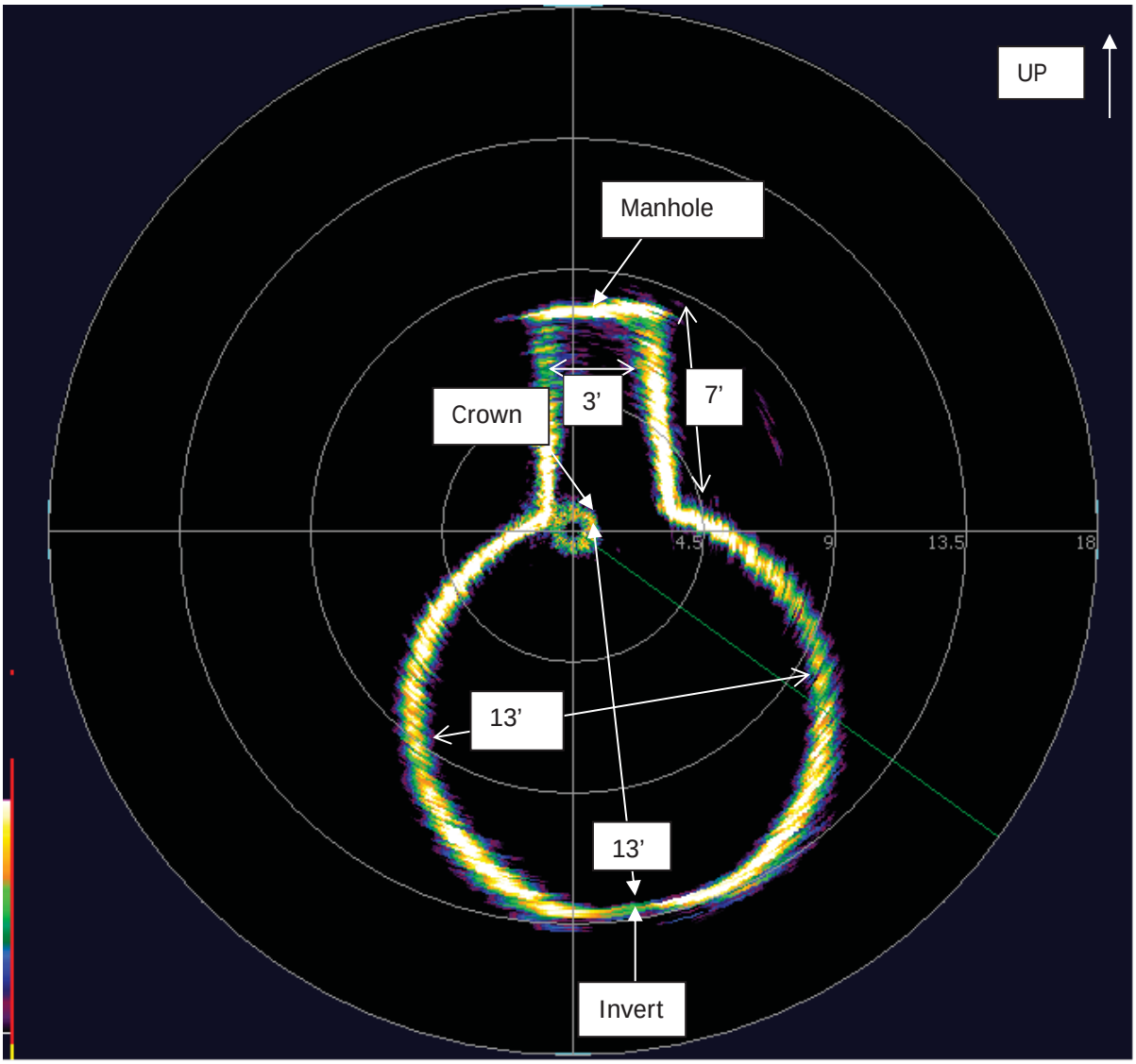

Figure 62 - Cross Section 46 - 2350 feet.

Cross section at 2400ft .

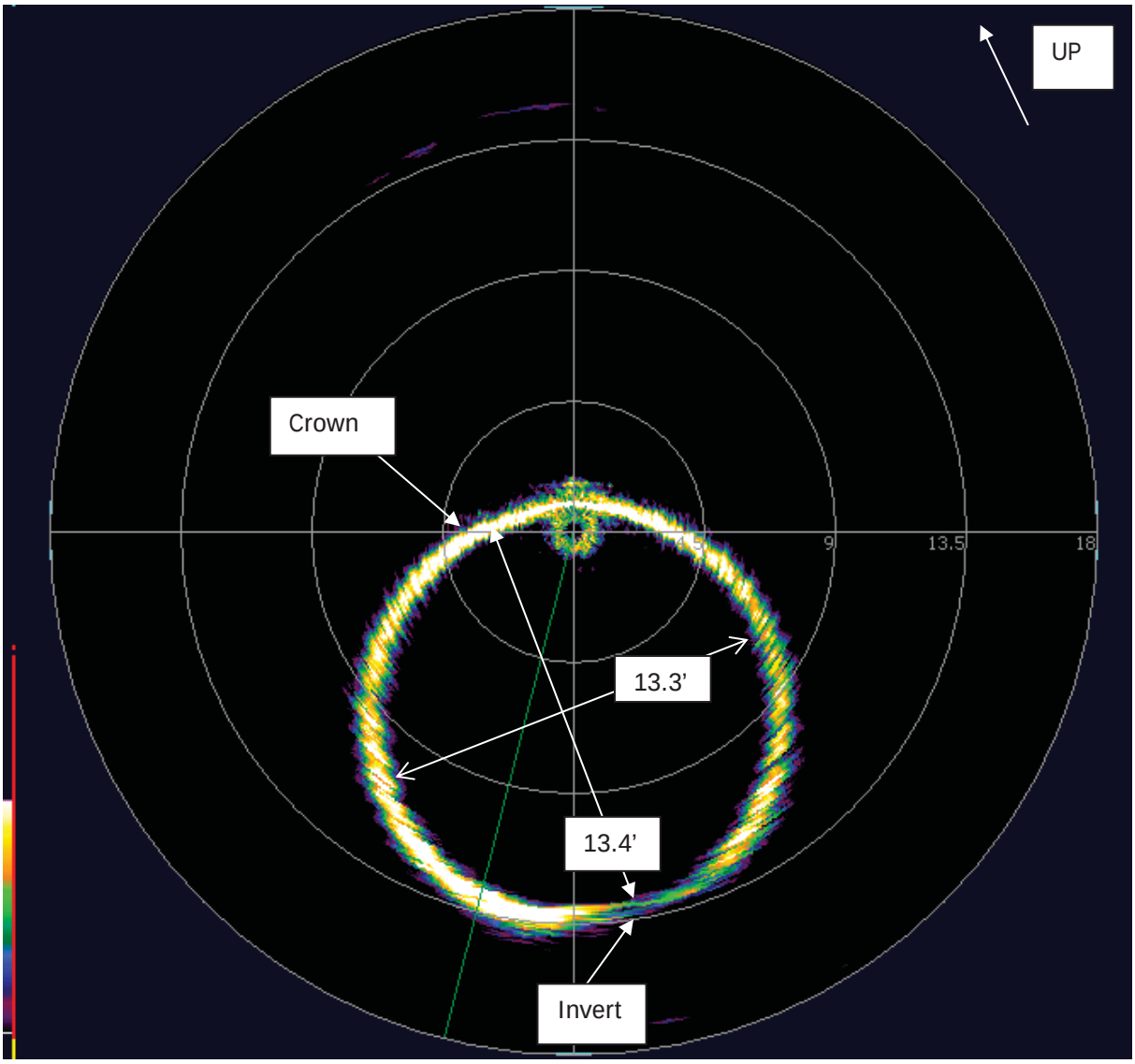

Figure 63 - Cross Section 47 - 2400 feet.

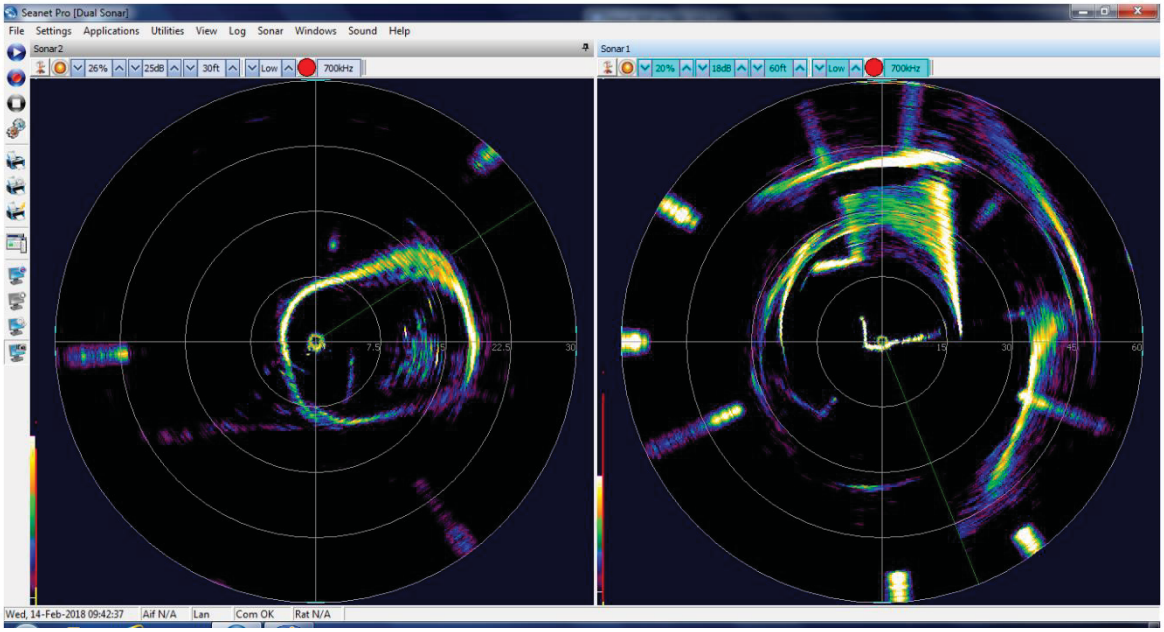

Figure 64 - Cross Section 48 – Sonar Images of Crib

3.2.2.2 Outfall

Cross section at 34 feet. The cross section measured 13.5’ tall with no sediment or scaling evident.

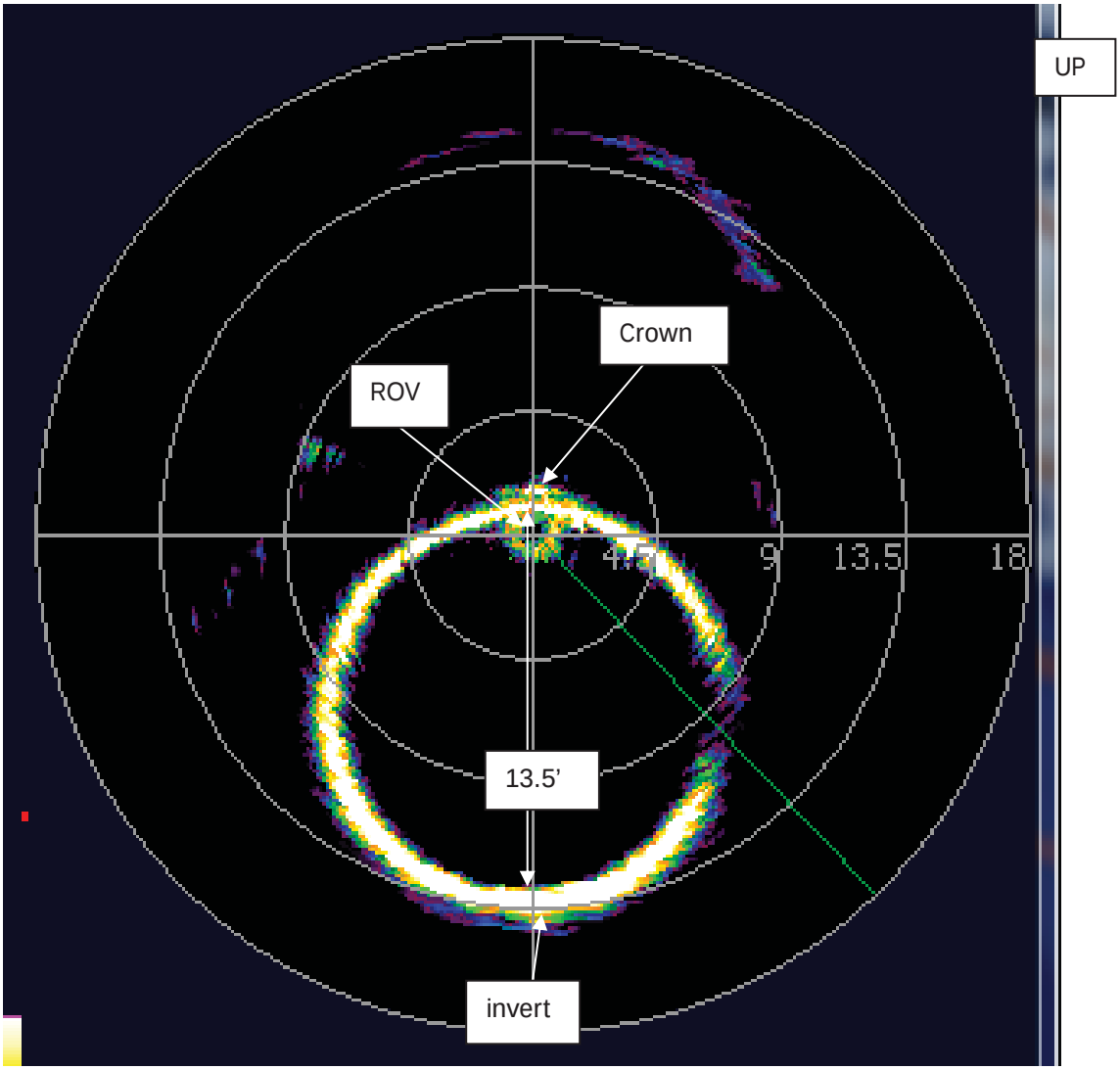

Figure 65 - Cross Section 1 - 34 feet. Square tunnel to round

Cross section at 100 ft. 13.5’ from invert to crown.

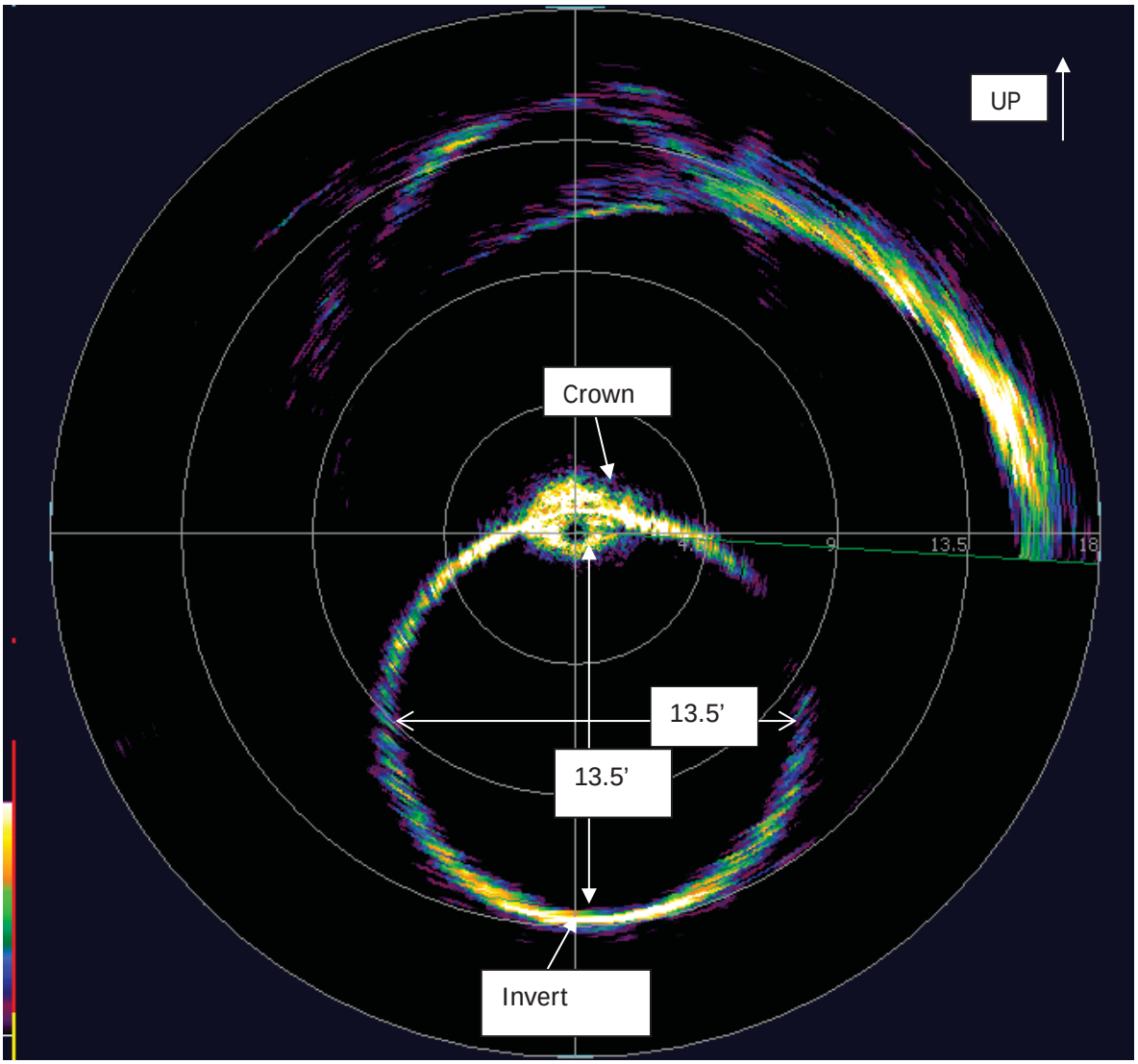

Figure 66 - Cross Section 28 - 100 feet.

Cross section at 150 ft 12.8’ from invert to crown.

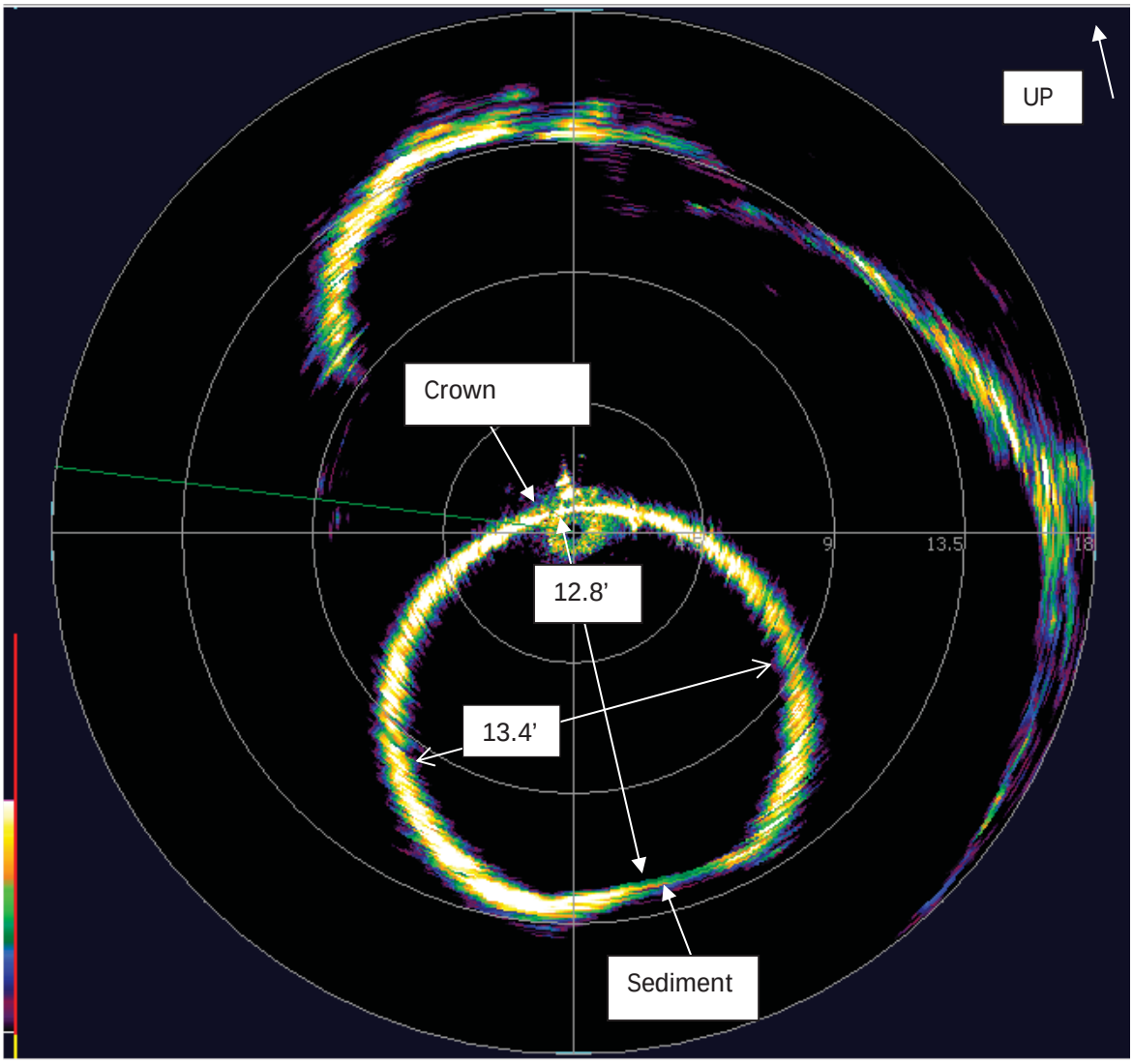

Figure 67 - Cross Section 29 - 150 feet.

Cross Section at 200 ft. 11.2’ from invert to crown.

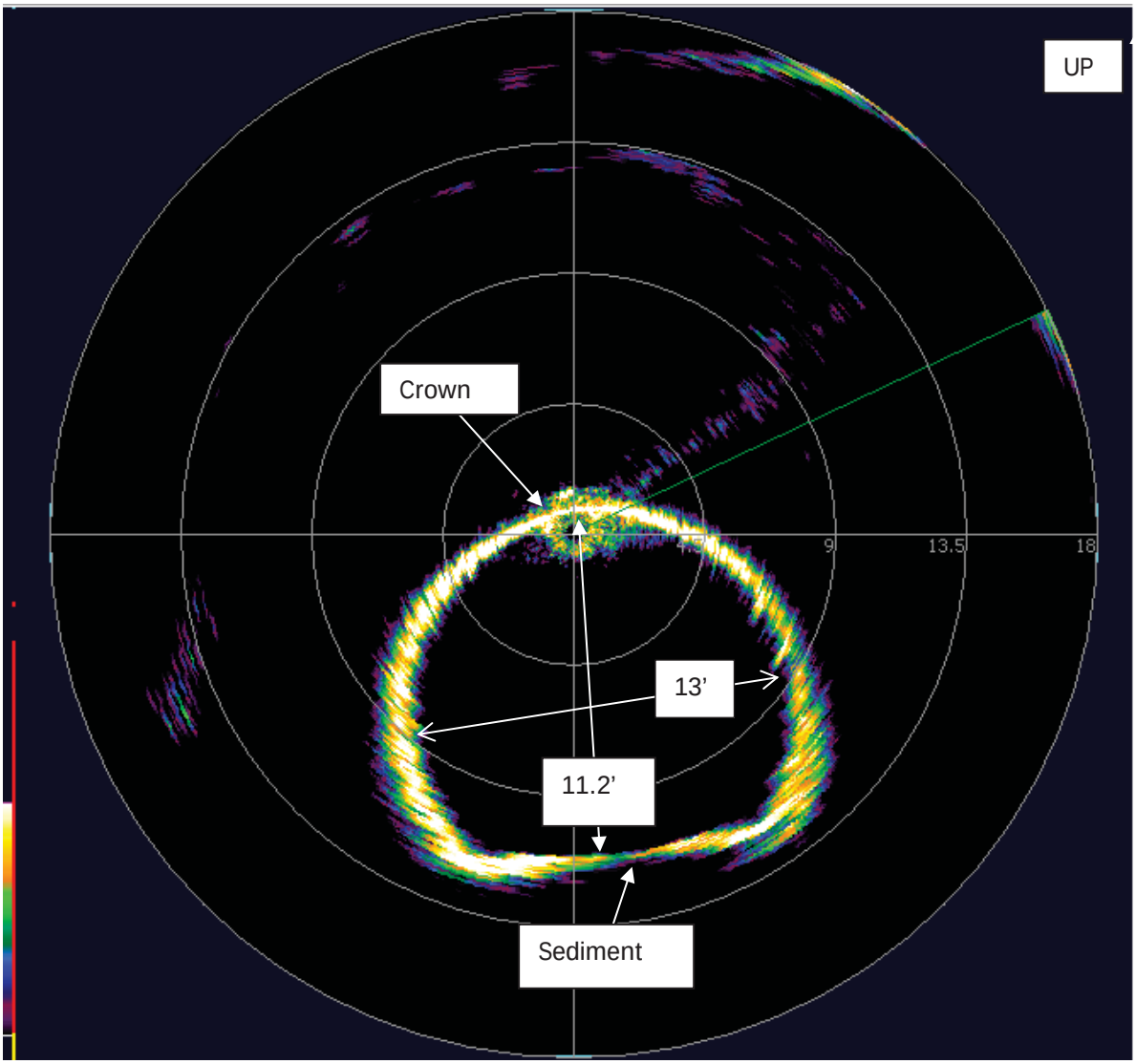

Figure 68 - Cross Section 30 - 200 feet.

Cross section at 250 ft. 12.6’ from invert to crown.

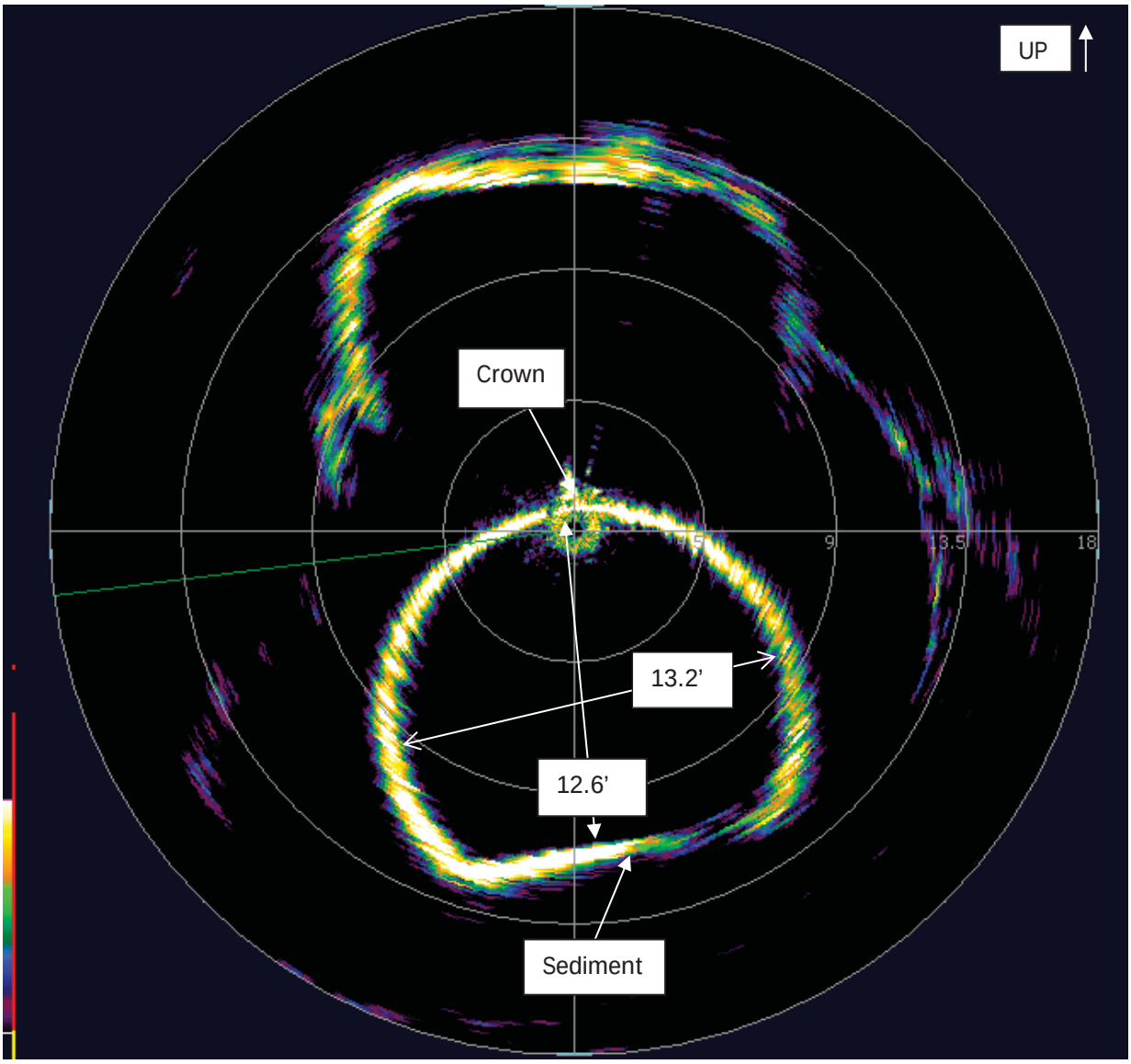

Figure 69 - Cross Section 31 - 250 feet.

Cross Section at 300 ft. 11.3’ from invert to crown.

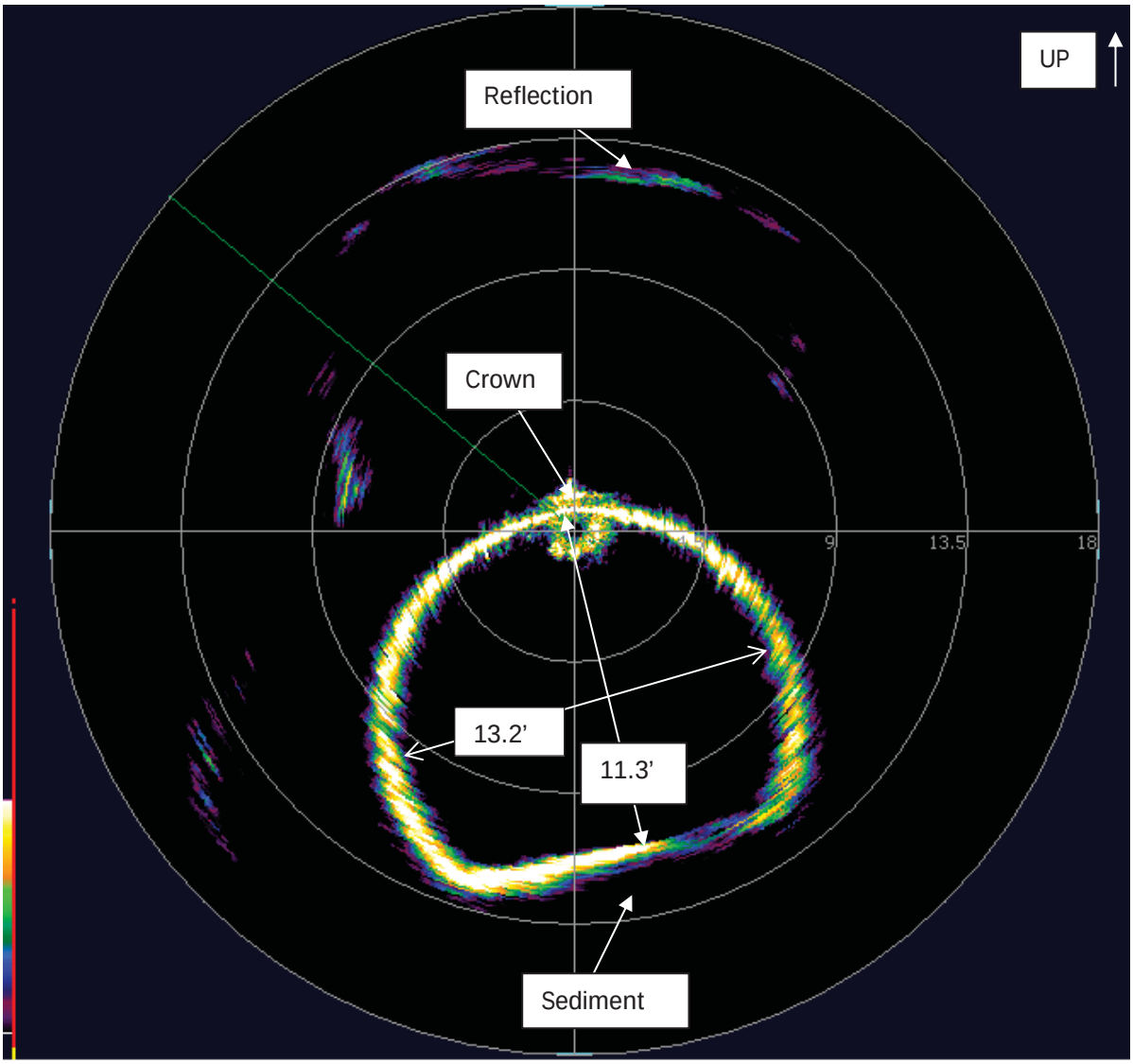

Figure 70 - Cross Section 32 - 300 feet.

Cross section at 350 ft. 11.6’ from invert to crown.

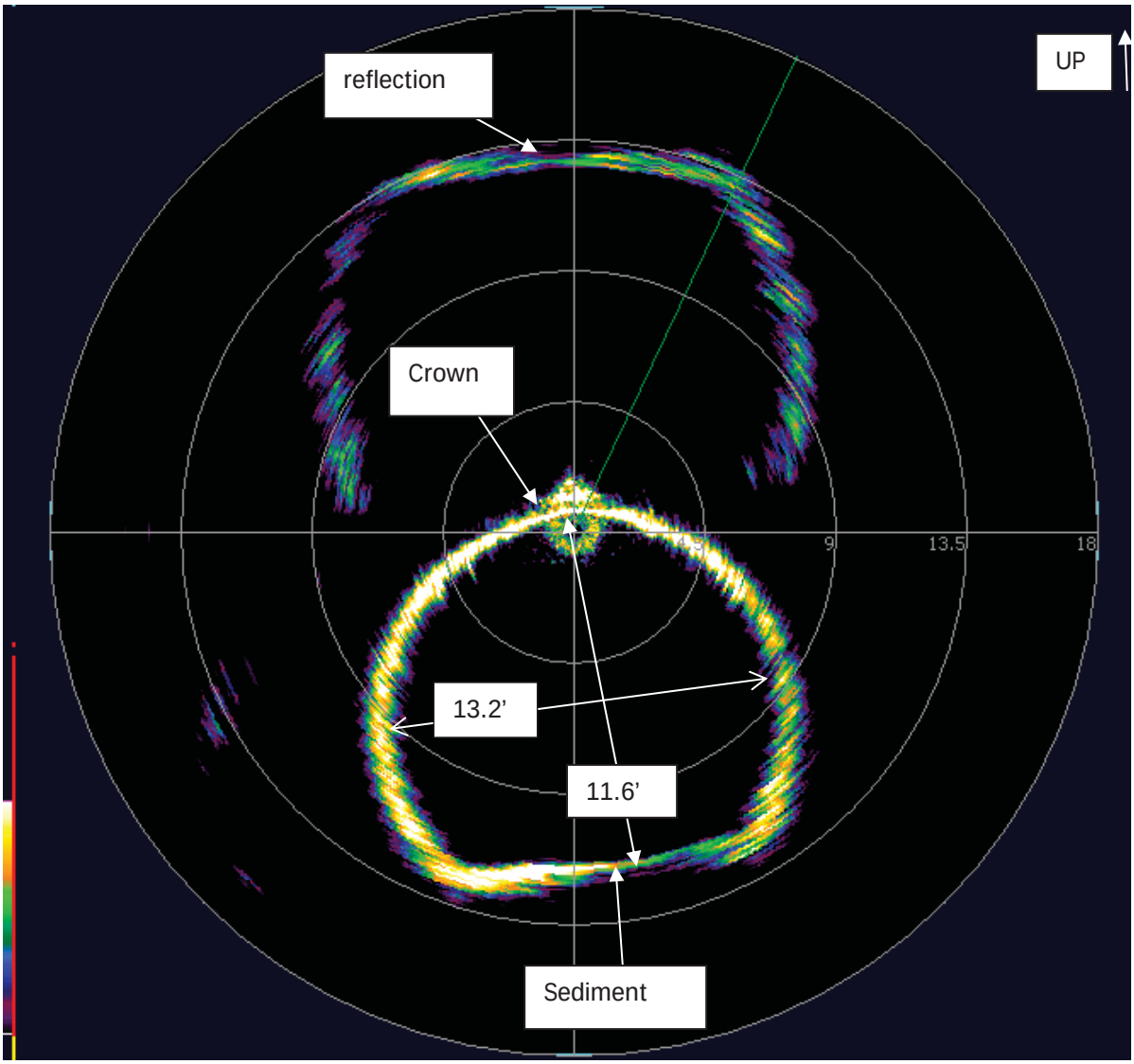

Figure 71 - Cross Section 33 - 350 feet.

Cross section at 400 ft.

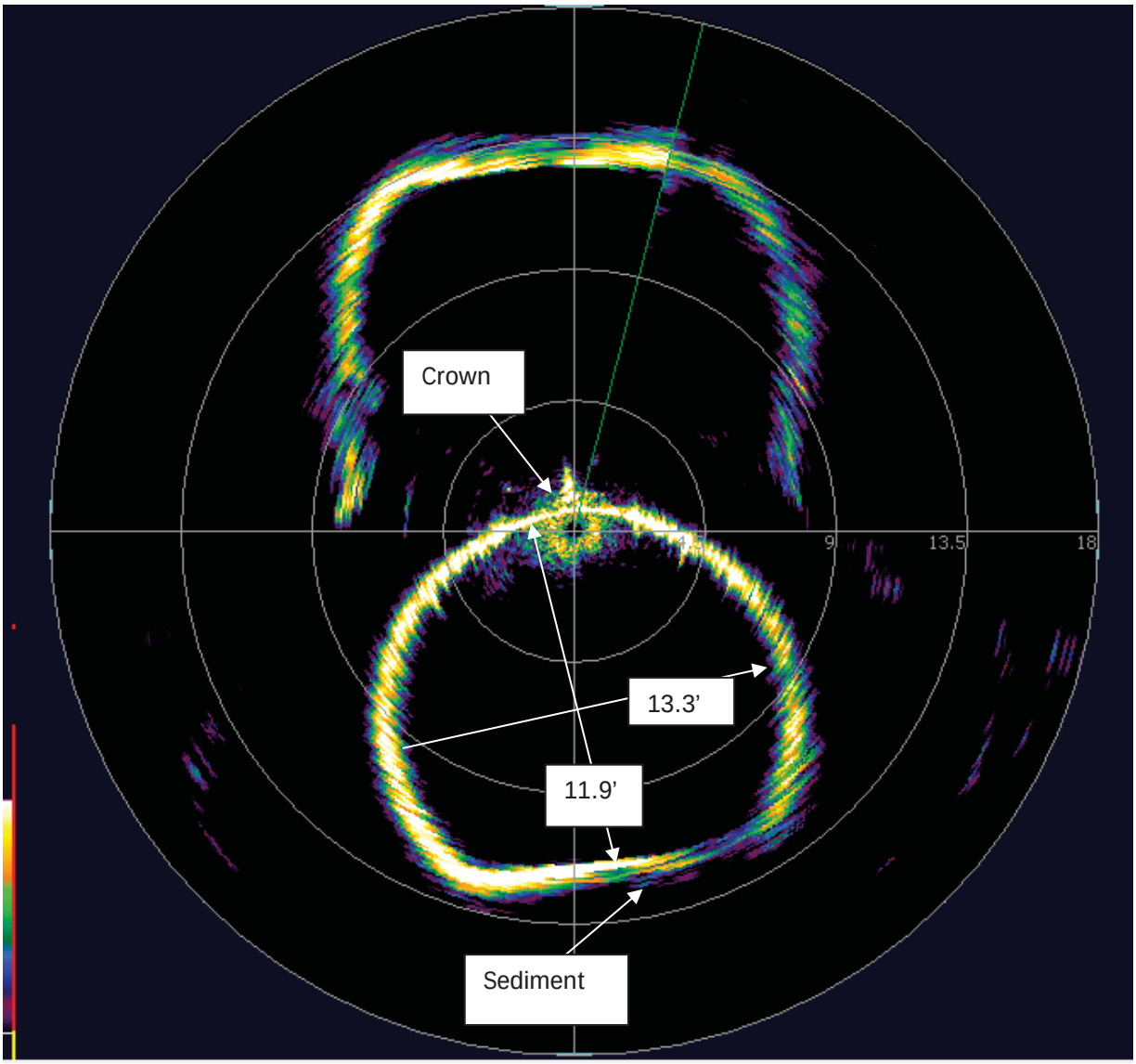

Figure 72 - Cross Section 34 - 400 feet.

Cross section at 450 ft. 13.5’ width by 11.7’ from invert to crown.

