WAVE EFFECTS ON THE

INTAKE STRUCTURE

AT

DIABLO CANYON, UNITS 1 AND 2

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SUMMARY

Test data from 1:45 scale model studies performed by Offshore Technology Corporation (OTC) for O. J. Lillevang and F. Raichlen were used to evaluate the adequacy of the intake structure to protect the auxiliary saltwater (ASW) pumps against the Design Flood Events if the breakwater is degraded to mean lower low water (MLLW).

The test results and analysis of the structure show that with few minor modifications implemented, the ASW pumps will be protected during events more severe than the Design Flood Events.

The principal observations were:

1) No slam pressures were detected on the front face of the curtain wall during any of the tests which were conducted.

2) The structural capacity of the front curtain wall is greater than the pressures that occur during frontal wave attack under conditions more severe than the design flood events.

3) Fillets installed in the interior of the intake structure between the front curtain wall and the deck slab mitigate slam pressures that may occur on the underside of the deck behind the curtain wall.

4) Strengthening the manholes to prevent excessive venting will assure that the wave forces on the ceiling of the auxiliary saltwater pump forebays will be below the structural capacity of the ceiling.

5) The intake structure top deck and modified air vent structure have sufficient structural capacity to resist conditions more severe than the design flood events.

6) There will be no uplift forces to unseat the water tight auxiliary saltwater pump access hatches during the design flood events.

7) Water velocity in the ASW pump forebays was determined to be too low for entrainment of missiles.

1.0 Introduction

Hydraulic model wave tests were conducted at Offshore Technology's facilities in Escondido, California, to review the adequacy of the intake structure to protect the auxiliary saltwater (ASW) pumps from extreme wave attack if the breakwater is degraded to mean lower low water (MLLW). The ASW pumps are located near the rear and close to the center of the structure. (See Figures 1 and 2).

The test results and analysis of the structure show that with a few minor modifications the ASW pumps will be protected during the design flood events if the breakwater is degraded to MLLW. The modifications required are being implemented.

The results of the wave testing program are documented in two consultants reports, "The Height Limiting Effect of Sea Floor Terrain Features and of Hypothetically Extensively Reduced Breakwaters on Wave Action at Diablo Canyon Sea Water Intake," by Omar J. Lillevang, Fredric Raichlen, Jack C. Cox, March 15, 1982, (Lillevang report), and "The Investigation of Wave-Structure Interactions For The Cooling Water Intake Structure of The Diablo Canyon Nuclear Power Plant", by Fredric Raichlen, December 1982, (Raichlen report).

The Lillevang report, which was submitted to the NRC on July 1, 1982, demonstrated the need for the addition of tubular air vents (snorkels) to prevent ingestion of water into the ASW pump rooms and measured the forces for which the air vent structures were analyzed. The forces were determined to be substantially less than the structural capacity of the modified ventilation structures. Construction work to install the snorkels and to strengthen the vent structures is complete for Unit 1 and is underway for Unit 2.

The Raichlen report, which was submitted to the NRC on January 10, 1983, documents studies of wave forces on the front curtain wall and on the ceiling of the auxiliary saltwater pump forebays. (See Figures 3, 4, 5 and 6). It also provides an analysis of the possibility of water borne missile entrainment in the auxiliary saltwater pumps forebays.

This report describes the use of the data in the Raichlen and Lillevang reports to evaluate the structural capacity of the intake structure to protect the ASW pumps during the design flood events should the breakwaters become degraded to MLLW.

2.0 Design Conditions

The testing program was developed to study conditions which result from the design flood events assuming the full length of the breakwater is degraded to MLLW as discussed with the NRC at the September 25, 1981 meeting. The flood events used in the testing program were more conservative than the proposed design flood events. The two design flood events and the more conservative conditions used in the testing program for the Raichlen report are as follows:

a) "Probable Maximum Tsunami" combined with storm waves of annual severity and high tide with anomaly.

This design flooding event is defined by Supplement 5 to the Safety Evaluation Report ("SER 5") as a Hosgri tsunami with a maximum wave height of 9.2 feet, combined astronomical and meteorological tidal height of 6.3 feet for a still water evaluation of 15.5 feet above MLLW not including storm wave effects. The annual storm wave considered in the SER was an 18-foot wave height outside the breakwater with a period of 15 seconds.

A still water height of +17 feet MLLW was used in the model tests with irregular waves simulating a storm more intense than the storm of January 28, 1981. The modeled storm would have an estimated return period of about 41 years and had a significant wave height of 26.8 feet. This storm is denoted herein and in the Raichlen report as the 1981 storm.

b) "Maximum Credible Wave Event" combined with high tide with anomaly.

The "Maximum Credible Wave Event" is not defined but the results of the previous model tests (Lillevang report) indicate that determination of the "Maximum Credible Wave Event" would have no significance for determination of the maximum wave effects at the intake structure. The previous tests found that the response wave within the intake basin reached a limited maximum height which did not increase further in response to increases in the offshore wave height. This is due to the effects of the natural terrain and the presence of the degraded breakwater. Therefore, it is the limited maximum response wave height in the basin in combination with the still water level in the basin that is of interest for assessing the maximum inundating effects and wave forces at the intake structure rather than some undetermined "Maximum Credible Wave Event" offshore. It is the effects of those limited waves which were studied.

