STRUCTURAL EVALUATION

AND REPAIR OF

SONGS 1

INTAKE STRUCTURE

Prepared by: O. Gurbuz

D. J. Haavik

W. R. Hughes

Reviewed by: S. C. R. Chan

Approved by: R. J. Kosiba

BECHTEL POWER CORPORATION
NORWALK, CALIFORNIA

October 2, 1984

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EXECUTIVE SUMMARY

The north pump well and screen well area of the San Onofre Generating Station Unit 1 intake structure could not be dewatered due to leakage at the north stop gate caused by concrete spalling adjacent to the stop gate. This resulted in an inspection program to examine the structure in greater detail.

Visual examination of concrete surfaces indicated discoloration in some areas, attributable to corrosion byproducts which had collected on the surface. Core samples were taken and they showed concrete delaminations between 3 and 4 in. in a plane parallel to the surface. Almost all of the initial samples contained either one or two delaminations on the sea water side which were generally filled with corrosion byproducts. Some cores passed through reinforcing steel, which showed either surface corrosion or, in some cases, extensive corrosion deterioration.

Additional cores were taken through the walls and base slab to determine the condition of the concrete and steel at the outside surface. All samples indicated sound concrete throughout and the reinforcing steel on the outside showed no signs of rust products nor delaminations. Additional through-wall cores were taken to confirm the exterior south wall reinforcing steel integrity in the wall adjacent to the discharge tunnel. These cores also indicated sound reinforcing steel and concrete on the discharge side, although chipping revealed structurally insignificant corrosion.

Concrete was chipped in specific areas selected because of their structural significance. These investigations led to the conclusion that reinforcing steel corrosion and concrete delaminations are primarily limited to the interior areas of the pump wells and screen wells. The areas between the screen wells and the tsunami gates indicated sound concrete with reinforcing steel in excellent condition, except for areas at the gate slots and areas with no structural significance. In the screen wells the reinforcing steel corrosion is still noticeable but significantly less degradation has occurred compared to areas in the pump wells.

The mechanism by which the reinforcing steel corrosion is initiated is dependent on the penetration and concentration of the chloride ions at the reinforcing steel surface. The high alkalinity of concrete results in a passive film on reinforcing steel surfaces which inhibits corrosion. When a sufficient concentration of chloride ions is present it will destroy the passive film protecting the reinforcing steel. Iron can then be oxidized as a result of local corrosion cell action and pass into solution. Limited field testing of the dewatered intake structure has confirmed the existence of the conditions necessary for electrochemical corrosion of reinforcing steel.

Tests have also been performed to determine chloride concentrations at various locations through the walls and the slab. The results of the chloride profiles show that the interior surface of the concrete contains chlorides penetrating to a depth of 14 in. Chloride contamination again is present near the outside surface but at a significantly lower concentration. Cores taken through the walls indicate sound concrete on the exterior and no evidence of reinforcing steel corrosion or concrete delamination. This is consistent with the observed lower chloride levels in the concrete near the outside of the walls resulting from the low chloride levels of the ground water adjacent to the wall, and the reduced availability of oxygen at this location. The service conditions of the interior and exterior walls are significantly different since the interior walls are exposed to continuously moving sea water while the exterior walls are exposed to fresh, relatively static, ground water.

Petrographic examinations have been made on samples of concrete taken from the structure to assess its in-place condition and the impact of its current condition upon the corrosion of the reinforcing steel. The examinations confirmed the presence of reinforcing steel corrosion products and the overall soundness of the concrete. Compression tests were performed on cores taken from the structure which showed excellent in-place strengths.

In conclusion, there is reinforcing steel corrosion and concrete delamination on the inside surface in the pump well area and corrosion and potential for concrete delamination in the screen well area. Structural evaluation has indicated that the existing condition would not be detrimental to proper functioning of the intake structure under normal operating condition. This conclusion is based on the fact that the concrete can develop the required strength as an unreinforced structural element. Although the structure would be expected to maintain overall integrity under the DBE loads, the integrity of the salt water cooling pumps could not be positively confirmed without an extensive assessment of the in-place capacity of the concrete walls with deteriorated reinforcement.

In order to restore the required design margins and to assure the structure will withstand the Design Basis Earthquake of 0.67g, a repair program was initiated which will be implemented prior to Return-to-Service. The design for the repair consists of a series of strap plates bolted onto the concrete surface to replace the positive moment load carrying capability of the deteriorated interior reinforcing steel. These plates are protected with a coal tar epoxy coating in addition to zinc anodes for corrosion protection. The bolt material selected is monel. The design philosophy was to replace the inside reinforcement in questionable local areas with the strap plate without taking any credit for the inside reinforcing steel. The outside reinforcing steel which provides negative moment resisting capacity has been found by core samples to be of high integrity and will perform its design function. Finally, remedial work was done on the gate slots as appropriate for operational considerations.

Enhanced surveillance requirements will be incorporated into existing station procedures to determine if reinforcing steel deterioration is continuing.

1. INTRODUCTION

The San Onofre Unit I Intake Structure is a large reinforced concrete structure which provides the structural transition from the pipes used to collect and discharge sea water for condenser cooling and for safety related salt water cooling. Gates have been provided to dewater segments of the structure and to control the flow. Bar screens are used for trash removal prior to allowing the water into the pump wells. See Figures 1-3 for the general layout and arrangement.