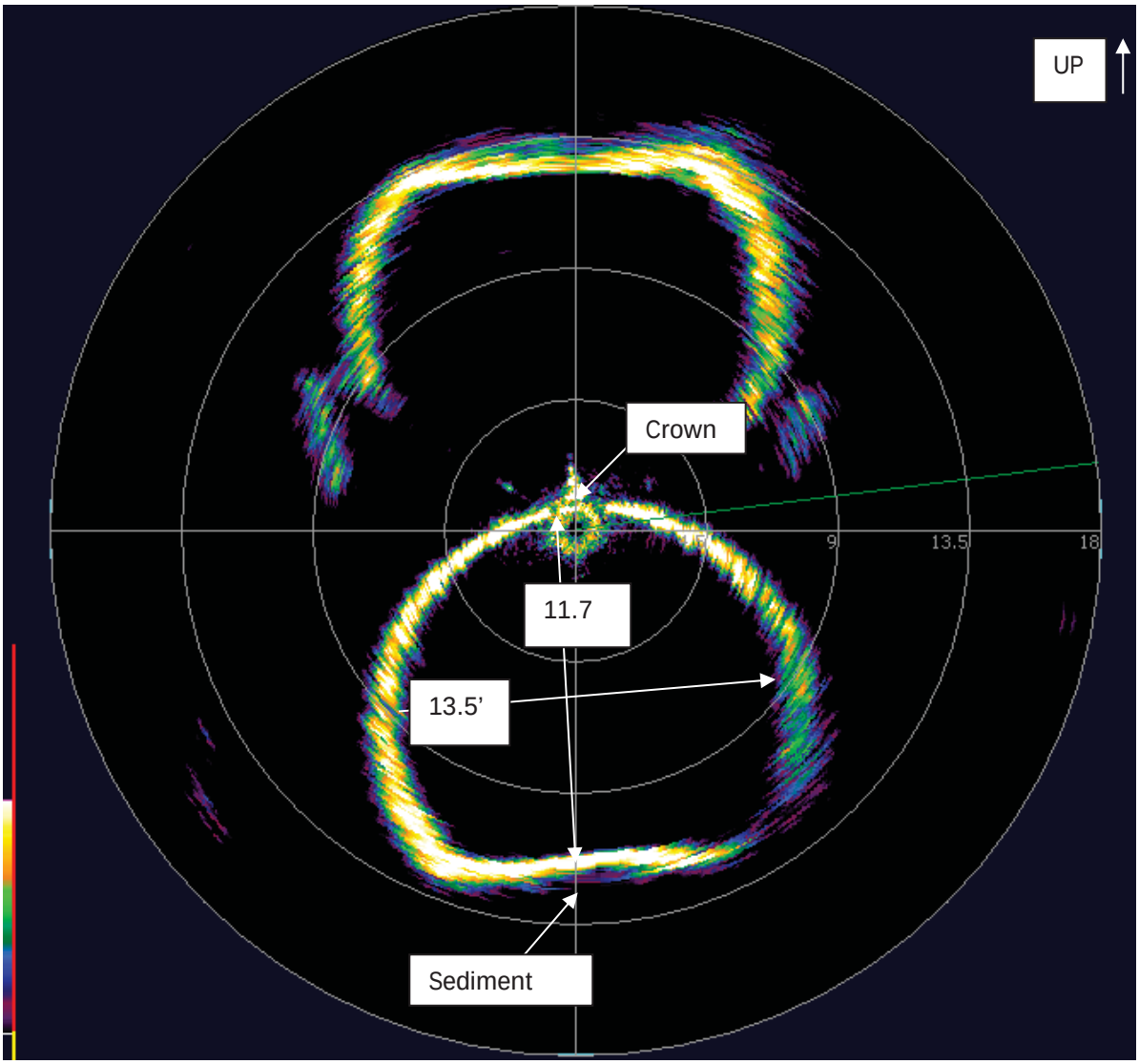

Figure 73 - Cross Section 35 - 450 feet.

Cross section at 500 ft. 13.5’ wide by 11.9’ from invert to crown.

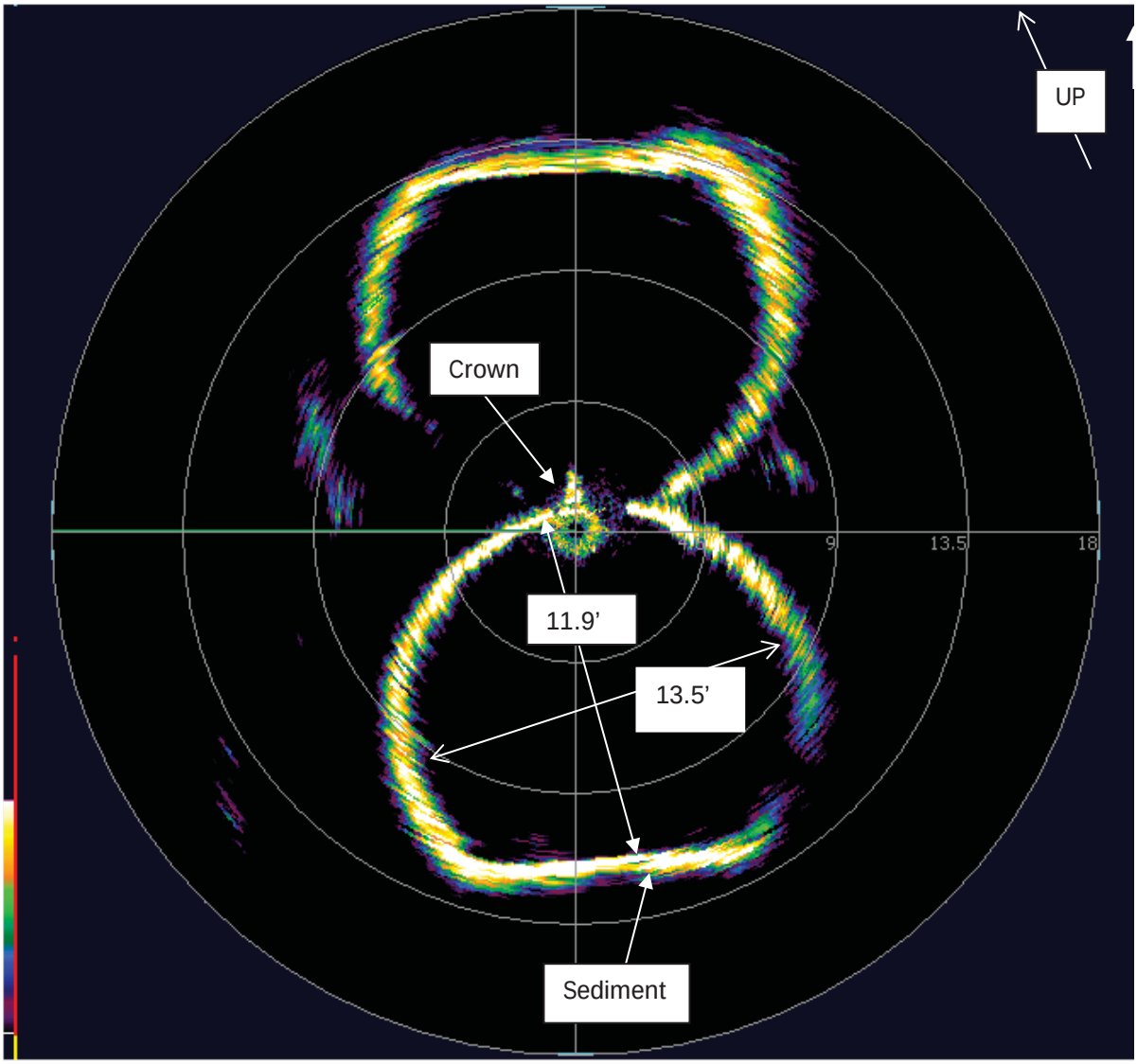

Figure 74 - Cross Section 36 - 500 feet.

Cross Section at 550 ft. 13.3’ wide by 12’ from invert to crown.

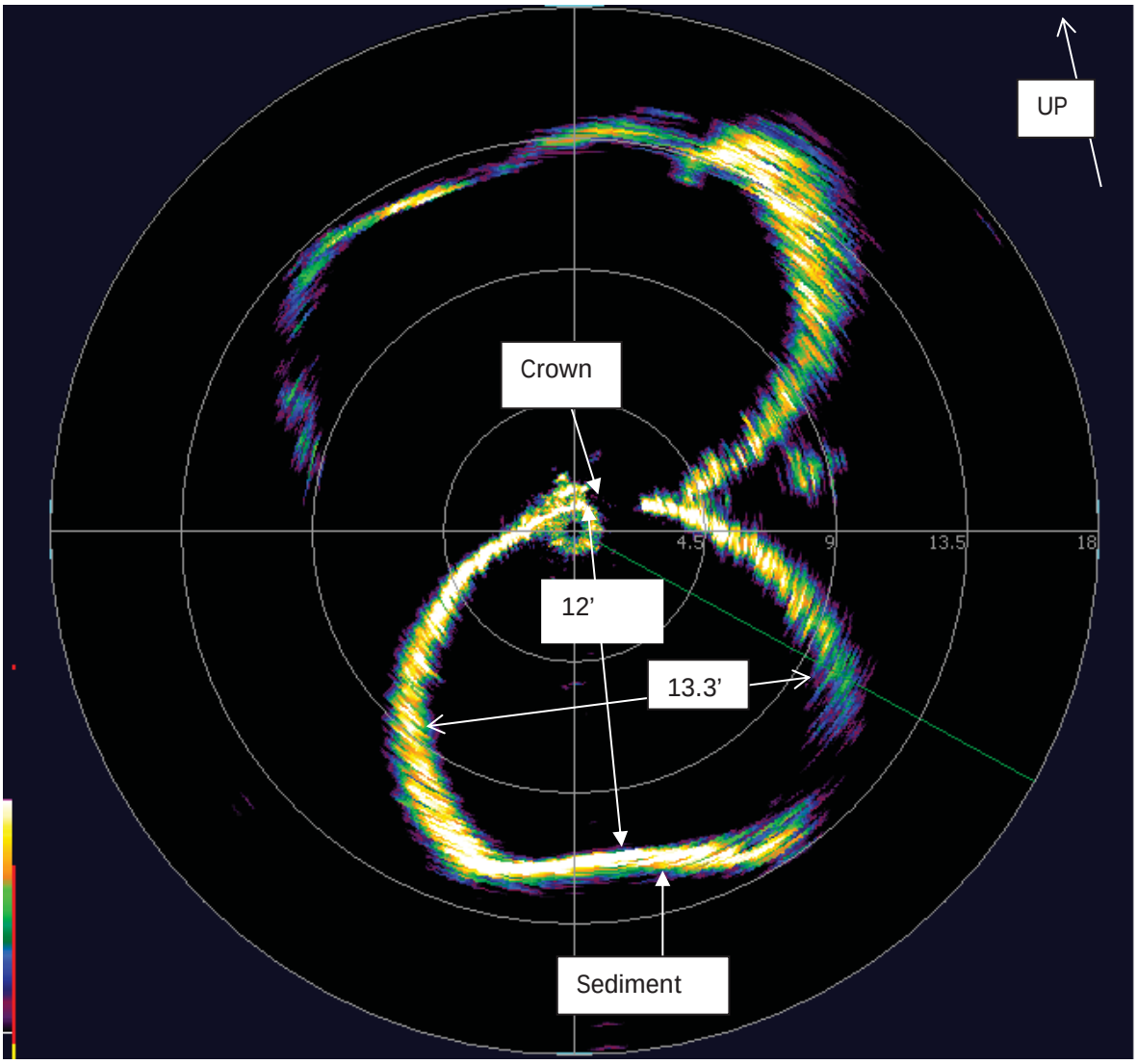

Figure 75 - Cross Section 37 - 550 feet.

Cross section at 600 ft. 13.3’ wide by 11.9’ from invert to crown.

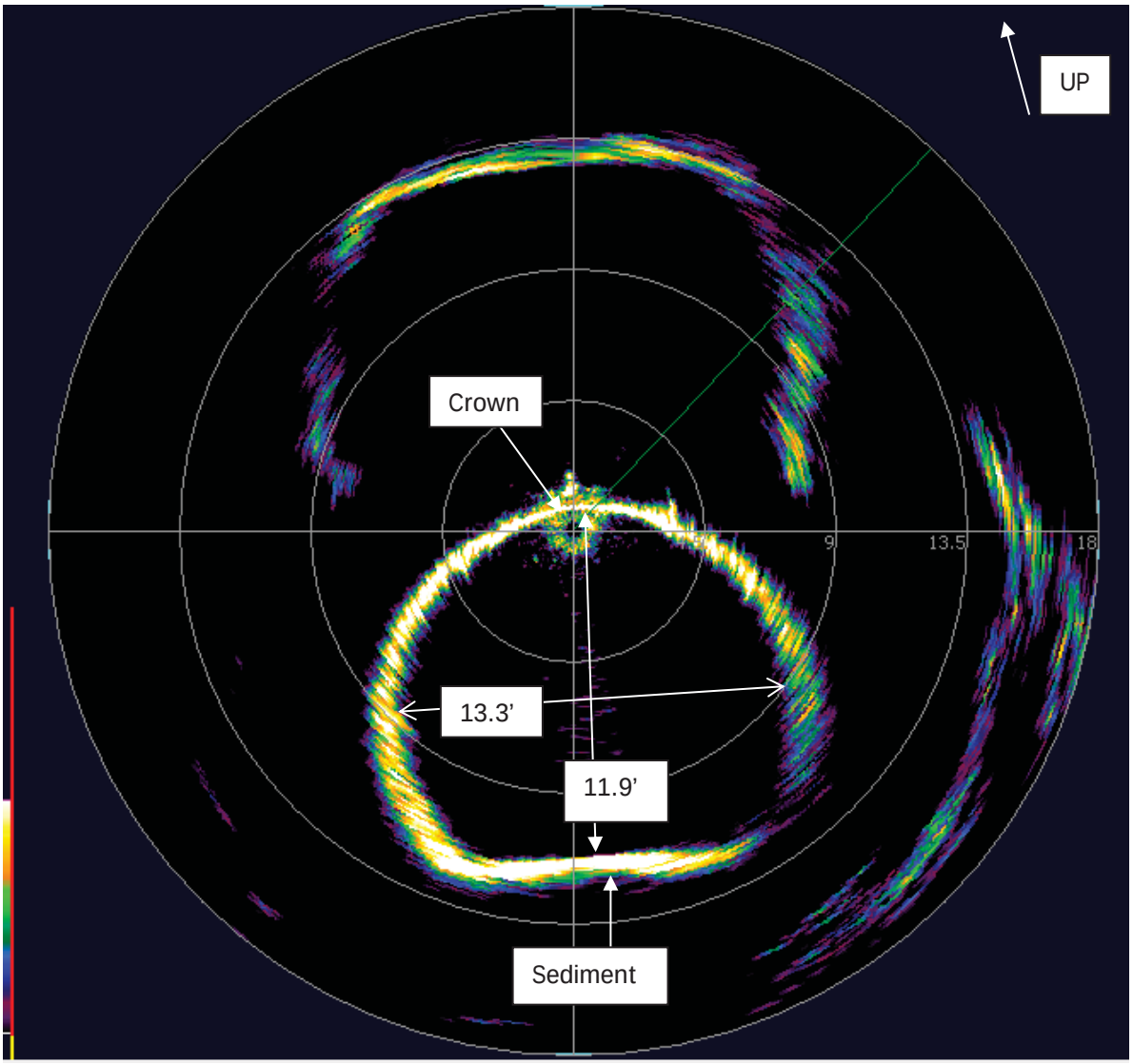

Figure 76 - Cross Section 38 - 600 feet.

Cross section at 650 ft. 13.4’ wide by 11.6’ from invert to crown.

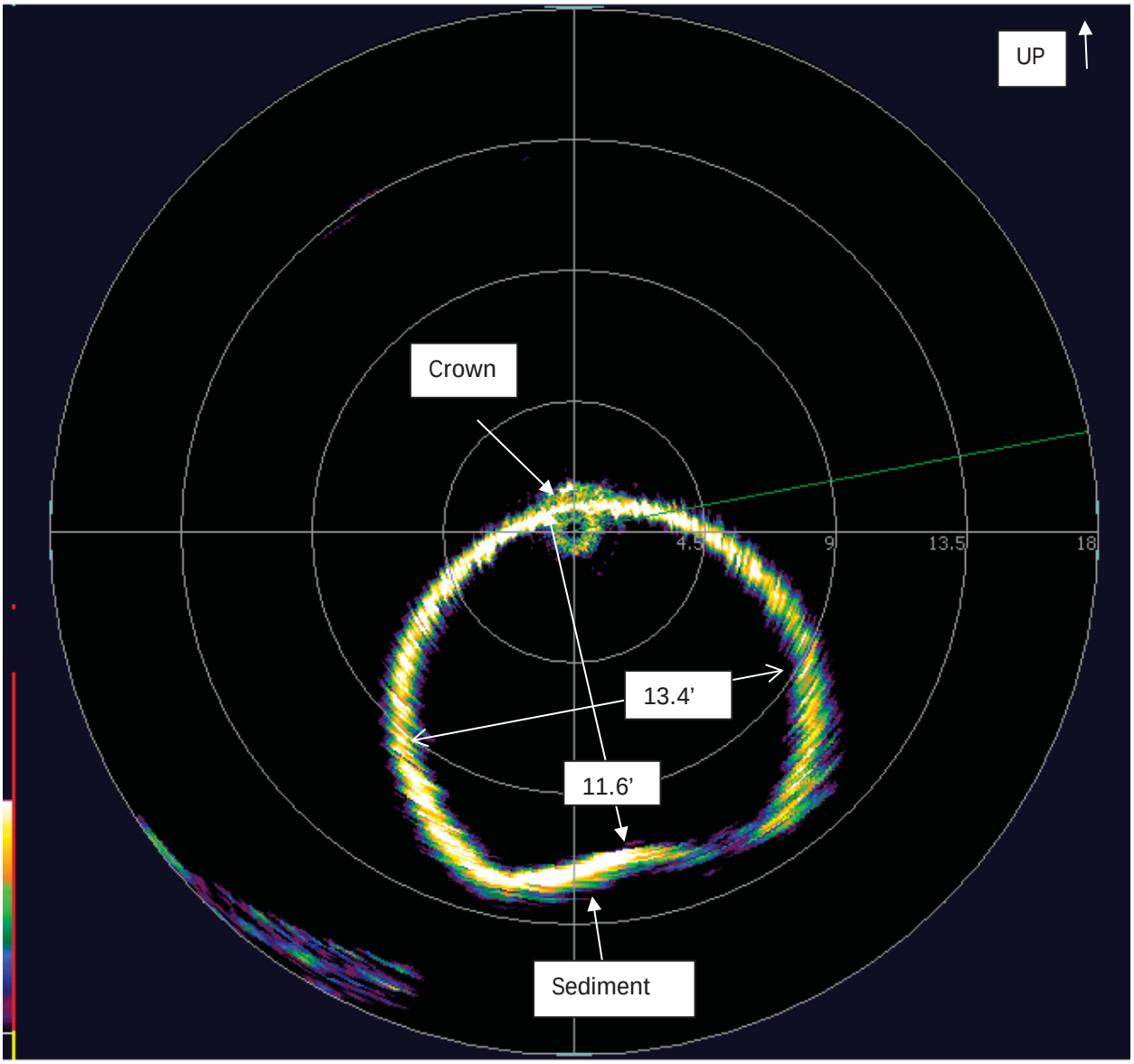

Figure 77 - Cross Section 39 - 650 feet.

Cross section at 700 ft. 13.4’ wide by 11.3’ from invert to crown. First location of manhole

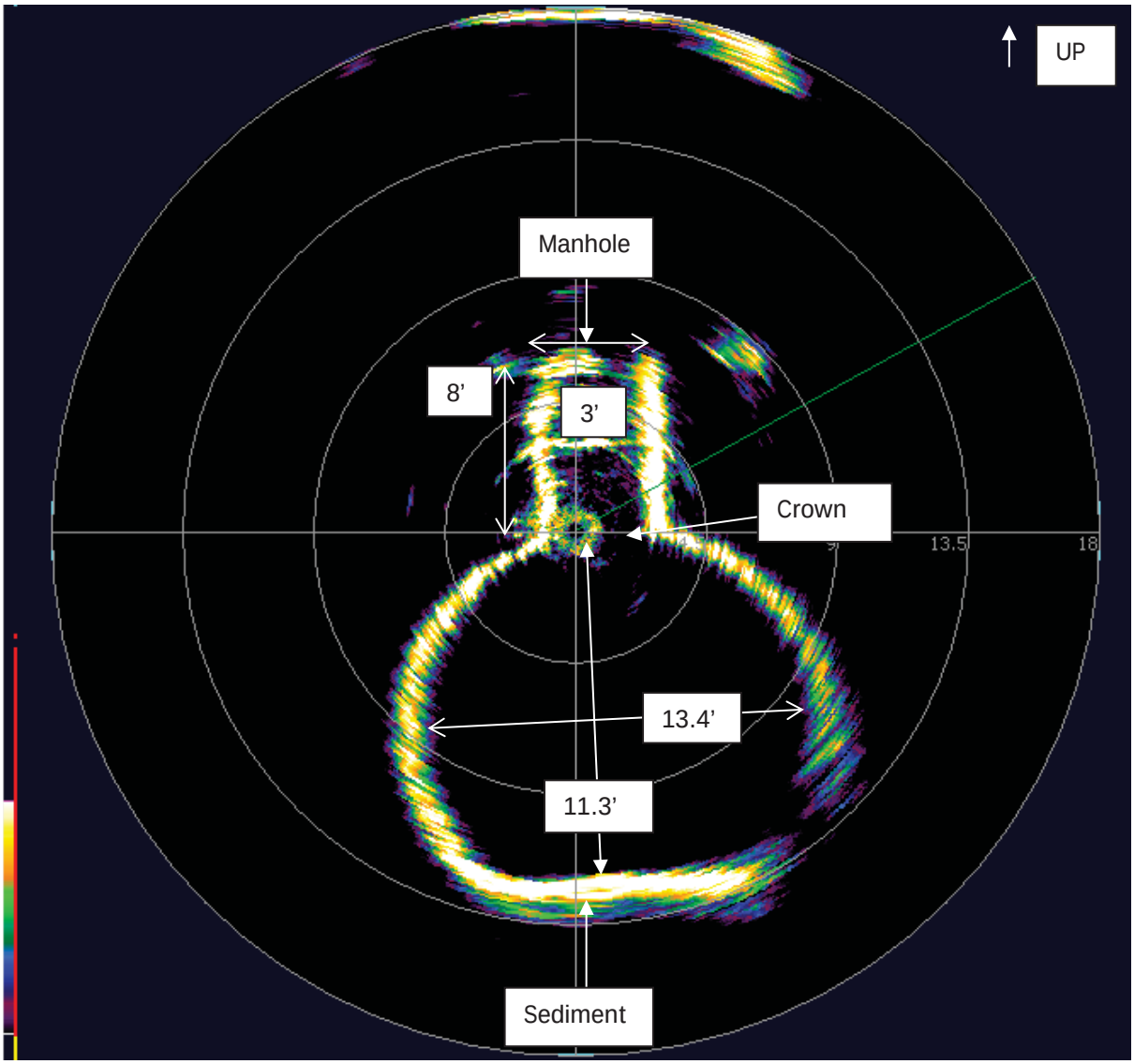

Figure 78 - Cross Section 40 - 700 feet.

Cross section at 750 ft. 13.1’ wide by 11.7’ from invert to crown.

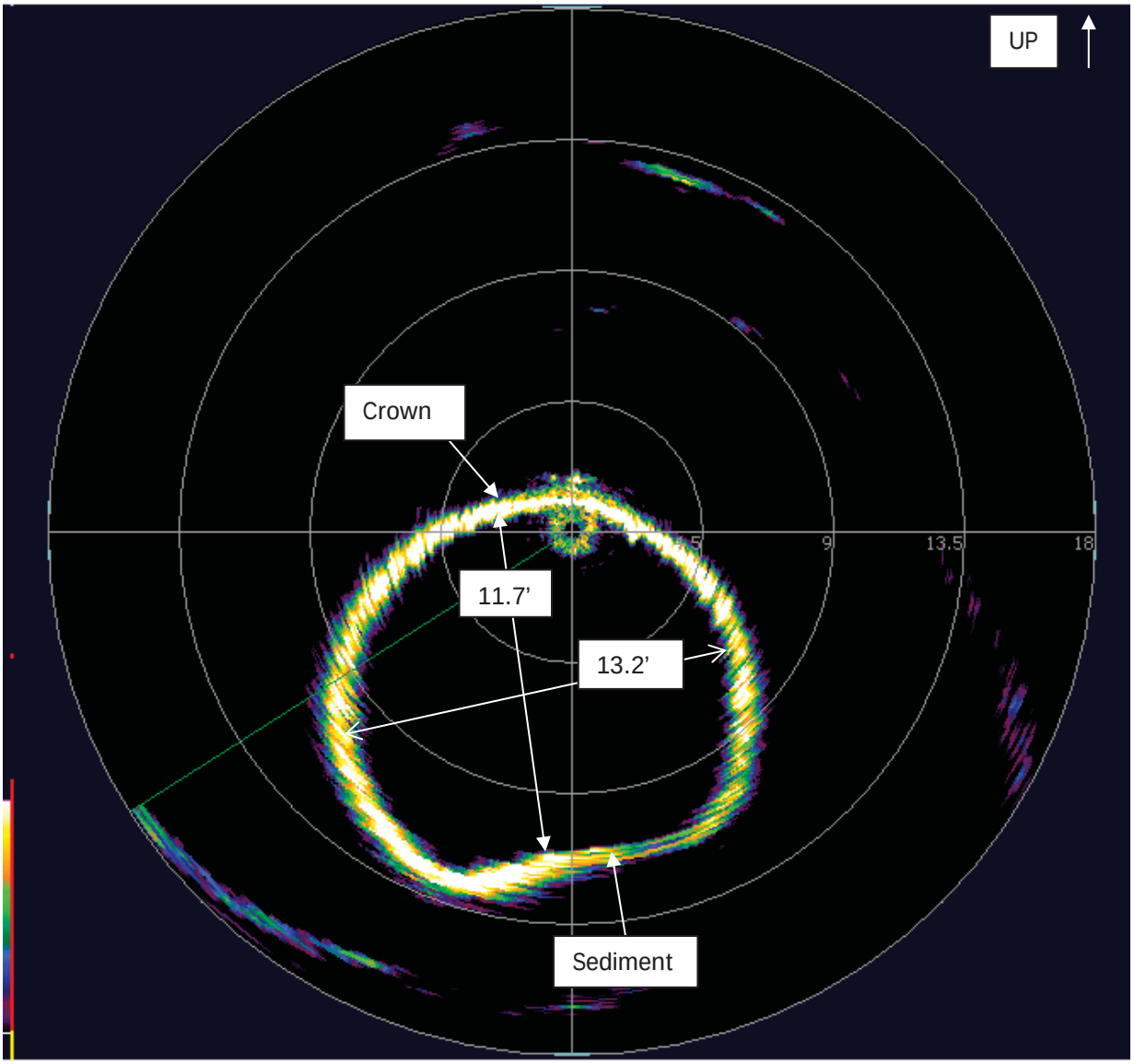

Figure 79 - Cross Section 41 - 750 feet.

Cross section at 800 ft. 13.3’ wide by 12’ from invert to crown.

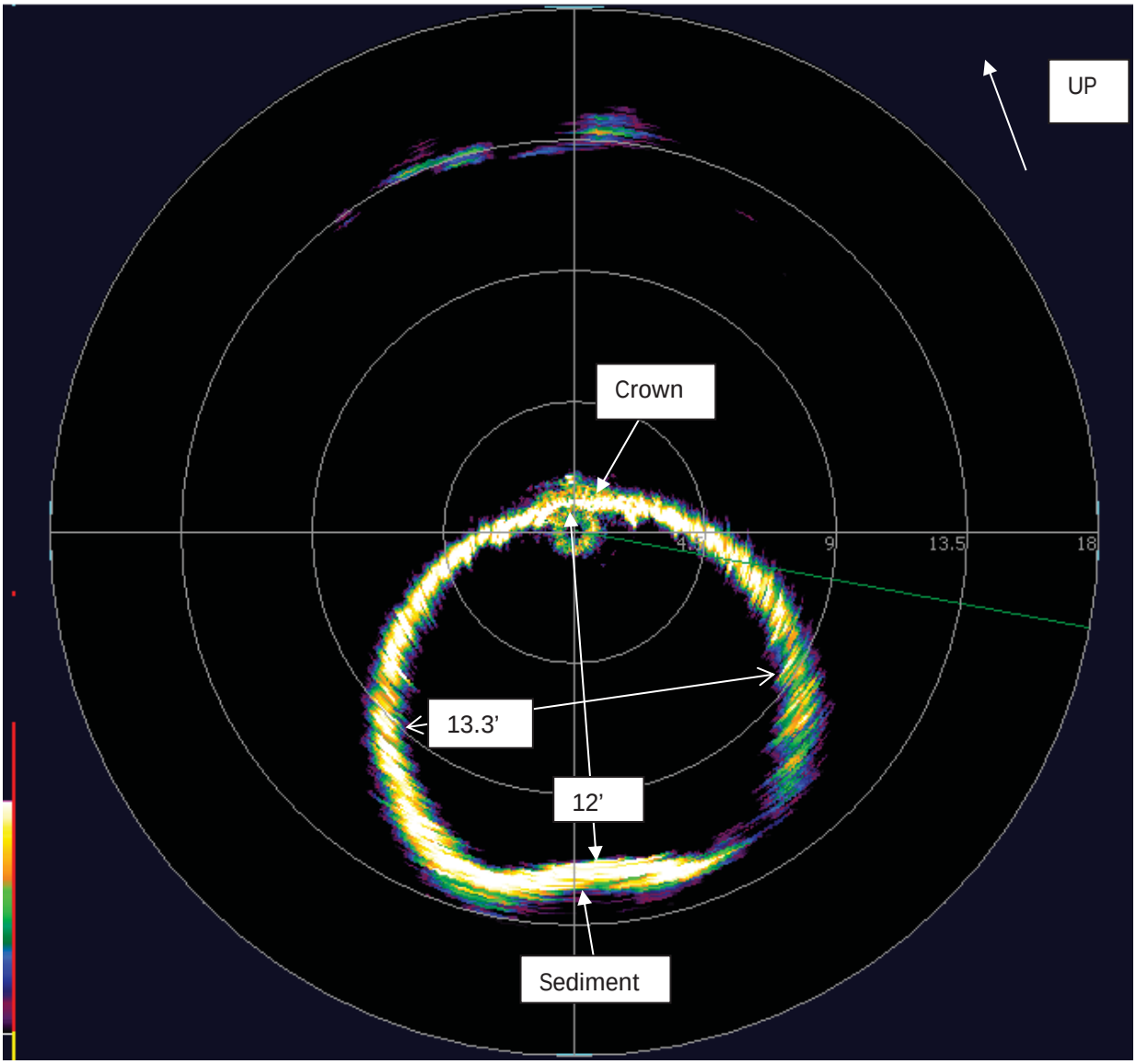

Figure 80 - Cross Section 42 - 800 feet.

Cross section at 850 ft.

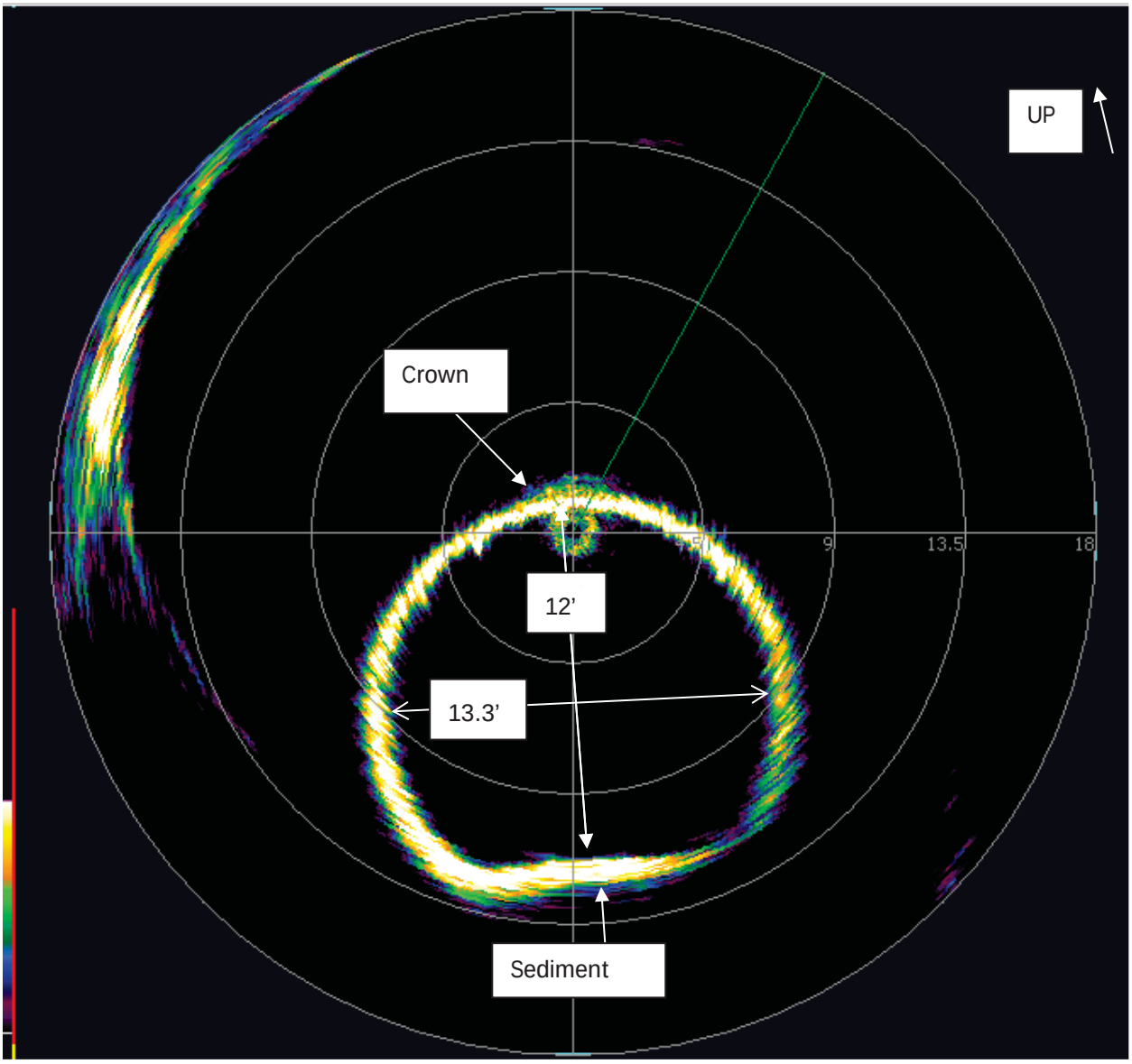

Figure 81 - Cross Section 43 - 850 feet.

Cross section at 900 ft.

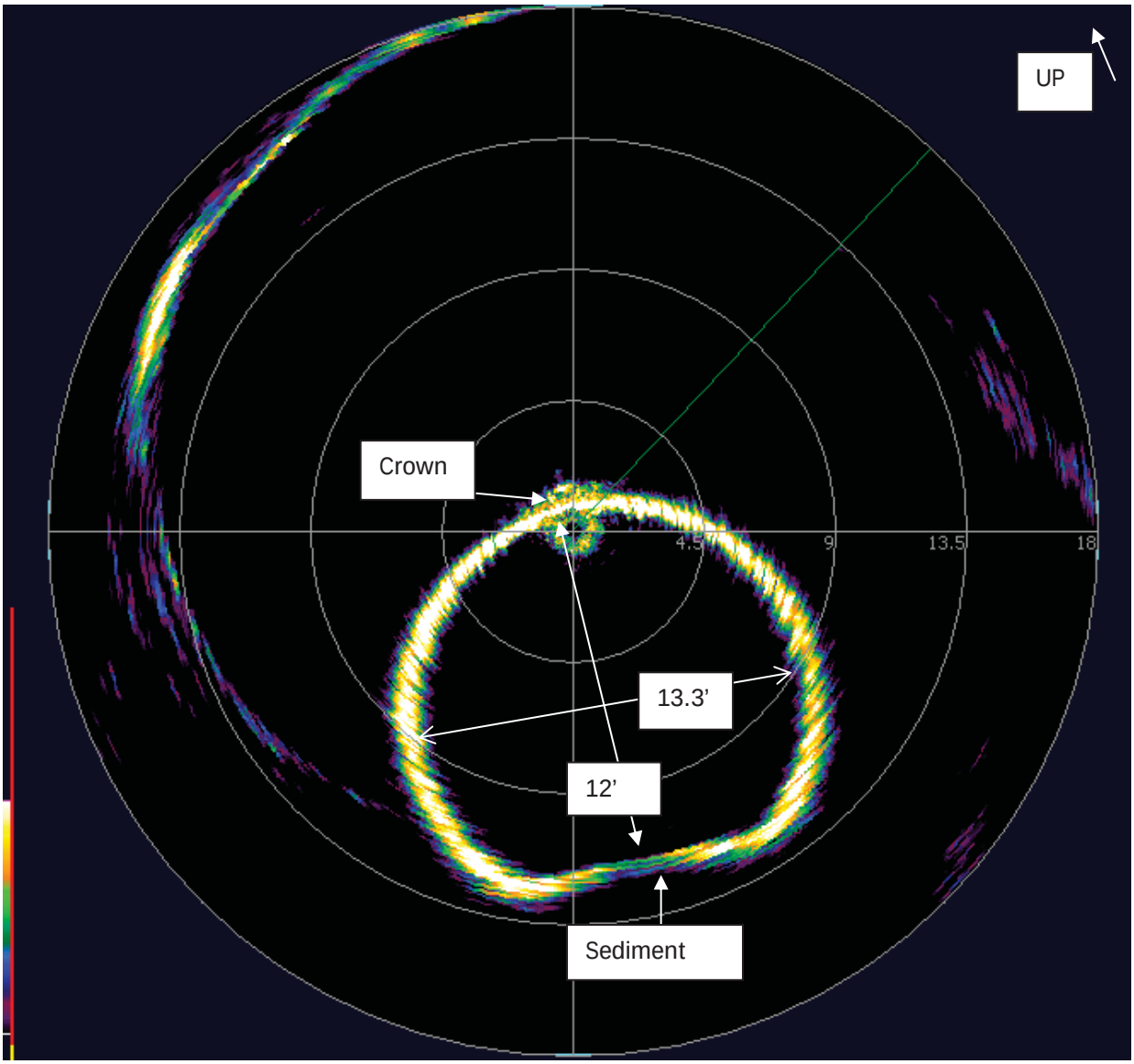

Figure 82 - Cross Section 44 - 900 feet.

Cross section at 950 ft.

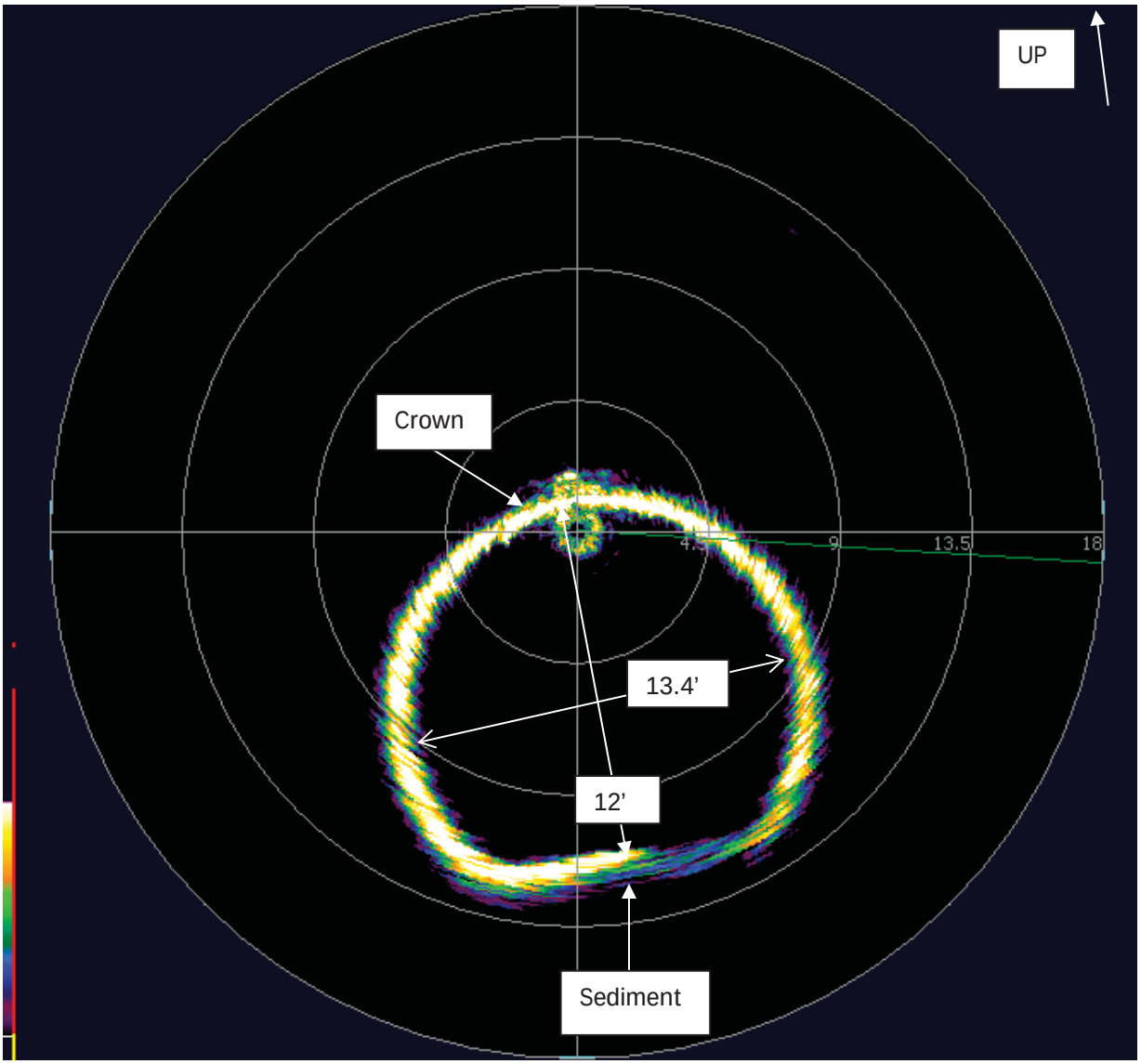

Figure 83 - Cross Section 45 – 950 feet.

Cross section at 1000 ft.