Model tests for this design flooding event were run with a simulated tide level at +7.5 feet MLLW which is greater than the 6.3 feet assumed in SER 5. The tests were conducted with waves ranging up to 50 feet in height and periods of 12 and 16 seconds.

During the testing program, other water levels and storm spectra were also tested where it was felt that these conditions might result in greater forces or higher velocities. The additional conditions investigated are described in the Raichlen report.

3.0 Criteria for the Evaluation of the Intake Structure

The capacity of the intake structure to resist the maximum wave induced loads as determined from the testing program was evaluated to the following criteria:

3.1 Loading Combinations

The loading combination used in the evaluation of the intake was:

$U = D + L + W_f$

U = Strength required to resist design loads

D = Dead Load

L = Live load

W_f = Wave forces associated with breakwater degraded to mean lower low water (MLLW).

3.2 Allowable Stresses

For concrete elements, the allowable stresses were based on ACI 318-71. The compressive strength of concrete and yield stress of reinforcing were based on the average of the actual tested values.

Compressive strength of concrete,	Fc' = 3630 psi
Yield stress of reinforcing,	Fy = 49600 psi

For structural steel, the allowable stresses were based on AISC Specification, 7th Edition. In the absence of the tested values, the minimum specified yield strength was used for the following allowable stresses:

Allowable bending stress,	Fb = Fy = 36000 psi
Allowable shear stress,	$Fv = Fy/\sqrt{3}$

4.0 Analysis Performed

The objective in the development of the testing program was to determine whether or not the ASW pumps would be protected during and following the design flooding events coincident with the entire breakwater degraded to MLLW. Tests were designed to develop data to be used in the evaluation of the following possible mechanisms:

(1) Wave forces on the ASW air ventilation structures.
(2) Water ingestion by the ASW air vents.
(3) Frontal wave forces on the curtain wall.
(4) Wave forces on the ceiling of the auxiliary saltwater pump forebays.
(5) Wave forces on the intake structure top deck.
(6) Water borne missiles in the ASW pump forebay.

The data for mechanisms (1) and (2) are compiled in the Lillevang report. The addition of the snorkels and modifications to the air vent structure resolved these concerns. The attached report "Investigation of Seawater Ingestion Into The Auxiliary Salt Water Pump Room Due to Splash-Runup During The Design Flood Events at Diablo Canyon", by Patrick J. Ryan,

provides additional analysis which further demonstrates that the ASW air vents, as modified, would not ingest excessive quantities of water during the design flood events.

The data for mechanisms (3), (4) and (6) are compiled in the Raichlen report. Data for mechanism (5) are based on observations during the tests for both reports.

5.0 Results of Analysis

5.1 Wave Forces on the ASW Air Ventilation Structures

The maximum total wave force that was measured on the air ventilation structures was 300 kips. The air ventilation structures as modified can resist a force of 780 kips applied at the top of the ventilation structure and occurring in any direction. Therefore, the ventilation structure has ample capacity to resist wave forces.

5.2 Frontal Wave Forces on the Curtain Wall

The ASW pumps are protected against frontal wave attack by a series of vertical walls. (See Figures 4, 5, and 6.) All the walls in series would have to sustain damage from frontal wave attack to affect the ASW pumps. Tests were devised to measure the pressures developed on the seaward curtain wall which is the first line of defense. Test results and structural analysis show that with minor modifications, which are being implemented, the seaward curtain wall will not sustain any structural damage and will, therefore, protect the ASW pump against frontal wave attack.

The curtain wall consists of 17 panels. All the panels, except the one at the centerline of the structure, extend from elevation +20.1' MLLW to elevation -5.14' MLLW. Except for center panel, seawater communicates freely through the intake openings below the curtain wall such that water is always on both sides of the wall and at the same surface level unless upset by wave action. (See Figures 4 and 5). The screen refuse bay is located behind the center panel. The bottom of this panel is at elevation +4.5 MLLW. (See Figure 6). The tests determined the front face pressure distribution from the top deck, elevation +20.1' MLLW to elevation -5.14' MLLW.

No "slam" (high magnitude, high frequency) pressures were detected on the front face of the curtain wall during any of the tests that were conducted. The testing also established that the wave induced loading on the front face can be approximated by a trapezoidal pressure distribution with the maximum pressure at the base and the slope paralleling the slope for hydrostatic pressure.

The maximum pressures were measured for the "Probable Maximum Tsunami" flood event which has a still water elevation of +17 feet

*See page 70 of the Raichlen report for explanation of determination of "probable maximum pressure."

MLLW. The maximum measured pressure on the front face was 44 feet of seawater. The "probable maximum pressure"* at the base of the typical curtain wall panel (El -5.14 MLLW) was determined to be about 59 feet of seawater. For the "Maximum Credible Wave Event" which has a still water elevation of +7.5 feet MLLW, the maximum pressure measured on the front face was about 26 feet of seawater.