During an attempt to dewater the north pump well and screen well, excessive leakage was observed. Divers sent below to examine the area reported local spalled concrete at the north pump gate slot and corrosion of the reinforcing steel. The intake structure was dewatered using the tsunami gates (closest to the sea intake) and an inspection program was initiated to determine the condition of the gate slots. Based on the preliminary results of this inspection, it was decided that inspection of the entire structure should be performed.

The investigation included limited concrete coring and chipping. This showed that some of the reinforcing steel near the concrete surfaces in contact with sea water was corroded. In order to define the problem, an extensive coring and chipping program was undertaken. Additional tests and examinations performed included half-cell testing, petrographic examinations, and chemical testing for chloride content.

All of these investigative efforts were directed to determine the cause and the extent of the problem, and to assure that integrity of the salt water pumps can be maintained. A 4600 gpm flow (1.3 percent of total flow) is required for safety related uses.

This report summarizes the work performed on the structure to determine the cause and extent of deterioration and the repair program that was subsequently implemented for Return-to-Service.

2. INITIAL CONDITION, INSPECTIONS, AND EXAMINATIONS

2.1 Initial Condition and Inspection Activities

2.1.1 Specified Concrete Requirements

The structure was designed and constructed in accordance with specifications consistent with the ACI-318-63 Building Code, which was in effect at the time of design.

For the Intake Structure, the specifications required that the concrete used have a minimum compressive strength of 3,000 psi at 28 days, a maximum aggregate size of 1-1/2 inch, a slump of 3 inches, and an air-entraining admixture. Based on these specification requirements, a concrete mix design with a water-cement ratio of 6.0 gallons per bag was prepared. The specification did not permit the use of other admixtures which could contain chlorides and the water used in the mix was required to be of potable quality. Tests for reactive aggregates were performed, in accordance with the specification requirements.

The ACI 318-63 Code does not directly specify requirements for structures exposed to sea water. Concrete that will be exposed to sulfate-containing or other chemically aggressive solutions is generally proportioned utilizing guidelines provided by ACI 613-54 "Recommended Practice for Selecting Proportions for Concrete." Guidance given in the ACI 613 Recommended Practice recommends a maximum water-cement ratio of 5.0 gallons per bag which may be increased to 5.5 gallons per bag if sulfate-resisting cement (Type V or Type II) is used. Type II cement was used throughout the San Onofre project.

The required concrete cover was specified to be 3 in., the minimum amount specified in ACI 318-63 for concrete cast against ground. The Code specified a minimum of 2 in. cover for formed concrete exposed to the ground or weather but called for suitably increased cover in extremely corrosive atmospheres or other severe exposures. More recent ACI documents, such as the "Guide to Durable Concrete" (ACI 201.2R-77) published in 1977, recommends a 3 in. minimum reinforcing steel cover for sea water exposure. Significant changes to these recommendations for sea water environment have not been made by ACI over the past two decades.

It is concluded that the specifications, as well as the original concrete and reinforcing steel details, represented the then-in-effect ACI code provisions and recommendations for the design environment. These code provisions have been essentially unchanged in the past twenty years and are still presently applicable.

2.1.2 Observations Before Dewatering

Excessive leakage was observed when attempting to dewater the north pump well and screen well. A divers' inspection report along with video tapes taken before dewatering showed evidence of localized reinforcing steel corrosion and spalled concrete at the gate slots. The divers' report and video were primarily limited to areas in and around the gates and gate slots.

After viewing the video tapes, an assessment was made of the spalled area and crack patterns around several gate slots. The most likely cause of the cracks was determined to be loading stresses from the gates and a preliminary procedure was prepared to allow repair of the damage. The intake structure was dewatered by use of the tsunami gates, to confirm the extent of the damage.

2.1.3 Dewatering

After preparing the gate slots and installing the tsunami gates, the water was drawn down to El (-)11 and held at that level. A visual inspection was made of the screen well down to that level by placing rubber rafts in the screen wells and continuing inspections as the water level dropped. The water was subsequently lowered to El (-)17 and inspections were conducted throughout the structure between the screen well and tsunami gates as well as the discharge structure. It was decided from those inspections that the water level could be lowered to the intake structure's bottom and inspection from the rafts was made on a continuous basis while the water level was dropped until the structure was substantially dewatered.

2.1.4 Visual Inspection after Dewatering

Upon dewatering, the structure was inspected to verify the findings of the divers' reports and videos, and to determine the extent of potential deterioration in portions of the structure required for safety-related salt water cooling.

A damaged area in the north wall just west of the traveling bar screen at the stop gate slots was noted. The concrete surface appeared sound with no crack pattern visible, except for fracturing that allowed the surface to spall locally. The reinforcing steel in this area of the wall was significantly corroded. Both the remaining surface and the mating surface of the spalled concrete showed various shades and degrees of red deposits turning to black underneath the red. Some black blotches also appeared on the surfaces. The reinforcing steel showed corrosion and was covered with a black deposit that was described as a muddy substance and identified as corrosion residue. Only a fraction of the reinforcing steel remained in this area. The cracking and damage of other areas examined was consistent with the videos taken prior to dewatering.

It was noted during and after dewatering that, except for discoloration of the walls and ceiling, and cracks noted at some gate slots and other areas, the structure appeared sound with little evidence of cracking. The discolorations were random blotches of black color noticed on many areas of the interior walls of the pump wells and, to a lesser degree, in the screen wells. In addition, spots of dark rusty deposits exuded from point locations on the concrete sometimes characterized by a moderate lump of rust congealed on the surface. These spots appeared frequently on the walls, floor and ceilings throughout the pump wells and screen wells of the structure. It was also noted that at a few of the areas showing rust colored deposits, moisture was weeping from within the concrete. A program of core drilling and chipping into several areas to determine the conditions within the concrete was inititated.