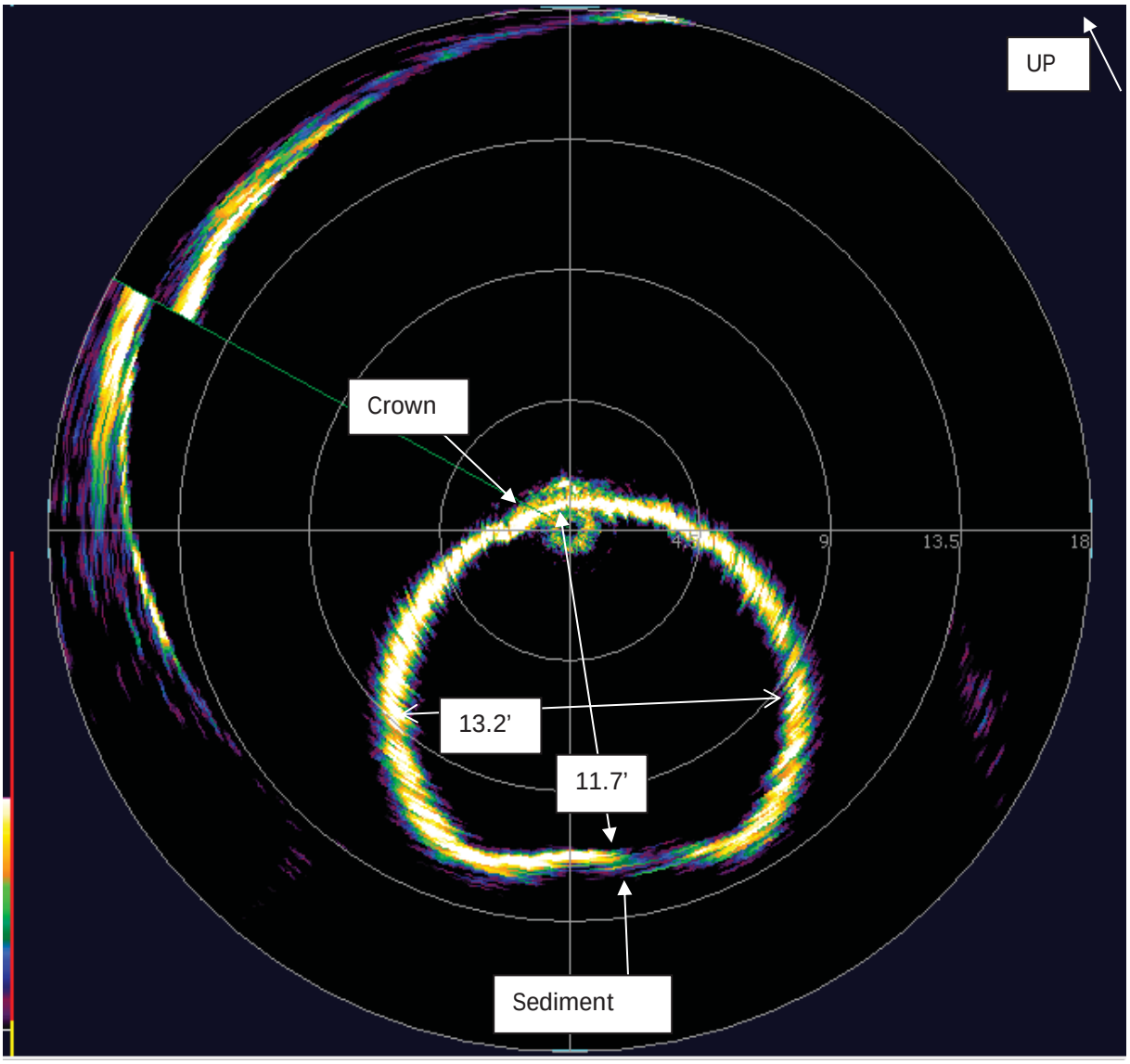

Figure 84 - Cross Section 46 - 1000 feet.

Cross section at 1050ft.

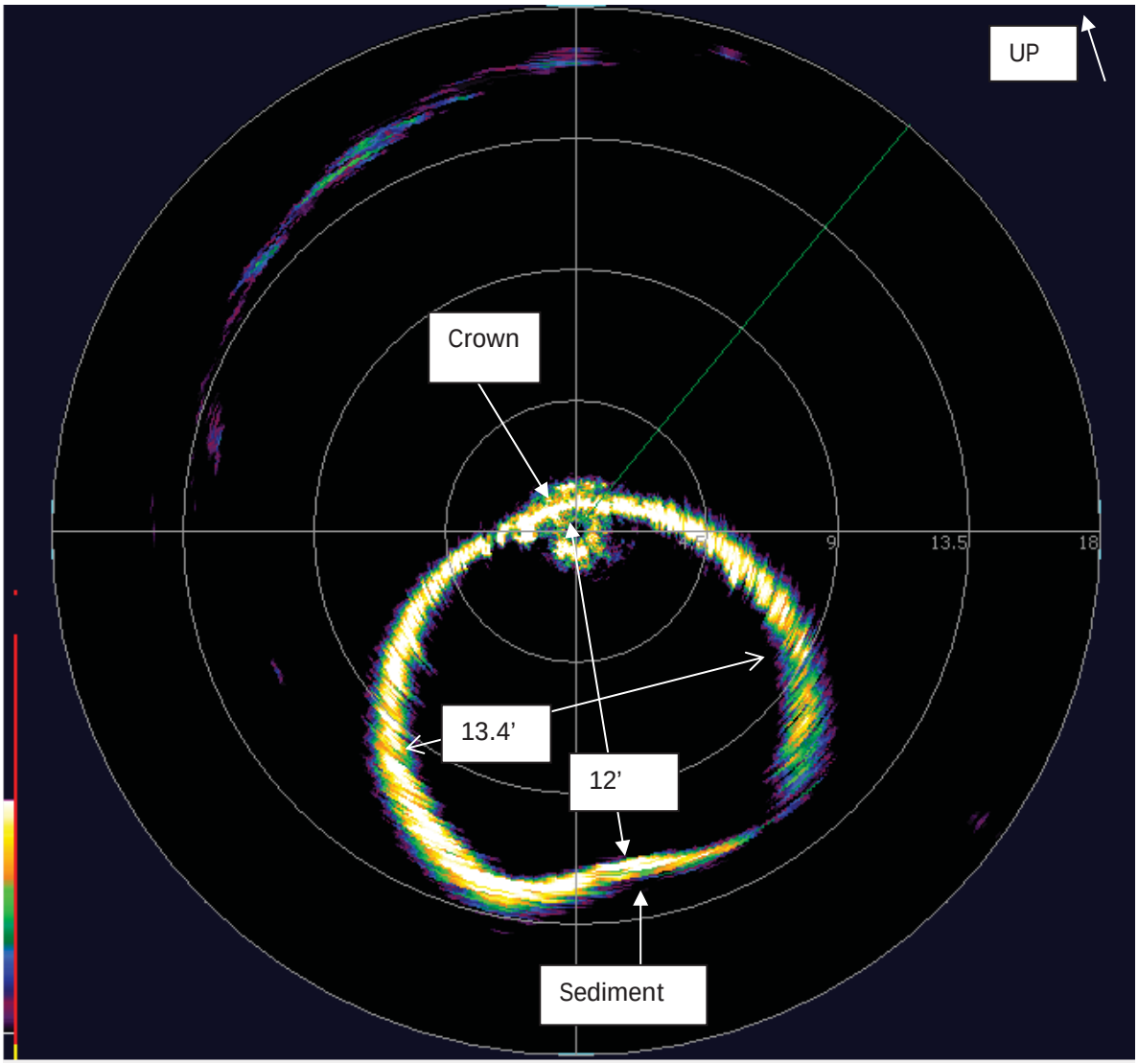

Figure 85 - Cross Section 47 - 1050 feet.

Cross section at 2300 ft.

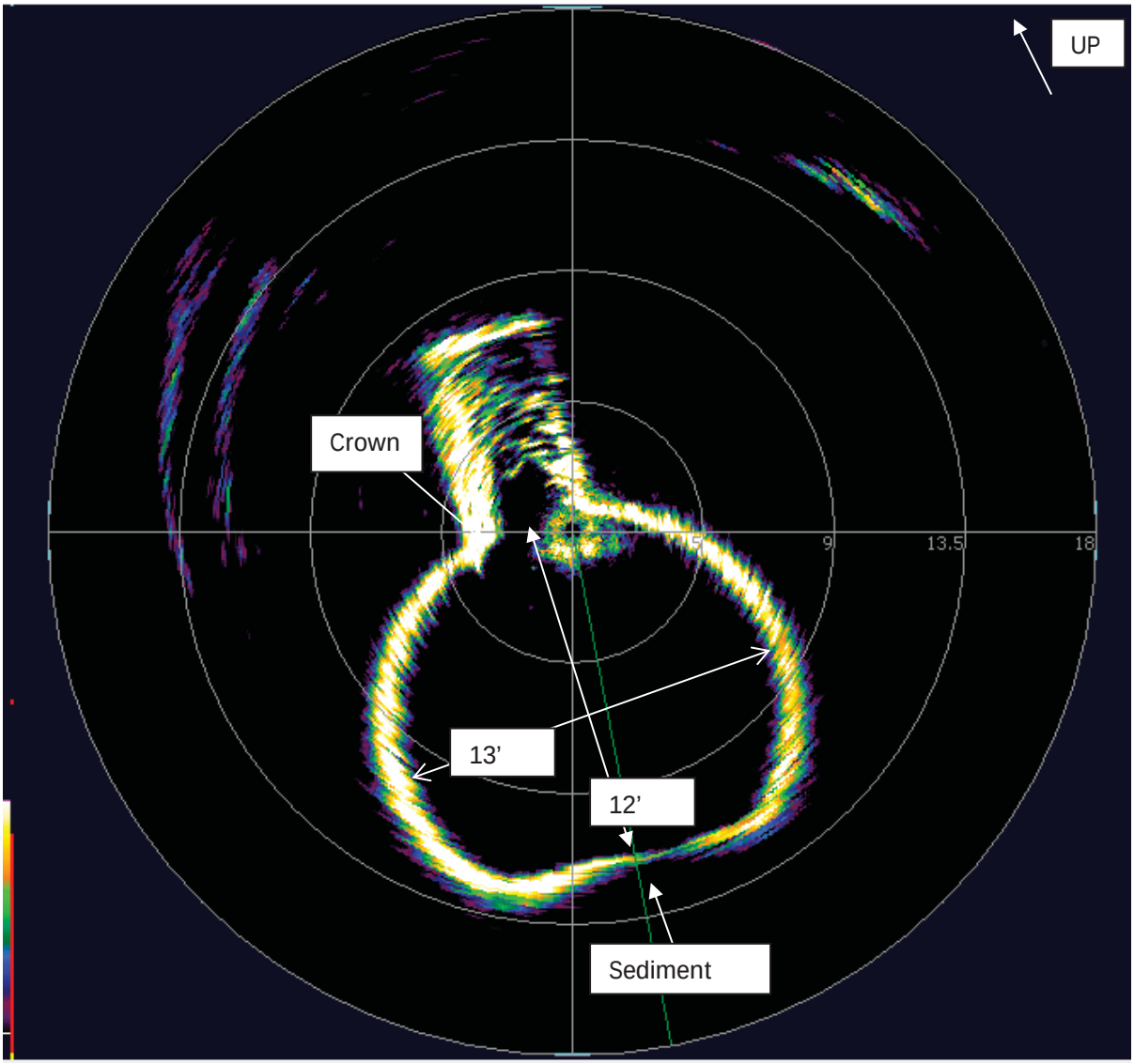

Figure 86 - Cross Section 48 - 1100 feet.

Cross section at 1150 ft.

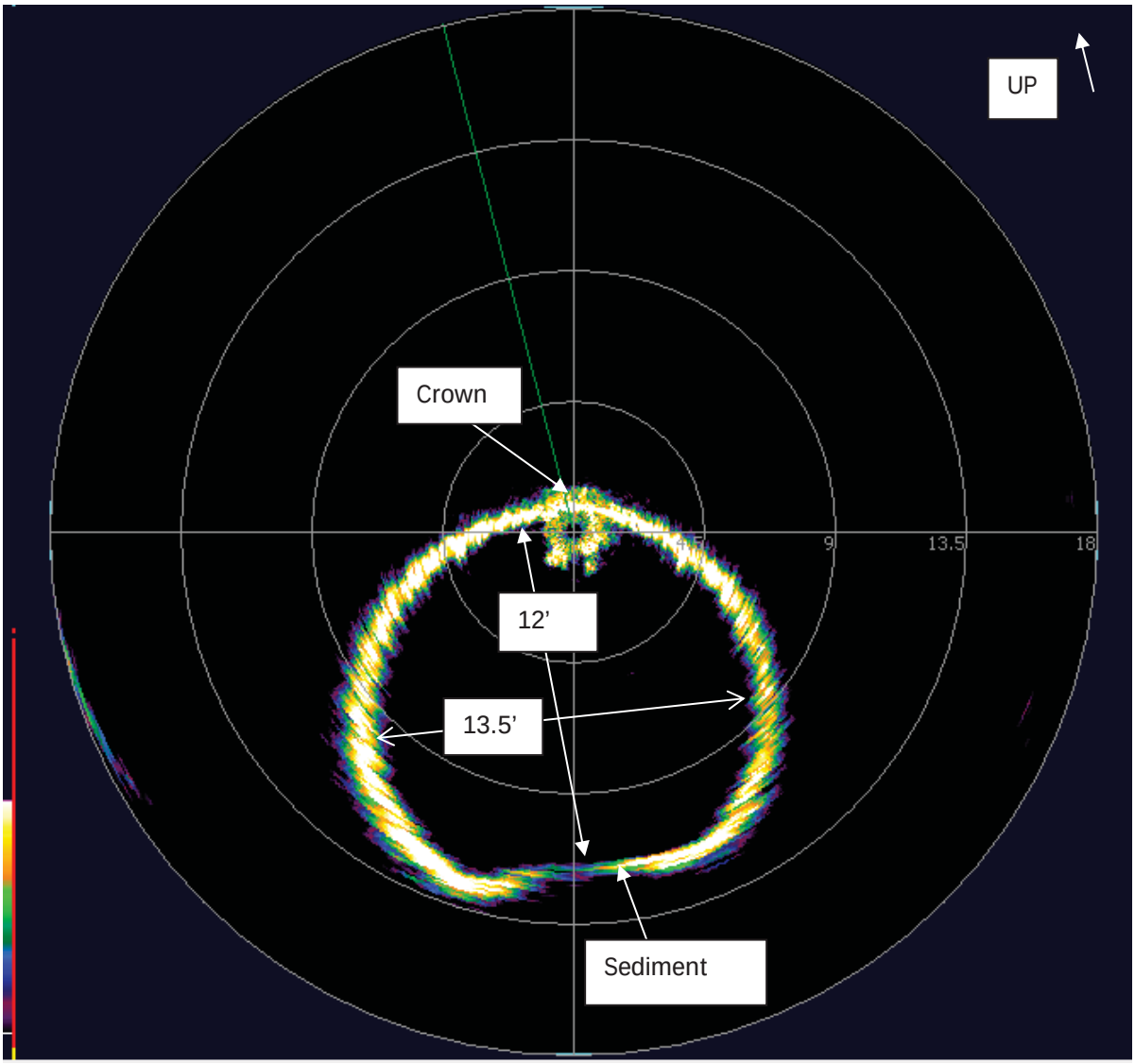

Figure 87 - Cross Section 49 - 1150 feet.

Cross section at 1200 ft.

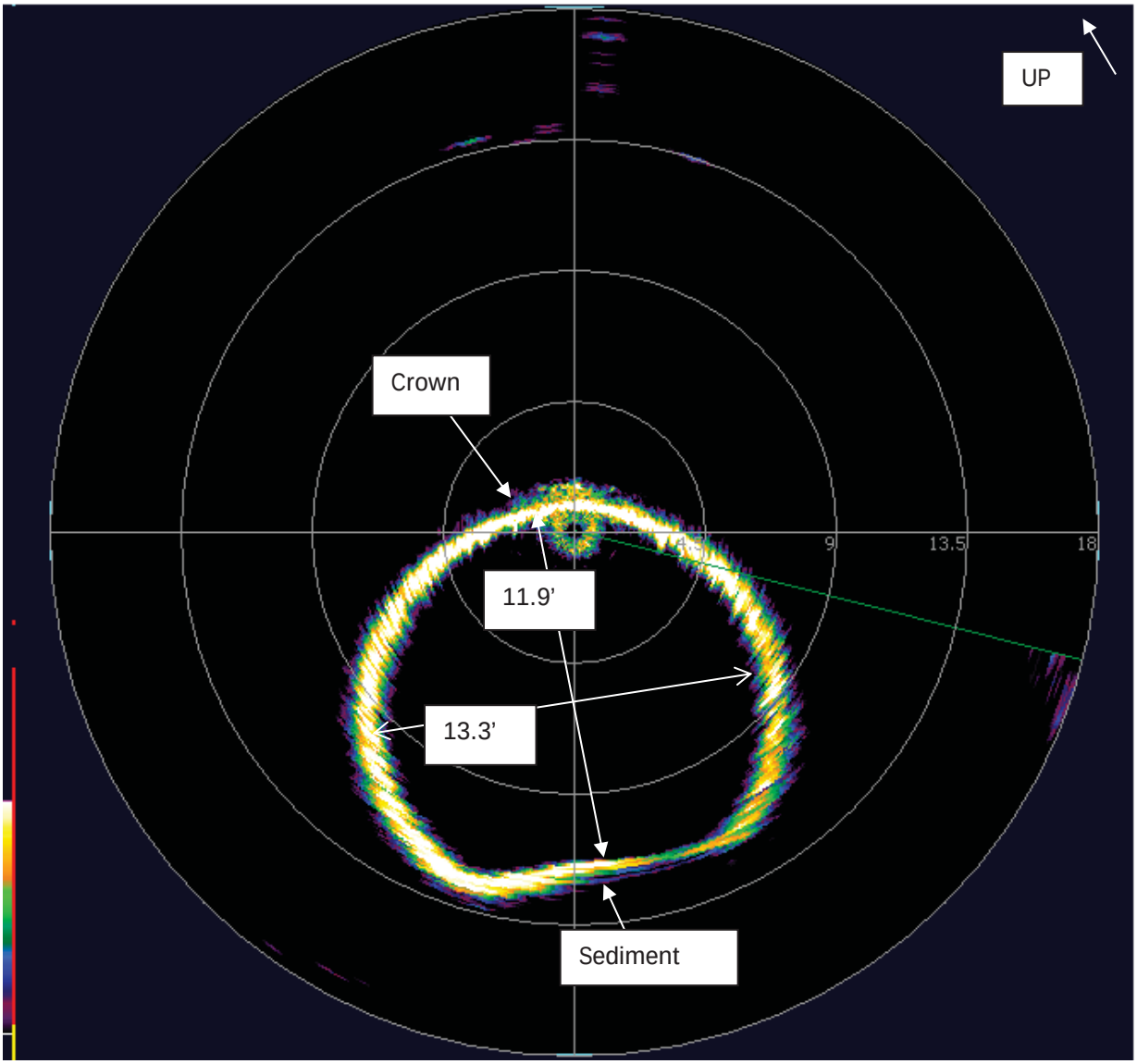

Figure 88 - Cross Section 50 -1200 feet.

Cross section at 1250 ft.

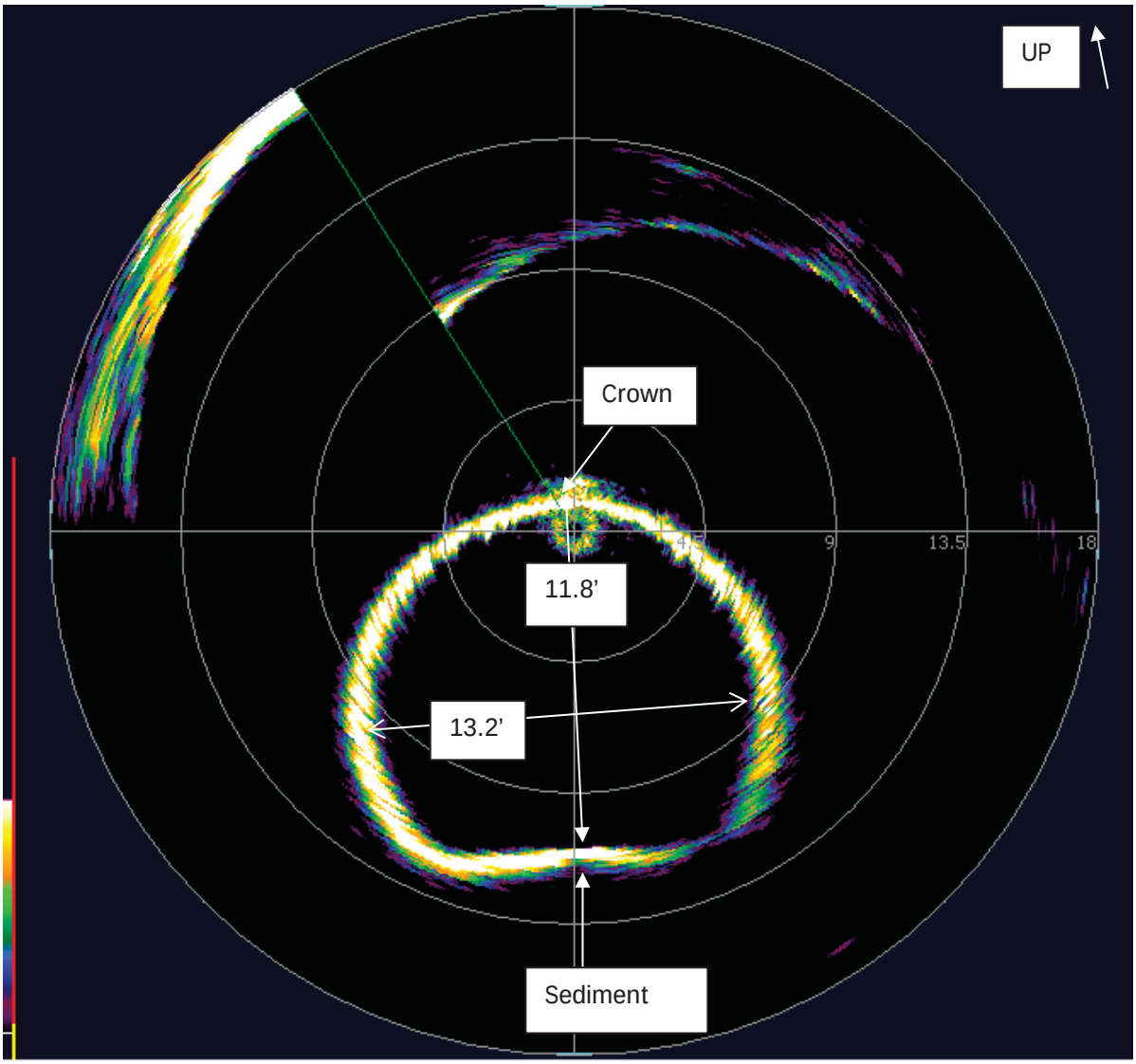

Figure 89 - Cross Section 51 - 1250 feet.

Cross section at 1300 ft.

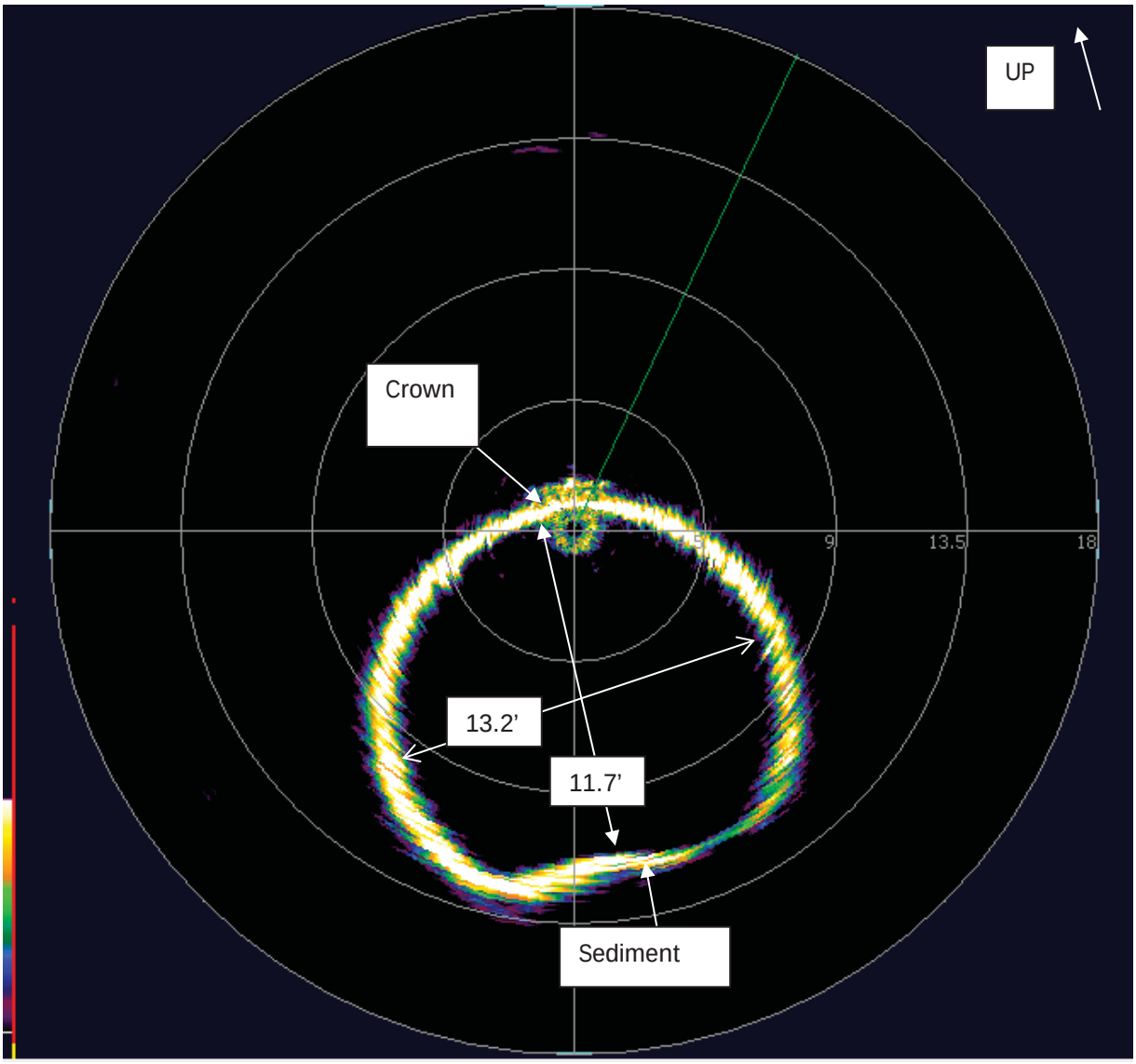

Figure 90 - Cross Section 52 - 1300 feet.

Cross section at 1350ft.

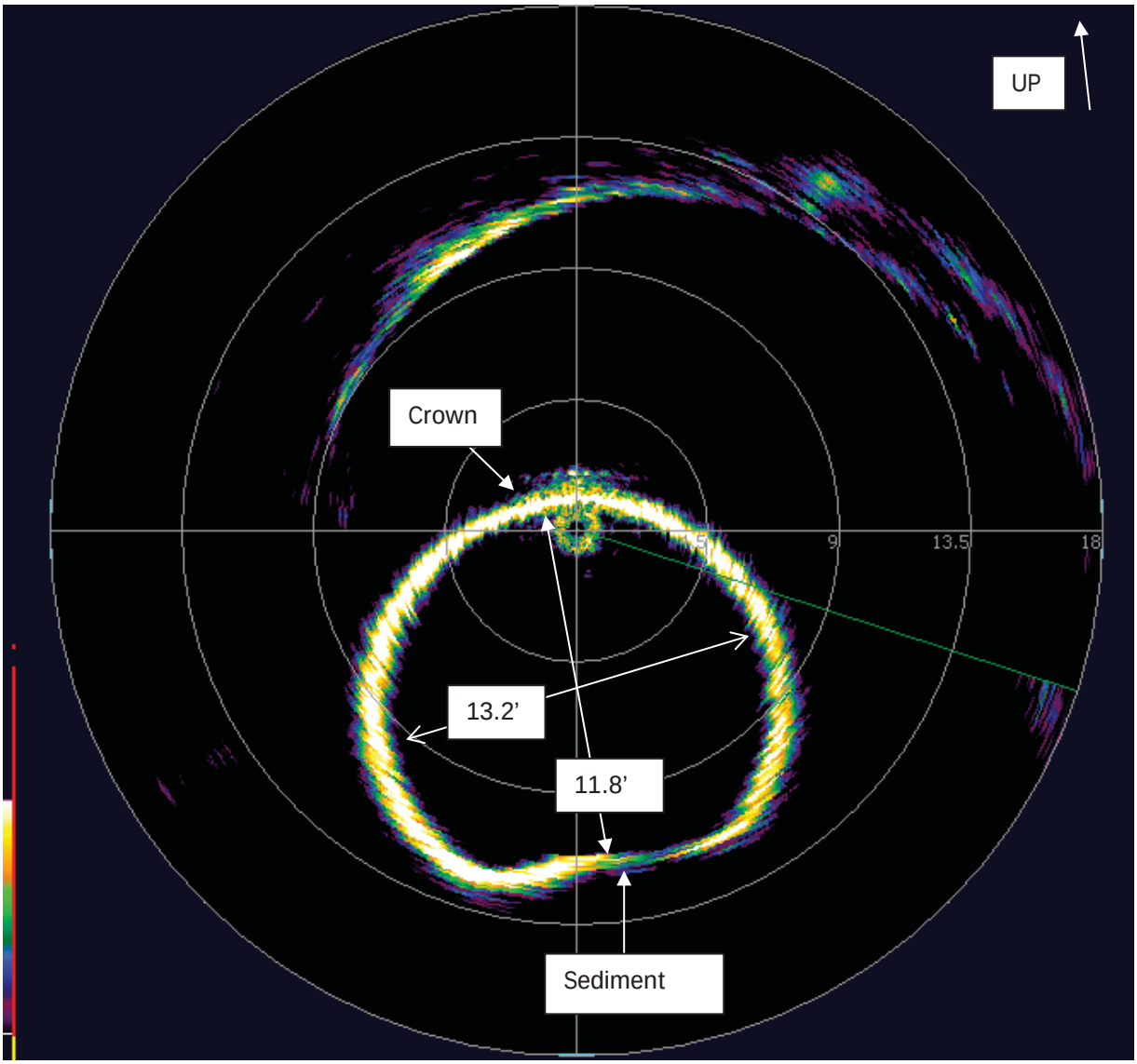

Figure 91 - Cross Section 53 - 1350 feet

Cross section at 1400 ft.

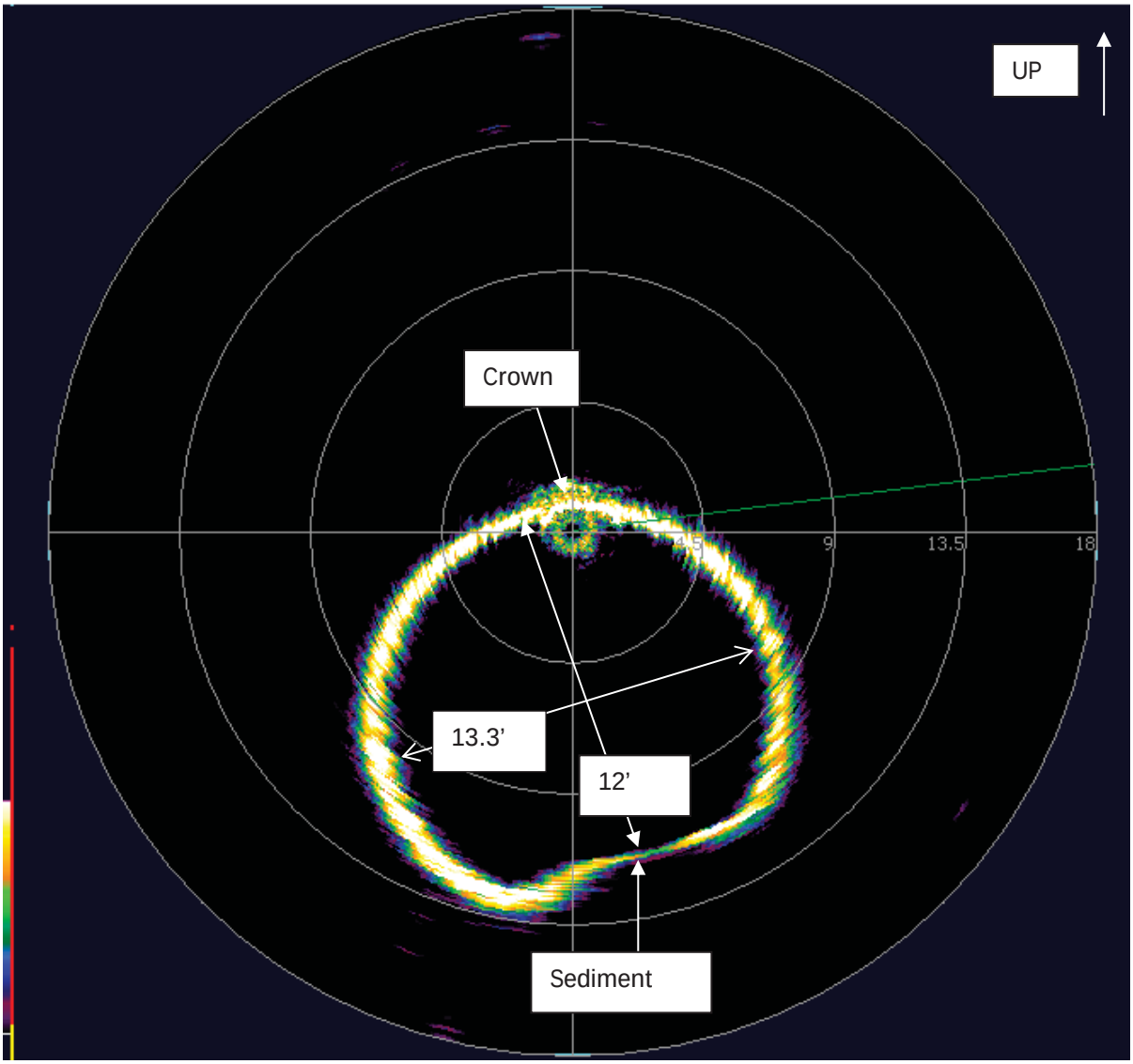

Figure 92 - Cross Section 54 - 1400 feet.

Cross section at 1450 ft.

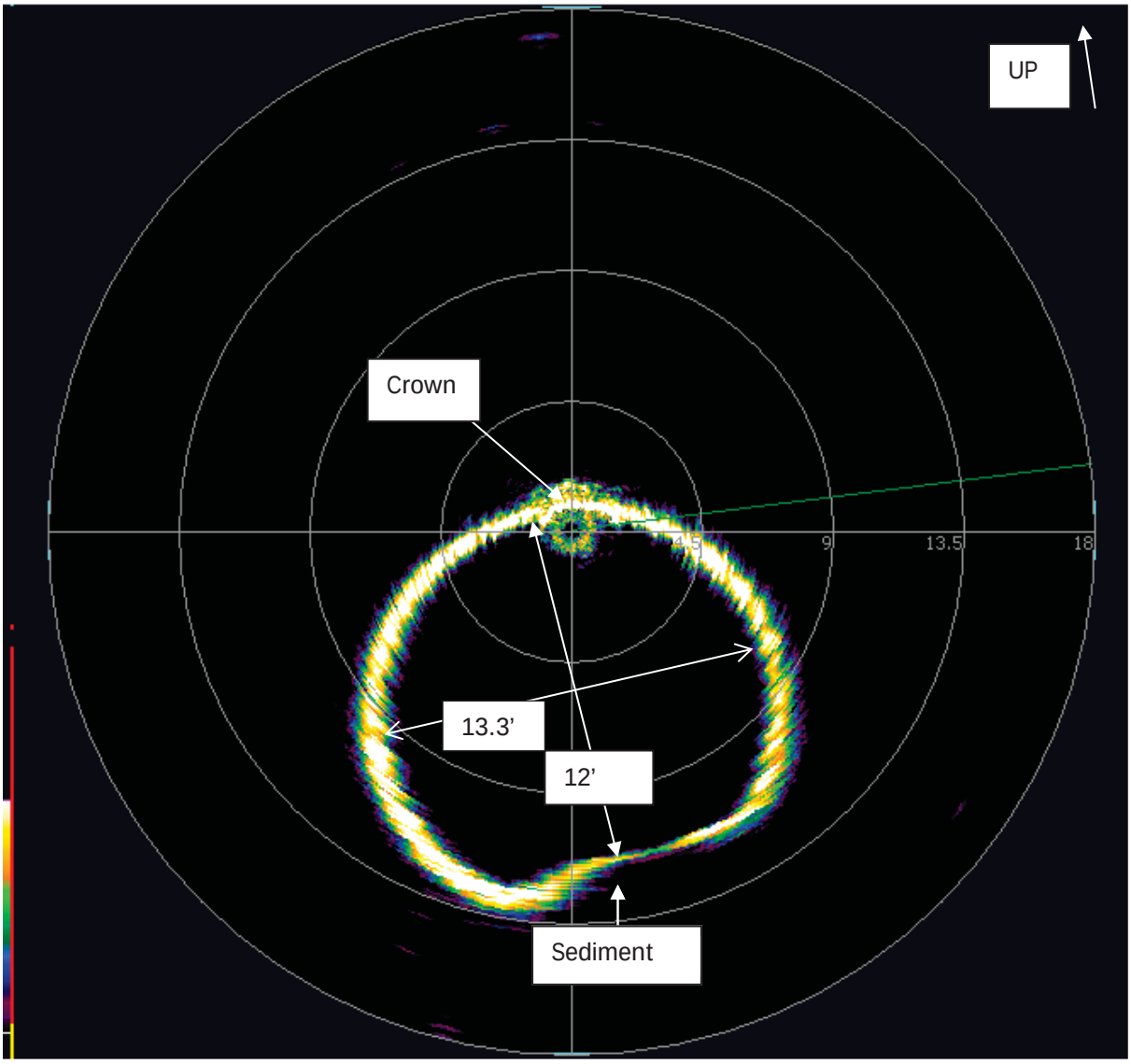

Figure 93 - Cross Section 29 - 1450 feet.

Cross section at 1500 ft.

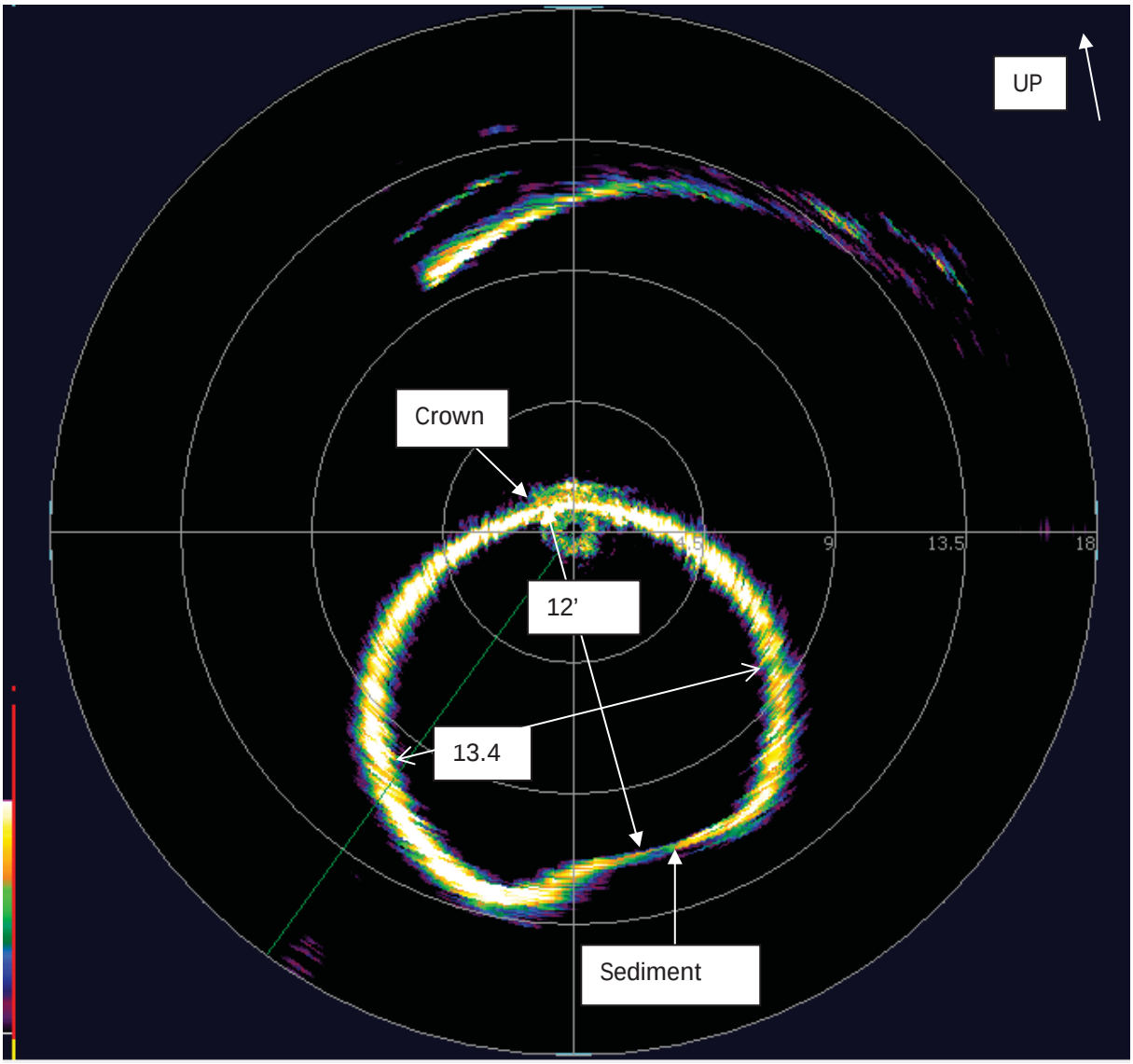

Figure 94 - Cross Section 30 - 1500 feet.

Cross section at 1550 ft.