The structural capacity of the curtain wall panels is greater than the front face pressures determined from the tests. The net pressure on the panels are lower than the front face pressures therefore the seaward curtain wall has ample capacity to resist frontal wave attack.

The net pressure on the curtain wall panels is the pressure on the front face minus the compensating interior pressure. When the typical panels are subjected to the "Probable Maximum Tsunami," it was determined that the net water surface elevation difference across the curtain wall conservatively approximates the maximum net pressure on the panels. The tests indicated that the probable maximum head difference is about 24 feet of seawater. Therefore for the typical panels, the maximum net pressure for "Probable Maximum Tsunami" is 24 feet of seawater, and the maximum net pressure for the "Maximum Credible Wave Event" will be less than 26 feet of seawater. The structural capacity of these panels of the curtain wall is 85 feet of seawater or 5.5 ksf.

For the center panel, the pressures are different from the typical curtain wall panel because the interior hydrostatic pressure will be less, and the base of the wall is at a higher elevation. The elevation of the base of the center panel is 9.5 feet higher than that for the typical curtain wall panel. Therefore, a conservative estimate of the probable maximum front face pressure at the base of this panel for the "Probable Maximum Tsunami" is 49.5 feet (59 feet - 9.5 feet). There are openings at El +15 MLLW through the curtain wall at the center panel. With the still water level at +17 feet MLLW, the water level would be higher than the openings and the refuse bay would flood. Therefore, the net pressure will be the pressure on the front face minus the hydrostatic pressure in the inside of the screen refuse bay. The interior hydrostatic pressure at the base of the curtain wall with the water level at +17 feet MLLW is 12.5 feet (17 feet - 4.5 feet). The net pressure on the center panel of the curtain wall for the "Probable Maximum Tsunami" is therefore 37 feet (49.5 feet - 12.5 feet). The net pressure for the "Maximum Credible Wave Event" is less than 26 feet. The capacity of this portion of the curtain wall is 3.9 ksf or 61 feet of water.

The test program demonstrated that the maximum net pressures that occur during frontal wave attack for conditions more severe than the design flood events are less than the structural capacity of the front curtain wall. The capacities of all the panels of the curtain wall are greater than the maximum pressure on the front face for either design flood event, therefore the panels have sufficient

structural capacity to resist the extreme wave forces even if the compensating interior pressures are neglected.

It was observed during the tests that slam pressures may occur on the underside of the deck behind the curtain wall. The deck slabs are not required to maintain the structural integrity of the intake structure. However, it was hypothesized that damage could occur to the front curtain wall due to pressures transmitted acoustically down the inside face of the wall. Therefore, tests were conducted which demonstrated that fillets installed in the corner formed by the wall and deck would mitigate the pressures. With the fillets, there would be no damage to the deck slab behind the curtain wall or to the curtain wall itself. Based on the test results, 45° concrete fillets have been designed and modification of the intake structure to install the fillets is underway.

Test results and structural analysis have shown the curtain wall with the afore described modification is capable of resisting frontal wave attack without suffering structural damage during the design flood events.

5.3 Wave Forces on the Ceiling of the Auxiliary Saltwater Pump Forebays:

Pressures on the ceiling of the ASW pump forebay were measured for various conditions to determine if the slab has the capacity to resist wave forces from below. Tests and analysis show that with minor modifications to preclude the forebays from venting freely through the manholes, the slab will not be damaged under severe wave attack and the ASW pumps will, therefore, be protected. Data from tests with vent openings equivalent to 5.5 inches diameter in the prototype (moderate venting case) and with no openings (non venting case) were used in the evaluation of the slab and the design of the modifications to the manholes.

The maximum pressure measured on the ceiling during the non or moderate venting test cases for all design events was 64 feet of seawater. The probable maximum pressure on the ceiling, which includes the dynamic and hydrostatic pressure, was determined to be between 88 feet and 97 feet of seawater. The slab was evaluated to have a capacity of 7.4 ksf or 116 feet of seawater.

Under test conditions that simulated open manholes 17 inches in diameter through the ceiling of the forebays, slam pressures were detected. The manholes have been modified to reduce venting and to withstand pressures greater than 97 feet of seawater.

The ASW pump, and connections to the slab for the ASW pumps and screen wash pumps were also checked for capacity to withstand the probable maximum pressure, and were found to be adequate without modification.

The tests results and structural analysis demonstrate that with minor modifications to the manholes, the ceiling of the ASW pump forebays is capable of resisting wave forces without suffering

structural damage during the design flood events and will therefore provide protection for the ASW pumps.

5.4 Wave Forces on the Intake Structure Top Deck

The water level over the deck at elevation +20.1 which forms the roof of the circulating water and auxiliary water pump chambers was observed to be generally lower than the top of the vent huts, El +34.6 MLLW. The water pressure on the deck would be hydrostatic and would be approximately 14.5 feet (34.6 - 20.1). Except for the hatches over the circulating water pumps, the capacity of the roof is 2.4 ksf or 37.5 feet. The hatches have a capacity of 1.15 ksf or 18.5 feet. Therefore the top deck has sufficient capacity for the design flood events.