2.2 Core Drilling

Over thirty concrete cores were taken in various locations over the duration of the investigation phase in order (1) to determine the structural condition and (2) to provide samples for compressive strength testing, petrographic analysis and chloride testing.

The cores were utilized in identifying delamination areas which were found to indicate the presence of corroded reinforcing steel nearby. The presence of delaminations and condition of reinforcing steel varied considerably through the structure, however the six cores drilled completely through the walls and floor to backfill consistently indicated that no corrosion of the reinforcing steel existed, nor was there any evidence of delaminations on the outside of the walls and floor against fill. Three additional cores were taken through the south wall which separates the intake from the discharge. These cores were taken to verify the condition where flow occurs across both faces of a single wall. Summaries of observations on the cores are given in Table 1. Locations of the cores are shown on Figures 1-3.

2.3 Concrete Chipping

Concrete chipping in designated areas allowed (1) an evaluation of the exposed reinforcing steel to estimate the functional amount remaining and (2) permitted assessment of the continuity of the delamination more easily than would be permitted by coring. The observed conditions vary considerably in different areas of the structure, ranging from total loss of the reinforcing steel by corrosion to reinforcing steel in excellent condition with no sign of corrosion. Summaries of the observations made at these chipped areas are given in Table 2. Locations of the chipped areas are also identified on Figures 1-3.

3. ASSESSMENT OF CONDITION PRIOR TO REPAIR

The condition of the structure prior to repair can be assessed by considering the available data:

- Visual examinations and observations of the structure
- Visual examination of the cores
- Visual examination of the chipped areas
- Laboratory testing and examination of cores
- Laboratory analyses of water
- Petrographic examination
- Half-cell measurements

The intake structure is essentially a reinforced concrete tunnel with the top slab open at the screen well. Due to geometry, embedment, and loading of the structure the outside steel reinforcement is required to resist negative moments, and the inside steel reinforcement is required to resist positive moments which are usually at midheight of the walls. Due to this loading configuration the chipping effort was concentrated at key locations to provide data for an accurate assessment of the existing load carrying capability of the structure.

The condition of the structure prior to repair was assessed by determining the condition of the concrete and reinforcing steel.

3.1 Concrete

Visual and petrographic (microscopic) examination of the concrete indicates that it is in very good to excellent condition with the exception of the delaminations on the inside surface. This conclusion was based on examinations of the core samples and chipped areas detailed in Tables 1 and 2. Both core samples and chipped pieces indicated that the concrete is sound and there is good adhesion of the constituents. The sampled concrete is moderately hard and neither significant bleeding nor segregation of the fresh concrete is indicated. Bleeding water (upward movement of water in fresh concrete) can be of significance because it can make weaker concrete by collecting on the underside of reinforcing bars and coarse aggregates, which reduces the bond between the mortar and those constituents. There is every indication that the concrete was well-consolidated and that the concrete is air entrained (air void content is estimated to be up to 1-2% by volume). Sound sand grains are evident, a condition indicating good bond on the cementitious matrix to the aggregate. Calcium hydroxide, a normal product of hydration of portland cement, is present and is characteristic of cement pastes of moderate water-cement ratios. There was no evidence of mineral admixtures such as fly ash or pozzolan. The general conclusions of the full cores indicated no evidence of a widespread alkali-silica reaction. The aggregate gradation is normal. Variations in concrete consolidation and mortar are consistent with normal construction variations. Any differences between the cores and pieces from different locations are not unusual.

Some samples are free from cracking while others have crack networks extending inward from the surface of the concrete that have been in place for a long time as evidenced by deposits. Cracks on the surface of the existing delaminated walls showing duress are not readily evident at the surface although extensive surface cleaning was not conducted at the time of examination. Even after subsequent surface cleaning with high-pressure water jets, cracking is not obvious on the wall surfaces. Some of these cracks may have been formed at the time the structure was constructed due to shrinkage caused by natural drying of the concrete prior to filling the structure with water. Other cracks may have been formed by dimensional changes due to cooling from the loss of heat of hydration shortly after the concrete was placed. Cracks exposed to the atmosphere on one concrete face have had a long time to dry out and promote shrinkage. Some cracks may have been formed as a part of the delamination process caused by internal pressures of the corrosion products. The petrographic analysis confirms that the cores show evidence of a moderate water-cement ratio which is consistent with the design water-cement ratio of 6.0 gallons per bag shown on the concrete mix design issued by the laboratory.

To provide additional objective evidence of the original concrete quality, eight concrete samples were tested for compressive strength. The results (after correction for shape of the sample) indicate the following compression test results:

Core No. (per Fig.1)	Core No. (per test report)	Compressive Strength, psi
2	2A	8180
2	2B	6690
2	2C	7530
3	3A	7220
3	3B	5680
4	4A	6900
4	4B	6700
5	5A	8980

As seen above, the lowest compressive strength obtained from core testing was 5680 psi, which is significantly higher than the specified minimum design strength of 3000 psi. The average of the compressive strengths obtained from core testing was 7240 psi. The existing concrete strength can be attributed to extensive curing in its service environment. The high chloride content near the surface does not adversely affect the concrete strength between the laminar plane and the surface.

A review of the concrete tests and examinations confirms that chlorides were not used as an additive in the original concrete mix because the chloride profiles indicate that low levels currently exist within the structure. This confirms that the structure was not initially contaminated with chloride beyond the low chloride levels naturally occurring in the basic concrete constituents.