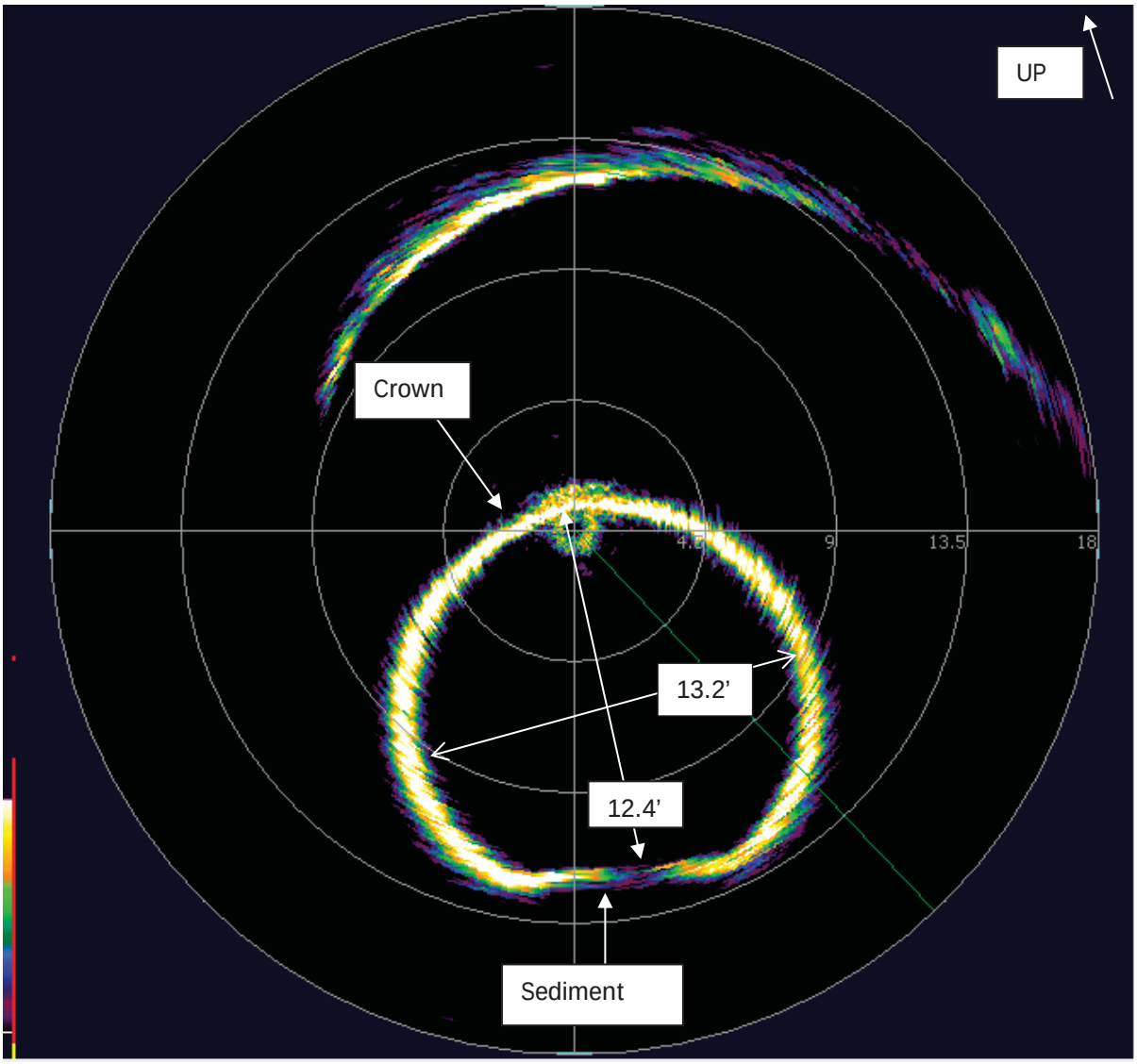

Figure 95 - Cross Section 31 - 1550 feet.

Cross section at 1600 ft.

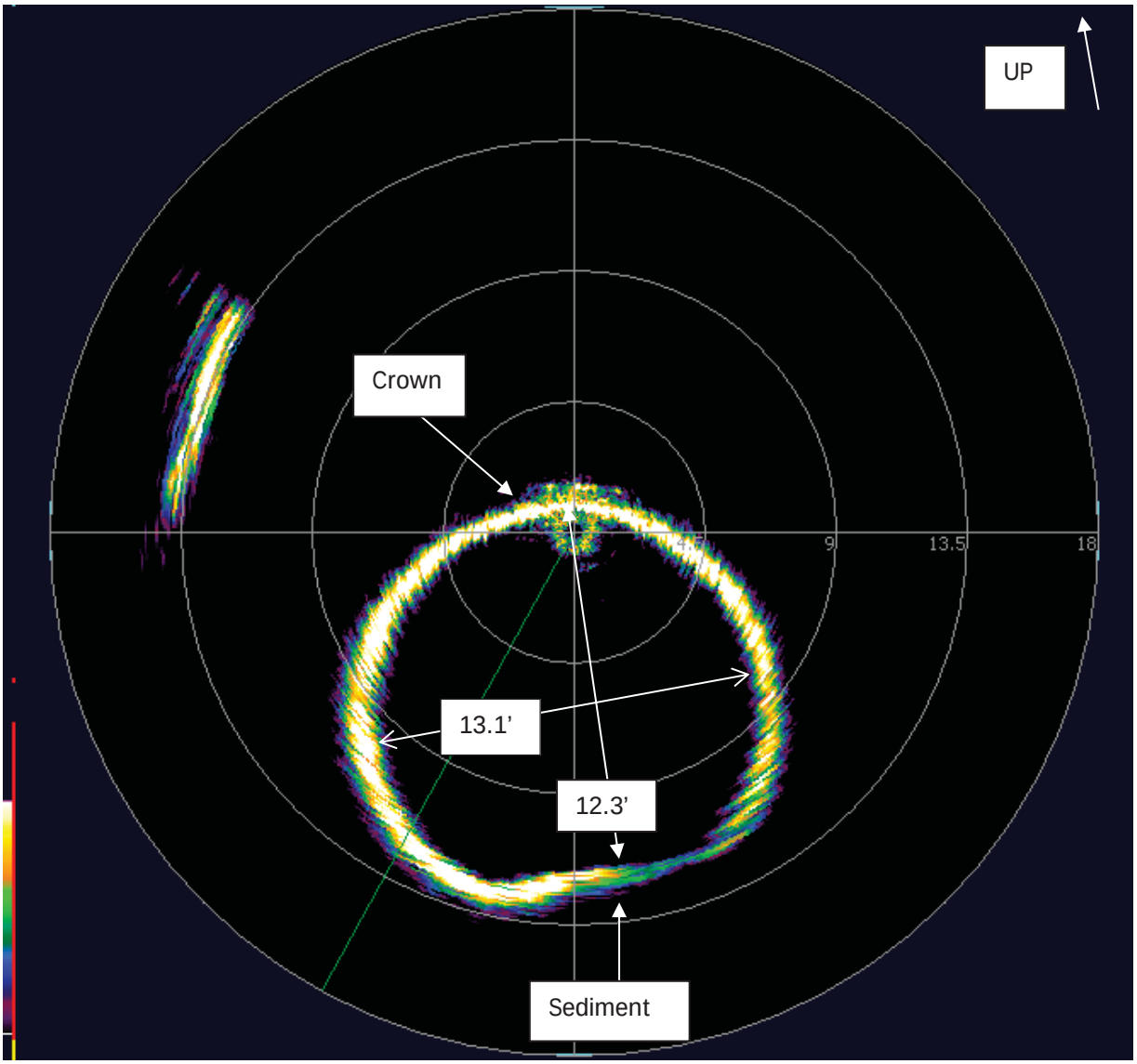

Figure 96 - Cross Section 32 - 1600 feet.

Cross section at 1650 ft.

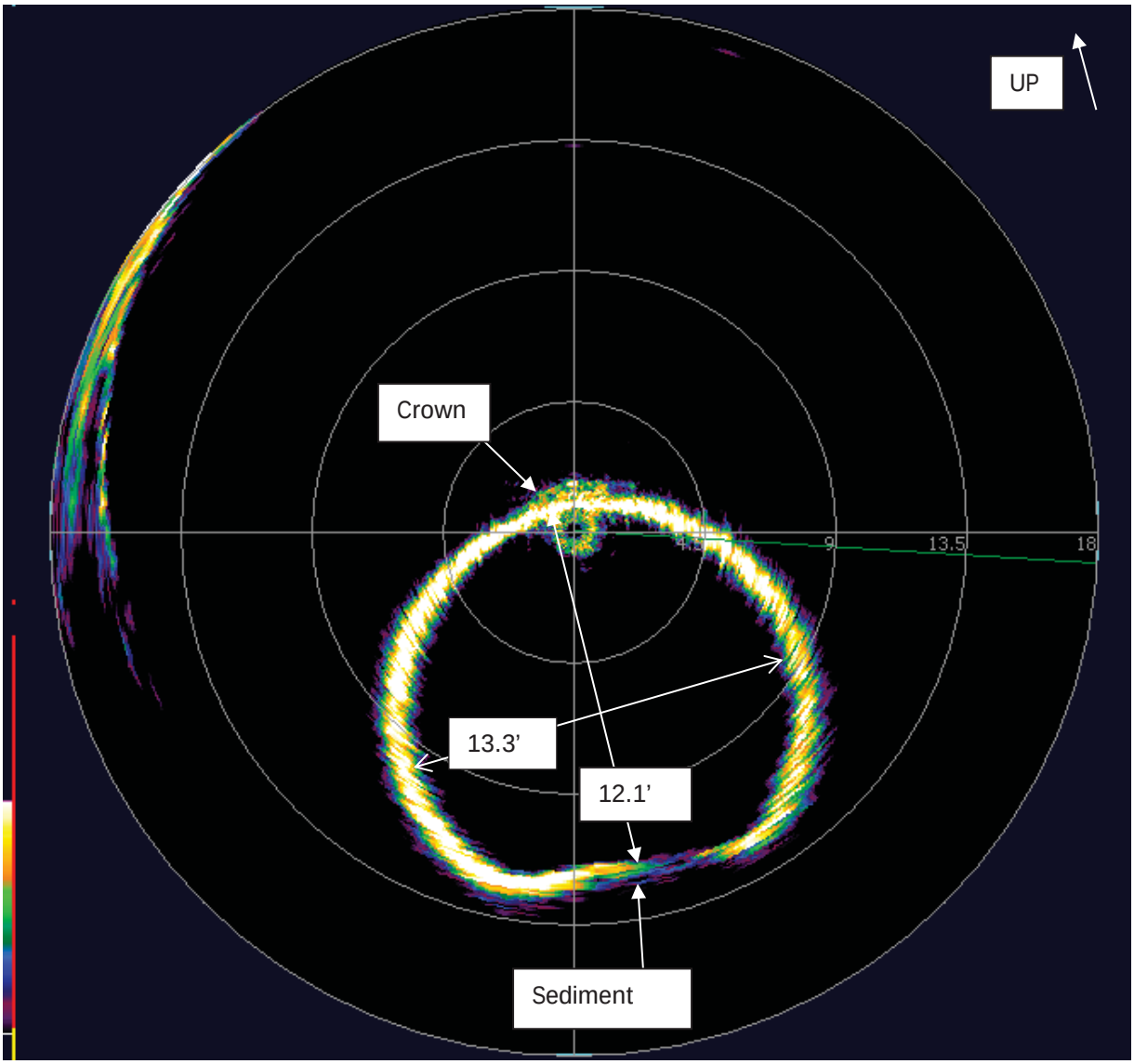

Figure 97 - Cross Section 33 - 1650 feet.

Cross section at 1700 ft. 13.5’ wide by 12.1’ from invert to crown.

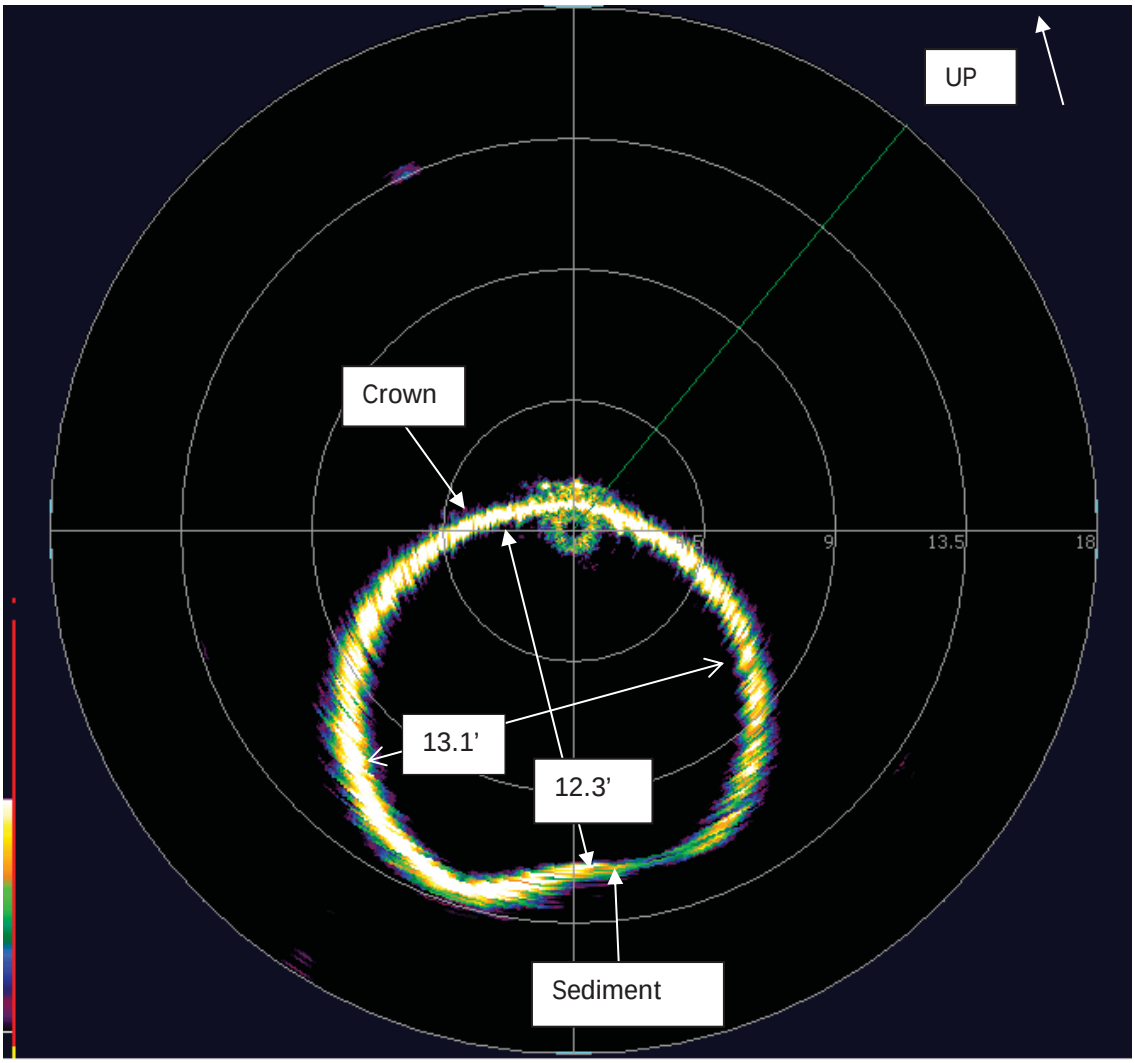

Figure 98 - Cross Section 34 - 1700 feet.

Cross section at 1750ft

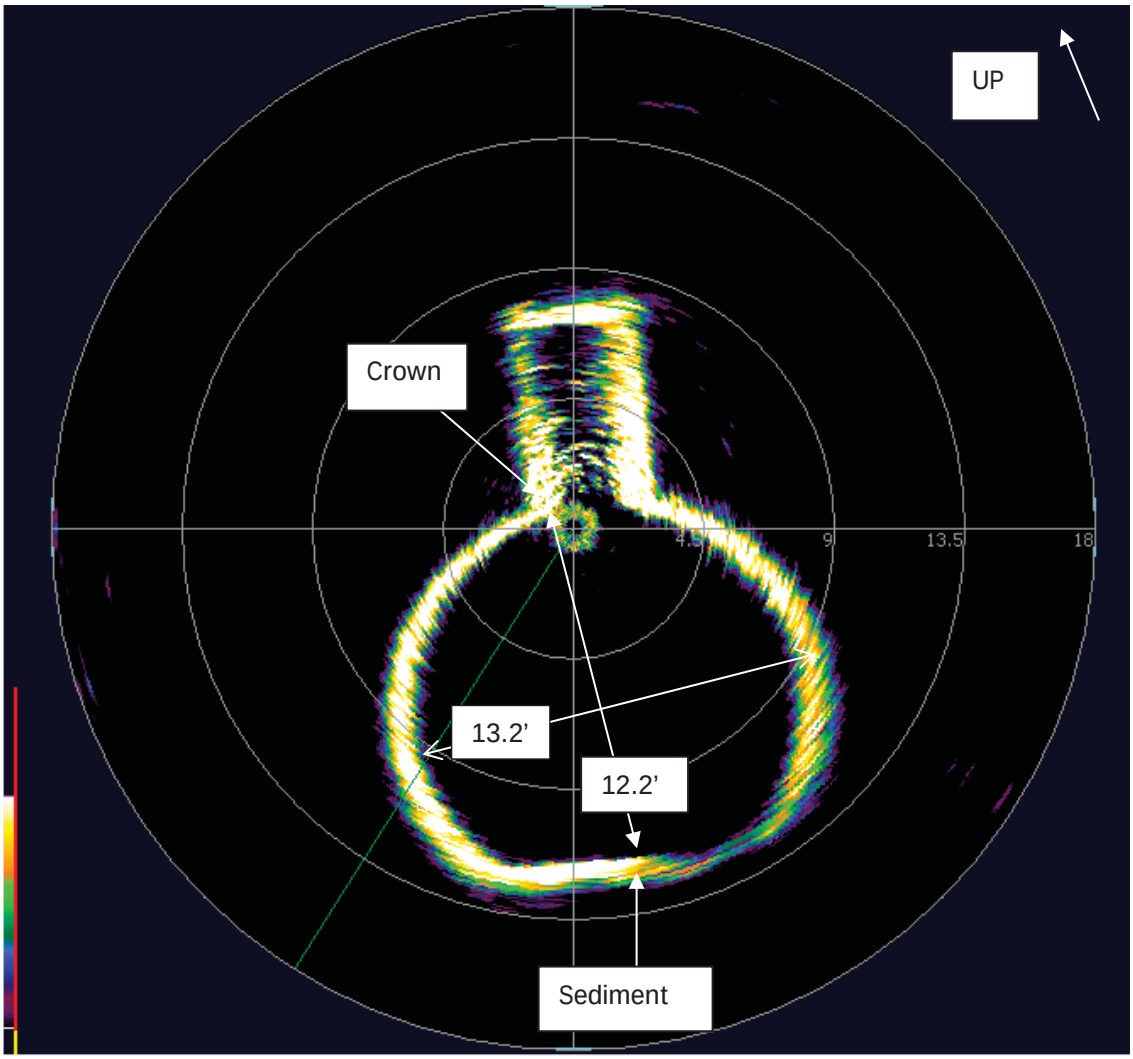

Figure 99 - Cross Section 35 - 1750 feet.

Cross section at 1800 ft..

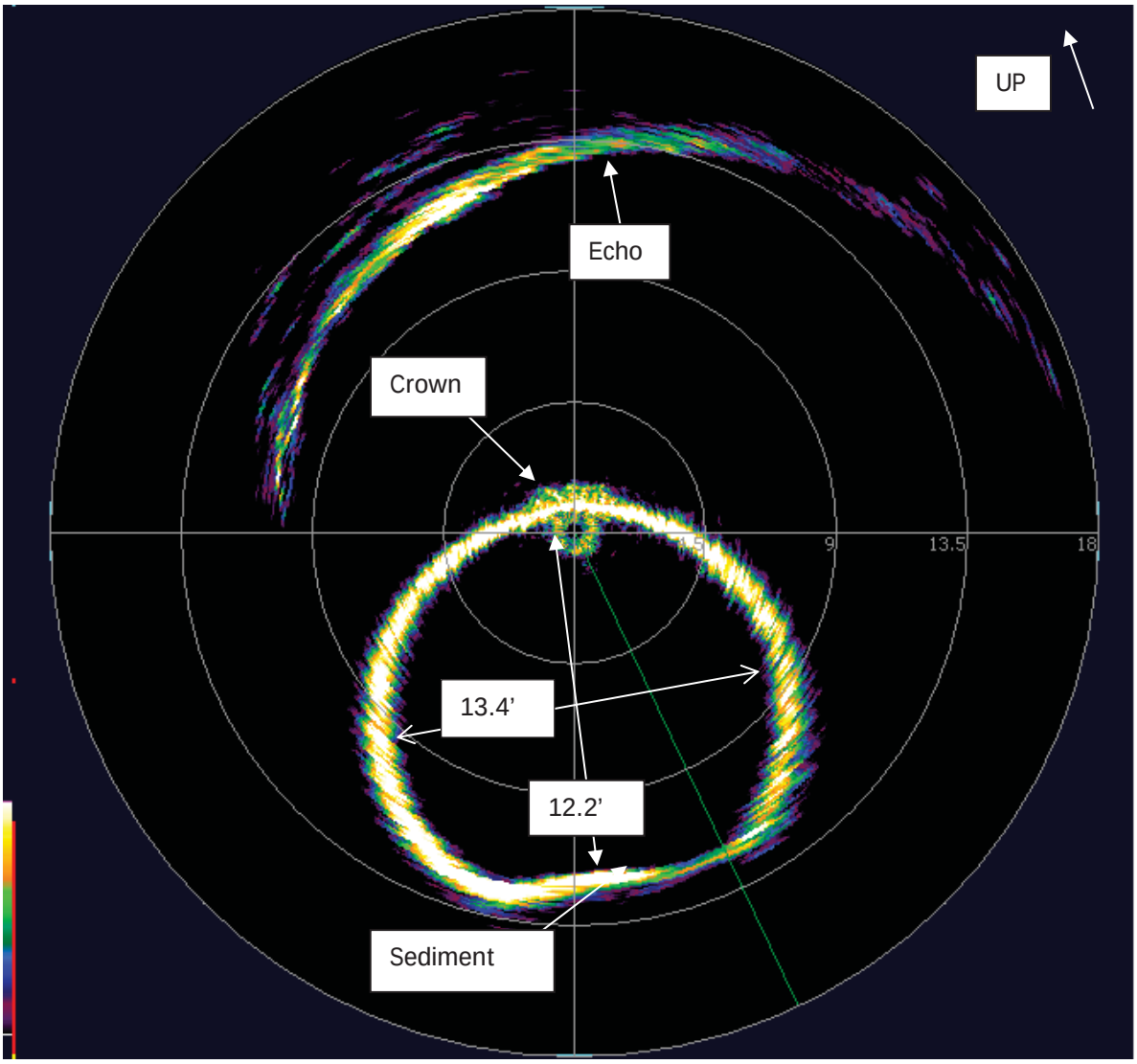

Figure 100 - Cross Section 36 - 1800 feet.

Cross section at 1850ft.

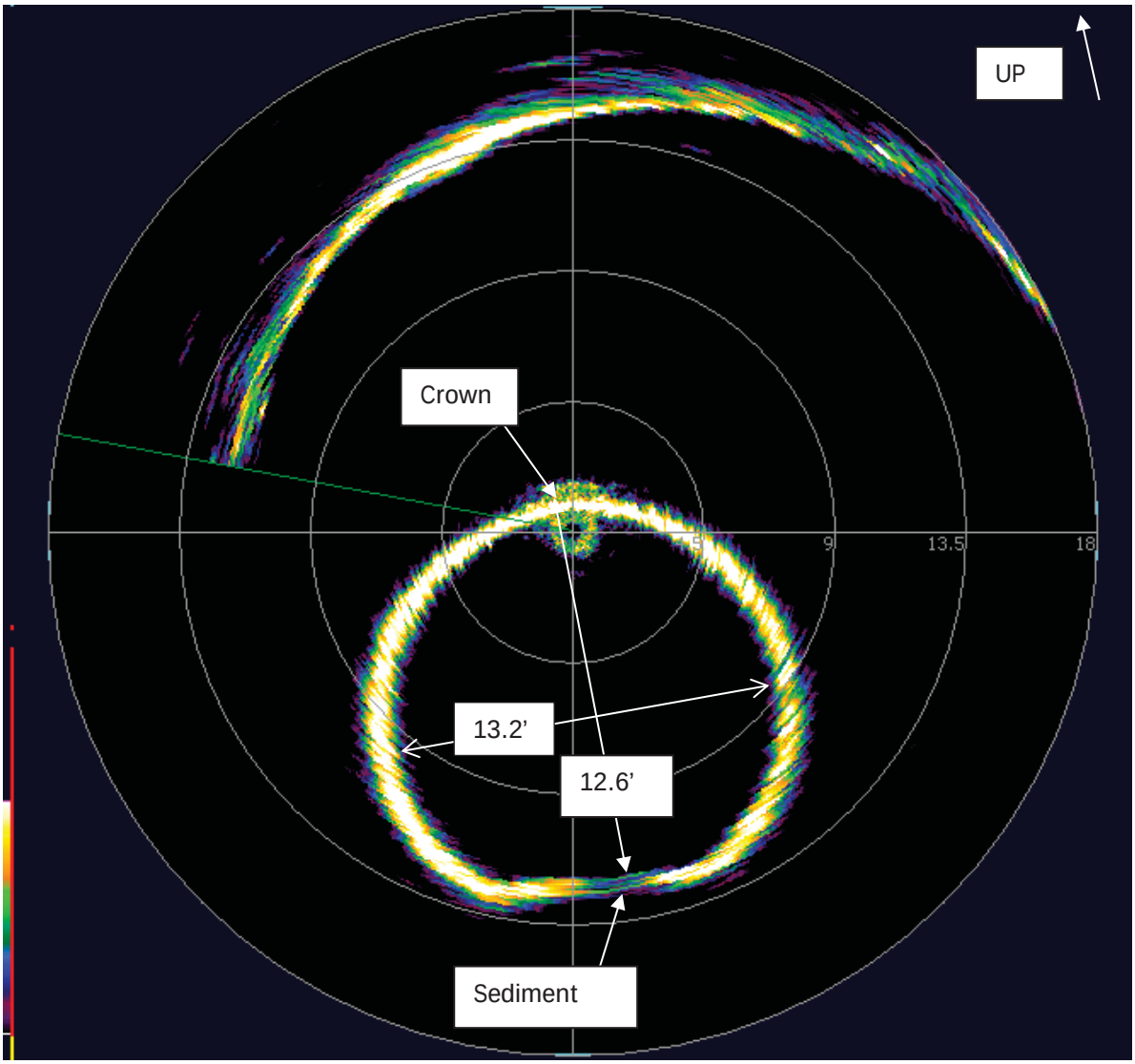

Figure 101 - Cross Section 37 - 1850 feet.

Cross section at 1900 ft.

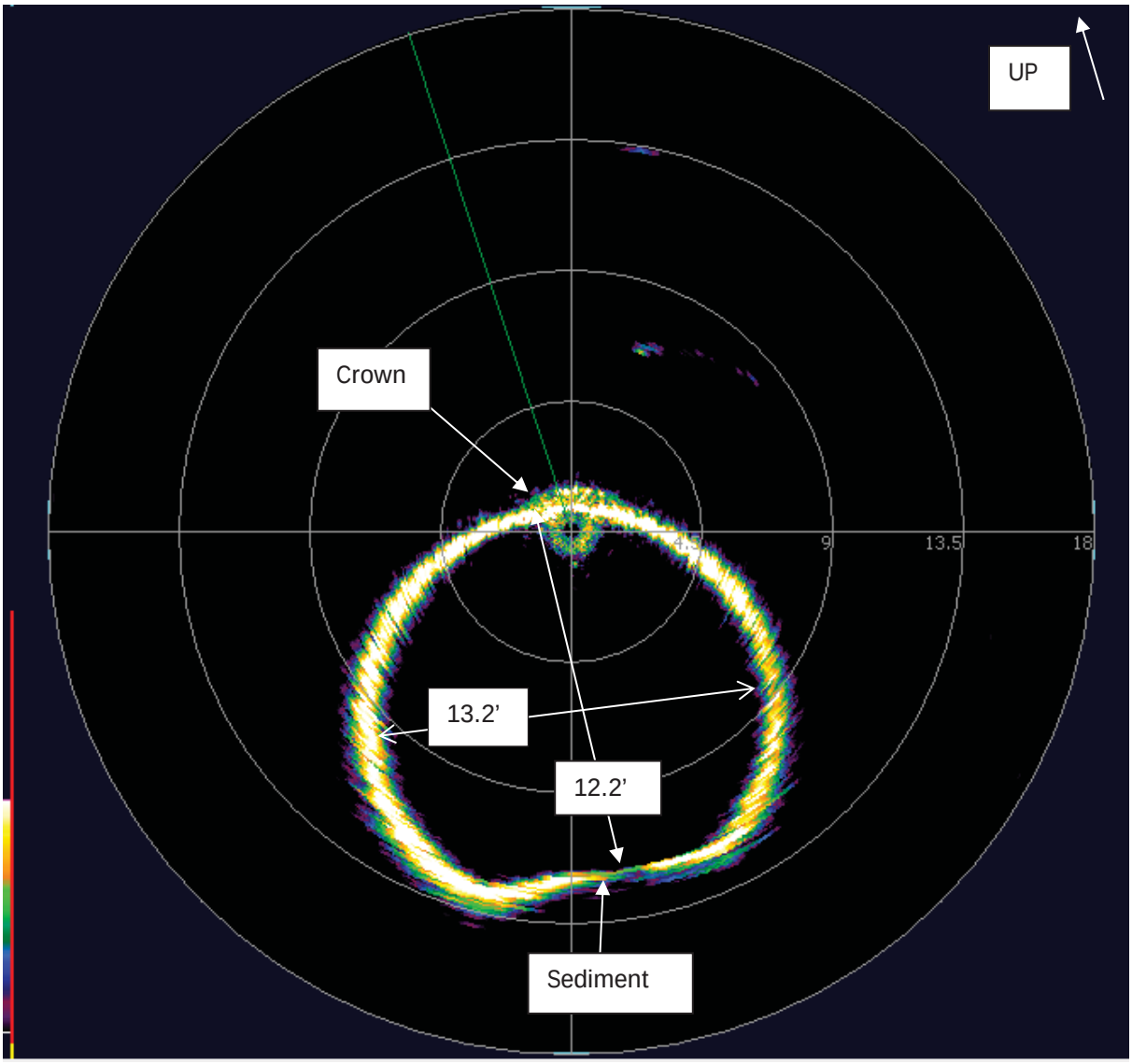

Figure 102 - Cross Section 38 - 1900 feet.

Cross section at 1950ft

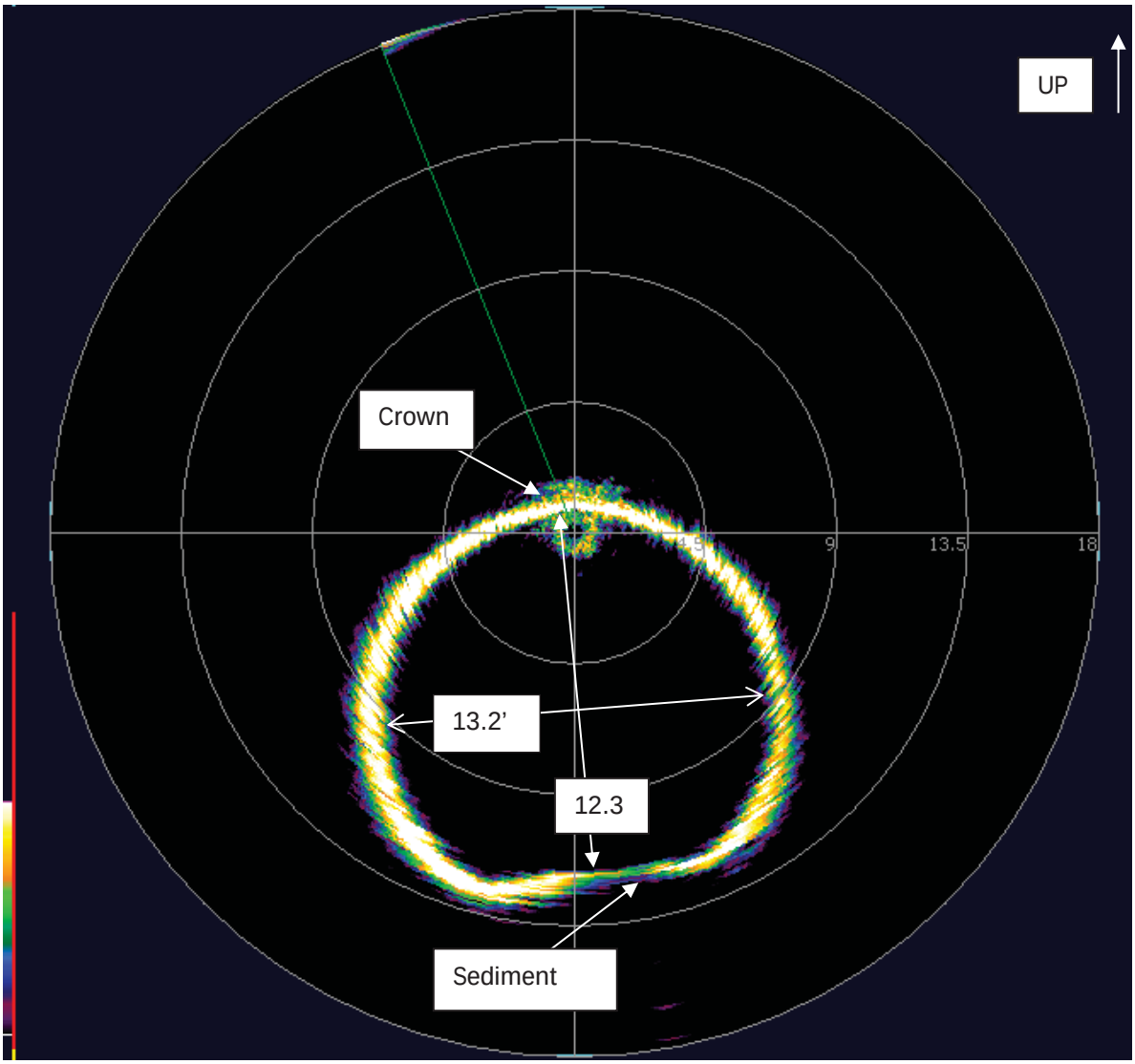

Figure 103 - Cross Section 39 - 1950 feet.

Cross section at 1990 ft.

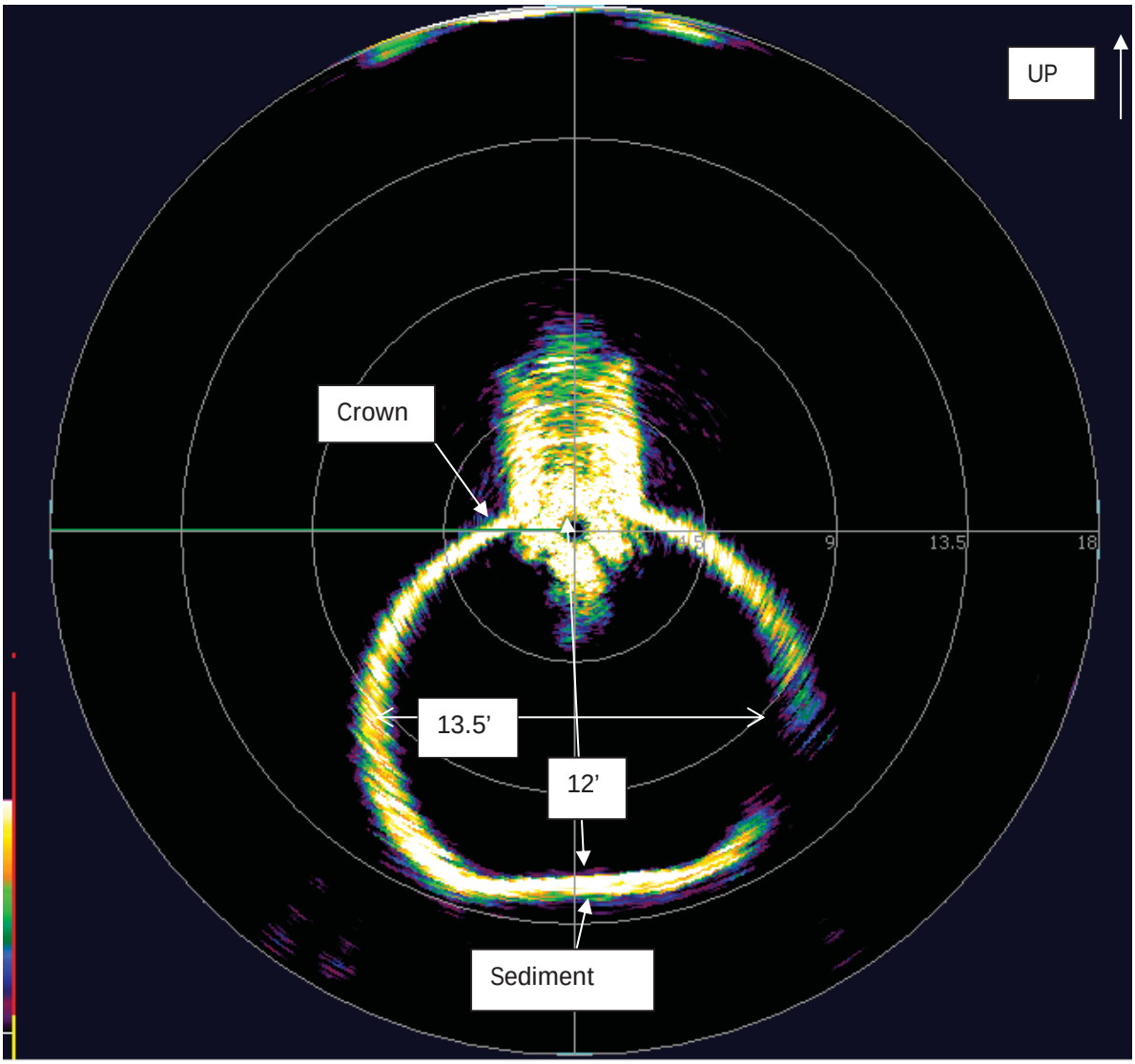

Figure 104 - Cross Section 39 - 1990 feet.

Cross section at 2050 ft.

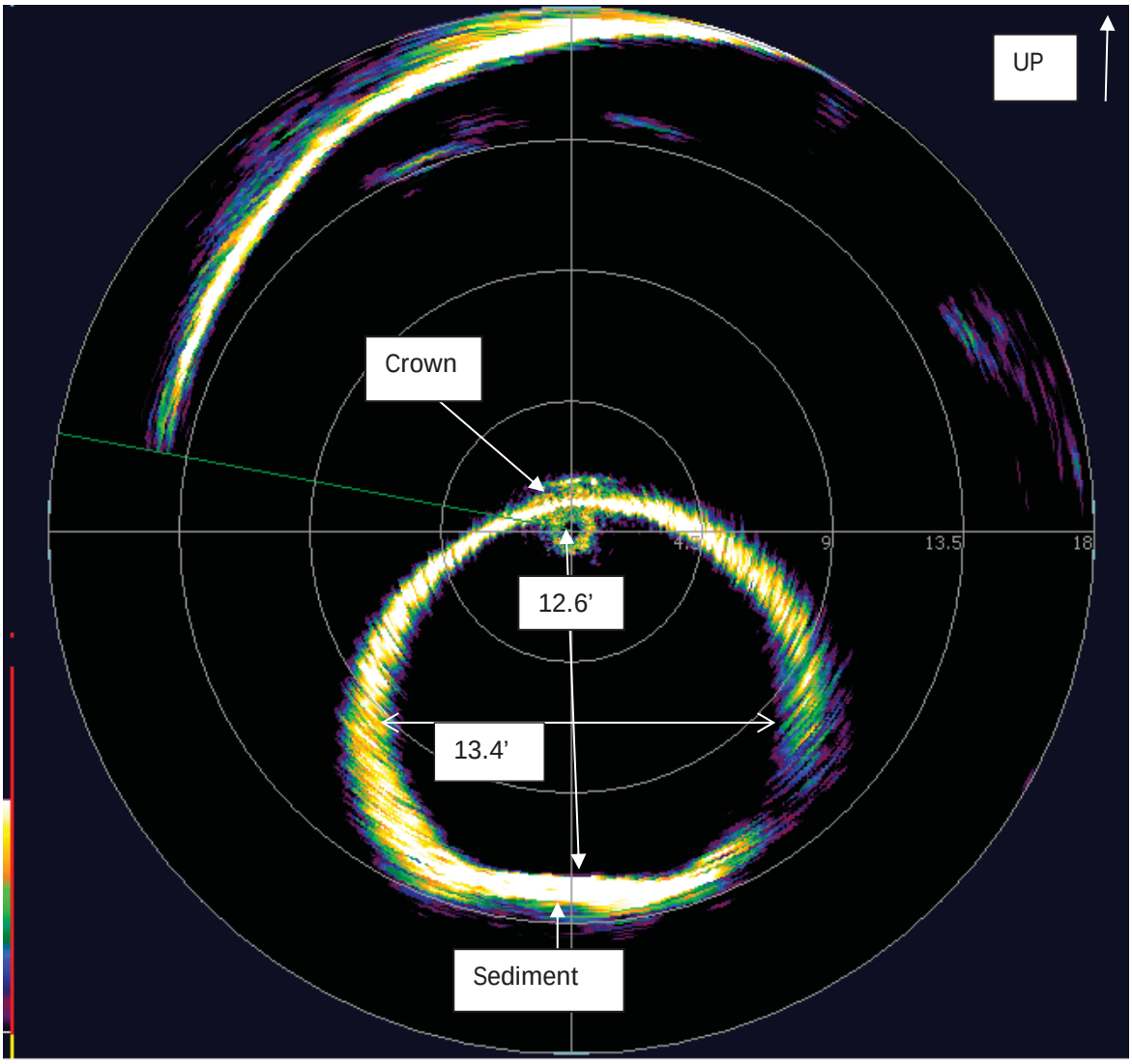

Figure 105 - Cross Section 41 - 2050 feet.

Cross section at 2100 ft.

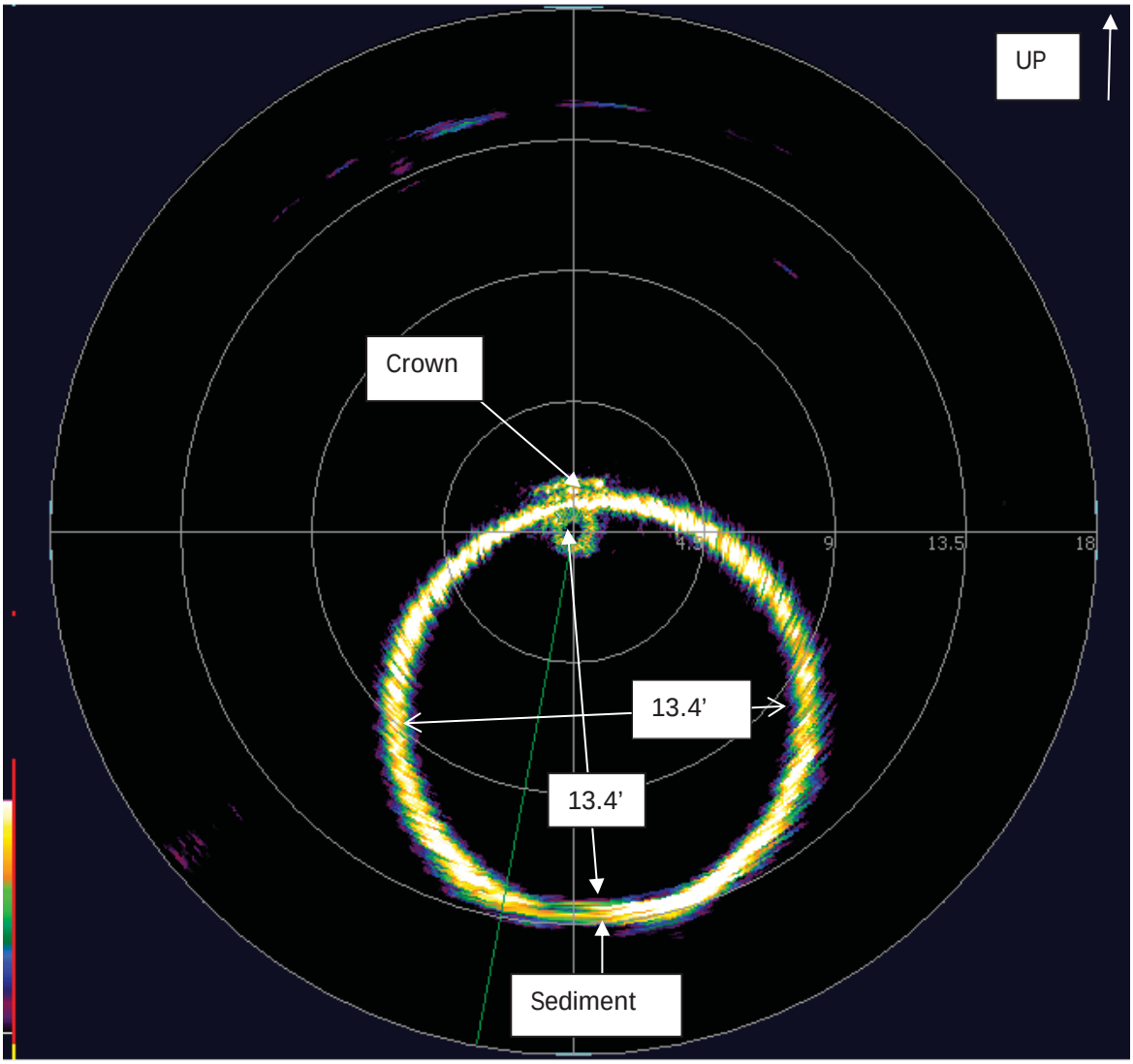

Figure 106 - Cross Section 41 - 2100 feet.