An evaluation was made to determine if water flowing across the deck during the design flood events would create uplift forces which would unseal the concrete ASW pump access hatches. It was determined that the access hatches would not be subjected to uplift forces and the weight of the concrete access hatches on its gasket is sufficient to provide a water tight seal. Therefore, it is concluded that water will not enter the auxiliary water pump chambers through the hatches during the design flood events.

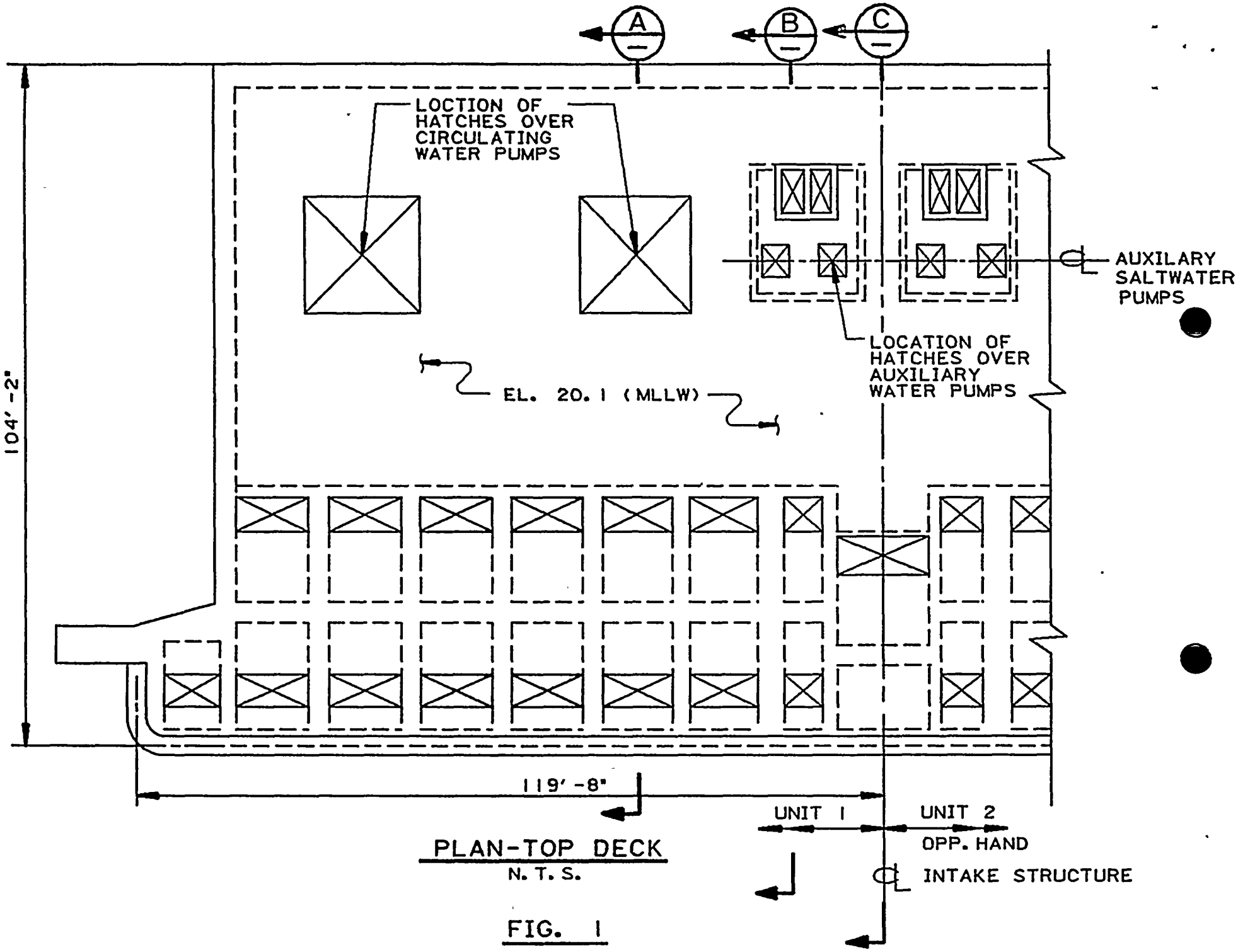
5.5 Missiles in the ASW Pump Forebay Chambers

The maximum measured water velocity at the ASW forebay entrance was 4 fps and the probable maximum velocity is between 4.0 fps and 7.8 fps. These velocities are too low to be a hazard to the ASW pumps. Furthermore, dye tests show that the velocities within the chamber are significantly less than those measured at the entrance.

In addition, observations were made of the motion of lucite cubes which have equivalent prototype dimensions of about 2 feet on a side and a specific gravity of 1.3. Irregular waves corresponding to the 1981 storm and a still water elevation of minus 2 feet MLLW were used. It was observed that the cubes moved very slowly into the forebay always along the floor of the chamber. In the length of time of exposure to waves from this storm (approximately 47 min prototype) the cubes moved only about one-third of the length of the forebay. This observation supports the conclusion that the velocities in the forebays due to waves are small, and the likelihood of entrainment of missiles is extremely remote.