Upon examining all available information, it is concluded that the concrete itself is in structurally sound condition, exhibiting appropriate mechanical strength to perform its structural function. The only problem with the concrete is that delaminations are evident in areas of the pump wells and screen wells on the inside surface of the structure and consequently the concrete no longer provides significant corrosion protection to the reinforcing steel in these areas.

3.2 Reinforcement

3.2.1 Inside Reinforcement

Some of the reinforcing steel on the structure's inside surface in the pump and screen well areas in contact with sea water was found by inspections to be corroded, with enough steel gone to remove the structural function of the bar. A chipping and coring program (described in Sections 2.2 and 2.3) defined the conditions found in areas examined in order to determine the extent of corrective action needed.

Table 2 lists the percentage of cross-sectional area remaining for the reinforcing steel exposed at each chipping location along with any other pertinent observations that could be made from the chipping. This table, along with the data for core drilling given in Table 1, indicates that the most severe corrosion is in the pump wells where sea water is exposed to the interior walls. In the screen well area the corrosion is less severe and it is concluded that about 80 percent of the original reinforcing steel remains. However, some of these bars are covered with rust and lamination planes are evident at these locations. West of the screen wells out to the tsunami gates the reinforcement was found to be free of structurally significant corrosion. The floor throughout the structure was also found to be free of structurally significant corrosion and the only corrosion found in the ceiling was in the smaller bars in the pump wells. There was no indication that the larger bars that function as local beam reinforcement in the structure were corroded.

Further usage of the inside reinforcing steel is based on these observations and may be summarized as follows: The inside reinforcing steel is still capable of functioning satisfactorily under all design loadings and is in very good to excellent condition with the exception of the wall and ceiling reinforcing steel (except for the area reinforced as beams adjacent to the circulating water pumps) in the pump well and the lower portion of the screen well areas. The present condition of this steel is structurally questionable and it would be considered satisfactory only if it is replaced with new reinforcing steel.

3.2.2 Outside Reinforcement

Cores taken completely through the walls and floor to soil consistently indicated that the outside reinforcing steel is in excellent condition with no sign of corrosion, nor are there any indications of delaminations of the concrete. The ground water on the outside of the walls and floor was tested for chloride content at each core drilled and showed a maximum chloride concentration less than 300 ppm, as opposed to the 19000 ppm generally accepted as the chloride content of sea water.

The observations from cores are detailed in Table 1 and indicated that the outside reinforcement is in excellent condition i.e., that its ability to perform the required structural function is unimpaired in all locations and design conditions.

3.2.3 Mechanism of Reinforcing Steel Corrosion

Concrete of high quality, as utilized in the San Onofre Unit 1 intake structure, is inherently a sound inhibitor of corrosion in reinforcing steel due to its natural highly alkaline environment (pH of 12.5 or more). Extensive experience shows that the normal alkalinity provides a passive environment protecting the reinforcing steel (Reference 1). The introduction of chlorides into the concrete over a long period of time, however, can lead to breakdown of the protection afforded by the alkaline environment. In small quantities (less than about 0.1 percent by weight) (Reference 2) the chloride concentration does not significantly alter the passive environment. As the chloride ion concentration increases it begins to penetrate the passive film, reacting with the iron ion and accelerating corrosion by allowing more iron ions to go into solution (some experts have proposed that a $FeCl_6^{-3}$ ion is intermediately formed).

The migration of the chlorides through the concrete surface is initiated principally by natural immersion through capillary and diffusion action under the influence of a chloride ion concentration gradient. The extent and timing of this action is dependent upon the concentration of chlorides in the ocean water (about 19,000 ppm) and the ground water outside the intake structure (less than 300 ppm at the time of testing) along with the mechanical properties of the concrete. Thus the permeability properties of the concrete become important in minimizing the intrusion of the chloride ions into the concrete as well as other corrosion-inducing factors: oxygen, carbon dioxide, and water. In addition, low permeability, to a moderate degree, increases the electrical resistivity of the concrete which assists in reducing the rate of corrosion by retarding the flow of electrical currents within the concrete that accompany electrochemical corrosion.

When salt intrudes into the concrete it will destroy the chemical passivity of the steel so that it can corrode (Reference 10).

In general, the corrosion process of the reinforcing steel in an initially salt free concrete will proceed in the following steps:

1. By capillary action, the concrete will absorb sea water until a corrosive amount of salt will build up in the concrete (Reference 11). The time for this to occur in contingent upon the concentration of chloride in the environment, the quality of the concrete and the thickness of the concrete cover (Reference 12).

2. After the passivity of the steel is destroyed by the chlorides, there must be sufficient moisture present in the concrete so that it is an electrolyte. For a corrosion cell to function there must be a driving voltage to cause a current to flow. This driving voltage is a combination of steel in salt free concrete connected to that which is in salt contaminated concrete. Oxygen must be present at the cathode (non-corroding location) on the steel. Sea water contains dissolved oxygen. Or, if all of the steel is in salt contaminated concrete, the driving voltage can be created by the steel being in concrete of differential moisture or salt content.

3. Once the corrosion of the steel begins, the rust that is developed can create a force of approximately 4700 psi (Reference 13). This force alone is sufficiently great to crack the concrete covering (Reference 13). Then, depending upon the amount of cover and the spacing of the reinforcing steel, the concrete will spall and/or delaminate in a line parallel to the reinforcing steel.