3.3 Marine growth throughout intake

Figure 107 - Marine growth at 100ft (intake)

Figure 108 - Marine growth at 500ft (intake)

Figure 109 - Marine growth at 1000ft (intake)

Figure 110 - Marine growth at 1500ft (intake)

Figure 111 - Marine growth at 2000ft (intake)

Figure 112 - Marine growth at 2450ft (intake)

3.4 Marine growth throughout discharge

Figure 113 - Marine growth at 100ft (Discharge), small air bubble

Figure 114 - Marine growth at 500ft (Discharge)

Figure 115 - Marine growth at 1000ft (Discharge)

Figure 116 - Marine growth at 1500ft (Discharge)

Figure 117 - Marine growth at 2000ft (Discharge)

3.5 Notable features throughout both intake and discharge

Figure 118 - Construction seam at 2484ft of intake

Figure 119 - Construction joint at 116ft of Discharge

Figure 120 - Large pocket of air at 423-429ft of Discharge

Figure 121 - Possible crack at 197ft of Discharge

Figure 122 - Two studs located at 2169ft of Discharge

4 Appendices

4.1 Appendix #1 – ROV Specification

LBV-6OO-6 MiniROV System

Little Benthic Vehicle

The compact size and increased depth capability of the LBV6006 make it an ideal rapid response, deep-water system. Four powerful forward thrusters and small diameter tether ensure true deep-water capability.

General
Depth Rating: 600m (2,000ft)
Length: 540mm (21.3in)
Width: 484mm (19.1in)
Height: 270mm (10.6in)
Diagonal: 554mm (21.8in)
Weight in air: 15.3kg (33.8lbs)
Configuration: 4 forward, 1 vertical, 1 lateral
Motor Type: Brushless DC direct drive
Bollard Thrust: 4.9kgf (10 ft. /lbs.) each
Speed at Surface: 3.5 knots (1.54 m/sec)
Max Operating Current: 2.5 knots (1.03m/sec)

Tether/Reel
Diameter: 8mm (0.3in) nominal
Length: 1,000m (3,280ft) standard
Buoyancy: Neutral in seawater

Cameras/Lighting
Range of View: 270 Degrees
Focus: Manual (90mm to infinity)
Format: NTSC or PAL
Cameras: 520 line High resolution color and 580 line Monochrome
Transmission Type: Fiber optic
Lighting: Internal 700 Lumen LED tracks color camera and additional internal 700 Lumen LED tracks monochrome camera
Control System
Configuration: Rugged case with weatherproof monitor and removable operator control unit
Monitor: 38cm (15”) LCD
Power Requirement: 3,000 watts, 100-240 VAC
Safety: Isolated input, circuit breaker, LIM, leak monitor Meets & exceeds ‘Code of Practice for the Safe use of Electricity in Water”
Auto Functions: Depth, heading, trim
Video Overlay: Depth, heading, lights, thruster gain, turns counter, camera angle, time, date, and user programmable characters

Additional Standard Equipment
Grabber: Manipulator with multiple jaw options
Sonar: Scanning, profiler
Console: Integrated Navigation and Control Console
Lights: 2 head ultra-bright LED

4.2 Appendix #2: Visualsoft DVR System

The video, sonar and written notes would be recorded with a computer such that all components of the inspection are synchronized. This simplifies reviewing the data after the inspection. The picture below shows the computer screen with two video channels, two sonar systems and written notes. By selecting a note in the list, all of the video and sonar channels are played from that location. This makes reviewing important features much faster without having to search through many disks or tapes. The data is supplied on a portable hard drive that plugs into a USB port on a PC computer. The portable drive includes viewing software.

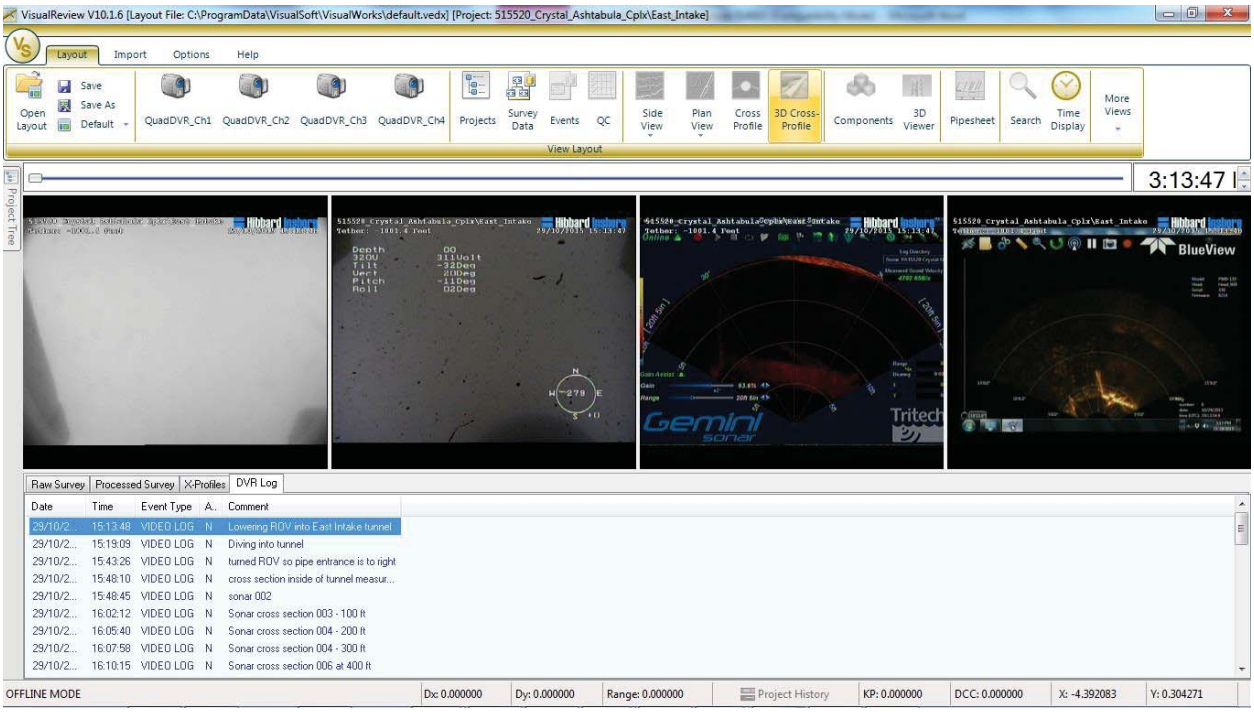

Figure 123 - VisualSoft Layout

The middle panes are the videos. If one of the videos is not showing ensure that the “QuadDVR_Ch1”, “QuadDVR_Ch2”, “QuadDVR_Ch3”, and “QuadDVR_Ch4 icons are selected in the top left pane of the layout tab. If you double click on any of the videos it will pull that video out as a separate window. Double clicking it again will return the video to its original spot.
To the left of the video panes is the “Project Tree” tab which allows for the selection of different projects or portions of projects. Note the “Projects” tab must be selected. Individual features can be selected in the lowest pane. The features are in chronological order by time that is overlayed on the video. Selecting a feature will jump the video to that feature.

When the mouse is hovering in front of any of the videos there are VCR-like controls visible that allow playing/pausing video, fast forward, rewind, frame forward, frame back, and play from start. The play from start button only plays from the start of the current 15 minute video section. All of the videos are linked, so selecting the controls on one video controls all the videos. The exception to this is the full screen button which will only expand the selected video to full screen

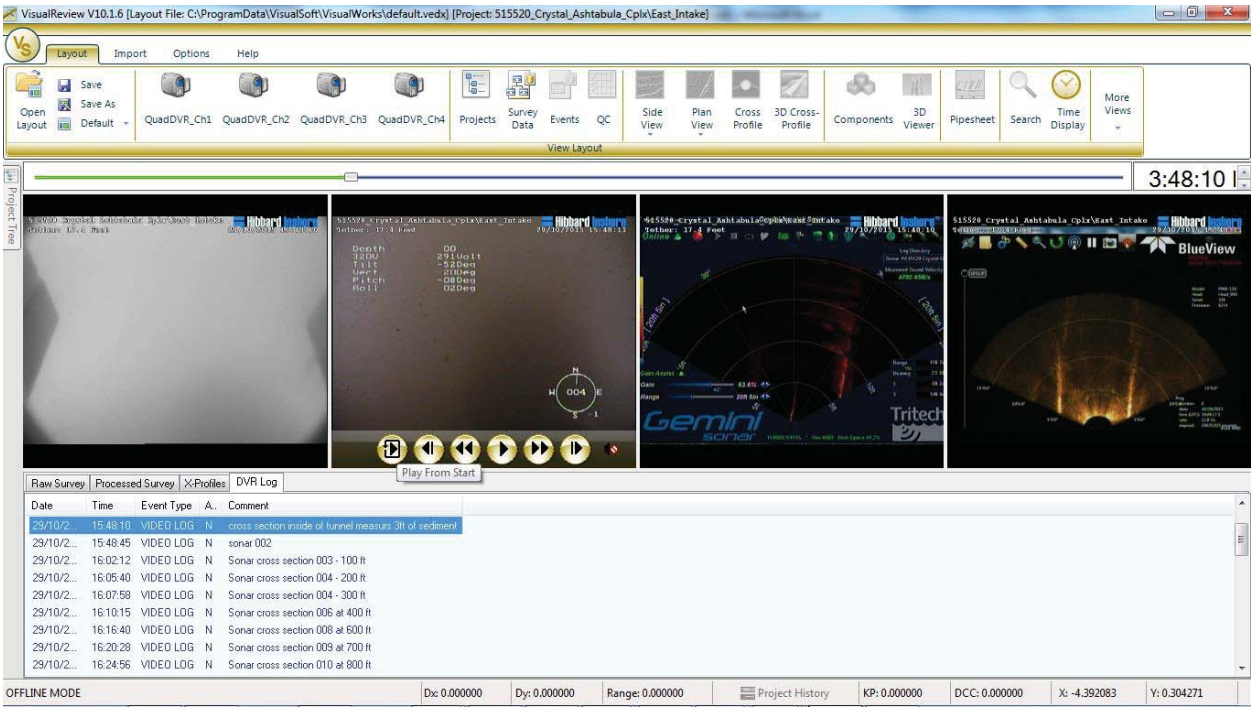

Figure 124 - VisualSoft Layout showing Video Controls

The blue and green bar above the videos displays the progress through the 15 minute section of video and pressing on any portion of the bar will jump to that location in the videos. The time shown next to the bar is the local time in 12 hour format for that place in the video.

4.3 Using the Data Reviewing Program: VisualReview.exe

4.3.1 Hard Drive Contents

All of the Data was delivered on a USB external hard drive. The drive also contains a program to view the data. Connect the drive to a Windows computer with the USB cable. Windows will recognize the external hard drive.

The drive contains the following file structure:

<Drive letter>:\Viewer
<Drive letter>:\VisualWorks
<Drive letter>:\Work

The folder titled “Viewer” contains the program to view the data from the inspection. The “VisualWorks” folder contains all of the data for the VisualReview program. The last folder titled “Work” contains data in standard formats that can be opened with standard programs. i.e. Microsoft Excel or Word.

4.3.2 Opening the Viewer Program

To open the “Viewer” program for the first time, you should open the folder “Viewer” and select “VisualReview Launcher” to start the application. This application will scan your computer to ensure all the required programs are on your computer and will allow you to download and install any missing prerequisites. Once this is done, you can simply open the folder “Viewer” and select “VisualReview” to start the program. Alternatively, you can copy the folder “Viewer” to the host computer and run the program. The program will run faster if run from the host computers local drive. A Windows XP or Windows 7 computer should be used. Once the program is opened, locate and load the data files. The drop down window should be used to locate the folder with the data files. Then the list of data sets will be visible, select as many data sets as desired to review.

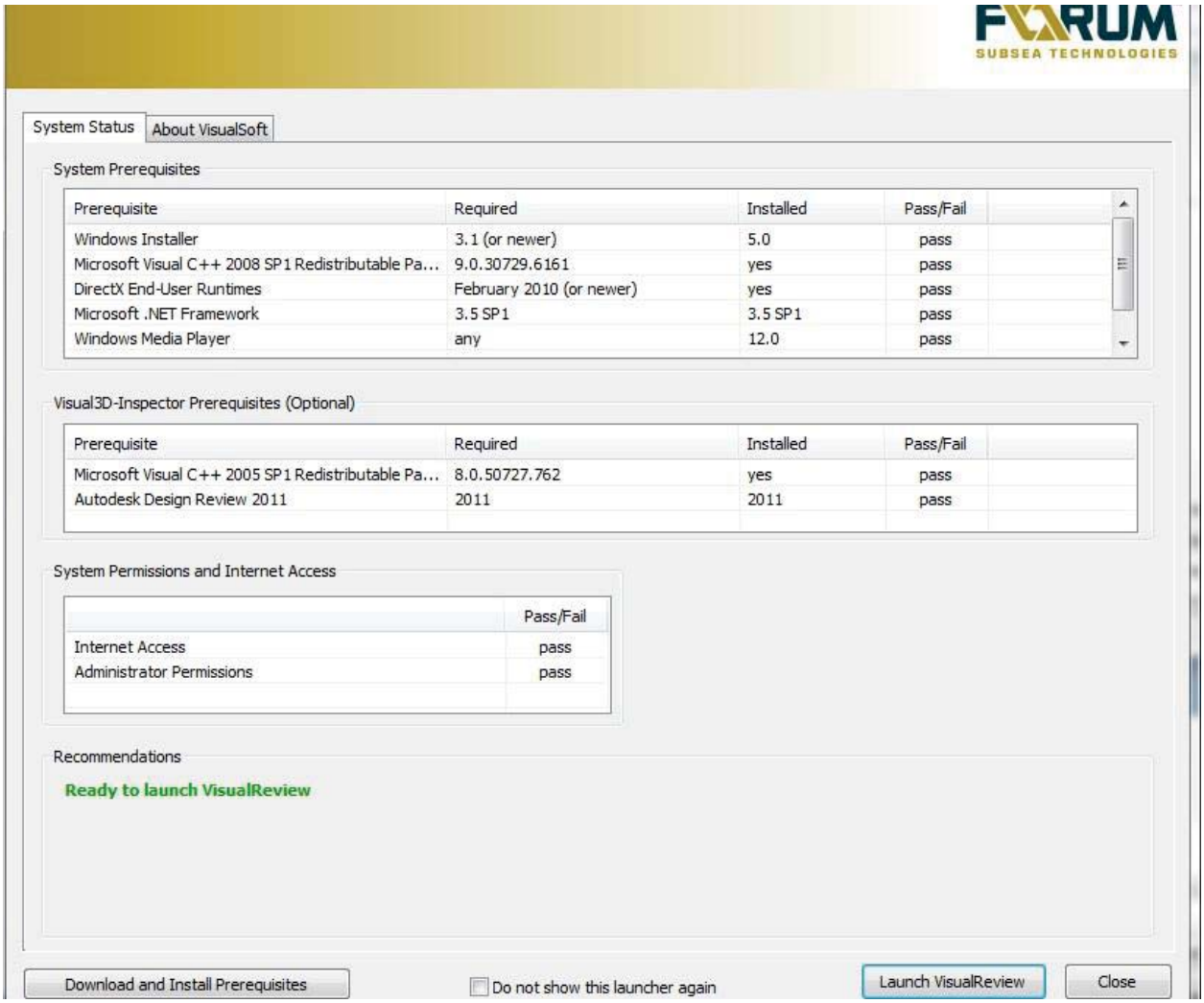

Figure 125 - VisualReview Launcher

4.3.3 Loading Data into Visual Review

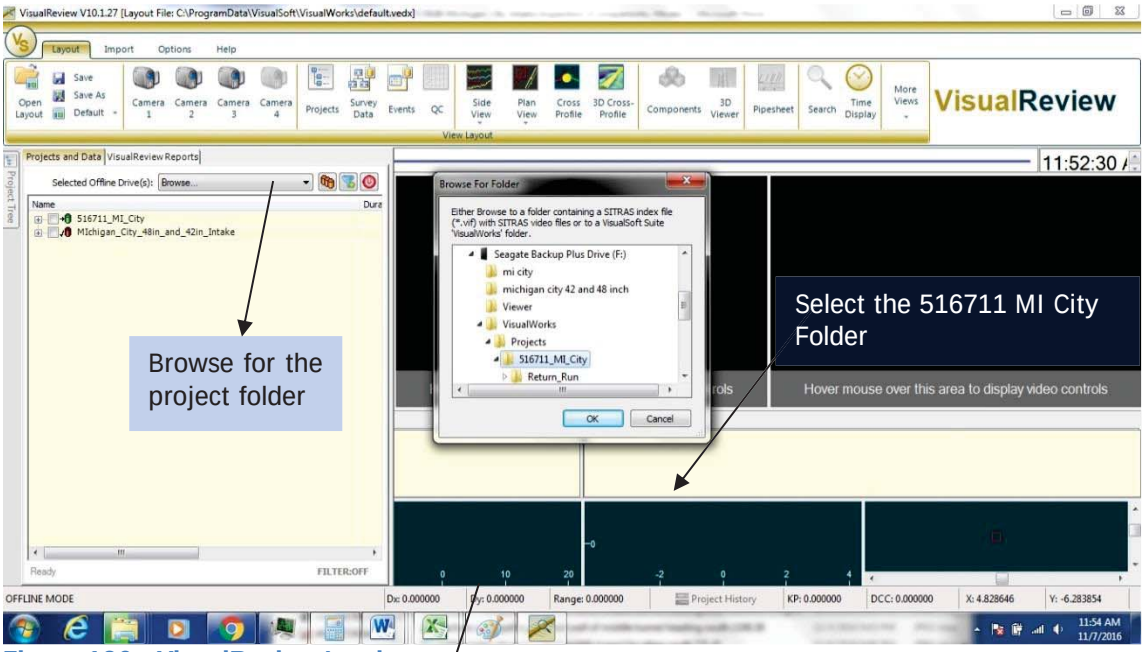

Figure 126 - VisualReview Load screen

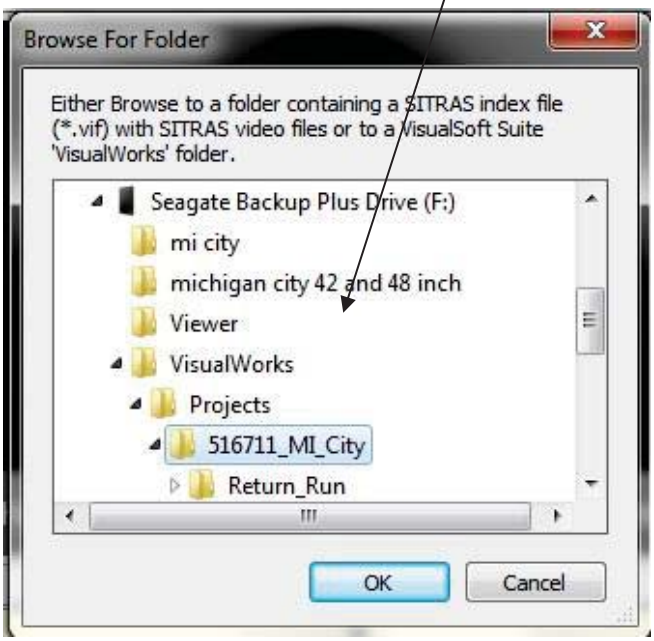

Figure 127 - Close up of Folder Selection

When the Data is loaded, there will be a list of data files in the window labeled “Projects”. If the “Projects” box is not visible, go to the top pane and select “Projects”. Select the check box to activate the data files. If the videos are not showing select “DualDVR_Ch1” and “DualDVR_Ch2” buttons in the top pane. The menu items that should be checked in the top pane are: “DualDVR_Ch1”, “DualDVR_Ch2”, “DualDVR_Ch3”, “DualDVR_Ch4”, “Projects”, “Survey Data”, and “Time Display”. At any time, a note can be double clicked in the “DVR Log” tab and all of the data will start to play from that location. The data can be paused, fast forwarded, etc. with the VCR style controls displayed over the videos. The notes

contain the umbilical payout, observations and ROV location. The umbilical payout is shown on all the camera overlays in feet. Depth is the water depth of the ROV in feet. The figure below shows the way that the data viewing program should look when reviewing data.

By right clicking on a video channel image, a still image can be grabbed or a video clip can be created for exporting to reports, etc. The notes were already exported to MS Excel as this functionality is not available in this free viewer. These notes are included with the rest of the data on the drive in the "Work" folder.

4.3.4 Review Data

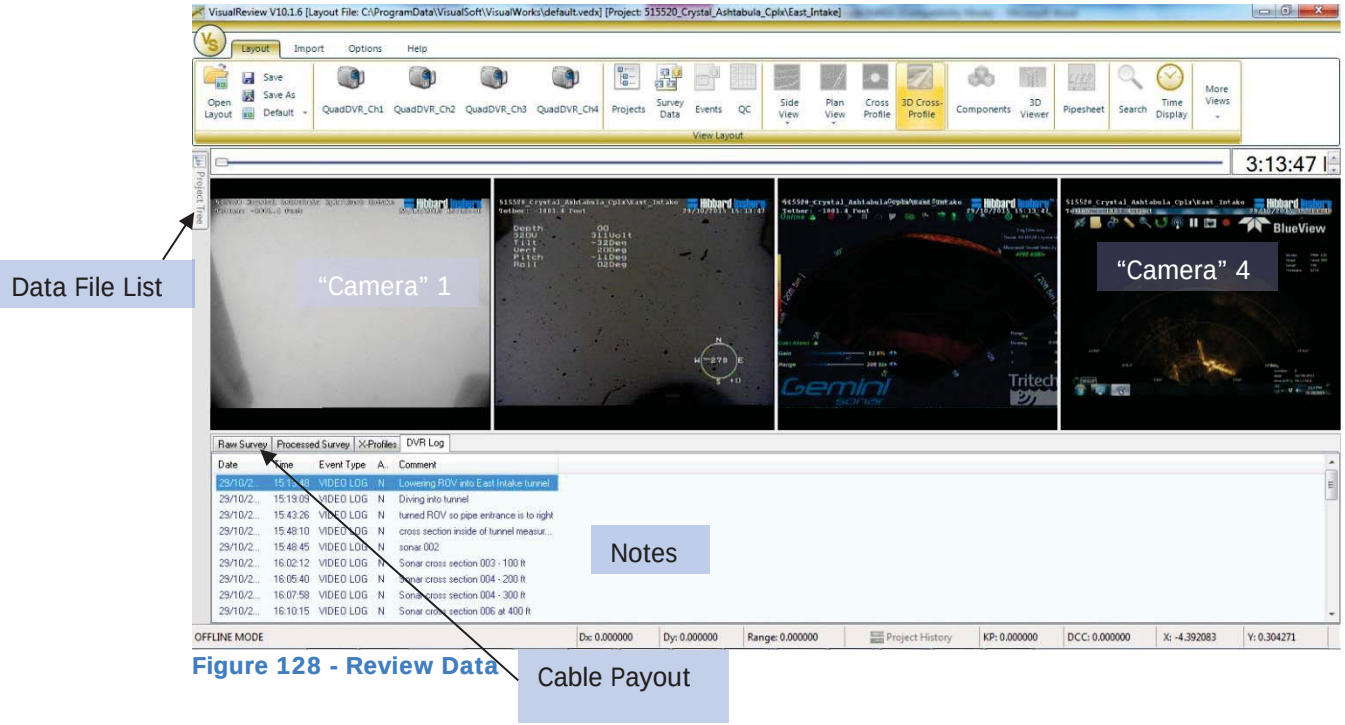

Figure 128 - Review Data

4.4 Appendix #3: Pilot Notes

Table 1 – Feb 13, 2018. Intake Tunnel.

Date	Time	Tether Ft	Comment
13/02/2018	0:26:28		mouth of intake plant side
13/02/2018	0:26:52		start of tunnel becoming visible on sonar
13/02/2018	0:27:16		transition from square to round tunnel
13/02/2018	0:30:52		50ft
13/02/2018	0:31:01	50.15	capturing profile
13/02/2018	0:31:27	50.15	crown to sediment level 12.1ft
13/02/2018	0:35:28	98.31	construction joint at 100ft
13/02/2018	0:36:49	105.21	right side of tunnel construction joint
13/02/2018	0:37:08	105.21	attempting full circumfrance
13/02/2018	0:41:08	105.21	cross section captured
13/02/2018	0:41:18	106.21	12.04 crown to sediment line
13/02/2018	0:42:10	106	note bluview connectivity resumed
13/02/2018	0:44:21	106	construction joint visible on multibeam
13/02/2018	0:44:46	106	jconstruction joint 100ft again
13/02/2018	0:46:18	151.83	150ft
13/02/2018	0:46:34	151.83	12.7ft crown to sediment
13/02/2018	0:49:06	213.15	200ft
13/02/2018	0:49:49	213.15	12.2 ft crown to sediment
13/02/2018	0:51:33	252.69	250ft
13/02/2018	0:52:30	257.98	12.6 crown to sediment
13/02/2018	0:54:47	265.75	300ft
13/02/2018	0:55:08	302.54	12.4ft crown to sediment
13/02/2018	0:57:36	351.3	350ft
13/02/2018	0:57:52	351.3	12.3ft crown to sediment
13/02/2018	0:59:43	403.69	400ft
13/02/2018	1:00:24	403.69	spring line to spring line 13.2
13/02/2018	1:01:10	403.69	12.0 ft crown to sediment
13/02/2018	1:03:23	452.18	450ft
13/02/2018	1:03:37	452.18	12ft crown to sediment
13/02/2018	1:03:53	452.18	13.1 spring line to spring line
13/02/2018	1:06:00	504.13	500ft
13/02/2018	1:06:22	504	13.3 width
13/02/2018	1:06:31	504	12.2 crown to sediment
13/02/2018	1:08:33	545	550ft

13/02/2018	1:09:01	550	13.3ft width
13/02/2018	1:09:08	550	12.3ft crown to sediment
13/02/2018	1:10:37	591	construction joint at 595ft
13/02/2018	1:11:49	595	joint seems to be in good condition. filled with marine growth at spring line and down
13/02/2018	1:14:12	595	12.7ft crown sediment
13/02/2018	1:14:32	595	13.2ft width
13/02/2018	1:15:01	595	12.8ft crown to sediment
13/02/2018	1:17:20	655	650ft
13/02/2018	1:17:44	655	13.3ft width
13/02/2018	1:18:01	655	12.1ft crown to sediment
13/02/2018	1:20:07	698	man hole at 698ft on tether counter
13/02/2018	1:20:31	698	manhole seems to be clean
13/02/2018	1:21:34	707	manhole visible in cross section
13/02/2018	1:21:45	707	bottom of manhole lid to crown of tunnel 8ft
13/02/2018	1:22:10	707	13.2 ft width
13/02/2018	1:22:28	707	12.3ft height
13/02/2018	1:24:19	715	750ft
13/02/2018	1:24:32	756.74	13.1 width
13/02/2018	1:24:40	756	12.4ft height
13/02/2018	1:26:15	777	potential construction joint heavy fouling
13/02/2018	1:27:09	805	800ft
13/02/2018	1:27:32	806	13ft width
13/02/2018	1:27:38	806	12.2ft height
13/02/2018	1:29:15	854	850ft
13/02/2018	1:29:41	855	12.3ft hieght
13/02/2018	1:29:52	855	13.4ft width
13/02/2018	1:31:46	901	600ft- should be 900
13/02/2018	13:31:58		fixed time from am to pm
13/02/2018	13:34:27		13ft width
13/02/2018	13:34:34		12.4ft height
13/02/2018	13:37:12		950ft
13/02/2018	13:37:31		13ft width
13/02/2018	13:37:39		12.2 ft hiehgt
13/02/2018	13:39:25	956	1000ft
13/02/2018	13:40:04	1005	13ft width
13/02/2018	13:40:10	1005	12.5ft tall
13/02/2018	13:42:05	1054	1050ft
13/02/2018	13:42:32	1058	12.5ft tall
13/02/2018	13:42:42	1058	13.2ft width
13/02/2018	13:45:11	1083	looking up vertical riser at bottom of manhole lid

13/02/2018	13:46:22	1110	6.5ft lid of manhole to crown
13/02/2018	13:47:09	1110	12.3ft height
13/02/2018	13:47:19	1110	13.1fft width
13/02/2018	13:52:41	1153	1150ft
13/02/2018	13:52:46	1153	12.2ft h
13/02/2018	13:52:52	1153	13.2ft width
13/02/2018	13:55:21	1153	1200ft
13/02/2018	13:55:50	1200	12.1ft H
13/02/2018	13:55:56	1203	13.3ft W
13/02/2018	13:58:08	1256	1250ft
13/02/2018	13:58:27	1256	12.3ft H
13/02/2018	13:58:38	1256	13.3ft W
13/02/2018	14:00:47	1308	1300ft
13/02/2018	14:01:12	1308	12.1ft H
13/02/2018	14:01:20	1308	13.2ft W
13/02/2018	14:03:36	1351	1350ft
13/02/2018	14:04:03	1351	12.4ft H
13/02/2018	14:04:14	1351	13.3ft W
13/02/2018	14:06:47	1404	1400ft
13/02/2018	14:07:24	1411	manhole #3
13/02/2018	14:09:31	1411	6.5ft vertical riser of manhole
13/02/2018	14:09:44	1423	3 to 3.5ft diameter manhole
13/02/2018	14:10:16	1423	13.3ft W
13/02/2018	14:10:34	1423	12.2ft H
13/02/2018	14:12:51	1451	1450ft
13/02/2018	14:13:13	1451	12.2ft H
13/02/2018	14:13:21	1451	13ft W
13/02/2018	14:17:02	1502	1500ft
13/02/2018	14:17:25	1502	12.5ft H
13/02/2018	14:17:33	1502	13ft W
13/02/2018	14:20:31	1556	1550ft
13/02/2018	14:20:52	1558	13.3ft W
13/02/2018	14:21:02	1558	12ft H
13/02/2018	14:22:47	1606	1600ft
13/02/2018	14:24:19	1608	12.3ft H
13/02/2018	14:24:26	1608	13.2ft W
13/02/2018	14:25:30	1608	possible change in marine build up
13/02/2018	14:25:48	1624	seems to be an increase in marine growth visually
13/02/2018	14:27:13	1671	1670ft
13/02/2018	14:27:28	1671	12.3ft h
13/02/2018	14:27:36	1671	13.3ft w

13/02/2018	14:29:31	1706	1700ft
13/02/2018	14:29:48	1706	manhole #4
13/02/2018	14:33:25	1707	lbv control lost

Table 2 – Feb 14, 2018. Intake Tunnel cont.

Date	Time	Tether Ft.	Comment
14/02/2018	8:00:39	1711	sending rov back to 1711
14/02/2018	8:03:53	1711	flying out on invert of tunnel observing sediment
14/02/2018	8:22:45	1711	manhole at 1700ft
14/02/2018	8:22:55	1711	light marine growth in manhole
14/02/2018	8:24:33	1730	12.1ft H
14/02/2018	8:24:58	1730	13.5ft W
14/02/2018	8:25:31	1730	3ft W manhole access
14/02/2018	8:25:44	1730	7ft bottom to manhole to crown
14/02/2018	8:27:02	1754	1750ft
14/02/2018	8:27:33	1755	13.7ft W
14/02/2018	8:27:41	1755	12.7ft H
14/02/2018	8:29:21	1805	1800ft
14/02/2018	8:29:47	1805	13.6ft W
14/02/2018	8:29:57	1806	correction 13.4ft W
14/02/2018	8:30:08	1806	12.5ft H
14/02/2018	8:31:26	1806	marine growth thicker in this location
14/02/2018	8:31:45	1866	1850ft
14/02/2018	8:32:52	1866	13.3ft W
14/02/2018	8:32:59	1866	12.9ft H
14/02/2018	8:35:26	1902	1900ft
14/02/2018	8:35:58	1902	12.5ft H
14/02/2018	8:36:07	1902	13.4ft W
14/02/2018	8:37:49	1972	manhole visible on multibeam
14/02/2018	8:38:03	1983	at manhole
14/02/2018	8:38:43	1983	1983ft at manhole
14/02/2018	8:39:58	1983	top of manhole
14/02/2018	8:44:30	1996	3ft diameter of manhole
14/02/2018	8:44:44	1996	6.5ft H of manhole
14/02/2018	8:44:54	1996	13.3ft W tunnel
14/02/2018	8:45:11	1996	13.5ft H
14/02/2018	8:46:26	1996	2000ft
14/02/2018	8:48:54	2045	alot more marine growth on crown
14/02/2018	8:49:29	2054	2050ft

14/02/2018	8:49:51	2054	13.4ft W
14/02/2018	8:49:59	2054	13.0ft H
14/02/2018	9:08:04	2112	2100ft
14/02/2018	9:08:32	2108	13.3ft H
14/02/2018	9:08:43	2108	13.4ft W
14/02/2018	9:09:40	2155	2150ft
14/02/2018	9:10:16	2155	13.5ft W
14/02/2018	9:10:22	2155	13.2ft H
14/02/2018	9:12:36	2206	2200ft
14/02/2018	9:12:58	2207	13.3ft W
14/02/2018	9:13:18	2207	12.9ft H
14/02/2018	9:15:22	2265	2250ft
14/02/2018	9:15:56	2265	13.7ft W
14/02/2018	9:16:03	2265	13.2ft H
14/02/2018	9:22:44	2317	2300ft
14/02/2018	9:22:52	2317	manhole visible
14/02/2018	9:23:09	2317	13.2ft W
14/02/2018	9:23:18	2317	13.4ft H
14/02/2018	9:24:56	2337	manhole #6
14/02/2018	9:26:22	2344	13.5ft W
14/02/2018	9:26:34	2344	13ft H
14/02/2018	9:27:02	2344	3ft Diameter of manhole
14/02/2018	9:27:23	2344	7ft tall stem
14/02/2018	9:29:58	2402	2400ft
14/02/2018	9:30:39	2402	13.3ft W
14/02/2018	9:30:52	2402	13.4ft H
14/02/2018	9:33:13	2451	intake crib visible on multibeam
14/02/2018	9:36:20	2451	12.9ft wide intake crib
14/02/2018	9:36:33	2451	21ft top of crown to back wall
14/02/2018	9:37:12	2451	heading into crib
14/02/2018	9:37:23	2451	2450ft
14/02/2018	9:38:17	2462	descending to bottom
14/02/2018	9:39:30	2468	alot of shell or rock on bottom
14/02/2018	9:42:23	2478	note crown of tunnel and crib of intake
14/02/2018	9:43:12	2478	bottom seems to be free of large debris on multibeam
14/02/2018	9:43:39	2484	inspevcting south east corner of crib bottom to top
14/02/2018	9:44:43	2484	Construction seam
14/02/2018	9:46:46	2484	seam is pretty universal and tight thus far
14/02/2018	9:48:22	2484	lid is removed from crib
14/02/2018	9:48:42	2484	correction bar racks still in place
14/02/2018	9:49:11	2484	seem fairly clean

14/02/2018	9:49:33	2484	trash rack/ grate free of any large debris
14/02/2018	9:54:29	2490	fixing time on LBV
14/02/2018	9:58:15	2490	going to come back on spring line of tunnel
14/02/2018	10:04:16	2490	rov pulling back

Table 3 – Feb 14, 2018. Outfall Tunnel

Date	Time	Tether Ft.	Comment
14/02/2018	11:04:39	0	at plant side of outfall tunnel entrance
14/02/2018	11:15:27	19	pointed at discharge
14/02/2018	11:15:43	0	attempting to locate proper entry
14/02/2018	11:17:00	37	change from square tunnel to round tunnel
14/02/2018	11:21:32	37	100ft
14/02/2018	11:22:16	110	construction joint
14/02/2018	11:23:16	116	attempting to follow construction joint down to spring line
14/02/2018	11:23:43	116	gap seems to be in good condition and tight. filled with marine growth past spring line
14/02/2018	11:27:24	116	100ft sonar grabs
14/02/2018	11:28:27	116	13.5ft W
14/02/2018	11:28:36	116	13.5ft H
14/02/2018	11:29:45	124	another construction joint roughly 20ft away from first spotted
14/02/2018	11:30:12	129	not much growth at crown but filled much past the crown
14/02/2018	11:31:07	153	150ft
14/02/2018	11:31:20	162	3rd joint spotted here at 150ft
14/02/2018	11:31:45	162	13.4ft W
14/02/2018	11:31:52	162	12.8ft H
14/02/2018	11:32:49	162	joint seems uniform here as well, half inch or less
14/02/2018	11:33:54	185	a lot of air in tunnel, most likely due to divers
14/02/2018	11:34:07	191	end of air bubble
14/02/2018	11:34:27		another large bubble and 4th construction joint only visible at crown
14/02/2018	11:35:02	197	possible crack spotted
14/02/2018	11:35:22	197	most likely over pour
14/02/2018	11:35:49	197	starting to see a rise in sediment levels
14/02/2018	11:36:04	197	11.7ft H
14/02/2018	11:36:11	197	13.1ft W
14/02/2018	11:38:01	262	250ft
14/02/2018	11:38:24	262	13.4ft W
14/02/2018	11:38:28	262	11.1ft H

14/02/2018	11:40:01	295	another construction joint
14/02/2018	11:40:37	299	note marine growth only extends 80% up walls possibly due to air on the crown frequently
14/02/2018	11:41:33	299	13.2ft W
14/02/2018	11:42:49	299	300ft
14/02/2018	11:42:59	299	11.3ft H
14/02/2018	11:44:14	321	construction joint
14/02/2018	11:45:39	363	350ft
14/02/2018	11:45:59	367	11.6ft H
14/02/2018	11:46:05	367	13.2ft W
14/02/2018	11:47:36	397	construction joint
14/02/2018	11:47:42	397	400ft
14/02/2018	11:47:52	397	13.3ft W
14/02/2018	11:47:58	397	11.9ft H
14/02/2018	11:48:51	397	small raised area possible form lines
14/02/2018	11:49:11	411	concrete seems to be in great condition
14/02/2018	11:49:45	423	very large air bubble
14/02/2018	11:49:56	425	1-2iinches deep on crown
14/02/2018	11:51:18	465	460ft
14/02/2018	11:51:43	465	11.4ft H
14/02/2018	11:51:51	465	height was from surface of water
14/02/2018	11:52:18	465	13.5ft W
14/02/2018	11:53:38	509	500ft
14/02/2018	11:54:20	517	13.5ft W
14/02/2018	11:55:06	517	11.9ft H
14/02/2018	11:55:47	525	joint
14/02/2018	11:56:13	532	forms lines
14/02/2018	11:56:23	536	still a lot of air
14/02/2018	11:57:14	557	550ft
14/02/2018	11:57:40	557	13.3ft W
14/02/2018	11:57:51	557	12ft H
14/02/2018	11:58:41	563	seam
14/02/2018	11:59:02	574	end of air bubble maybe
14/02/2018	11:59:15	580	construction seams or lines are seen often
14/02/2018	11:59:47	602	600ft
14/02/2018	11:59:52	602	construction joint visible
14/02/2018	12:00:10	602	11.9 ft H
14/02/2018	12:00:18	602	13.3ft W
14/02/2018	12:00:57	602	joint looks good at crown
14/02/2018	12:01:40	617	joint
14/02/2018	12:02:02	638	marine growth starting on crown

14/02/2018	12:02:27	656	650ft
14/02/2018	12:02:45	656	13.5ft W
14/02/2018	12:02:52	659	11.6ft H
14/02/2018	12:04:46	699	700ft manhole
14/02/2018	12:05:43	610	manhole seems to be partly filled with air
14/02/2018	12:14:43	710	with cross section of sonar it appears that the surface of the water against the airpocket in man hole is 3ft to 4ft deep
14/02/2018	12:15:31	710	3ft from air to crown
14/02/2018	12:15:40	710	3ft wide
14/02/2018	12:15:45	710	11.4ft H
14/02/2018	12:15:55	710	13.4ft W
14/02/2018	12:17:55	754	750ft
14/02/2018	12:18:45	754	13.2ft W
14/02/2018	12:18:55	754	11.7ft H
14/02/2018	12:20:37	756	800ft
14/02/2018	12:21:08	812	13.3ft W
14/02/2018	12:21:27	812	12ft H
14/02/2018	12:22:37	854	850ft
14/02/2018	12:23:07	860	13.3ft
14/02/2018	12:23:24	860	12ft H
14/02/2018	12:25:52	897	900ft
14/02/2018	12:27:12	917	13.3ft W
14/02/2018	12:27:22	917	12ft H
14/02/2018	12:29:03	961	950ft
14/02/2018	12:29:32	961	13.5ft W
14/02/2018	12:29:37	961	11.8ft H
14/02/2018	12:34:39	1000	1000ft
14/02/2018	12:34:55	1000	13.2ft W
14/02/2018	12:35:00	1000	11.7ft H
14/02/2018	12:40:13	1051	1050ft
14/02/2018	12:40:43	1053	13.4ft W
14/02/2018	12:40:48	1053	12ft H
14/02/2018	12:41:51	1088	manhole approaching
14/02/2018	12:42:03	1102	1100ft
14/02/2018	12:42:18	1105	at manhole
14/02/2018	12:43:17	1108	can’t see lid
14/02/2018	12:43:57	1108	6.5ft stem length
14/02/2018	12:44:15	1108	3ft wide
14/02/2018	12:44:21	1108	13.0ft W
14/02/2018	12:44:27	1108	12ft H
14/02/2018	12:46:49	1154	1150ft

14/02/2018	12:47:16	1154	13.5ft W
14/02/2018	12:47:24	1154	12ft H
14/02/2018	12:49:37	1202	1200ft
14/02/2018	12:49:56	1202	13.3ft W
14/02/2018	12:50:08	1202	11.9ft H
14/02/2018	12:52:30	1252	1250ft
14/02/2018	12:53:17	1254	11.8ft H
14/02/2018	12:53:30	1254	13.2ft H
14/02/2018	12:54:55	1303	1300ft
14/02/2018	12:55:42	1305	13.2ft W
14/02/2018	12:55:49	1305	11.7ft H
14/02/2018	12:57:22	1352	1350ft
14/02/2018	12:57:46	1353	13.2ft H
14/02/2018	12:57:58	1353	11.8ft H
14/02/2018	12:59:15	1402	1400ft continuing to manhole
14/02/2018	13:00:49	1414	manhole appears in good condition
14/02/2018	13:01:12	1419	6.5ft length of stem
14/02/2018	13:01:35	1419	3ft wide
14/02/2018	13:01:39	1419	13.3ft W
14/02/2018	13:01:47	1419	12.0ft H
14/02/2018	13:04:10	1500	1500ft
14/02/2018	13:05:30	1501	13.2ft W
14/02/2018	13:05:42	1501	12ft H
14/02/2018	13:07:09	1551	1550ft
14/02/2018	13:07:59	1552	13.2ft W
14/02/2018	13:08:05	1552	12.4ft H
14/02/2018	13:11:04	1603	1600ft
14/02/2018	13:11:40	1603	13.1ft W
14/02/2018	13:11:49	1603	12.3ft H
14/02/2018	13:13:18	1651	1650ft
14/02/2018	13:13:42	1651	13.3ft W
14/02/2018	13:13:47	1651	12.1ft H
14/02/2018	13:15:12	1703	1700ft
14/02/2018	13:17:06	1703	13.3ft W
14/02/2018	13:17:12	1703	12ft H
14/02/2018	13:19:49	1721	manhole at 1727ft roughly
14/02/2018	13:20:10	1730	lid of manhole appears clean
14/02/2018	13:20:18	1730	7ft length of stem 3 ft wide
14/02/2018	13:20:34	1730	13.2ft W
14/02/2018	13:20:43	1730	12.2ft H
14/02/2018	13:22:03	1749	most of marine growth consists of mussels instead of coral type

			growths
14/02/2018	13:23:24	1813	1800
14/02/2018	13:23:33	1813	13.4ft W
14/02/2018	13:23:57	1813	12.2ft H
14/02/2018	13:25:23	1854	1850ft
14/02/2018	13:25:49	1858	13.2ft W
14/02/2018	13:25:59	1858	12.6ft H
14/02/2018	13:27:39	1901	1900ft
14/02/2018	13:28:10	1901	13.2ft W
14/02/2018	13:28:18	1901	12.2ft H
14/02/2018	13:29:57	1951	1950ft
14/02/2018	13:30:27	1953	12.3ft H
14/02/2018	13:30:37	1953	13.2ft W
14/02/2018	13:30:52	1953	manhole on multibeam
14/02/2018	13:31:28	1980	manhole
14/02/2018	13:31:33	1980	air in this manhole
14/02/2018	13:32:39	1990	1990ft at manhole
14/02/2018	13:33:47	1990	13.5ft W
14/02/2018	13:33:54	1990	12ft H
14/02/2018	13:34:32	1990	5.4ft stem so 1.6ft air
14/02/2018	13:41:01	1990	2050ft
14/02/2018	13:41:33	2054	13.4ft W
14/02/2018	13:41:42	2054	12.6ft H
14/02/2018	13:43:02	2101	2100ft
14/02/2018	13:43:11	2105	light visible from end of tunnel
14/02/2018	13:43:26	2105	13.4ft W
14/02/2018	13:43:36	2105	13.4ft H
14/02/2018	13:45:40	2150	2150ft at end of tunnel
14/02/2018	13:46:17	2158	headed to top of crib
14/02/2018	13:46:41	2158	joint on east wall of crib
14/02/2018	13:47:35	2169	following joint
14/02/2018	13:47:41	2169	less growth on this crib
14/02/2018	13:48:21	2169	lost joint
14/02/2018	13:49:05	2169	on grating
14/02/2018	13:49:23	2169	core sample location at discharge tower
14/02/2018	13:49:44	2169	note studs
14/02/2018	13:50:31	2169	grating is half off at this time
14/02/2018	13:50:41	2169	coming down west wall for inspection
14/02/2018	13:51:04	2169	joint in view
14/02/2018	13:51:55	2169	another joint
14/02/2018	13:52:23	2169	on bottom

14/02/2018	13:52:44		bottom is free of any major debris. just shells and sand

ATTACHMENT J

Sub Terra Memo

Civil & Mining Engineering
Tunnel Engineering & Construction Management
Rapid Excavation/Support Systems
Active/Abandoned Mine Subsidence
Geotechnical Instrumentation
Structural Monitoring
Blast Optimization / Vibration Monitoring
Mine & Quarry Permitting
Material Science / Laboratory Testing

Friday, March 29, 2019

Patrick Crain
Poseidon Water
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Direct: 760 655 3994
Mobile: 760 889 2975
(Sent via e-mail to pcrain@poseidonwater.com)

Re: Huntingdon Beach, CA – Desalination Project: Desk Top Evaluation of Underground Construction Issues for an Offshore Intake

Dear Mr. Crain:

Thanks for involving us in the above noted project and the subordinated task which we understand focuses on developing input on underground construction of an offshore intake for use in a desk top evaluation of intake alternatives for the Huntingdon Beach, CA – Desalination Project.