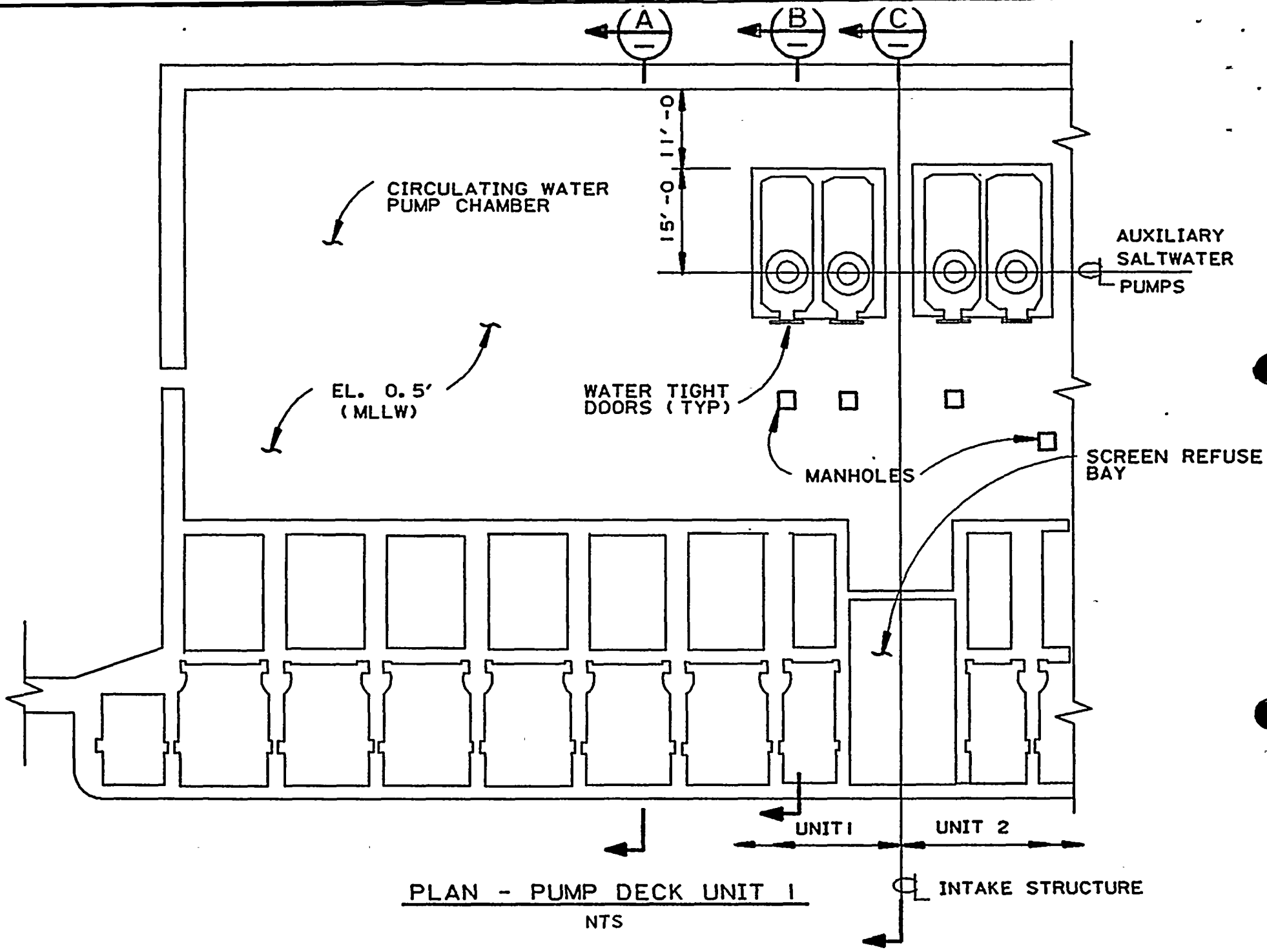
6.0 Conclusion

The testing and analysis of the intake structure show that for the design flood events with the breakwater entirely degraded to the MLLW elevation, there is an ample margin of safety for the protection of the ASW pumps with the minor modifications described herein completed.



PLAN-TOP DECK
N. T. S.