An excellent description of the typical reactions which occur at the anode and cathode is quoted as follows (Reference 14):

Anodic reaction:

$$Fe \dashrightarrow Fe^{++} + 2e^- \quad (1)$$

Cathodic reaction:

$$2H^+ + 2e^- \dashrightarrow H_2 \quad (2)$$

Depolarization reactions:

$$4H^{++} + 4e^- + O_2 \dashrightarrow 2H_2O \quad (3a)$$

$$O_2 + 2H_2O + 4e^- \dashrightarrow 4OH^- \quad (3b)$$

The tendency of a metal to oxidize to a metal ion in an aqueous solution of normal ionic activity at standard temperature is given by its position in the electromotive force series. Iron, which is relatively high in this series, has a substantial tendency to enter into solution. The area where the ions go into solution is known as the anode. Electrons are generated to maintain ionic balance [Equation (1)]. This is often called the primary stage of the corrosion reaction.

The anodic region then has an excess of electrons and to maintain equilibrium an equal amount of electrons are consumed at adjacent surfaces of the metal to form hydrogen [Equation (2)]. This results in a thin protective film of hydrogen around the cathode which inhibits further progress of the reaction. The cathode is said to be polarized and no further reaction is possible unless the protective hydrogen film is removed (depolarized).

The destruction of the film may occur in one of two ways: (a) hydrogen evolved as a gas [Equation (2)] or (b) oxygen depolarization at the cathode [Equation (3a, 3b)].

These cathodic reactions, often called secondary reactions, are the controlling reactions on the rate of corrosion for iron and structural steels.

Consequently, any environmental condition which influences these reactions will likewise influence the rate of corrosion.

The reaction shown in Equation (2) is generally not characteristic of steel in concrete because the process is normally quite slow. Corrosion process characterized by cathodic depolarization from oxygen are the most common [Equation (3a,3b)] because the oxygen acts to prevent the buildup of hydrogen gas by consuming the free electrons.

The secondary reactions permit the primary reaction to proceed with the accumulation in solution of ferrous ions which are oxidized in the presence of water and oxygen and precipitated as rust. Two states of oxidation may exist depending mainly on the availability of oxygen. The first state, ferrous hydroxide [Equation (4)] which has a whitish color, is usually formed directly at the metal surface. Ferrous hydroxide is usually converted to ferric oxide [Equation (5)] at a little distance from the surface where it is in contact with more oxygen to produce the familiar reddish-brown rust.

$$2Fe + 2H_2O + O_2 \rightarrow 2Fe\,(OH)_2 \qquad (4)$$

$$4Fe(OH)_2 + 2H_2O + O_2 \rightarrow 4Fe(OH)_3 \qquad (5)$$

The magnitude of the electrochemical potential determines the tendency of the reaction to proceed, whereas the rate of corrosion is determined mainly by resistance to the continued process set up by certain of the corrosion by-products.

3.2.4 Reinforcing Steel Corrosion in the Intake Structure

The visual and petrographic examinations of the structure readily lead to the conclusion that the delaminations in the otherwise sound concrete were a direct result of corrosion of the reinforcing steel and the formation of chemical products that occupy a larger volume than the reinforcing steel consumed in the reactions.

For the intake structure chloride ions first entered the concrete due to the concrete's natural permeability and through existing cracks. It is known that once the chloride ions reached the reinforcing steel in sufficient concentration, they depassified the surface of the reinforcing steel, allowing the classic galvanic electrochemical corrosion process. The pressure buildup due to corrosion by-products is believed to have caused the delamination cracking at the reinforcing steel mat which then allowed free access to the reinforcing steel for additional chloride ions and oxygen.

The investigation found that the pump wells are the most severely corroded areas of the structure with the screen wells being less severely corroded. This is thought to occur for several reasons, all of which relate to the creation of an environment more conducive to corrosion:

- The intake area (pump wells and screen wells) has been dewatered for maintenance perhaps a dozen times in its life. Dewatering allows access of oxygen and drying of concrete surfaces while in the dewatered conditions. The pump well and screen well areas are normally the only portions of the structure that are dewatered.

- The circulating water pump bells provide a large mass of stainless steel in the pump bays and can function as the cathode in a galvanic cell since the pumps are not electrically isolated.

- Oxygen concentrations in the water are probably increased at the corrosion locations. With access at the screen wells, oxygen can be dissolved in the water due to the action of the traveling screens as well as the turbulence resulting in the pump bays from pump operation.

- Oxygen starved environments are known to cause corrosion of stainless steel screens and pumps. In the past during plant outages when such conditions are believed to be present, air lances have been inserted into the pump wells to bubble air (and oxygen) thus mitigating corrosion of the stainless steel circulating water pumps. In the screen well area, exposure to oxygen was maintained by operating the stainless steel travelling screens.

The petrographic analysis revealed cracking in some of the concrete samples and has indicated a moderate water-cement ratio. The most impermeable concretes would not have any cracking and would have a low water-cement ratio. Additionally, the moderate water-cement ratio probably results in an electrical resistivity which permitted a stronger galvanic potential between the steel rebar (anode) and the stainless steel pumps (cathode).

Surface cracking can also occur if the cover is small, however, the structure, with a 3 in. cover, did not exhibit surface cracking that could be noted by visual observation.