We discussed the project and issues in conceptual detail via the telephone during the week of March 11. Based on that discussion, and our subsequent internet based communications, we developed a Scope of Work to include:

1. A review of data and information that you have provided.
2. Develop and provide conceptual method descriptions for offshore construction of an intake at alternate location D2 using underground methods. Discuss the technical feasibility of alternates, the availability of equipment and qualified Contractor's, and opine on other factors that may assist Poseidon Water in their comparison of alternatives.
3. Reference, as appropriate, data and available construction information for other seawater intakes in similar conditions.
4. Provide ROM magnitude cost and schedule information for feasible trenchless options.

SubTerra, Inc. specializes in the design and construction of trenchless options (tunnels and micro-tunnels) for installing pipe of the size contemplated by this project (e.g., 12 to 14-ft inside diameter and previously provided these services for the Carlsbad, CA

Post Office Box 520, 218 East North Bend Way, North Bend, WA 98045
Phone: (425) 888-5425 Web Address: www.subterra.us Email: subterra@subterra.us Fax: (425) 888-2725

desalination project as well as other water projects throughout the US. Sub T Engineers, their sister company, provides similar services in Israel.

1. Questions Asked by RQQCB

Poseidon has requested us to develop a document that responds to questions and comments from a telephone call on February 26, 2019.

Reference is made to the 26th of February Conference Call regarding Poseidon's Response to Water Board Staff's Letter dated February 4, 2019, Item 6:

> *Technological, additional clarification to add to Responses #12 and/or #14 – Please discuss (1) whether other configurations for extending the intake to Stations U2 or D2 other than parallel to the beach onshore or offshore of the proposed HBDP have/have not been considered (e.g., intake extending diagonally from the existing onshore intake structure) and why; and (2) whether an intake could be constructed at either of the alternative intake locations using two or more smaller diameter pipes instead of one large pipe, which could be constructed of HDPE, and could be jacked under the beach/offshore and cleaned using a pigging method. Please discuss and provide justification why or why not this option was/was not considered when assessing an alternative intake location at Stations U2 or D2.*

2. Conceptual Description of Alternatives

Consideration is given to three separate alternatives for in-taking water and disposal of reject brine from the proposed Huntingdon Beach Facility. Figure 1 shows an overview of the three different intake alignments:

1. The existing power station intake.
2. An option that runs from the existing intake, 1.2 miles along the beach to a location just short of the Santa Ana river estuary, then offshore to Intake D2.
3. An option that runs from the Desalination Plant site, under the Pacific Coast highway to a receiving shaft located on the beach then continues in a straight line to Intake D2. This option could also be constructed from the same location on the existing intake as used for item 2 above.

2.1. Conceptual Intake Alignment 01 – Beach Crossing Option

Figure 2 (aerial and topography) shows the plan layout of the Beach Crossing Option and Figure 2B shows a cross section along the alignment. This alignment would start at the existing intake and run along the beach parallel to the shore to an Access Shaft just north of the Santa Ana River. The pipeline would then run from the access shaft to Intake D2 as shown on Figure 2B.

The beach line could be installed at the same depth as the pipeline from the intake structure or at a higher elevation depending on system flow characteristics. It is anticipated that the beach line would be constructed inside a sheet piled wall.

A very short section of the pipeline to Intake D2 could be constructed using micro-tunneling methods, however, concerns over release of bentonite and slurry would limit

the offshore extent where this technology could be used. A significant part of the alignment would therefore need to be completed using a trestle or alternative surface based technology including connection to Intake D2.

This Alignment could be constructed at a lower elevation placing the top of the pipe at Intake D2 with its top 4-ft below the ground surface at that location. This would require lowering the pipeline by 10-ft and positioning Intake D2 with its base at least 20-ft below ground surface, possibly deeper. Under this general arrangement, a micro-tunnel might be advanced an additional 250-ft if soil/pipe friction forces could be reliably mitigated (see Section 3 below).

2.2. Conceptual Intake Alignment 02 –Direct Option

Figure 3 (aerial and topography) shows the plan layout of the Direct, Plant to Intake D2 Option and Figure 3B shows a cross section along the alignment. The initial part of this alignment would be constructed using trenchless technologies to an Access Shaft on the beach. Alternatively, the alignment could be started from the same location on the existing intake as used for Alignment 01. The pipeline to D2 would then be constructed from the Access Shaft to Intake D2.

Micro-tunneling is considered a feasible technology for under crossing the Pacific Coast highway at the depths and in the materials expected to be encountered. A micro-tunnel could also extend from the Access Shaft a distance of perhaps 1,500 to 2,000-ft where it would need to be recovered underwater. The remainder of the alignment would need to be completed using a trestle or alternate surface based technology.

This Alignment could also be constructed at a lower elevation placing the top of the pipe at Intake D2 with its top 4-ft below the ground surface at that location, as was the case for Alignment 01. This would again require lowering the pipeline by 10-ft and positioning Intake D2 with its base at least 20-ft below ground surface, possibly deeper. Under this general arrangement, a micro-tunnel might be advanced an additional 500-ft if soil/pipe friction forces could be reliably mitigated (see Section 3 below). An additional access shaft would also likely be required to minimize individual drive lengths and all access shafts would need to be driven deeper.

Section 3 outlines and discusses the limitations associated with pipe jacking and micro-tunneling that would limit the effectiveness of these technologies with regard to constructing the pipelines for this project.

2.3. Anticipated Geotechnical Conditions at the Project Site

The feasibility of constructing shafts and tunnels or micro-tunnels for this project will depend on the geotechnical conditions that will be encountered along the alignment. Although an explicit, micro-tunnel site characterization study has not been performed, Poseidon commissioned a study by Geosyntec[1] in 2013 which included a characterization of the subsea sediments in the project area. The following paragraphs contain summary geotechnical information from that study.

[1] Geosyntec, 2013. Feasibility Assessment of Shoreline Subsurface Collectors, Huntingdon Beach Seawater Desalination Project, Huntingdon Beach, California. Prepared for Poseidon Resources, 79pp.

ES-1: "*The Facility Site is in a coastal lowland area known as the Talbert Gap, an erosional gap in the uplifted coastal margin. The hydrogeology of the Talbert Gap has been studied extensively by the Department of Water Resources (DWR) and Orange County Water District (OCWD) because of seawater intrusion of the coastal margin aquifers. The Talbert Aquifer extends offshore through the Talbert Gap and facilitates discharge of groundwater to the sea and inland intrusion of seawater (DWR, 1967; Herndon 1992; Edwards et al., 2009). In the coastal portion of the Los Angeles and Orange County Groundwater Basins, including the Talbert Gap, the coarse-grained units in the upper aquifer system are overlain by fine-grained sediments of low permeability. Near the coastal margin in the vicinity of the project site, the top of the Talbert Aquifer is approximately 90 feet deep*."

P 4: *The report indicates that the Facility Site lies within a former estuarine environment. Depth to groundwater ranged from 7 to 10 ft. The stratigraphy consisted of approximately 9 ft fill / 4 ft clay / interbedded fine-grained sand with interbeds of clay and silt to 72 ft bgs (marine and estuarine) underlain by fluvial deposits (sand, silt & clay) from 72 to 90 ft. The transition between non-marine below and marine above was estimated at an elevation of approximately 60 ft below mean sea level (msl).*

P 8: *Borings by the Los Angeles District of ACOE in 1978 in Borrow Area B (offshore of Huntington Harbor) document that the shallow seafloor sediments consist of silty sand, and silt or clay intervals with thickness of 0.5 to 6 ft are present at depths ranging from 3 to over 20 feet*.

P 15: Section 3.5 Integration of on shore and off shore data notes: *Numerous seafloor samples and shallow cores document that the shallow sediment offshore also is predominantly silty fine-grained sand. In fact, as discussed further below, nearly all of the San Pedro Shelf is covered by this "muddy sand" (Wong et al., 2012) as is illustrated by Figure 11. The fine-grained ("muddy"), low permeability, shallow sediments function as a confining layer above the Talbert Aquifer, which limits the hydraulic connection between the Talbert Aquifer and overlying groundwater or seawater.*

The anticipated geotechnical conditions therefore include: fine sands, sandy silt, silty sand, silty fine-grained sand, and silt or clay intervals with thickness of 0.5 to 6 ft to the depths under consideration for micro-tunneling.

3. Construction Feasibility of Alternatives

Tunneling and micro-tunneling technologies have been applied around the world to construct intakes and outfalls for power stations, sewers, fish farms, and desalination plants. However a majority of the applications of these technologies has been to complete tunnels in competent sedimentary rock formations and in hard rock.

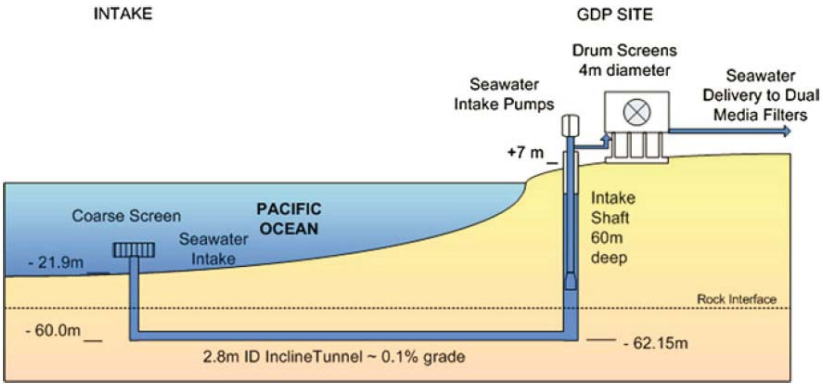
A smaller number of facilities have used micro-tunneling in unconsolidated formations to install pipelines of the sizes being contemplated for this project and horizontal directional drilling to install much smaller diameter pipelines.

3.1. General Approach using Trenchless Technologies in Rock Formations

The general arrangement includes construction of a shaft on shore that is socketed at least 50-ft into bedrock and a coffer dam off shore at the required intake location. A tunnel boring machine is then installed in the shaft and excavates and lines a rock tunnel to the location of the off shore cofferdam. Both shafts are constructed and sealed into the underlying rock mass so that the TBM can be launched and recovered in the dry.

The intake structure is constructed inside the coffer dam, the pipeline is flooded and the cofferdam removed. A large pump is installed in the onshore shaft to pump water to the desalination plant.

The schematic below is based on a paper by Craig [2] which provides a detailed description of two Australian SWRO Desalination projects and illustrates the concepts presented above.



We are not aware of any offshore intakes that have been completed in unconsolidated soils using this general arrangement.

3.2. General Approach and Limitations of Trenchless Technologies in Sandy Soils

Pipe jacking is a general term describing a method of installing a pipe in the ground using a set of hydraulic rams to push, or jack, the pipe along its alignment. Various technologies are used to excavate and remove the soil from in front of and inside the pipe including:

1. Auger Boring: Drag picks or cutters arranged in a pattern on the face of a cutter head that breaks up the soil which is ingested and transported inside the pipe using an auger.
2. Micro Tunneling: Drag picks or cutters arranged in a pattern on the face of a cutter head that breaks up the soil which is mixed with slurry inside the cutter

[2] Craig, K. (2013). Sydney and the Gold Coast Intakes. Thuwal, Saudi Arabia: KAUST Intakes and Outfall Seminar Presentation.

head and removed by pumping the slurry through lines attached to the inside wall of the pipe.

Auger boring is limited to runs of several hundred feet that are located above the water table. The length of a Micro Tunnel is limited by friction forces acting on the pipe, the ability to mitigate those forces using bentonite lubrication, the ability to introduce intermediate jacking stations and the ability to operate the bentonite lubrication and slurry mucking system in areas of shallow soil cover.

3.2.1. Micro-tunnel Operations and Equipment Summary

Figure 4 illustrates the subsurface components of a micro-tunnel system including:

1. A sheet piled shoring system. Other types of water tight shaft ground support may also be used including secant piles and diaphragm walls. The minimum inside width or length is typically 40-ft due to the size of the jacking frame.
2. A forward seal that allows the micro-tunnel to penetrate the shaft wall with minimum leakage.
3. The jacking frame, jacks, and micro-tunnel boring machine (MTBM)
4. Crane (not shown) that lowers in the pipe sections and lifts the equipment in and out.
5. Surface control structures.

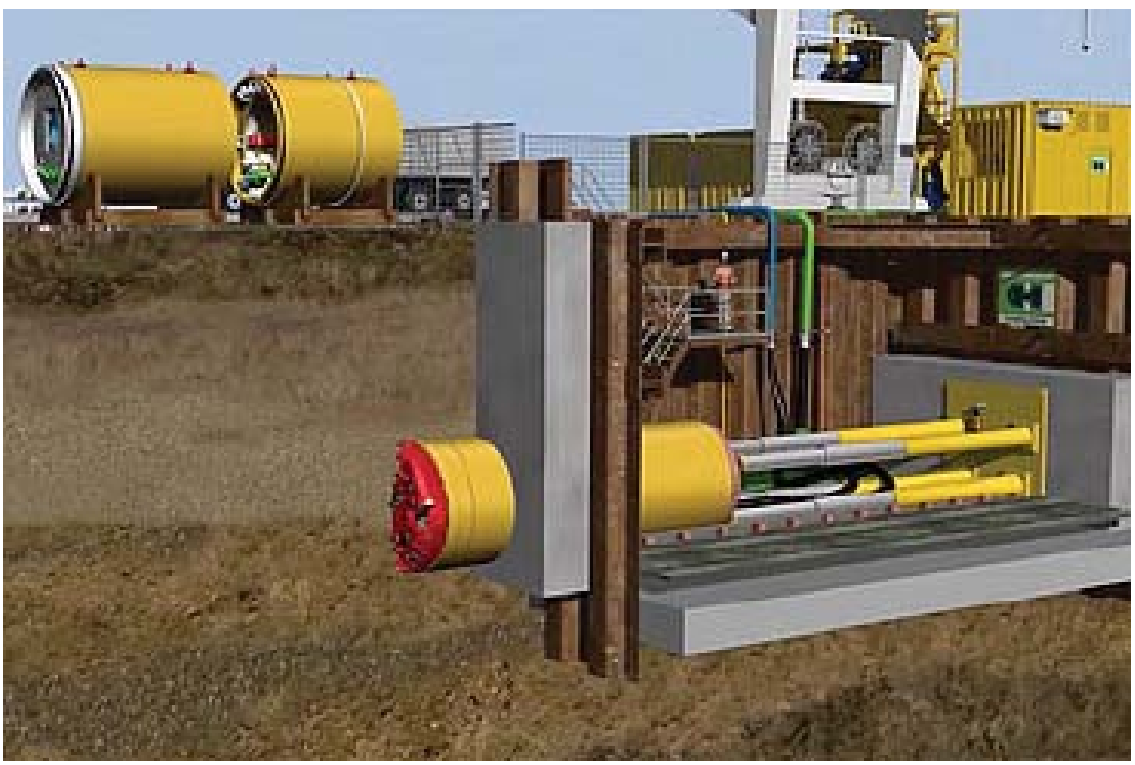


Figure 4: Illustration of the Subsurface Components of a Micro-tunnel System

In addition, a large area is required for siting the slurry processing equipment. A typical set up is shown below in Figure 5.

Figure 5: General Arrangement for a Microtunnel Slurry Separation Plant

The silty, sand laden slurry would be pumped from the tunnel face to the shaft and then to the primary separation circuit that removes gravel and coarse sand. Secondary and tertiary systems including desilting cones, mud cleaners and centrifuges will be required to recover fines. Recovered coarse grained materials are typically routed to a location where they are picked up by a front end loader and loaded to waiting trucks. The trucks then move the recovered sand to a nearby landfill or other licensed disposal facility.

A large quanity of silt is anticipated to be present in the micro-tunneled spoils based on the data summarized in Section 2.3. This will require the use of mud cleaners and centrifuges that will produce large quantities of “mud” that will require hauling and landfilling.

3.2.2. Access Shaft, Jacking Frame and Launch Seal

Figures 6 shows a jacking frame set up in a 40-ft long, sheet piled shaft that was constructed in silty sand with dewatering. Dewatering is required to allow installation of a relatively thin concrete base slab. Otherwise, a thick base slab is required to resist uplift forces. Such a dewatering system requires baker tanks set up around the shaft to retain the water prior to disposal

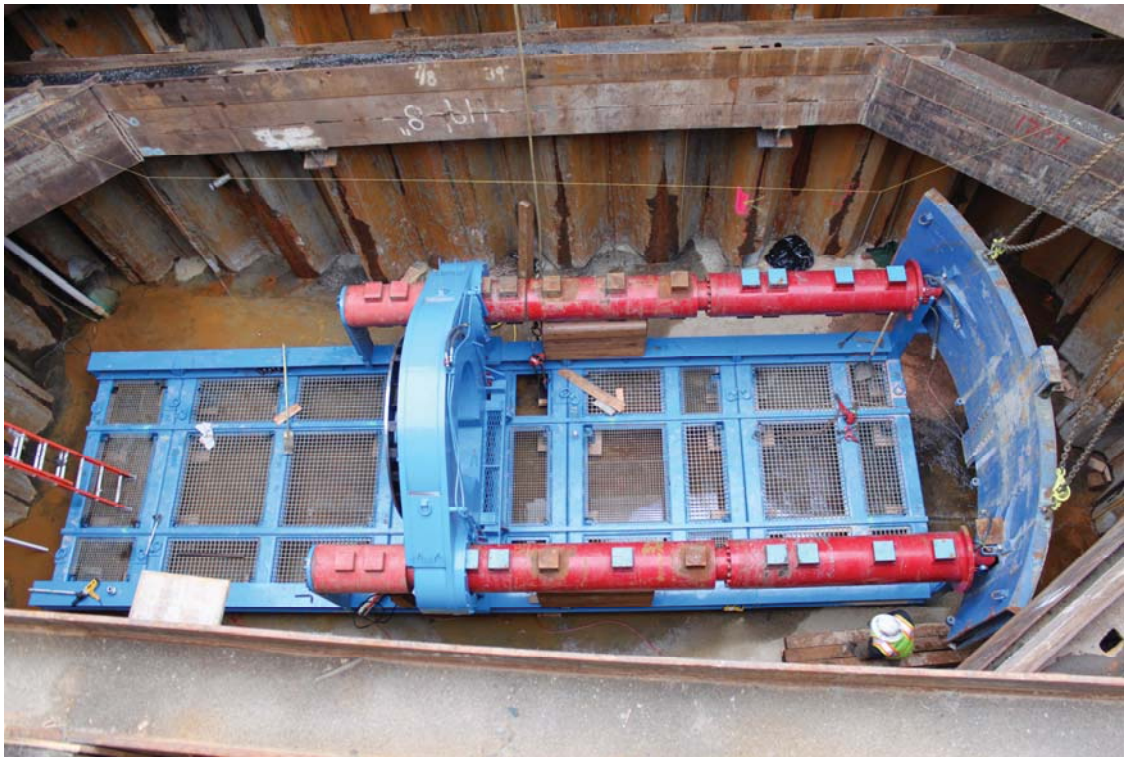


Figure 6: Sheet Piled Shafts with Dewatering

Figure 7 shows a jacking frame set up and entry seal in a secant pile supported shaft housing two micro-tunnels. Construction may be accomplished with or without dewatering.

Figure 7: Jacking Frame and Seal Set in a Secant Pile Shaft

3.2.3. Pipe Jacking Forces

During pipe jacking, the outside of the pipe is subject to friction forces which will need to be managed using injected bentonite and polymers compatible with a seawater environment. An effective system is required as high friction forces can significantly limit the length of pipeline that can be installed using micro-tunnel methods. Sandy and gravelly soils are typically associated with high friction factors which may limit runs to 300-ft without the use of intermediate jacking stations. Specialized, high viscosity bentonite mixtures intended to displace the sand without mixing are typically required for long drives (600 to 1,500ft).

The seawater environment and the presence of loose, unconsolidated soils pose special problems for both the pipe lubrication and slurry transport systems. The primary issue is the potential for these fluids to escape the micro-tunnel alignment and enter the sea.

3.3. Example Installations in Unconsolidated Soils

This section of the report describes several projects where trenchless technologies have been used or are planned for use in unconsolidated soils offshore.

3.3.1. Fish Farm in Portugal

Herrenknecht, a German micro-tunnel manufacturer, recently completed six, short micro-tunnels off of the coast of Portugal as intakes for a fish farm. Each intake was constructed by micro-tunneling from an undeveloped shoreline to several hundred meters off shore where the soil cover over the MTBM was about 5m. The pipe behind the MTBM was then flooded and the MTBM removed. Each installation was completed with a 90 degree elbow and a pipe extended several meters above the surface of the sea bed.

The use of a proprietary, extremely thick and viscous pipe lubricant was necessary to control the loss of bentonite and control soil/pipe friction. We are unaware of a domestic, US project that has utilized this technology.

3.3.1.1. Soreq Desalination Intake

We contacted Huchison Water's Soreq representative in Israel to request construction details for the micro-tunnels[3] and were told that Huchison is in a competitive bid and cannot provide process details until that competition is over. However, we obtained the following input from individuals who worked on the project and from reviews of available, published information:

1. The intake pipeline was 3.1m diameter and 2.5 km long.
2. Two micro-tunnels (one for each 75 MCM/yr plant) were mined from two shafts.
3. The micro-tunnels did not extend all the way from the shore to the intake structure. One individual stated that they were mined about 500m from each shaft.

[3] Ages, B., 2019. Personal Communication with Gil Doron, Huchison Water, March 15, 2019.

4. The intake pipelines were then completed using a suction dredge to uncover each micro-tunnel boring machine after each bored pipeline section had been weighted down. Each micro-tunnel boring machine was then retrieved and the pipeline extended in a dredged trench to each Intake Structure.
5. A Huchison Water slide show also indicates that the micro-tunnels may have been completed in a lithified sandstone formation locally known as Kurkar and not in unconsolidated sands and silts.

Separately, our understanding is that trenchless technology was required to avoid beach disturbance as the beach in this area is a nesting site for endangered turtle species.

3.4. Technology Selection for California Desalination Facilities

There are several existing and planned SWRO Desalination facilities either under construction or in planning along the California coastline; several of the intakes proposed for these projects are reviewed below.

3.4.1. Santa Cruz Desalination Intake

URS completed a seawater intake facility conceptual design report for the city of Santa Cruz in 2013[4] where they evaluated alternative technologies for intake construction in sandy soils and in bedrock. Technical Memorandum 3 from this study concluded that (highlight added):

> *"Intake pipeline construction methods were evaluated including dredging, hydraulic directional drilling (HDD), micro-tunneling (MT) and tunnel boring machine (TBM). In sandy offshore soil areas, dredging is the recommended construction method. In bedrock areas, MT or TBM is the recommended construction method. Some sites may require a combination of construction methods. Specific construction approaches were developed for each site."*

3.4.2. Monterey Bay Regional Water Project

The description for this regional project has been excerpted from an online summary by the developer, Deep Water Desal [5]

> *"The seawater intake/discharge system consists of underwater intake and discharge structures, pipelines, and a pump station. The intake structure would be located offshore and southwest of the Moss Landing Harbor entrance at approximately 130 feet below the surface in the head of the Monterey Submarine Canyon. The discharge structure would be located on the seafloor of the shelf north of the Monterey Canyon. It would be*

[4] URS, 2013. Seawater intake facility conceptual design report, Scwd2 regional seawater Desalination project. Prepared for: scwd2 Desalination Program 212 Locust Street, Suite C, Santa Cruz, CA 95060, May 8, 2013.

[5] Deep Water Desal, 2019. Internet reference - https://www.deepwaterdesal.com/monterey-bay-regional-water-project-summary.htm.

placed on the seafloor at a water depth of about 70 feet and be covered with ballast and armor stone.

Pipelines would be installed to these locations using HDD technology. HDD technology uses a surface launched drill rig to drill a pilot bore to the target site (discharge and intake locations). The pilot bore is enlarged by a back reamer(s) to pre-ream to the size required for the product pipe. The assembled pipeline is pulled back through during the final reaming process."

This example has been included to illustrate where trenchless technologies have been considered for intake development in geologic conditions similar to those anticipated for the Huntingdon Beach project.

The fundamental difference between the Monterey Bay project and the proposed Huntingdon Beach project is the required size of intake and water transfer pipelines. The Monterey Bay project pipelines included two 3.5-ft diameter intake lines and one 3-ft diameter discharge line planned for completion to depths of 130-ft and 70-ft respectfully. Offshore installations plan to use HDD technology, however, the feasibility of constructing using this technology has not yet been accepted and the project remains in the development stage.

On the other hand, the Huntingdon Beach project intends to use a 12-ft to 14-ft diameter intake which is beyond the range of HDD technology at the current time. Ignoring, for the moment, flow profiles, average flow velocities and the impact of fouling on the pipe area available for flow while making a comparison based solely on initial pipe area, it would take 12, 3.5-ft diameter pipes to replace one 12-ft diameter pipe. It would take 15, 3.5-ft diameter pipes to replace one 12-ft diameter pipe after 3-inches of fouling accumulation.

3.5. Summary of Alternatives

One of the major issues with regard to using trenchless technology is the need to occupy large sections of the pristine beach for building shafts, siting cranes, control structures and separation plants, and removing processed soils.

Similar impositions are associated with surface construction methods including sheet piling with direct burial however these methods have a small travelling footprint, produce uncontaminated sand that may be used elsewhere for beach restoration and can generally be completed at a faster rate than can trenchless installations.

In our opinion, avoiding installation along or across the pristine beach, using either trenching or trenchless technologies, should be avoided if at all possible.

The ranking of alternatives with regard to construction requirements and risks is therefore:

1. Use of the existing intake is the preferred alternative for the reasons noted above.
2. Alternative 1 is the second placed alternative as installation using surface methods is preferred based on surface occupation and disturbance. In addition, surface methods will not introduce bentonite, polymers or other additives to the sand meaning that the sand can be re-used locally without treatment. A short

section of twin micro-tunnels could be installed across the beach as shown on Figure 3B but this would not be a prudent use of this technology as surface methods would be used before and after any micro-tunnel run.

3. Alternative 2 is the least preferable, in our opinion, even when starting at the same location on the existing intake as Option 1. Although a section of micro-tunnel could be installed between the Beach Shaft and a location just beyond the shoreline, surface based technologies would be required to complete the intake pipeline to the Intake structure.

This ranking is consistent with the findings presented by URS in their study for the City of Santa Cruz where they concluded that ..”in sandy offshore soil areas, dredging is the recommended construction method”.

4. Report use and Disclaimer

The report is written for and intended for the exclusive use of Poseidon Resources as input to their response to the Water Board staff’s letter and for no other purpose. The work was performed in accordance with SubTerra, Inc.’s contract with Poseidon Resources in a manner consistent with the level of care and skill ordinarily exercised by other professionals in similar circumstances at the time the Services were performed. No other warranties exist or apply.

If you have any questions please don’t hesitate to call us at 425-888-5425 or 425-471-0879.

Chris D. Breeds, PE, PhD

President, SubTerra, Inc.

FIGURES

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Figure 1
Overview Map
Alignment Options and Estimates

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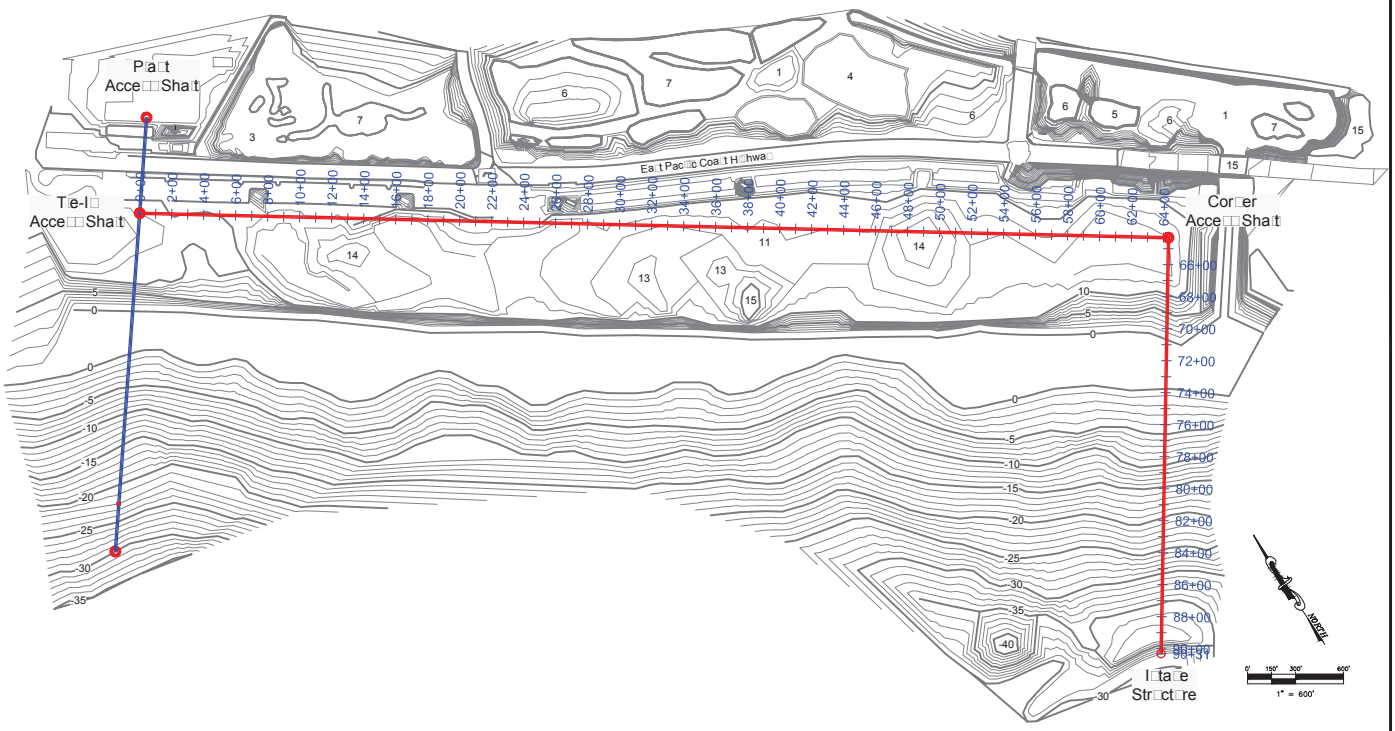

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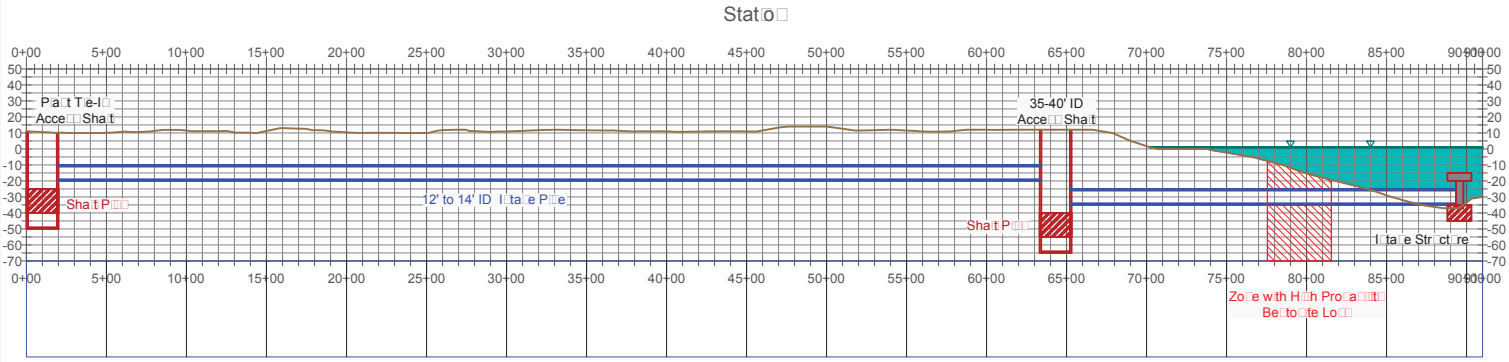

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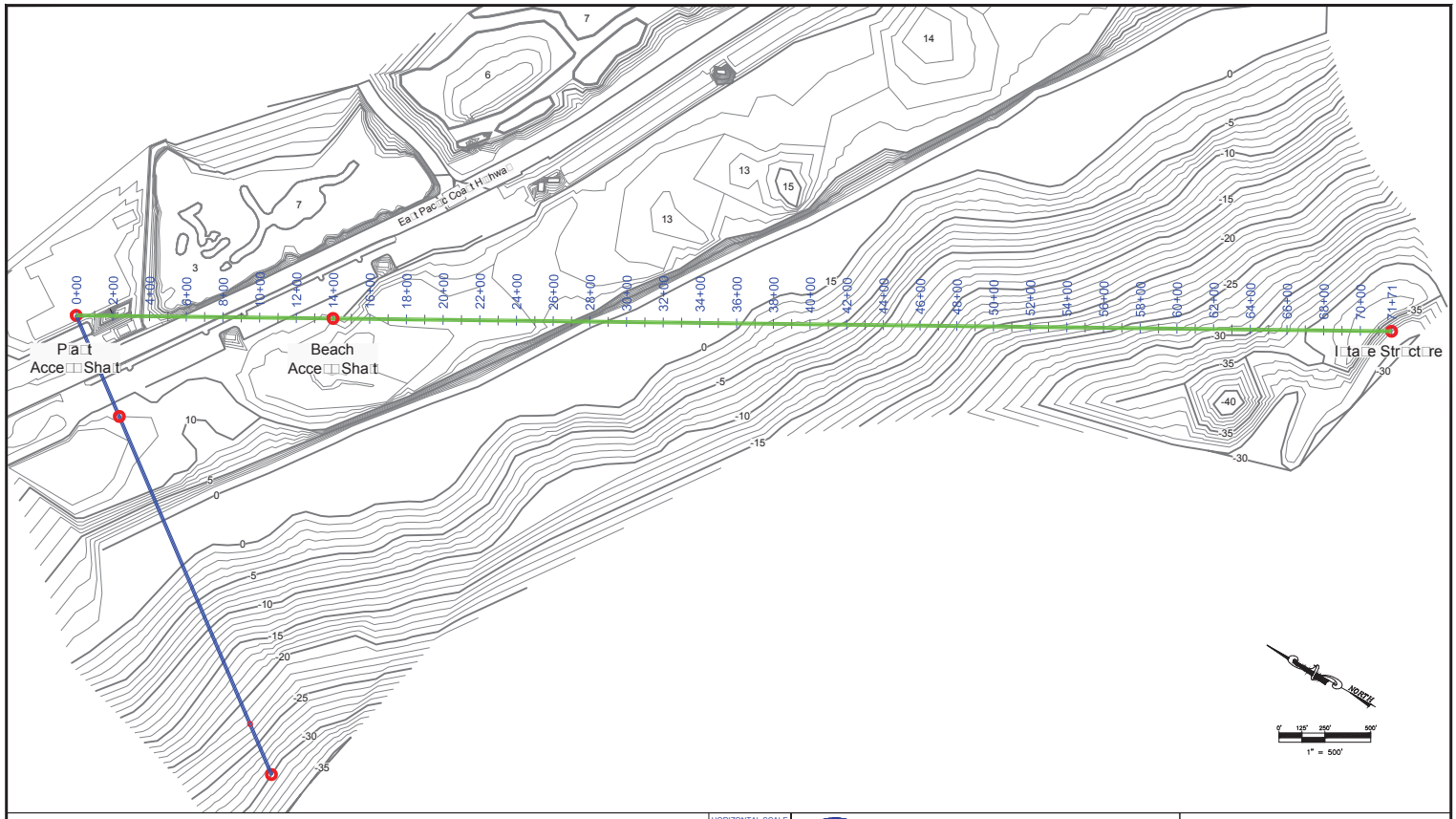