FIG. 1

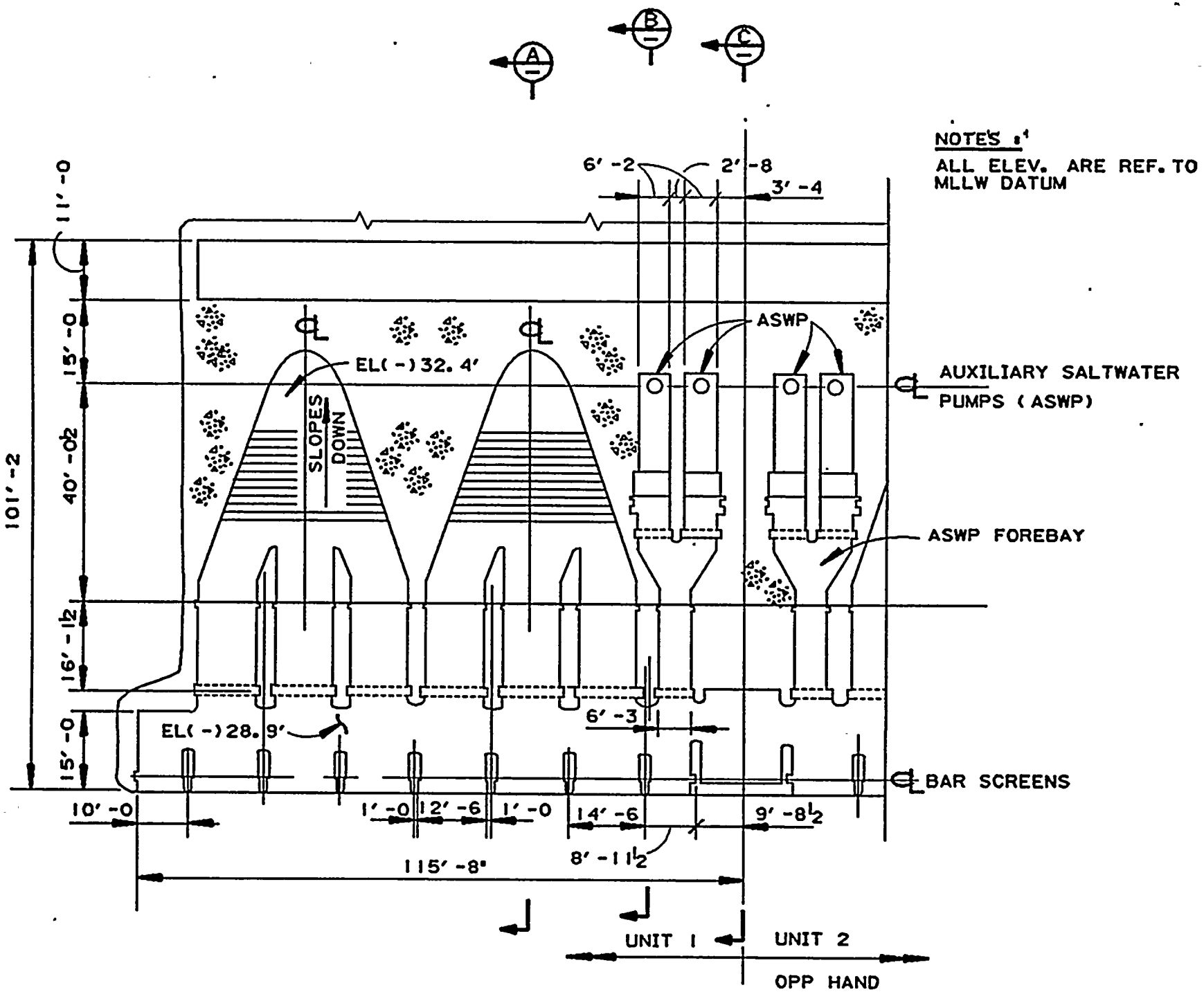


PLAN - PUMP DECK UNIT 1

NTS

FIG. 2

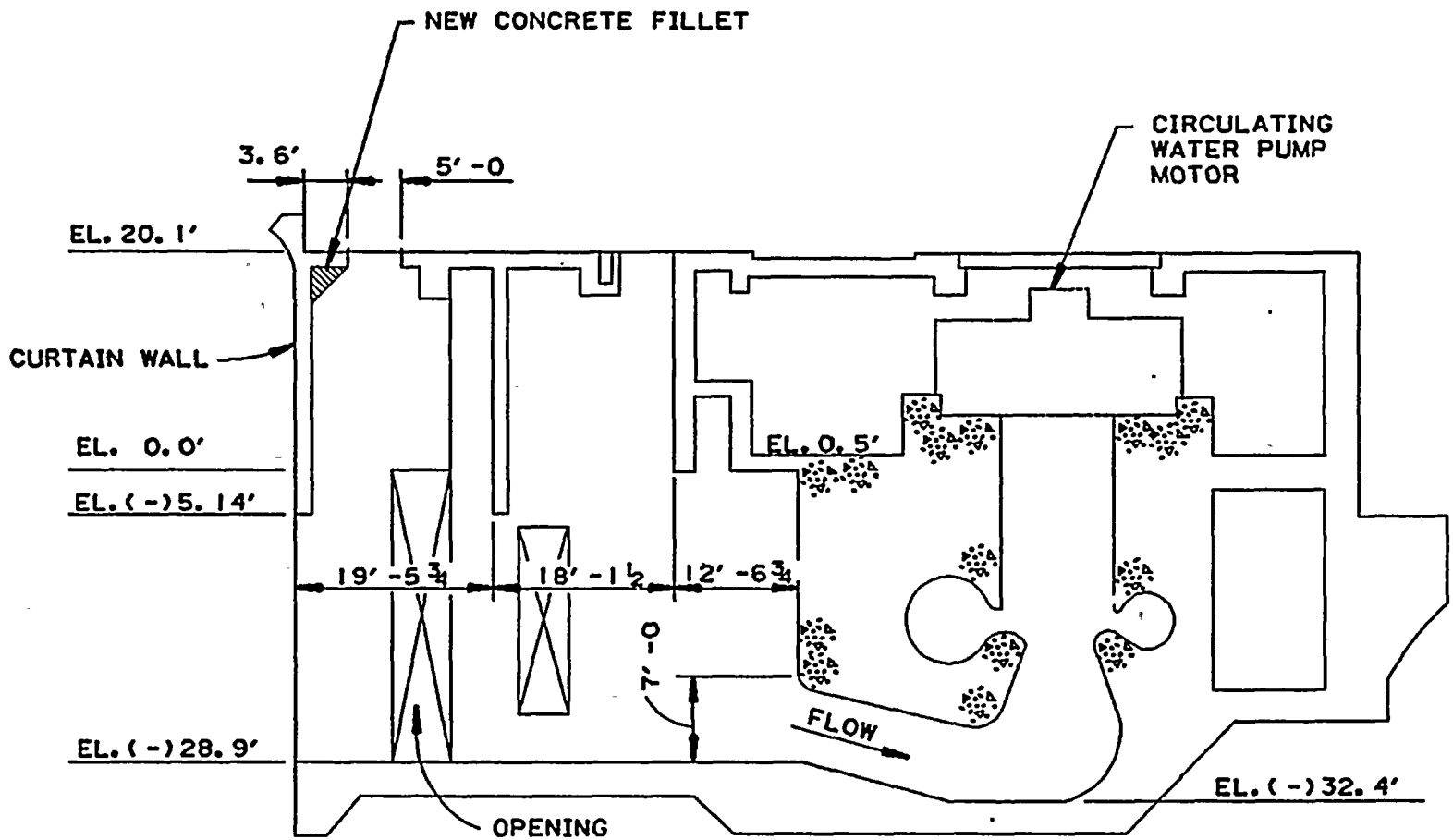
020E82 FIG52.DGN 1