The visual and petrographic examinations indicate extensive amounts of iron hydroxides resulting from corrosion of embedded steel. The iron-oxide deposits have been found to be a mixture of black, magnetic oxide (magnetite Fe_3O_4) and brown ferric iron oxide hydrates ($Fe_2O_3.nH_2O$) with traces of green ferrous iron oxides. All the deposits of iron-oxide rust in the core samples contain water soluble chloride ions (confirmed by testing with a silver nitrate solution). While deposits vary in composition along the microcrack surfaces, the deposits include magnesium hydroxide ($Mg(OH)_2$), ettringite, calcite (calcium carbonate $CaCO_3$), calcium hydroxide ($Ca(OH)_2$), and trace amounts of alkalic silica gel, a minor contributor. Cores ranging from 12 in. deep to the full thickness of the wall or floor were cut to permit chloride analysis which allowed verification of the degree of contamination of the concrete by chloride ions. Plots of these chloride profiles showing percentage of chloride ion across the thickness of the structural member are given in Figures 4, 5, and 6.

A review of this data clearly indicates that the corrosion of the interior reinforcing steel has started and is in advanced stages in several areas within the pump wells and is approaching that state in areas of the lower screen wells. The operating environment will continue to provide oxygen from the sea water in solution. Extensive corrosion is not prevalent throughout the structure. The damage is most prevalent in the pump wells and lower screen wells. Areas west of the screen wells (nearly one-half of the structure) have been chipped and cored with the result showing sound concrete and reinforcing steel in areas required for structural integrity.

The pump well and lower screen well areas were repaired by use of steel strap plates as discussed in Section 5. Because inspections revealed that sound concrete and reinforcing steel exists west of the screen wells (i.e., the gate structure), no repair was required in that area. The most prevalent degradation of the reinforcing steel is limited to areas in the pump wells close to the stainless steel pumps. Areas away from these pumps have generally not shown deterioration and have been demonstrated to be sound. Another contributing factor to the locations of corrosion may be the potential differences which are created in various parts of this extensive structure by differences in concrete porosity and resulting chloride concentrations, normal impurities in the steel, etc.

The tests conducted on the full concrete thru-wall cores clearly indicated that the reinforcing steel on the exterior face (backfill side) is sound. Chloride levels at the exterior surface are low. Furthermore, there is a limited source of incoming oxygen to initiate and propagate the rusting process as well as a limited source of additional chloride ions (less than 300 ppm) in the fresh water against the exterior surface.

In summary, the reinforcing steel corrosion was caused by the breakdown of the alkaline protective environment through chloride ion intrusion penetrating the concrete due to permeability and microcracking followed by oxidation of the reinforcing steel from electrochemical reactions. The corrosion process is a classical failure mechanism which has been observed in reinforced concrete structures exposed to salt (e.g., parking structures, bridge decks, etc.) and, in more limited cases, in structures attacked by sea water.

The extent of deterioration is limited to inside reinforcing steel (sea water side) on the pump well and lower screen well structures. Areas west of the screen wells and the area south of the south pump well wall at the discharge tunnel have been inspected and will not affect the safety-related function of the salt water cooling pumps. Exterior wall surfaces of the structure are unaffected. The exterior surface of these walls have excellent integrity due to significantly lower chloride levels, as well as their freshwater environment, and the limited amount of oxygen.

4. ASSESSMENT OF EXISTING STRUCTURAL CAPACITY

After determining the physical condition of the structure, it becomes appropriate to compare the current structural capacity to that required by the original design requirements. An assessment of the results of the physical investigation must first be made as they will govern the structural analysis assumptions.

4.1 Theory of Structural Function

Reinforced concrete flexural members carry loads in such a way that compressive forces are resisted by the concrete, and tensile forces are resisted by the reinforcing steel. The reinforcing steel, in turn, develops its strength through bond. The bond stresses occur at the interface between the reinforcing steel and concrete and are due to adhesion, friction, and bearing on the surface as well as the deformations that are a part of the reinforcing steel.

Considering the reinforcing steel and concrete in the walls, failure mechanism as a result of corrosion may be due to either one or both of the following:

- o Reduction of reinforcing steel area due to corrosion.
- o Reduction in bond due to delamination of concrete and corrosion of reinforcing steel deformations which otherwise would provide bond resistance through interlocking with surrounding concrete.

This system must be intact to assure the integrity of the concrete.

The mechanism of bond failure usually involves eventual splitting of the surrounding concrete which allows the reinforcing steel to pull out without developing load transfer between the concrete and the steel. From this brief discussion, it is seen that both the cross sectional area and bond transfer capability are essential for proper functioning of the reinforcing steel and the concrete structural elements.

4.2 Criteria for Assessment of Existing Reinforcing Steel

If the existing wall reinforcing steel is utilized, an estimation of the structural capacity of existing walls must consider the following:

- Remaining cross-sectional area
- Remaining bond
- Projection of the remaining area and bond to the end of the design life

It is relatively easy to estimate the remaining cross-sectional area of the exposed bar after removing the rust. Calculation of the remaining bond is more difficult, although it can be stated that, for bond failure to occur, only inplane cracks are needed. Thus, if inplane cracking (delaminations) exists, the bond capacity of the rebar becomes questionable. Furthermore, corrosion of the deformations will cause additional loss of bond since no significant bond stress can be developed through bearing against the deformations.

As such, the bond capacity is unreliable if there are visible delaminations or if the reinforcing steel has lost most of its deformations and the bond capacity must conservatively be assumed to be negligible.

4.3 Structural Analysis Methods

The analysis of the circulating water intake structure consists of determining moments in the various structural elements for applied static and dynamic loads. All major walls except the central interior wall retain soil and are therefore subjected to soil and hydrostatic pressure. In addition, the forces due to a seismic event, principally dynamic soil and hydrodynamic pressures and inertial forces act upon these walls. The central interior wall is subject to hydrodynamic and inertial forces.