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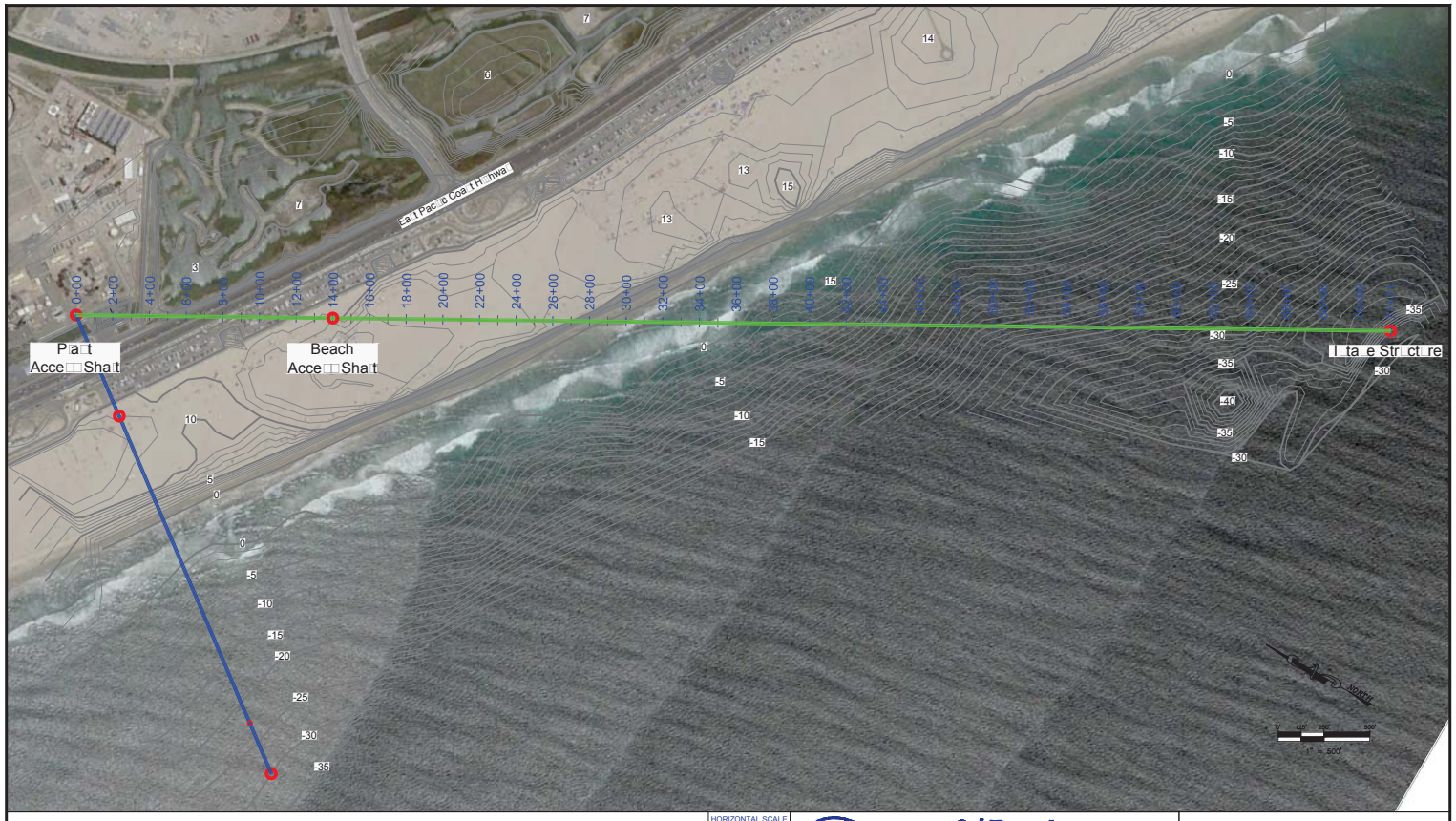

HORIZONTAL SCALE 1"=500'	SubTerra, Inc. P.O. Box 520, 218 East North Bend Way, North Bend WA 98045 Telephone: (425) 888-5425 E-mail: SubTerra@subterra.us Fax: (425) 888-2725	Figure 3 Conceptual Intake Alignment 02 Horizontal to Beach Alignment Direct Plan
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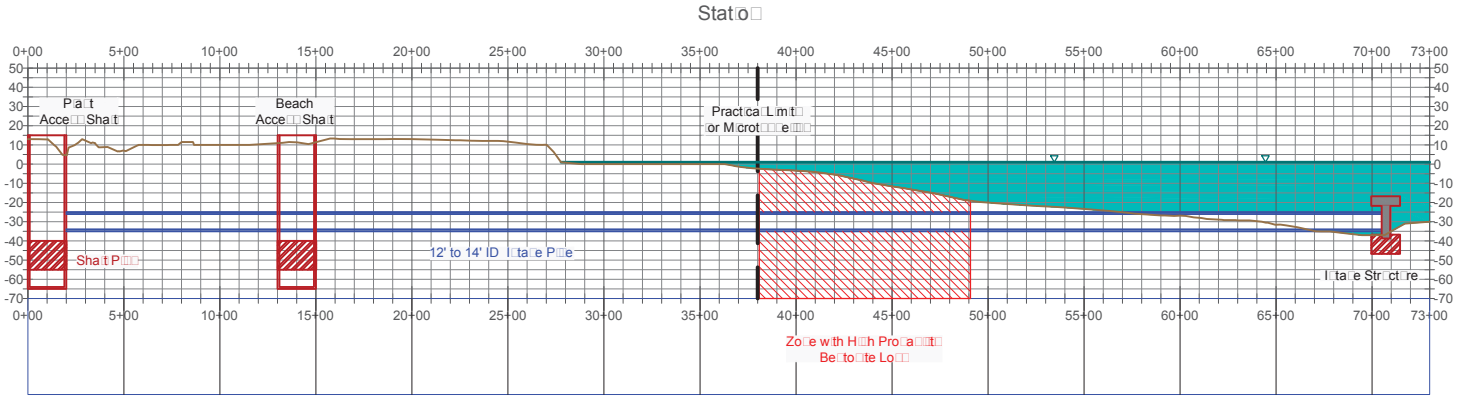

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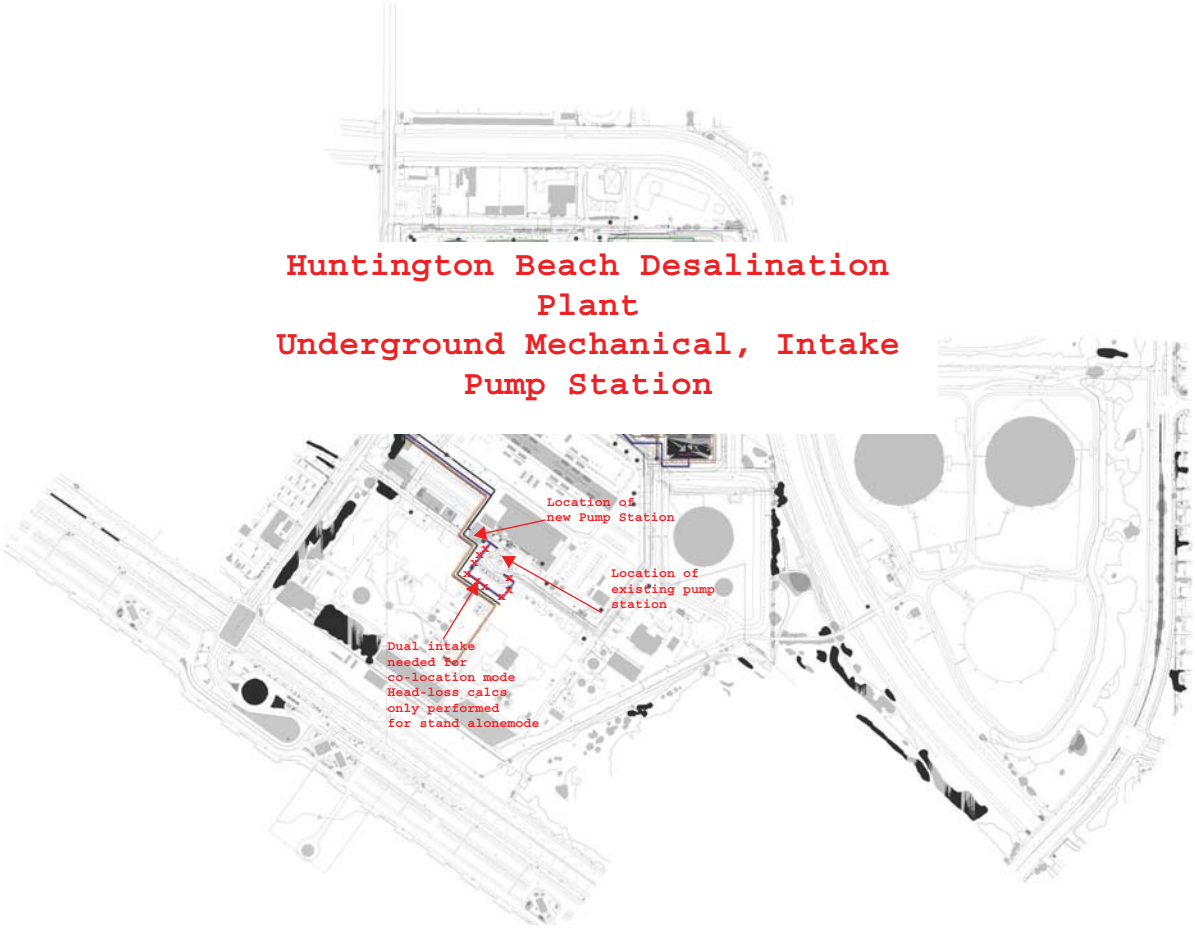
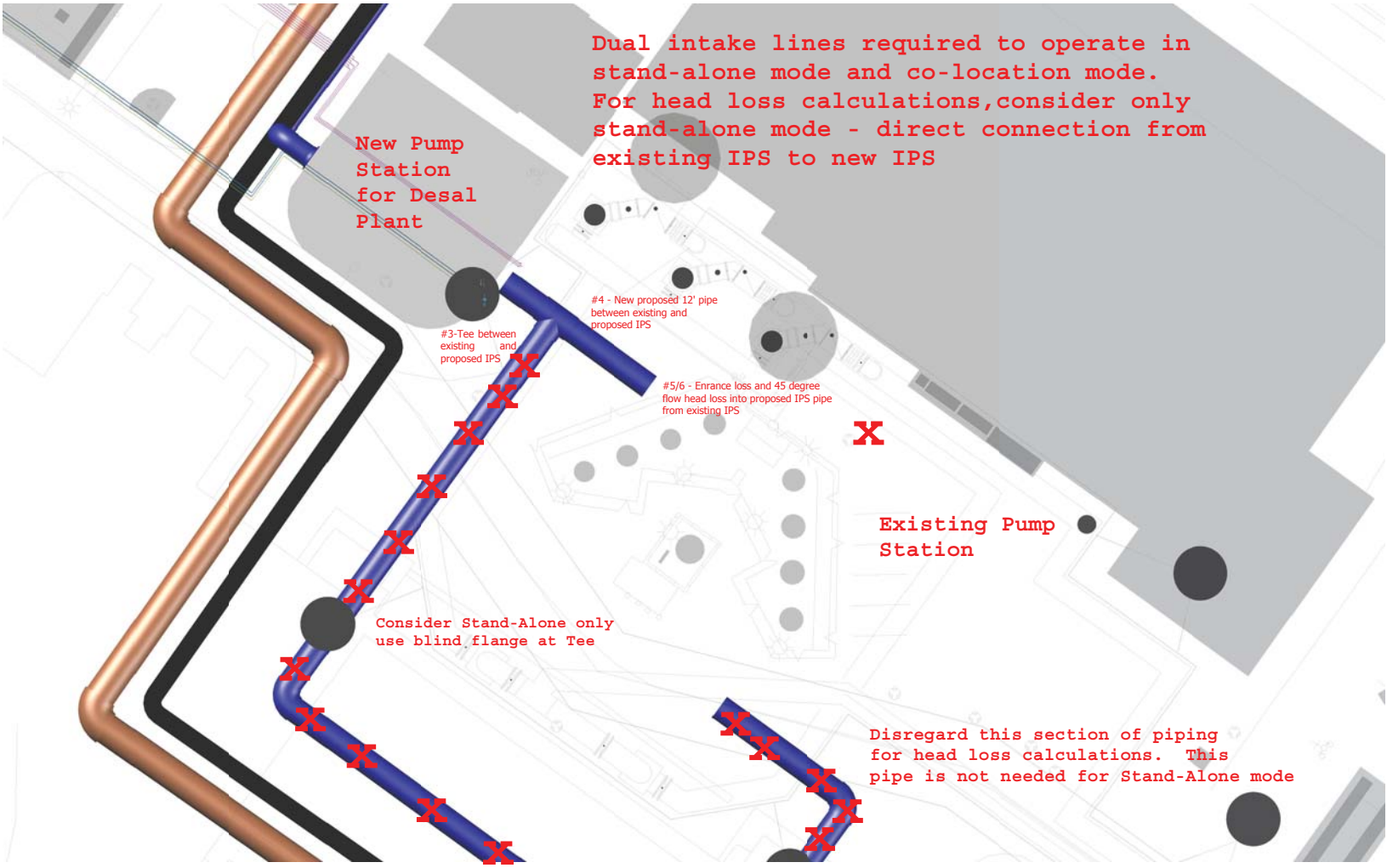
ATTACHMENT K

Facility Layout Drawings

Huntington Beach Desalination Plant
Underground Mechanical, Intake Pump Station




Dual intake lines required to operate in stand-alone mode and co-location mode. For head loss calculations,consider only stand-alone mode - direct connection from existing IPS to new IPS
New Pump Station for Desal Plant
#4 - New proposed 12' pipe between existing and proposed IPS
#3-Tee between existing and proposed IPS
#5/6 - Enrance loss and 45 degree flow head loss into proposed IPS pipe from existing IPS
Existing Pump Station
Consider Stand-Alone only use blind flange at Tee
Disregard this section of piping for head loss calculations. This pipe is not needed for Stand-Alone mode

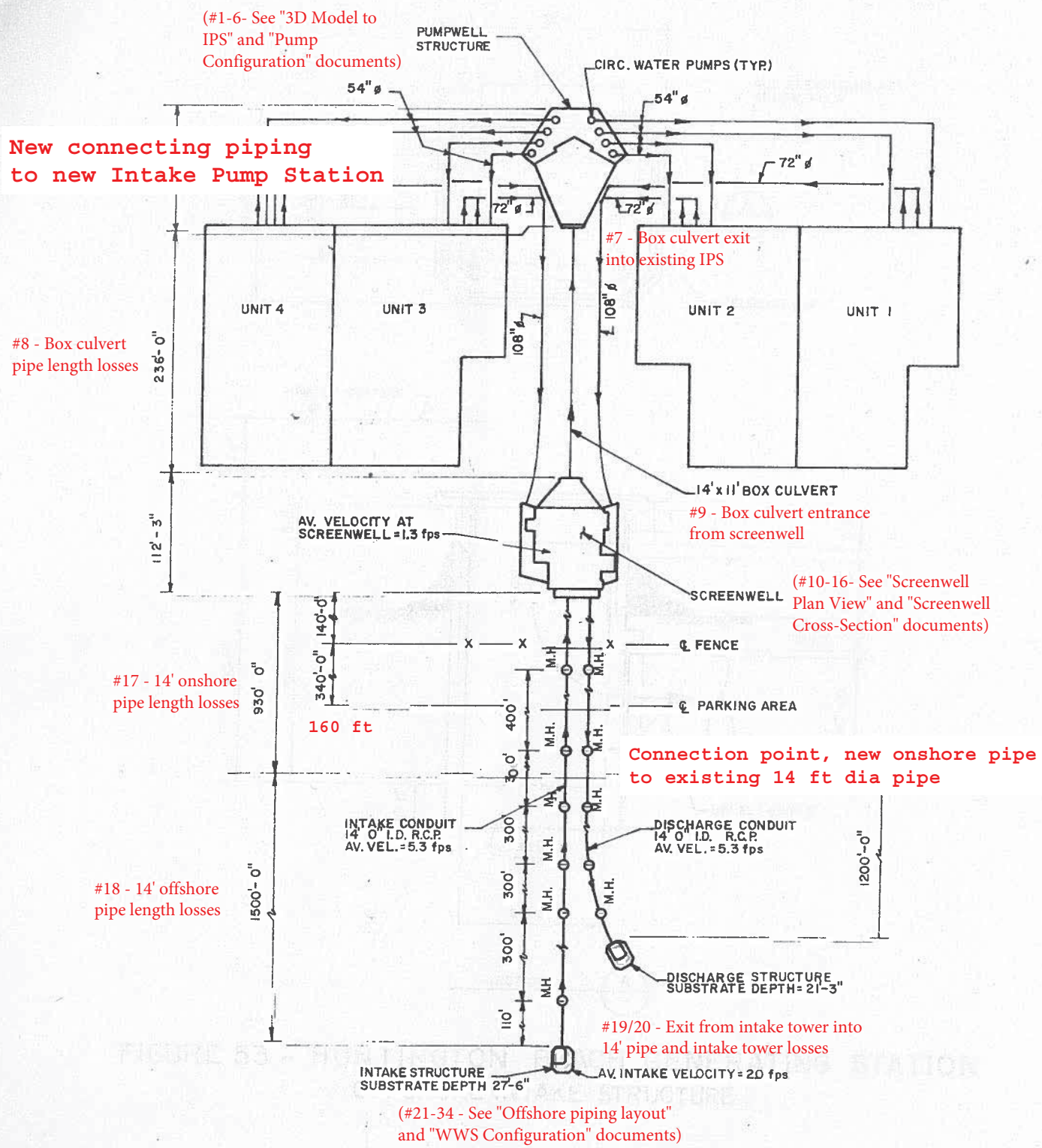


FIGURE 52.- HUNTINGTON BEACH GENERATING STATION
CIRCULATING WATER SYSTEM

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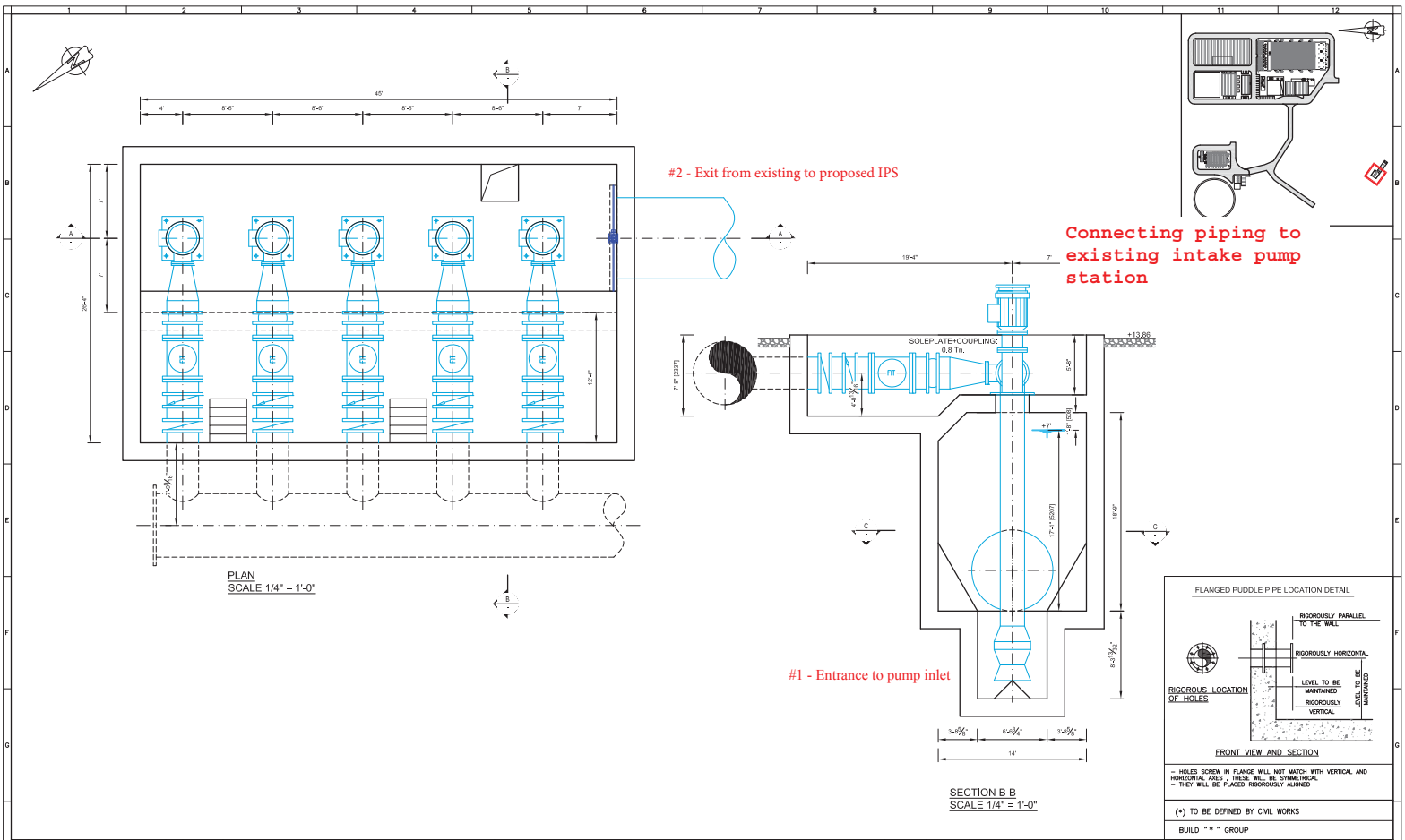

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						KSD	acciona Agua / TETRA TECH	Drawn: A.R.L.			Sheet Number: 1 of 1	
								Reviewed: M.M.H.				
								Approved: J.I.	Title of plant: PUMP STATION PLAN AND SECTION	Drawing Number: 04 130 000 0001		
								Scale: 1/4" = 1'-0"				
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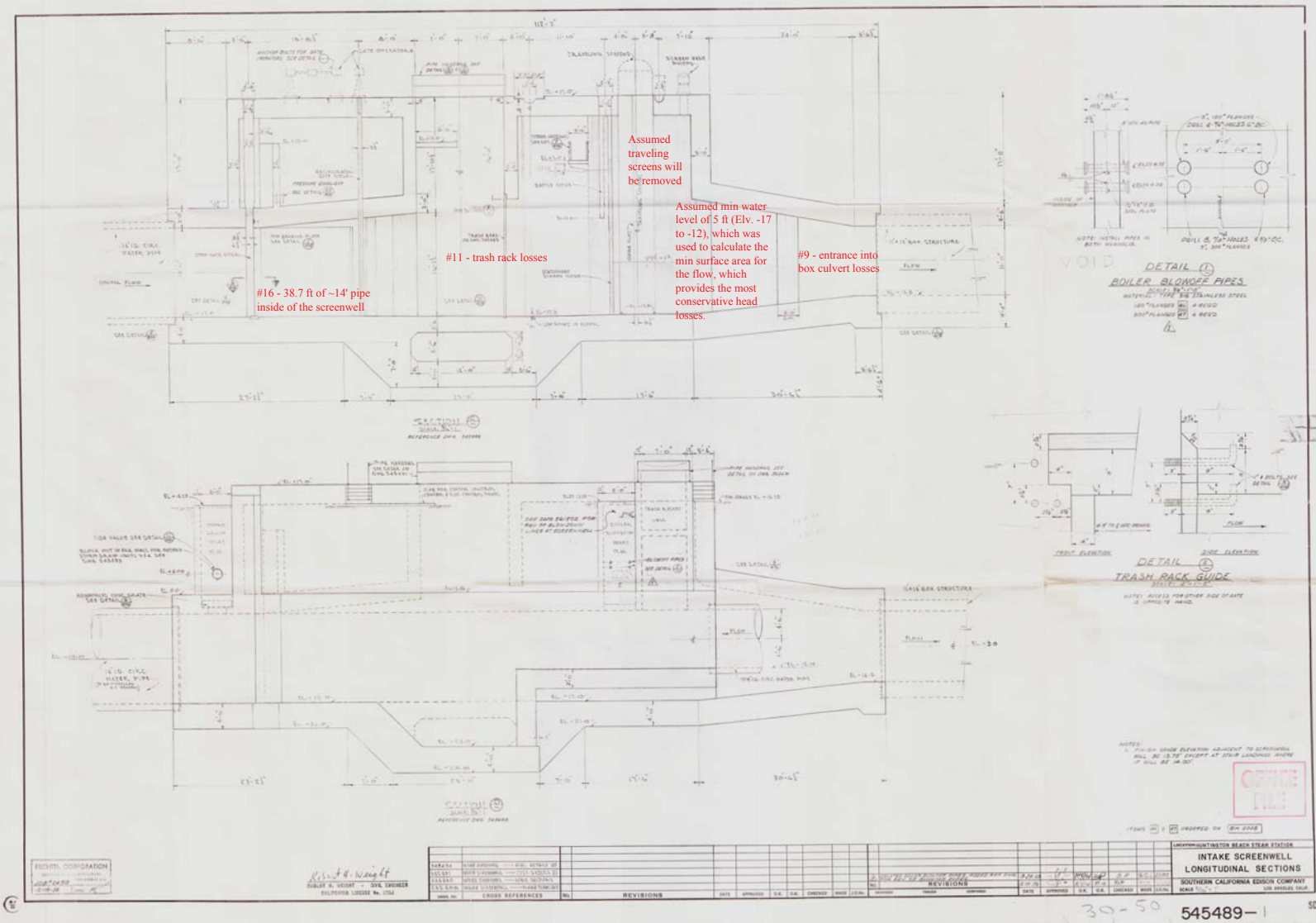
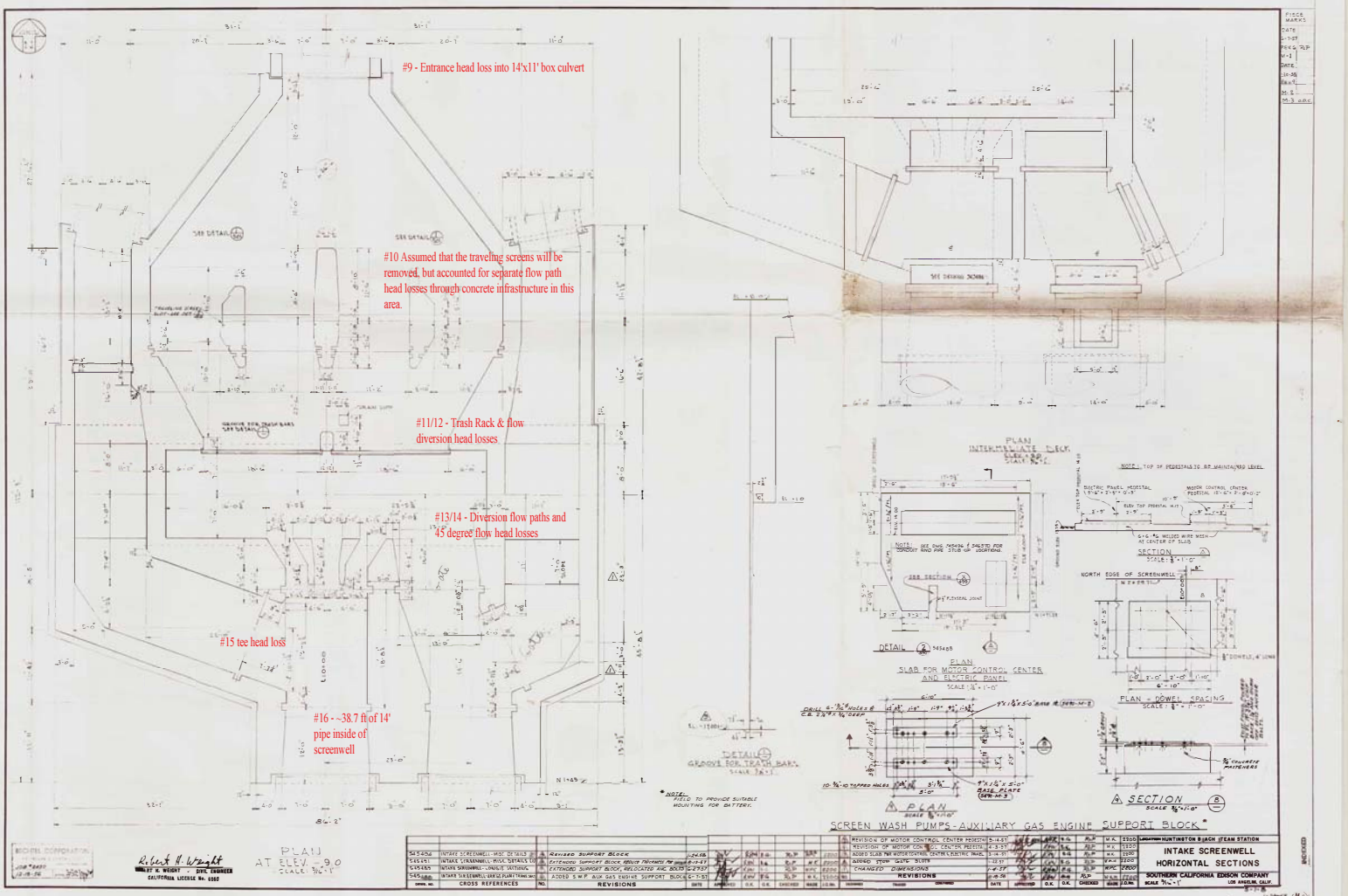
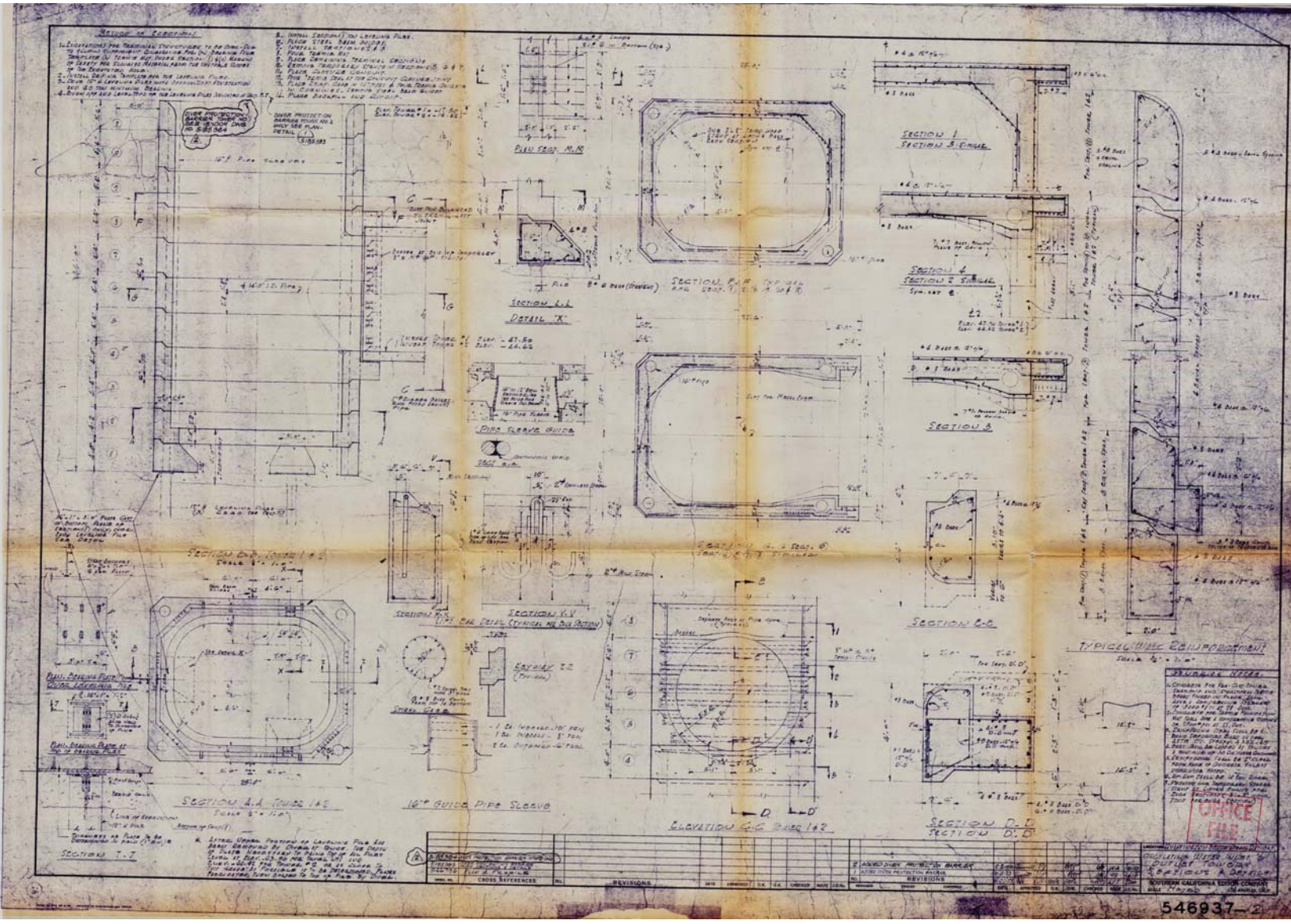

Assumed traveling screens will be removed
Assumed min water level of 5 ft (Elv. -17 to -12), which was used to calculate the min surface area for the flow, which provides the most conservative head losses.
#11 - trash rack losses
#9 - entrance into box culvert losses
#16 - 38.7 ft of ~14' pipe inside of the screenwell
DETAIL
BOILER BLOWOFF PIPES
TRASH RACK GUIDE
INTAKE SCREENWELL
LONGITUDINAL SECTIONS
SOUTHERN CALIFORNIA EDISON COMPANY
545489-1


#9 - Entrance head loss into 14'x11' box culvert
#10 Assumed that the traveling screens will be removed, but accounted for separate flow path head losses through concrete infrastructure in this area.
#11/12 - Trash Rack & flow diversion head losses
#13/14 - Diversion flow paths and 45 degree flow head losses
#15 tee head loss
#16 - ~38.7 ft of 14' pipe inside of screenwell
PLAN
INTERMEDIATE BLOCK
SECTION
NORTH EDGE OF SCREENWELL
DETAIL
PLAN
SLAB FOR MOTOR CONTROL CENTER AND ELECTRIC PANEL
PLAN - DOWEL SPACING
PLAN
SECTION
DETAIL
GROOVE FOR TRASH BARS
SCREEN WASH PUMPS-AUXILIARY GAS ENGINE SUPPORT BLOCK
PLAN
AT ELEV. -9.0
Robert H. Wright
CALIFORNIA LICENSE No. 6960
REVISIONS
CROSS REFERENCES
INTAKE SCREENWELL
HORIZONTAL SECTIONS
SOUTHERN CALIFORNIA EDISON COMPANY
545490- 9



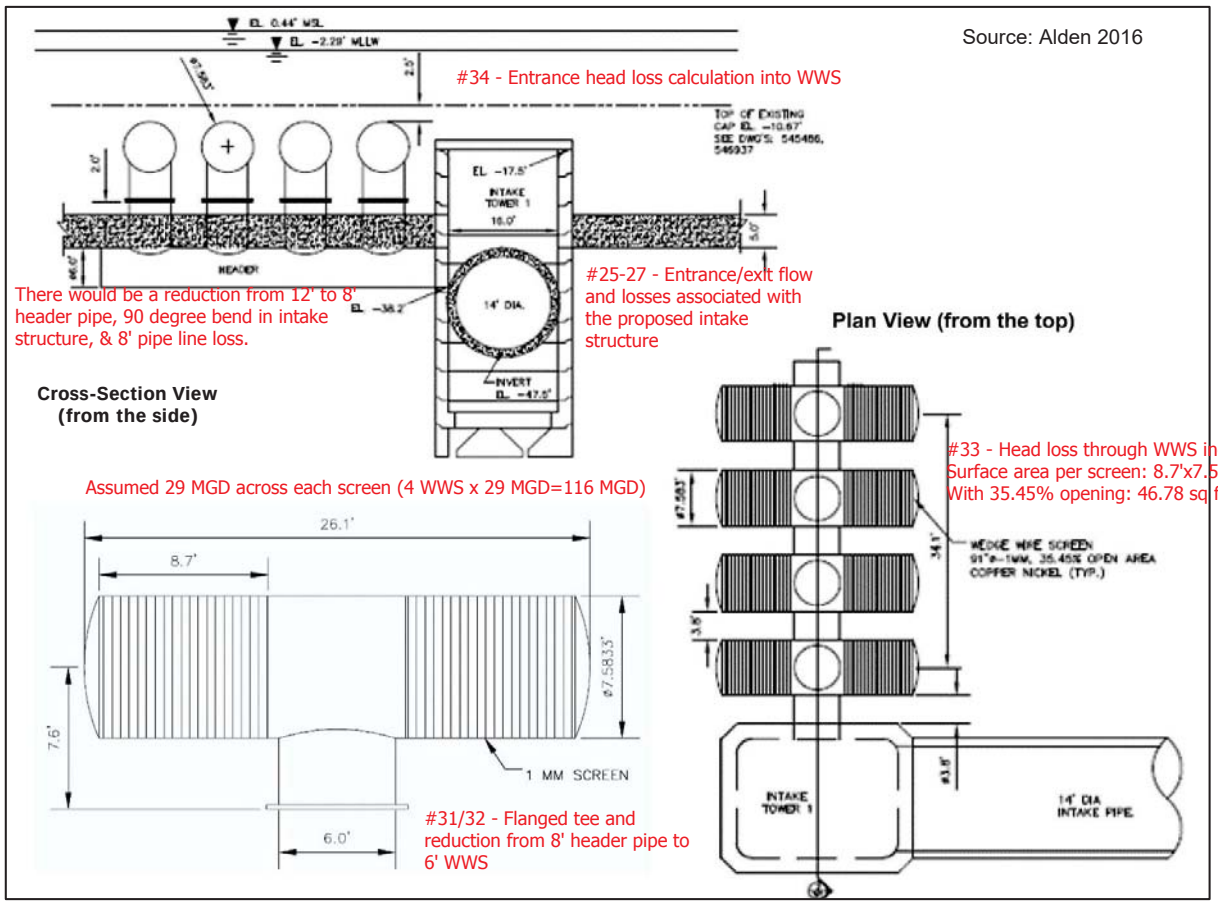
Method of Erection
Plan Sect. M.M
Section L.L
Detail "X"
Section F.F
Section 1
Section 3 Similar
Section 4
Section 2 Similar
Section 3
Pipe Sleeve Guide
Section E.B
Section V.V
Section A.A Tower 1&2
16" Guide Pipe Sleeve
Elevation C.C Tower 1&2
Section D.D
Section D'.D'
Section T.T
Typical Wall Reinforcement
Generic Notes
Cross References
Revisions
Southern California Edison Company
Office File
546937-2

#18 - 1,500 ft existing 14' offshore intake pipe

#19-23 - Entrance/exit, intake tower, reduction from 14' to 12' pipe & 90 degree bend from intake tower to proposed 12' pipe

#24 - 12' proposed offshore concrete intake pipe

Figure 2-7a. Wedgewire Screen Shape and Dimensions

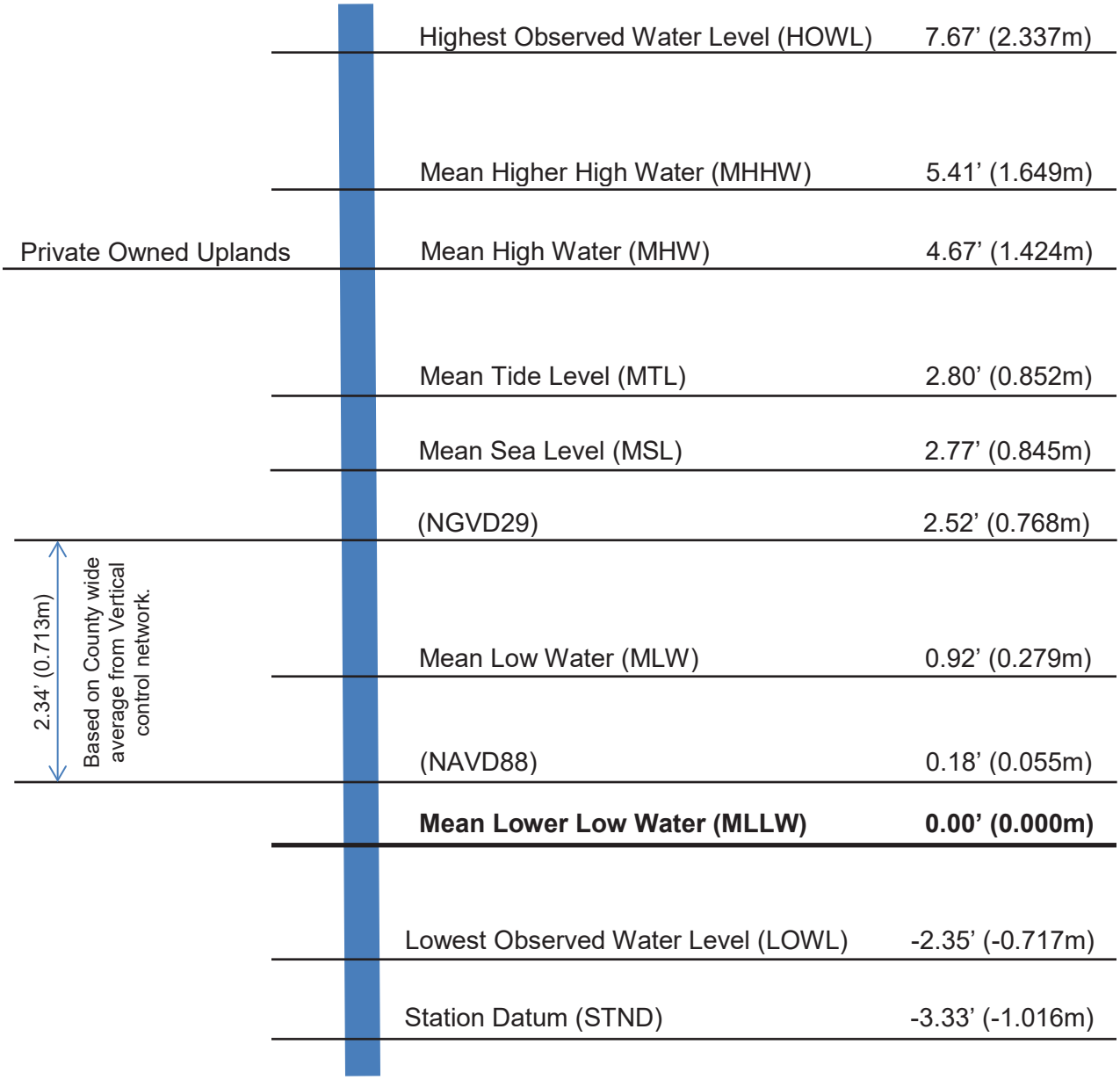


County of Orange / OC Public Works / OC Survey - Geodetic Unit

Tidal Datums for Newport Beach, Newport Bay Entrance CA

The chart below displays tidal datums for Newport Bay Entrance, Newport Beach and are based on the following:

Station ID	9410580
Status	Accepted (Apr 17 2003)
Length of Series	14 years
Time Period	1980-1994
Tidal Epoch	1983-2001
Datum	STND



Information was taken from National Ocean Service (NOS), Tides and Currents website October 29, 2015

ATTACHMENT L

Head Loss Calculations

PROPOSED INTAKE CONDITION ANALYSIS (OFFSHORE ONLY, 12 FT PIPE)