PLAN - INVERT ELEVATION (-) 28. 9 ft MLLW

N T S

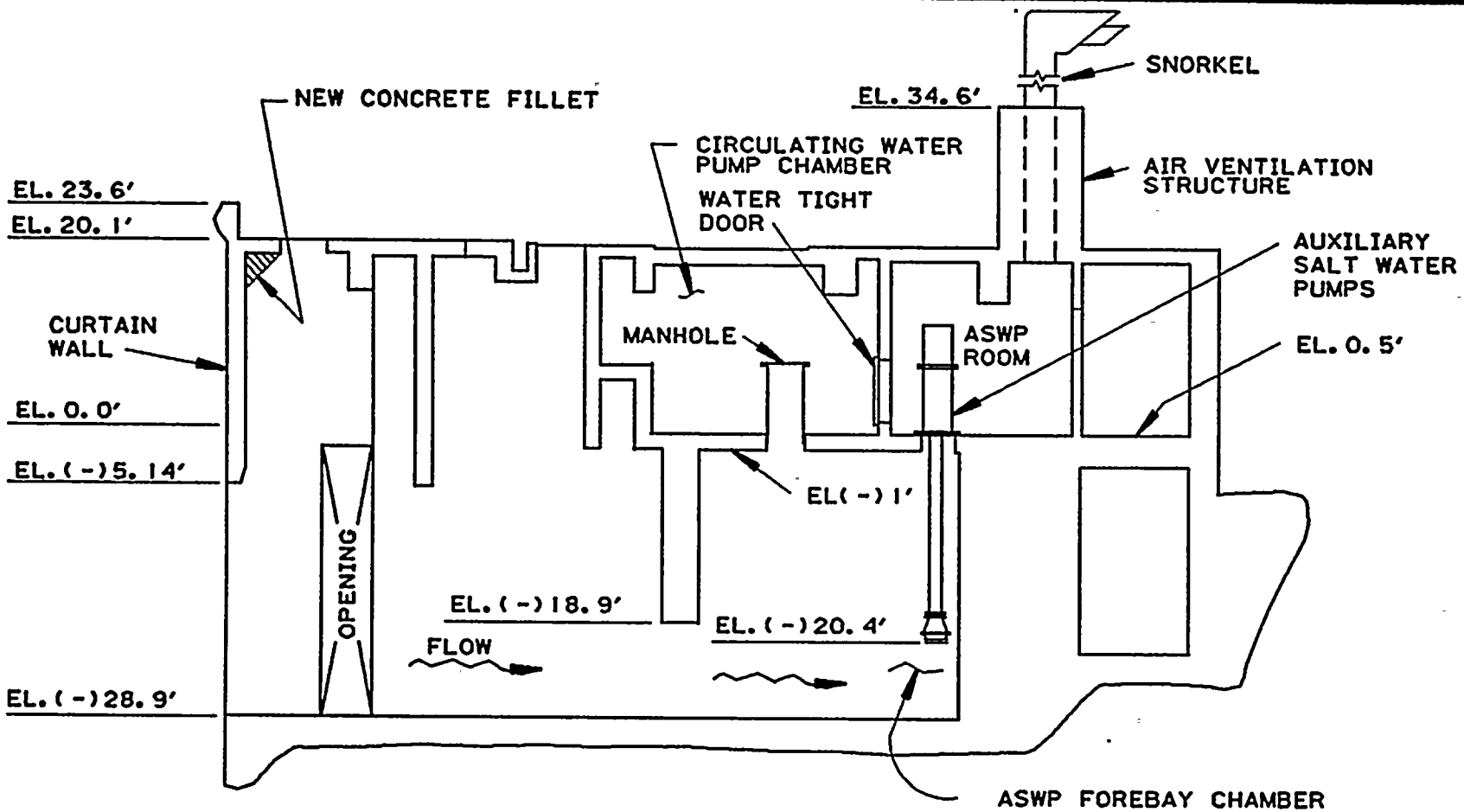
FIG. 3



SECTION A
NTS

NOTES:
ALL ELEV. ARE REF. TO MLLW DATUM

FIG. 4