The static loads are calculated using the conventional methods. The dynamic loads are taken from the soil report entitled "Balance of Plant SONGS Unit 1 Soil Structure Interaction Methodology Report," Revision 1, July 1978 (Reference 4). The dynamic incremental earth pressure given in that report was based on a dynamic analysis.

All lateral loads are expressed in terms of lateral pressure diagrams, consistent with the Balance of Plant Seismic Upgrade Calculations. These pressure diagrams, together with inertial loads, are used in determining the design forces and moments.

The elements of the intake structure that are evaluated are identified on Figures 7 and 8. Conventional methods are used in the analysis of these elements, i.e., the intake and discharge culverts are treated as rigid boxes and are analyzed by the moment distribution method. The other elements are analyzed as a one-way or two-way slab between supports, as appropriate.

The assumption of a rigid box implies that the walls and slabs are integral and that sufficient reinforcement exists on both faces to assure continuity. Investigations have shown that the exterior reinforcement is in satisfactory condition. In the case of inside reinforcement in outside walls, where reinforcing steel deterioration has taken place, this assumption is still valid since negative moment (with inside reinforcing steel under compression) exists at the corners under all loading conditions. This occurs because there will always be active or passive soil pressure, together with hydrostatic effects, pushing inward on the exterior walls. So far as the interior wall is concerned, where reinforcement at both faces has corroded, the calculated moments are smaller than the unreinforced moment capacity of the wall. Thus, no cracking is expected under the DBE conditions assuming that both pump wells are full. Furthermore, the shear capacity of this wall considering only the concrete is also adequate to transfer the shear loads.

The above review of various structural elements indicates that the assumption of rigid box behavior is appropriate and that the design forces and moments are calculated utilizing adequate techniques.

4.4 Determination of Moment Capacity

The moment capacity of various elements is computed on the following basis:

- For those elements that are in direct contact with the sea water no credit is taken for the three inches of concrete cover on the inside face. This is a conservative assumption for those areas where no delamination exists.

- The reinforcement as shown on the original design drawings has been verified as adequate and used in determining the moment capacity.

- Concrete compressive strength is taken to be 4500 psi which takes credit for the inplace wet-cured conditions supported by the compressive tests. Considering that the lowest corrected compressive core strength from the test program is 5680 psi and the average of all the compessive strength tests is 7240 psi, this is a very conservative assumption.

Bechtel Structural Analysis program (BSAP OPTCON) gives moment versus axial load interaction plots and was used to obtain capacity of various elements.

4.5 Loading Conditions

The following loading condition was investigated for each structural element being evaluated, consistent with the Balance of Plant Structures Seismic Reevaluation Program (BOPSSR) (Reference 8).

D + L + E'

where D = dead load,
L = live load, including surcharge and hydrostatic,
E' = loads due to design basis earthquake of 0.67g horizontal and 0.44g vertical

It should be noted that the circulating water structure as modified in accordance with BOPSSR program, is considered in the present analysis.

4.6 Structural Analysis Results

The design forces and moments as discussed in Subsection 4.3 and the section capacities as discussed in Subsection 4.4 were used to obtain the data given in Table 3. The numbers in the column under the heading "Required Reinforcement Percent" are the percentages of original reinforcement (reinforcement shown on the drawings) which will be required to sustain the design loading condition. The "Available Reinforcement Percent" values are based on Table 2, reflecting the condition that existed before repairs.

The data shown in Table 3 indicates three distinct regions of the intake structure in regard to both observations and remedial action:

(1) Pump well areas: The interior reinforcing steel is heavily deteriorated on the walls and therefore requires remedial action. The ceiling reinforcing steel (underneath the pump deck) is sound with local corrosion only. The interior floor (basemat) reinforcement is not required for structural integrity. The outside wall reinforcement is in excellent condition as demonstrated by the core drilling results. Therefore, no remedial action is necessary. Assessment of the structural condition prior to undertaking repairs would be that the structure would function satisfactorily under normal operating loads, considering the code allowables. Under the DBE conditions the structure would be expected to maintain overall integrity, although the integrity of the salt water cooling pumps could not be positively confirmed without an extensive study of the in-place capacity of the concrete walls with deteriorated reinforcing bars.

(2) Screen well areas: The wall reinforcement at lower levels is about 20 percent corroded. However, the potential for eventual delamination exists with rust deposits on the surface. Since the effectiveness of the bond is questionable, repair of the lower portion of this area is considered prudent. Assessment of the structural condition prior to undertaking repairs would be that the structure would function satisfactorily under seismic loading if the deterioration process did not accelerate.

(3) Intake area: The structure from the pump wells to the tsunami gates is in good condition in areas required for structural integrity and adequate reinforcement exists to resist the design loads. Therefore, no remedial action is necessary in this area.

5. CORRECTIVE MEASURES AND LONG TERM PROTECTION

5.1 Repair Objectives

The Final Safety Analysis requires that the intake structure meet Seismic Category A requirements, including the DBE, to the extent that water is always available to the Salt Water Cooling Pumps. To assure this requirement is met, the pump well area and the screen well area will be repaired in order (1) to properly support the pumps, (2) to prevent them from being flooded, and (3) to assure that the 1.3 percent flow is available to the pumps. The Salt Water Cooling Pumps are located on the Pump Room slab and are thus subject to flooding if any of the walls or the slab fail in that area.

The final consideration for determining the nature of the required repair was the condition of the reinforcement. The repair scheme was based on conclusions of the condition of the reinforcing steel and the concrete as determined from the inspections, tests, and examinations discussed in Sections 2 and 3, and an assessment of existing structural capacity as discussed in Section 4.