ID#	DESCRIPTION	ID in	ID ft	Area SF	Length ft	Q mgd	Q CFS	V fps	HW Coeff	K Minor Loss	H_L ft	ASSUMPTIONS
1	Entrance Into Pump Inlet	29.0	2.42	4.59		29	44.87	9.78		1.00	1.486	Entrance losses into pump intake (assumed 29 MGD per pump, 116 MGD total)
2	Exit into IPS	132.0	11.00	95.03		116	179.48	1.89		0.50	0.028	Exit losses from 12' pipe into IPS
3	Tee, Flanged	132.0	11.00	95.03		116	179.48	1.89		1.00	0.055	Tee inline with pipe from existing IPS to proposed IPS
4	12' Proposed Intake Pipe to IPS	132.0	11.00	95.03	56.5	116	179.48	1.89	100		0.007	Proposed 12' concrete pipe between existing IPS to IPS
5	Entrance into IPS Pipe	156.0	13.00	132.73		116	179.48	1.35		1.00	0.028	Entrance losses from existing IPS to IPS pipe
6	45° Bend into IPS Pipe	156.0	13.00	132.73		116	179.48	1.35		0.45	0.013	Assuming 45 degree change in direction as it exits existing IPS
7	Box Culvert Exit	156.0	13.00	132.73		116	179.48	1.35		0.50	0.014	Exit losses from 14' culvert to existing IPS
8	14' Box Culvert	156.0	13.00	132.73	236.0	116	179.48	1.35	100		0.012	Box culvert distance between screenwell and existing IPS
9	Box Culvert Entrance	156.0	13.00	132.73		116	179.48	1.35		1.00	0.028	Entrance losses into 14' box culvert
10	Flow Diversion Around Concrete Pillars	NA	NA	167.51		116	179.48	1.07		1.00	0.053	Flow diversion around 3 concrete pillars
11	Trash Racks	NA	NA	185.00		116	179.48	0.97		N/A	0.670	Manufacturer Assumed Head Loss Across Trash Rack
12	Flow Diversion Through 2 Trash Racks	NA	NA	185.00		116	179.48	0.97		1.00	0.029	Assuming tee for flow direction diversion to 2x trash racks (18.5'x5'[min water level]x2)
13	45° Bend into Screenwell	48.1	4.01	12.63		39	59.83	4.74		0.45	0.313	Assuming 45 degree change in direction from 2x 4'1.5" lines
14	Flow Diversion Through 3 Flow Paths	NA	NA	66.51		116	179.48	2.70		1.00	0.113	Assuming tee for flow direction diversion to 2x 4'1.5" and 1x 5'0.5" lines
15	Tee Inside Entrance of Screenwell	156.0	13.00	119.24		116	179.48	1.51		1.00	0.035	Tee fitting inside of screenwell area prior to screens
16	Pipe Inside Intake Screenwell Structure	156.0	13.00	119.24	38.7	116	179.48	1.51	100		0.002	14' pipe inside the entrance of the screenwell
17	14' Existing Concrete Onshore Intake Pipe	156.0	13.00	119.24	930.0	116	179.48	1.51	100		0.059	14' pipe between screenwell and shoreline
18	14' Existing Offshore Intake Pipe	156.0	13.00	119.24	1500.0	116	179.48	1.51	100		0.096	Existing 14' concrete pipe, assuming up to 6" of marine growth & 2 ft of sediment on the bottom, located between existing 14' onshore pipe and existing intake structure
19	Exit Into Offshore Intake Pipe	156.0	13.00	119.24		116	179.48	1.51		0.50	0.018	Exit losses from intake tower to existing offshore 14' pipe
20	Intake Tower	NA	NA	490.00		116	179.48	0.37		1.00	0.0021	Losses inside of intake tower between existing 14' and proposed 12' pipe
21	Entrance into Intake Tower	132.0	11.00	95.03		116	179.48	1.89		1.00	0.055	Entrance losses from proposed 12' pipe into intake tower
22	Sudden Reduction 14' to 12' D1 =	156.0	13.00	119.24								Assuming reduction from existing 14' intake
	D2 =	132.0	11.00	95.03		116	179.48	1.89		0.06	0.004	structure to proposed 12' line to intake location (D2)
23	90° Bend from 14' to 12' Pipe	132.0	11.00	95.03		116	179.48	1.89		0.30	0.017	Assuming 90 degree elbow at existing 14' intake structure to proposed 12' line to new intake location
24	12' Proposed Offshore Concrete Intake Pipe	132.0	11.00	95.03	6474.0	116	179.48	1.89	100		0.765	Proposed 12' concrete pipe, assuming up to 6" of marine growth, located between existing 14' pipe to intake location (D2)
25	Exit Into Proposed 12' Intake Pipe	132.0	11.00	95.03		116	179.48	1.89		0.50	0.028	Exit losses from new WWS intake tower to proposed 12' pipe
26	New WWS Intake Tower	NA	NA	490.00		116	179.48	0.37		1.00	0.002	Losses inside of new WWS intake tower between proposed 12' pipe and 8' WWS header pipe
27	Entrance into New WWS Intake Tower	84.0	7.00	38.48		116	179.48	4.66		1.00	0.338	Entrance losses from 8' header pipe to proposed WWS intake tower
28	Sudden Reduction 12' to 8' D1 =	132.0	11.00	95.03								Pipe reduction from header pipe (12') to WWS
	D2 =	84.0	7.00	38.48		116	179.48	4.66		2.16	0.729	header pipe (8')
29	90° Bend from 12' to 8' Pipe	84.0	7.00	38.48		116	179.48	4.66		0.30	0.101	Assuming 90 degree elbow from 12' pipe at proposed WWS intake structure to 8' WWS header pipe at new intake location (D2)
30	8' Proposed WWS Header Pipe	84.0	7.00	38.48	45.0	116	179.48	4.66	100		0.048	Proposed 8' concrete WWS header pipe, assuming up to 6" of marine growth, located at intake location (D2)
31	Sudden Reduction 8' to 6' D1 =	84.0	7.00	38.48								Pipe reduction from header pipe (8') for 4 WWS
	D2 =	60.0	5.00	19.63		29	44.87	2.29		0.92	0.299	tees (6')
32	Tee, Flanged	60.0	5.00	19.63		29	44.87	2.29		1.00	0.324	Assuming tee on wedgewire screens to main header pipe (x4)
33	Wedge wire screens	NA	NA	46.78		29	44.87	0.96		4.15	0.237	Head losses across wedge wire screens pore space (x4)
34	Entrance	60.0	5.00	19.63		29	44.87	2.29		1.00	0.324	Entrance losses into wede wire screens (x4)
							Sub-Total Head Loss			Σ =	6.336	Miscellaneous losses to account for missing fittings and flow obstacles not shown or specified on as-built records
							Miscellaneous		20%		1.267	
							Total Head Loss			**Σ =**	**7.603**	

Datum	Value	Description
MSL	2.78	Mean Sea Level
MLLW	0.0	Mean Lower-Low Water
NAVD88	0.18	North American Vertical Datum of 1988
Min Tide	-2.35	Lowest Observed Tide
Min Level	-10	Minimum water level at lowest tide and max flow
Max Head Loss	**7.65**	**Maximum allowable head loss**

Hazen-Williams Equation:

$H_L = 3.022 \times L \times (V/HW)^{1.85}/D^{1.17}$

Minor Loss Equation:

$H_L = K \times (V^2/2g)$

Notes:

L= Length of pipe (ft)

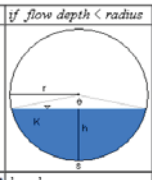
V= Velocity (ft/sec)

HW= Hazen-Williams Coefficient

D= Inner Pipe Diameter (ft)

K= Minor Loss Coefficient

g= Acceleration due to gravity (32.2 ft/s^2)

step	solve for	*if flow depth < radius*
		
1	circular segment height	$h = d$
2	central angle	$\theta = 2\arccos\left(\frac{r-h}{r}\right)$
3	circular segment area	$K = \frac{r^2(\theta - \sin\theta)}{2}$
4	arc length	$s = r \times \theta$
5	flow area	$A = K$
6	wetted perimeter	$P_w = s$
7	hydraulic radius	$R_h = \frac{A}{P_w}$

Assumed 2 ft of sand in the bottom of the existing 14' pipe to screenwell

14 ft pipe

Θ= 1.550387

K= 13.48958

Θ= Central Angle

K= Assumed Entrained Sand Area

Note: I subtracted the assumed entrained sand area from the Area (SF) calculations

PROPOSED INTAKE CONDITION ANALYSIS (OFFSHORE ONLY, 10 FT PIPE)

ID#	DESCRIPTION	ID in	ID ft	Area SF	Length ft	Q mgd	Q CFS	V fps	HW Coeff	K Minor Loss	H_L ft	ASSUMPTIONS
1	Entrance Into Pump Inlet	29.0	2.42	4.59		29	44.87	9.78		1.00	1.486	Entrance losses into pump intake (assumed 29 MGD per pump, 116 MGD total)
2	Exit into IPS	108.0	9.00	63.62		116	179.48	2.82		0.50	0.062	Exit losses from 10' pipe into IPS
3	Tee, Flanged	108.0	9.00	63.62		116	179.48	2.82		1.00	0.124	Tee inline with pipe from existing IPS to proposed IPS
4	10' Proposed Intake Pipe to IPS	108.0	9.00	63.62	56.5	116	179.48	2.82	100		0.018	Proposed 10' concrete pipe between existing IPS to IPS
5	Entrance into IPS Pipe	156.0	13.00	132.73		116	179.48	1.35		1.00	0.028	Entrance losses from existing IPS to IPS pipe
6	45° Bend into IPS Pipe	156.0	13.00	132.73		116	179.48	1.35		0.45	0.013	Assuming 45 degree change in direction as it exits existing IPS
7	Box Culvert Exit	156.0	13.00	132.73		116	179.48	1.35		0.50	0.014	Exit losses from 14' culvert to existing IPS
8	14' Box Culvert	156.0	13.00	132.73	236.0	116	179.48	1.35	100		0.012	Box culvert distance between screenwell and existing IPS
9	Box Culvert Entrance	156.0	13.00	132.73		116	179.48	1.35		1.00	0.028	Entrance losses into 14' box culvert
10	Flow Diversion Around Concrete Pillars	NA	NA	167.51		116	179.48	1.07		1.00	0.053	Flow diversion around 3 concrete pillars
11	Trash Racks	NA	NA	185.00		116	179.48	0.97		N/A	0.670	Manufacturer Assumed Head Loss Across Trash Rack
12	Flow Diversion Through 2 Trash Racks	NA	NA	185.00		116	179.48	0.97		1.00	0.029	Assuming tee for flow direction diversion to 2x trash racks (18.5'x5'[min water level]x2)
13	45° Bend into Screenwell	48.1	4.01	12.63		39	59.83	4.74		0.45	0.313	Assuming 45 degree change in direction from 2x 4'1.5" lines
14	Flow Diversion Through 3 Flow Paths	NA	NA	66.51		116	179.48	2.70		1.00	0.113	Assuming tee for flow direction diversion to 2x 4'1.5" and 1x 5'0.5" lines
15	Tee Inside Entrance of Screenwell	156.0	13.00	119.24		116	179.48	1.51		1.00	0.035	Tee fitting inside of screenwell area prior to screens
16	Pipe Inside Intake Screenwell Structure	156.0	13.00	119.24	38.7	116	179.48	1.51	100		0.002	14' pipe inside the entrance of the screenwell
17	14' Existing Concrete Onshore Intake Pipe	156.0	13.00	119.24	930.0	116	179.48	1.51	100		0.059	14' pipe between screenwell and shoreline
18	14' Existing Offshore Intake Pipe	156.0	13.00	119.24	1500.0	116	179.48	1.51	100		0.096	Existing 14' concrete pipe, assuming up to 6" of marine growth & 2 ft of sediment on the bottom, located between existing 14' onshore pipe and existing intake structure
19	Exit Into Offshore Intake Pipe	156.0	13.00	119.24		116	179.48	1.51		0.50	0.018	Exit losses from intake tower to existing offshore 14' pipe
20	Intake Tower	NA	NA	490.00		116	179.48	0.37		1.00	0.0021	Losses inside of intake tower between existing 14' and proposed 10' pipe
21	Entrance into Intake Tower	108.0	9.00	63.62		116	179.48	2.82		1.00	0.124	Entrance losses from proposed 10' pipe into intake tower
22	Sudden Reduction 14' to 10' D1 =	156.0	13.00	119.24								Assuming reduction from existing 14' intake
	D2 =	108.0	9.00	63.62		116	179.48	2.82		0.76	0.094	structure to proposed 10' line to intake location (D2)
23	90° Bend from 14' to 10' Pipe	108.0	9.00	63.62		116	179.48	2.82		0.30	0.037	Assuming 90 degree elbow at existing 14' intake structure to proposed 10' line to new intake location
24	10' Proposed Offshore Concrete Intake Pipe	108.0	9.00	63.62	6474.0	116	179.48	2.82	100		2.034	Proposed 10' concrete pipe, assuming up to 6" of marine growth, located between existing 14' pipe to intake location (D2)
25	Exit Into Proposed 10' Intake Pipe	108.0	9.00	63.62		116	179.48	2.82		0.50	0.062	Exit losses from new WWS intake tower to proposed 10' pipe
26	New WWS Intake Tower	NA	NA	490.00		116	179.48	0.37		1.00	0.0021	Losses inside of new WWS intake tower between proposed 10' pipe and 8' WWS header pipe
27	Entrance into New WWS Intake Tower	84.0	7.00	38.48		116	179.48	4.66		1.00	0.338	Entrance losses from 8' header pipe to proposed WWS intake tower
28	Sudden Reduction 10' to 8' D1 =	108.0	9.00	63.62								Pipe reduction from header pipe (10') to WWS
	D2 =	84.0	7.00	38.48		116	179.48	4.66		0.43	0.144	header pipe (8')
29	90° Bend from 10' to 8' Pipe	84.0	7.00	38.48		116	179.48	4.66		0.30	0.101	Assuming 90 degree elbow from 10' pipe at proposed WWS intake structure to 8' WWS header pipe at new intake location (D2)
30	8' Proposed WWS Header Pipe	84.0	7.00	38.48	45.0	116	179.48	4.66	100		0.048	Proposed 8' concrete WWS header pipe, assuming up to 6" of marine growth, located at intake location (D2)
31	Sudden Reduction 8' to 6' D1 =	84.0	7.00	38.48								Pipe reduction from header pipe (8') for 4 WWS
	D2 =	60.0	5.00	19.63		29	44.87	2.29		0.92	0.299	tees (6')
32	Tee, Flanged	60.0	5.00	19.63		29	44.87	2.29		1.00	0.324	Assuming tee on wedgewire screens to main header pipe (x4)
33	Wedge wire screens	NA	NA	46.78		29	44.87	0.96		4.15	0.237	Head losses across wedge wire screens pore space (x4)
34	Entrance	60.0	5.00	19.63		29	44.87	2.29		1.00	0.324	Entrance losses into wede wire screens (x4)
							Sub-Total Head Loss			Σ =	7.346	Miscellaneous losses to account for missing fittings and flow obstacles not shown or specified on as-built records
							Miscellaneous		20%		1.469	
							Total Head Loss			**Σ =**	**8.815**	

Datum	Value	Description
MSL	2.78	Mean Sea Level
MLLW	0.0	Mean Lower-Low Water
NAVD88	0.18	North American Vertical Datum of 1988
Min Tide	-2.35	Lowest Observed Tide
Min Level	-10	Minimum water level at lowest tide and max flow
Max Head Loss	**7.65**	**Maximum allowable head loss**

Hazen-Williams Equation:

$H_L = 3.022 \times L \times (V/HW)^{1.85}/D^{1.17}$

Minor Loss Equation:

$H_L = K \times (V^2/2g)$

Notes:

L= Length of pipe (ft)

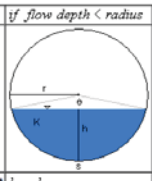
V= Velocity (ft/sec)

HW= Hazen-Williams Coefficient

D= Inner Pipe Diameter (ft)

K= Minor Loss Coefficient

g= Acceleration due to gravity (32.2 ft/s^2)

step	solve for	if flow depth < radius
		
1	circular segment height	$h = d$
2	central angle	$\theta = 2\arccos\left(\frac{r-h}{r}\right)$
3	circular segment area	$K = \frac{r^2(\theta - \sin\theta)}{2}$
4	arc length	$s = r \times \theta$
5	flow area	$A = K$
6	wetted perimeter	$P_W = s$
7	hydraulic radius	$R_h = \frac{A}{P_W}$

Assumed 2 ft of sand in the bottom of the existing 14' pipe to screenwell

14 ft pipe

Θ= 1.550387

K= 13.48958

Θ= Central Angle

K= Assumed Entrained Sand Area

Note: I subtracted the assumed entrained sand area from the Area (SF) calculations

PROPOSED INTAKE CONDITION ANALYSIS (ONSHORE 14 FT - OFFSHORE ONLY, 12 FT)

ID#	DESCRIPTION	ID in	ID ft	Area SF	Length ft	Q mgd	Q CFS	V fps	HW Coeff	K Minor Loss	H_L ft	ASSUMPTIONS
1	Entrance Into Pump Inlet	29.0	2.42	4.59		29	44.87	9.78		1.00	1.486	Entrance losses into pump intake (assumed 29 MGD per pump, 116 MGD total)
2	Exit into IPS	132.0	11.00	95.03		116	179.48	1.89		0.50	0.028	Exit losses from 12' pipe into IPS
3	Tee, Flanged	132.0	11.00	95.03		116	179.48	1.89		1.00	0.055	Tee inline with pipe from existing IPS to proposed IPS
4	12' Proposed Intake Pipe to IPS	132.0	11.00	95.03	56.5	116	179.48	1.89	100		0.007	Proposed 12' concrete pipe between existing IPS to IPS
5	Entrance into IPS Pipe	156.0	13.00	132.73		116	179.48	1.35		1.00	0.028	Entrance losses from existing IPS to IPS pipe
6	45° Bend into IPS Pipe	156.0	13.00	132.73		116	179.48	1.35		0.45	0.013	Assuming 45 degree change in direction as it exits existing IPS
7	Box Culvert Exit	156.0	13.00	132.73		116	179.48	1.35		0.50	0.014	Exit losses from 14' culvert to existing IPS
8	14' Box Culvert	156.0	13.00	132.73	236.0	116	179.48	1.35	100		0.012	Box culvert distance between screenwell and existing IPS
9	Box Culvert Entrance	156.0	13.00	132.73		116	179.48	1.35		1.00	0.028	Entrance losses into 14' box culvert
10	Flow Diversion Around Concrete Pillars	NA	NA	167.51		116	179.48	1.07		1.00	0.053	Flow diversion around 3 concrete pillars
11	Trash Racks	NA	NA	185.00		116	179.48	0.97		N/A	0.670	Manufacturer Assumed Head Loss Across Trash Rack
12	Flow Diversion Through 2 Trash Racks	NA	NA	185.00		116	179.48	0.97		1.00	0.029	Assuming tee for flow direction diversion to 2x trash racks (18.5'x5'[min water level]x2)
13	45° Bend into Screenwell	48.1	4.01	12.63		39	59.83	4.74		0.45	0.313	Assuming 45 degree change in direction from 2x 4'1.5" lines
14	Flow Diversion Through 3 Flow Paths	NA	NA	66.51		116	179.48	2.70		1.00	0.113	Assuming tee for flow direction diversion to 2x 4'1.5" and 1x 5'0.5" lines
15	Tee Inside Entrance of Screenwell	156.0	13.00	119.24		116	179.48	1.51		1.00	0.035	Tee fitting inside of screenwell area prior to screens
16	Pipe Inside Intake Screenwell Structure	156.0	13.00	119.24	38.7	116	179.48	1.51	100		0.002	14' pipe inside the entrance of the screenwell
17	14' Existing Concrete Onshore Intake Pipe	156.0	13.00	119.24	640.0	116	179.48	1.51	100		0.041	Existing 14' pipe, assuming up to 6" of marine growth & 2 ft of sediment on the bottom, located between screenwell and connection point
18	90° Bend from 14' Pipe	156.0	13.00	132.73		116	179.48	1.35		0.30	0.009	Assuming 90 degree elbow at onshore 14' pipe to proposed 14' onshore line
19	14' Proposed Onshore Intake Pipe	156.0	13.00	132.73	5917.0	116	179.48	1.35	100		0.310	Proposed 14' onshore pipe, assuming up to 6" of marine growth, located between existing 14' onshore pipe and offshore 12' line
20	Exit Into Proposed 14' Intake Pipe	156.0	13.00	132.73		116	179.48	1.35		0.50	0.014	Exit losses from concrete vault to proposed 14' pipe
21	Onshore Concrete Vault	NA	NA	490.00		116	179.48	0.37		1.00	0.0021	Losses inside of new onshore concrete vault between proposed 14' and 12' pipe
22	Entrance into Onshore Concrete Vault	132.0	11.00	95.03		116	179.48	1.89		1.00	0.055	Entrance losses from proposed offshore 12' to proposed onshore concrete vault
23	Sudden Reduction 14' to 12' D1 =	156.0	13.00	132.73								Assuming reduction from proposed onshore 14'
	D2 =	132.0	11.00	95.03		116	179.48	1.89		0.16	0.009	pipe to proposed 12' line to intake location (D2)
24	90° Bend from 14' to 12' Pipe	132.0	11.00	95.03		116	179.48	1.89		0.30	0.017	Assuming 90 degree elbow at onshore 14' pipe to proposed 12' line to new intake location (D2)
25	12' Proposed Offshore Concrete Intake Pipe	132.0	11.00	95.03	2598.0	116	179.48	1.89	100		0.307	Proposed 12' concrete pipe, assuming up to 6" of marine growth, located between onshore 14' pipe to intake location (D2)
26	Exit Into Proposed 12' Intake Pipe	132.0	11.00	95.03		116	179.48	1.89		0.50	0.028	Exit losses from new WWS intake tower to proposed 12' pipe
27	New WWS Intake Tower	NA	NA	490.00		116	179.48	0.37		1.00	0.0021	Losses inside of new WWS intake tower between proposed 12' pipe and 8' WWS header pipe
28	Entrance into New WWS Intake Tower	84.0	7.00	38.48		116	179.48	4.66		1.00	0.338	Entrance losses from 8' header pipe to proposed WWS intake tower
29	Sudden Reduction 12' to 8' D1 =	132.0	11.00	95.03								Pipe reduction from header pipe (12') to WWS
	D2 =	84.0	7.00	38.48		116	179.48	4.66		2.16	0.729	header pipe (8')
30	90° Bend from 12' to 8' Pipe	84.0	7.00	38.48		116	179.48	4.66		0.30	0.101	Assuming 90 degree elbow from 12' pipe at proposed WWS intake structure to 8' WWS header pipe at new intake location (D2)
31	8' Proposed WWS Header Pipe	84.0	7.00	38.48	45.0	116	179.48	4.66	100		0.048	Proposed 8' concrete WWS header pipe, assuming up to 6" of marine growth, located at intake location (D2)
32	Sudden Reduction 8' to 6' D1 =	84.0	7.00	38.48								Pipe reduction from header pipe (8') for 4 WWS
	D2 =	60.0	5.00	19.63		29	44.87	2.29		0.92	0.299	tees (6')
33	Tee, Flanged	60.0	5.00	19.63		29	44.87	2.29		1.00	0.324	Assuming tee on wedgewire screens to main header pipe (x4)
34	Wedge wire screens	NA	NA	46.78		29	44.87	0.96		4.15	0.237	Head losses across wedge wire screens pore space (x4)
35	Entrance	60.0	5.00	19.63		29	44.87	2.29		1.00	0.324	Entrance losses into wede wire screens (x4)
							Sub-Total Head Loss			Σ =	6.083	
							Miscellaneous		20%		1.217	Miscellaneous losses to account for missing fittings and flow obstacles not shown or specified on as-built records
							Total Head Loss			**Σ =**	**7.300**	

Datum	Value	Description
MSL	2.78	Mean Sea Level
MLLW	0.0	Mean Lower-Low Water
NAVD88	0.18	North American Vertical Datum of 1988
Min Tide	-2.35	Lowest Observed Tide
Min Level	-10	Minimum water level at lowest tide and max flow
Max Head Loss	**7.65**	**Maximum allowable head loss**

Hazen-Williams Equation:

$H_L = 3.022 \times L \times (V/HW)^{1.85}/D^{1.17}$

Minor Loss Equation:

$H_L = K \times (V^2/2g)$

Notes:

L= Length of pipe (ft)

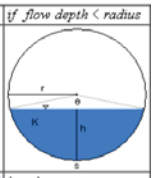
V= Velocity (ft/sec)

HW= Hazen-Williams Coefficient

D= Inner Pipe Diameter (ft)

K= Minor Loss Coefficient

g= Acceleration due to gravity (32.2 ft/s^2)

step	solve for	*if flow depth < radius*
		
1	circular segment height	$h = d$
2	central angle	$\theta = 2\arccos\left(\frac{r-h}{r}\right)$
3	circular segment area	$K = \frac{r^2(\theta - \sin\theta)}{2}$
4	arc length	$s = r \times \theta$
5	flow area	$A = K$
6	wetted perimeter	$P_w = s$
7	hydraulic radius	$R_h = \frac{A}{P_w}$

Assumed 2 ft of sand in the bottom of the existing 14' pipe to screenwell

14 ft pipe

Θ= 1.550387

K= 13.48958

Θ= Central Angle

K= Assumed Entrained Sand Area

Note: I subtracted the assumed entrained sand area from the Area (SF) calculations

PROPOSED INTAKE CONDITION ANALYSIS (ONSHORE 12 FT - OFFSHORE ONLY, 12 FT)

ID#	DESCRIPTION	ID in	ID ft	Area SF	Length ft	Q mgd	Q CFS	V fps	HW Coeff	K Minor Loss	H_L ft	ASSUMPTIONS
1	Entrance Into Pump Inlet	29.0	2.42	4.59		29	44.87	9.78		1.00	1.486	Entrance losses into pump intake (assumed 29 MGD per pump, 116 MGD total)
2	Exit into IPS	132.0	11.00	95.03		116	179.48	1.89		0.50	0.028	Exit losses from 12' pipe into IPS
3	Tee, Flanged	132.0	11.00	95.03		116	179.48	1.89		1.00	0.055	Tee inline with pipe from existing IPS to proposed IPS
4	12' Proposed Intake Pipe to IPS	132.0	11.00	95.03	56.5	116	179.48	1.89	100		0.007	Proposed 12' concrete pipe between existing IPS to IPS
5	Entrance into IPS Pipe	156.0	13.00	132.73		116	179.48	1.35		1.00	0.028	Entrance losses from existing IPS to IPS pipe
6	45° Bend into IPS Pipe	156.0	13.00	132.73		116	179.48	1.35		0.45	0.013	Assuming 45 degree change in direction as it exits existing IPS
7	Box Culvert Exit	156.0	13.00	132.73		116	179.48	1.35		0.50	0.014	Exit losses from 14' culvert to existing IPS
8	14' Box Culvert	156.0	13.00	132.73	236.0	116	179.48	1.35	100		0.012	Box culvert distance between screenwell and existing IPS
9	Box Culvert Entrance	156.0	13.00	132.73		116	179.48	1.35		1.00	0.028	Entrance losses into 14' box culvert
10	Flow Diversion Around Concrete Pillars	NA	NA	167.51		116	179.48	1.07		1.00	0.053	Flow diversion around 3 concrete pillars
11	Trash Racks	NA	NA	185.00		116	179.48	0.97		N/A	0.670	Manufacturer Assumed Head Loss Across Trash Rack
12	Flow Diversion Through 2 Trash Racks	NA	NA	185.00		116	179.48	0.97		1.00	0.029	Assuming tee for flow direction diversion to 2x trash racks (18.5'x5'[min water level]x2)
13	45° Bend into Screenwell	48.1	4.01	12.63		39	59.83	4.74		0.45	0.313	Assuming 45 degree change in direction from 2x 4'1.5" lines
14	Flow Diversion Through 3 Flow Paths	NA	NA	66.51		116	179.48	2.70		1.00	0.113	Assuming tee for flow direction diversion to 2x 4'1.5" and 1x 5'0.5" lines
15	Tee Inside Entrance of Screenwell	156.0	13.00	119.24		116	179.48	1.51		1.00	0.035	Tee fitting inside of screenwell area prior to screens
16	Pipe Inside Intake Screenwell Structure	156.0	13.00	119.24	38.7	116	179.48	1.51	100		0.002	14' pipe inside the entrance of the screenwell
17	14' Existing Concrete Onshore Intake Pipe	156.0	13.00	119.24	640.0	116	179.48	1.51	100		0.041	Existing 14' pipe, assuming up to 6" of marine growth & 2 ft of sediment on the bottom, located between screenwell and connection point
18	Sudden Reduction 14' to 12' D1 =	156.0	13.00	119.24								Assuming reduction from proposed onshore 14'
	D2 =	132.0	11.00	95.03		116	179.48	1.89		0.06	0.004	pipe to proposed 12' onshore line
19	90° Bend on Onshore 12' Pipe	132.0	11.00	95.03		116	179.48	1.89		0.30	0.017	Assuming 90 degree elbow after reduction from onshore 14' pipe to proposed 12' onshore line
20	12' Proposed Onshore Intake Pipe	132.0	11.00	95.03	5917.0	116	179.48	1.89	100		0.700	Proposed 12' concrete pipe, assuming up to 6" of marine growth, located between existing 14' onshore pipe and proposed offshore 12' line to intake structure
23	Exit Into Proposed 12' Intake Pipe	132.0	11.00	95.03		116	179.48	1.89		0.50	0.028	Exit losses from concrete vault to proposed 12' pipe
24	Onshore Concrete Vault	NA	NA	490.00		116	179.48	0.37		1.00	0.0021	Losses inside of new onshore concrete vault between proposed onshore and offshore 12' pipe
25	Entrance into Onshore Concrete Vault	132.0	11.00	95.03		116	179.48	1.89		1.00	0.055	Entrance losses from proposed offshore 12' to proposed onshore concrete vault
21	90° Bend on Offshore 12' Pipe	132.0	11.00	95.03		116	179.48	1.89		0.30	0.017	Assuming 90 degree elbow at onshore 12' pipe to proposed 12' line to new intake location (D2)
22	12' Proposed Offshore Concrete Intake Pipe	132.0	11.00	95.03	2598.0	116	179.48	1.89	100		0.307	Proposed 12' concrete pipe, assuming up to 6" of marine growth, located between onshore 12' pipe to intake location (D2)
23	Exit Into Proposed 12' Intake Pipe	132.0	11.00	95.03		116	179.48	1.89		0.50	0.028	Exit losses from new WWS intake tower to proposed 12' pipe
24	New WWS Intake Tower	NA	NA	490.00		116	179.48	0.37		1.00	0.002	Losses inside of new WWS intake tower between proposed 12' pipe and 8' WWS header pipe
25	Entrance into New WWS Intake Tower	84.0	7.00	38.48		116	179.48	4.66		1.00	0.338	Entrance losses from 8' header pipe to proposed WWS intake tower
26	Sudden Reduction 12' to 8' D1 =	132.0	11.00	95.03								Pipe reduction from header pipe (12') to WWS
	D2 =	84.0	7.00	38.48		116	179.48	4.66		2.16	0.729	header pipe (8')
27	90° Bend from 12' to 8' Pipe	84.0	7.00	38.48		116	179.48	4.66		0.30	0.101	Assuming 90 degree elbow from 12' pipe at proposed WWS intake structure to 8' WWS header pipe at new intake location (D2)
28	8' Proposed WWS Header Pipe	84.0	7.00	38.48	45.0	116	179.48	4.66	100		0.048	Proposed 8' concrete WWS header pipe, assuming up to 6" of marine growth, located at intake location (D2)
29	Sudden Reduction 8' to 6' D1 =	84.0	7.00	38.48								Pipe reduction from header pipe (8') for 4 WWS
	D2 =	60.0	5.00	19.63		29	44.87	2.29		0.92	0.299	tees (6')
30	Tee, Flanged	60.0	5.00	19.63		29	44.87	2.29		1.00	0.324	Assuming tee on wedgewire screens to main header pipe (x4)
31	Wedge wire screens	NA	NA	46.78		29	44.87	0.96		4.15	0.237	Head losses across wedge wire screens pore space (x4)
32	Entrance	60.0	5.00	19.63		29	44.87	2.29		1.00	0.324	Entrance losses into wede wire screens (x4)
							Sub-Total Head Loss			Σ =	6.489	Miscellaneous losses to account for missing fittings and flow obstacles not shown or specified on as-built records
							Miscellaneous		20%		1.298	
							Total Head Loss			**Σ =**	**7.787**	

Datum	Value	Description
MSL	2.78	Mean Sea Level
MLLW	0.0	Mean Lower-Low Water
NAVD88	0.18	North American Vertical Datum of 1988
Min Tide	-2.35	Lowest Observed Tide
Min Level	-10	Minimum water level at lowest tide and max flow
Max Head Loss	**7.65**	**Maximum allowable head loss**

Hazen-Williams Equation:

$H_L = 3.022 \times L \times (V/HW)^{1.85}/D^{1.17}$

Minor Loss Equation:

$H_L = K \times (V^2/2g)$

Notes:

L= Length of pipe (ft)

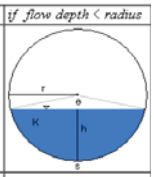
V= Velocity (ft/sec)

HW= Hazen-Williams Coefficient

D= Inner Pipe Diameter (ft)

K= Minor Loss Coefficient

g= Acceleration due to gravity (32.2 ft/s^2)

step	solve for	*if flow depth < radius*
		
1	circular segment height	$h = d$
2	central angle	$\theta = 2\arccos\left(\frac{r-h}{r}\right)$
3	circular segment area	$K = \frac{r^2(\theta - \sin\theta)}{2}$
4	arc length	$s = r \times \theta$
5	flow area	$A = K$
6	wetted perimeter	$P_w = s$
7	hydraulic radius	$R_h = \frac{A}{P_w}$

Assumed 2 ft of sand in the bottom of the existing 14' pipe to screenwell

14 ft pipe

Θ= 1.550387

K= 13.48958

Θ= Central Angle

K= Assumed Entrained Sand Area

Note: I subtracted the assumed entrained sand area from the Area (SF) calculations

PROPOSED INTAKE CONDITION ANALYSIS (ONSHORE 12 FT - OFFSHORE ONLY, 10 FT)

ID#	DESCRIPTION	ID in	ID ft	Area SF	Length ft	Q mgd	Q CFS	V fps	HW Coeff	K Minor Loss	H_L ft	ASSUMPTIONS
1	Entrance Into Pump Inlet	29.0	2.42	4.59		29	44.87	9.78		1.00	1.486	Entrance losses into pump intake (assumed 29 MGD per pump, 116 MGD total)
2	Exit into IPS	108.0	9.00	63.62		116	179.48	2.82		0.50	0.062	Exit losses from 10' pipe into IPS
3	Tee, Flanged	108.0	9.00	63.62		116	179.48	2.82		1.00	0.124	Tee inline with pipe from existing IPS to proposed IPS
4	10' Proposed Intake Pipe to IPS	108.0	9.00	63.62	56.5	116	179.48	2.82	100		0.018	Proposed 10' concrete pipe between existing IPS to IPS
5	Entrance into IPS Pipe	156.0	13.00	132.73		116	179.48	1.35		1.00	0.028	Entrance losses from existing IPS to IPS pipe
6	45° Bend into IPS Pipe	156.0	13.00	132.73		116	179.48	1.35		0.45	0.013	Assuming 45 degree change in direction as it exits existing IPS
7	Box Culvert Exit	156.0	13.00	132.73		116	179.48	1.35		0.50	0.014	Exit losses from 14' culvert to existing IPS
8	14' Box Culvert	156.0	13.00	132.73	236.0	116	179.48	1.35	100		0.012	Box culvert distance between screenwell and existing IPS
9	Box Culvert Entrance	156.0	13.00	132.73		116	179.48	1.35		1.00	0.028	Entrance losses into 14' box culvert
10	Flow Diversion Around Concrete Pillars	NA	NA	167.51		116	179.48	1.07		1.00	0.053	Flow diversion around 3 concrete pillars
11	Trash Racks	NA	NA	185.00		116	179.48	0.97		N/A	0.670	Manufacturer Assumed Head Loss Across Trash Rack
12	Flow Diversion Through 2 Trash Racks	NA	NA	185.00		116	179.48	0.97		1.00	0.029	Assuming tee for flow direction diversion to 2x trash racks (18.5'x5'[min water level]x2)
13	45° Bend into Screenwell	48.1	4.01	12.63		39	59.83	4.74		0.45	0.313	Assuming 45 degree change in direction from 2x 4'1.5" lines
14	Flow Diversion Through 3 Flow Paths	NA	NA	66.51		116	179.48	2.70		1.00	0.113	Assuming tee for flow direction diversion to 2x 4'1.5" and 1x 5'0.5" lines
15	Tee Inside Entrance of Screenwell	156.0	13.00	119.24		116	179.48	1.51		1.00	0.035	Tee fitting inside of screenwell area prior to screens
16	Pipe Inside Intake Screenwell Structure	156.0	13.00	119.24	38.7	116	179.48	1.51	100		0.002	14' pipe inside the entrance of the screenwell
17	14' Existing Concrete Onshore Intake Pipe	156.0	13.00	119.24	640.0	116	179.48	1.51	100		0.041	Existing 14' pipe, assuming up to 6" of marine growth & 2 ft of sediment on the bottom, located between screenwell and connection point
18	Sudden Reduction 14' to 12' D1 =	156.0	13.00	119.24								Assuming reduction from proposed onshore 14'
	D2 =	132.0	11.00	95.03		116	179.48	1.89		0.06	0.004	pipe to proposed 10' onshore line
19	90° Bend on Onshore 12' Pipe	132.0	11.00	95.03		116	179.48	1.89		0.30	0.017	Assuming 90 degree elbow at onshore 14' pipe to proposed 12' onshore line
20	12' Proposed Onshore Intake Pipe	132.0	11.00	95.03	5917.0	116	179.48	1.89	100		0.700	Proposed 12' concrete pipe, assuming up to 6" of marine growth, located between existing 14' onshore pipe and proposed offshore 10' line to intake structure
23	Exit Into Proposed 12' Intake Pipe	132.0	11.00	95.03		116	179.48	1.89		0.50	0.028	Exit losses from concrete vault to proposed 12' pipe
24	Onshore Concrete Vault	NA	NA	490.00		116	179.48	0.37		1.00	0.002	Losses inside of new onshore concrete vault between proposed 12' and 10' pipe
25	Entrance into Onshore Concrete Vault	108.0	9.00	63.62		116	179.48	2.82		1.00	0.124	Entrance losses from proposed offshore 10' to proposed onshore concrete vault
18	Sudden Reduction 12' to 10' D1 =	132.0	11.00	95.03								Assuming reduction from proposed onshore 12'
	D2 =	108.0	9.00	63.62		116	179.48	2.82		0.24	0.030	pipe to proposed 10' offshore line
21	90° Bend on Offshore 10' Pipe	108.0	9.00	63.62		116	179.48	2.82		0.30	0.037	Assuming 90 degree elbow after onshore 12' pipe to proposed 10' line to new intake location (D2)
22	10' Proposed Offshore Concrete Intake Pipe	108.0	9.00	63.62	2598.0	116	179.48	2.82	100		0.816	Proposed 10' concrete pipe, assuming up to 6" of marine growth, located between onshore 12' pipe to intake location (D2)
23	Exit Into Proposed 10' Intake Pipe	108.0	9.00	63.62		116	179.48	2.82		0.50	0.062	Exit losses from new WWS intake tower to proposed 10' pipe
24	New WWS Intake Tower	NA	NA	490.00		116	179.48	0.37		1.00	0.002	Losses inside of new WWS intake tower between proposed 10' pipe and 8' WWS header pipe
25	Entrance into New WWS Intake Tower	84.0	7.00	38.48		116	179.48	4.66		1.00	0.338	Entrance losses from 8' header pipe to proposed WWS intake tower
26	Sudden Reduction 10' to 8' D1 =	108.0	9.00	63.62								Pipe reduction from header pipe (10') to WWS
	D2 =	84.0	7.00	38.48		116	179.48	4.66		0.43	0.144	header pipe (8')
27	90° Bend from 10' to 8' Pipe	84.0	7.00	38.48		116	179.48	4.66		0.30	0.101	Assuming 90 degree elbow from 10' pipe at proposed WWS intake structure to 8' WWS header pipe at new intake location (D2)
28	8' Proposed WWS Header Pipe	84.0	7.00	38.48	45.0	116	179.48	4.66	100		0.048	Proposed 8' concrete WWS header pipe, assuming up to 6" of marine growth, located at intake location (D2)
29	Sudden Reduction 8' to 6' D1 =	84.0	7.00	38.48								Pipe reduction from header pipe (8') for 4 WWS
	D2 =	60.0	5.00	19.63		29	44.87	2.29		0.92	0.299	tees (6')
30	Tee, Flanged	60.0	5.00	19.63		29	44.87	2.29		1.00	0.324	Assuming tee on wedgewire screens to main header pipe (x4)
31	Wedge wire screens	NA	NA	46.78		29	44.87	0.96		4.15	0.237	Head losses across wedge wire screens pore space (x4)
32	Entrance	60.0	5.00	19.63		29	44.87	2.29		1.00	0.324	Entrance losses into wede wire screens (x4)
							Sub-Total Head Loss			Σ =	6.680	
							Miscellaneous		20%		1.336	Miscellaneous losses to account for missing fittings and flow obstacles not shown or specified on as-built records
							Total Head Loss			**Σ =**	**8.015**	

Datum	Value	Description
MSL	2.78	Mean Sea Level
MLLW	0.0	Mean Lower-Low Water
NAVD88	0.18	North American Vertical Datum of 1988
Min Tide	-2.35	Lowest Observed Tide
Min Level	-10	Minimum water level at lowest tide and max flow
Max Head Loss	**7.65**	**Maximum allowable head loss**

Hazen-Williams Equation:

$H_L = 3.022 \times L \times (V/HW)^{1.85}/D^{1.17}$

Minor Loss Equation:

$H_L = K \times (V^2/2g)$

Notes:

L= Length of pipe (ft)

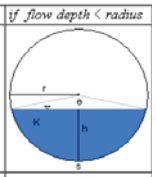
V= Velocity (ft/sec)

HW= Hazen-Williams Coefficient

D= Inner Pipe Diameter (ft)

K= Minor Loss Coefficient

g= Acceleration due to gravity (32.2 ft/s^2)

step	solve for	*if flow depth < radius*
		
1	circular segment height	$h = d$
2	central angle	$\theta = 2\arccos\left(\frac{r-h}{r}\right)$
3	circular segment area	$K = \frac{r^2(\theta - \sin\theta)}{2}$
4	arc length	$s = r \times \theta$
5	flow area	$A = K$
6	wetted perimeter	$P_W = s$
7	hydraulic radius	$R_h = \frac{A}{P_W}$

Assumed 2 ft of sand in the bottom of the existing 14' pipe to screenwell

14 ft pipe

Θ= 1.550387

K= 13.48958

Θ= Central Angle

K= Assumed Entrained Sand Area

Note: I subtracted the assumed entrained sand area from the Area (SF) calculations