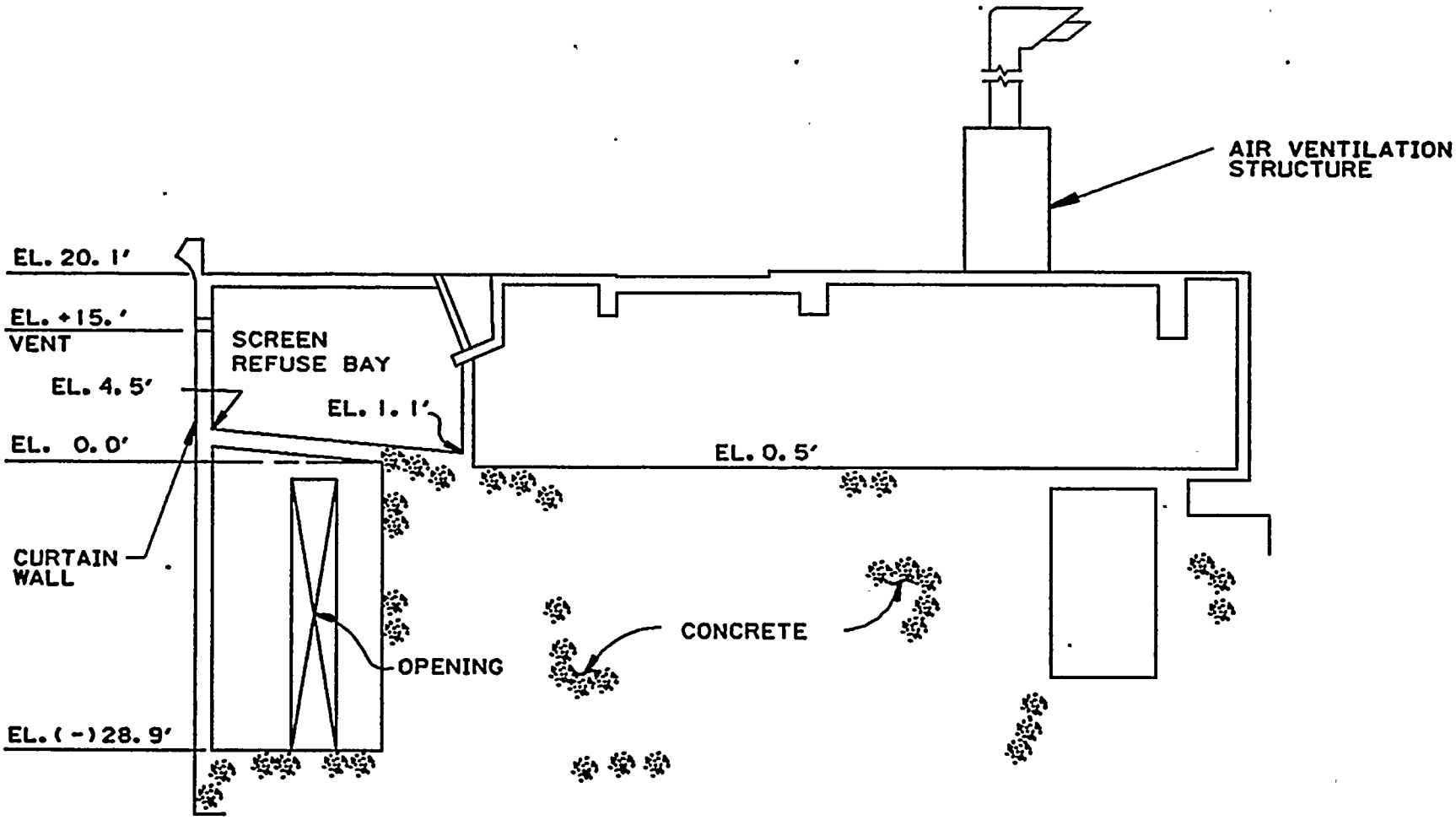
SECTION B

NOTES:

ALL ELEV. ARE REF. TO MLLW DATUM

FIG. 5

17N082 FIG55.DGN



SECTION C
NTS

NOTES:

ALL ELEV. ARE REF. TO MLLW DATUM

FIG. 6