The conclusions are as follows:

1. The inside face reinforcing steel in the walls of the pump well and lower screen well areas is corroded and no credit is taken for this steel.

2. The bottom reinforcement in the pump deck is only locally corroded but has conservatively been assumed to be unusable in most areas except for the heavily reinforced area adjacent to the circulating water pumps which was shown by chipping to be in good condition.

3. The interior reinforcing steel in the base slab is not required for structural integrity and therefore no remedial action is required.

4. The outside face rebar is in excellent condition and can be expected to resist the design loads, therefore no remedial action is required.

5. Reinforcement that is above the sea water level is in excellent condition and will resist design loads, no remedial action is required.

6. The reinforcement west of the screen wells is also in very good condition in areas required for structural integrity. No remedial action is required.

7. The reinforcement in the walls of the discharge structure is not required for structural integrity, no remedial action is required.

5.2 Restoration of Design Margins

5.2.1 Description of Repairs

The walls and slab of the pump well and lower screen well area are strap plated with carbon steel plate strips coated with a coal tar epoxy coating to protect them from corrosion. Sacrificial anodes are added to provide additional protection. Monel anchor bolts are used to attach the plates to the wall. The anchor bolts are fabricated from corrosion resistant material such as ASTM B-164 and will be grouted in place. The delaminated 3 to 4 in. of concrete is left in place and will be kept in place by pretensioning the bolts to generate compression at the delaminated interface. The delaminated 3 to 4 in. thick concrete will readily resist the compressive stresses induced since the concrete strength is unaffected by the corrosion of reinforcing steel.

The strap plates were used on the walls and ceilings wherever credit for the existing reinforcement could not be taken for the design loads (see Figures 9 and 11). Additional chipping was done in some areas where heavy reinforcement exists to verify its adequacy (see Table 2). In areas where reinforcement is not required for structural integrity or was determined to be adequate and no lamination of concrete exists, a strap plate was not provided.

Surveillance requirements to monitor the existing reinforcement for which credit is taken will be incorporated into existing station procedures.

Details of the repairs are shown on Figures 9-11.

5.2.2 Design of Strap Plates and Anchor Bolts

The frames and sections of the intake structure were analyzed under the design loads for 0.67g. After calculating the maximum positive moments, the strap plate was then designed assuming composite action between the plate and existing concrete. Strap plate sizes were standardized; the spacing was adjusted on the basis of actual moments.

Composite action between the strap plates and concrete is assured using shear connectors. For this purpose monel anchor bolts were grouted in place, with a bond breaker, and then torqued using the AISC provisions to introduce pretensioning in the bolts. This, in turn, provides compressive stresses between the plate and concrete and also at the laminar surface.

Design of the anchor bolts is based on both the ACI shear friction methodology and the AISC composite design methodology. In the shear friction concept, normal reinforcement is provided to develop shear friction between the strap plate and concrete. The area of bolts that will function as normal reinforcement is determined from

$$A_{vf} = \frac{V_n}{\phi f_y \mu}$$

where V_n = shear strength required, conservatively taken as the yield strength of the plate; ϕ = strength reduction factor, 0.85; f_y = yield point of the bolt material, and μ = coefficient of friction. There is no specific value of coefficient of friction for a case where concrete is placed against coal tar epoxy coated steel plate. The ACI code specifies 0.70 for concrete against bare steel. Considering the shear friction mechanism where the resistance to shear forces is developed by a concrete wedge pushed against the plate which, in turn, is kept in place by tensioning of the normal reinforcement (i.e., the bolts), a 0.7 factor is used.

Following the AISC code composite design procedures and extrapolating the existing test data on common shear connectors to the bolt design selected, approximately the same number of required bolts was obtained to develop the strength of the plate.

To further assure composite action of the strap plate and concrete without any significant slip between the two elements, the following requirements were incorporated:

(1) Pretension the bolts using the turn of the nut method. This assures early activation of "shear friction resistance."

(2) Minimize the gap between the plate and bolt by using hardened plastic spacers which will, at the same time, act as electrical insulators.

These provisions will assure composite action between the strap plates and and concrete. Any additional loads on the wall will be resisted by both the concrete and the plates.

The question of preloading the strap plates by means of forced displacement of the walls prior to plate installation was also considered. Such a provision was not deemed necessary because the calculated deflections are very small, the maximum value being less than one-sixteenth of an inch.

This summarizes the significant aspects of the design involving strap plates.

6. SUMMARY AND CONCLUSIONS

6.1 Summary

The investigation and evaluation demonstrated that the pump well and screen well areas of the intake structure had deteriorated sufficiently to possibly impair their structural function. The driving mechanism of deterioration was determined to be classic electrochemical corrosion of reinforcing steel initiated by the intrusion of chloride through the concrete to the reinforcing steel.

Structural analysis dictated that repairs be made to replace the function of the corroded reinforcing steel in the deteriorated areas and thus provide assurance that the portions of the structure important to safety would withstand a Ø.67g earthquake. This was done by attaching steel plates and anchor bolts with an appropriate corrosion-protection system to the surface of the deteriorated areas.

6.2 Conclusions

The repairs as described in this report are all that is necessary to restore the structure to conformance with design requirements.

As noted above, electrochemical corrosion precipitated by intrusion of chloride ions is the cause of the structural deterioration, which has occured primarily in the pump well and screen well areas of the structure. Remaining areas of the structure are in satisfactory condition and will be monitored by surveillance requirements which will be developed and incorporated into existing station procedures.

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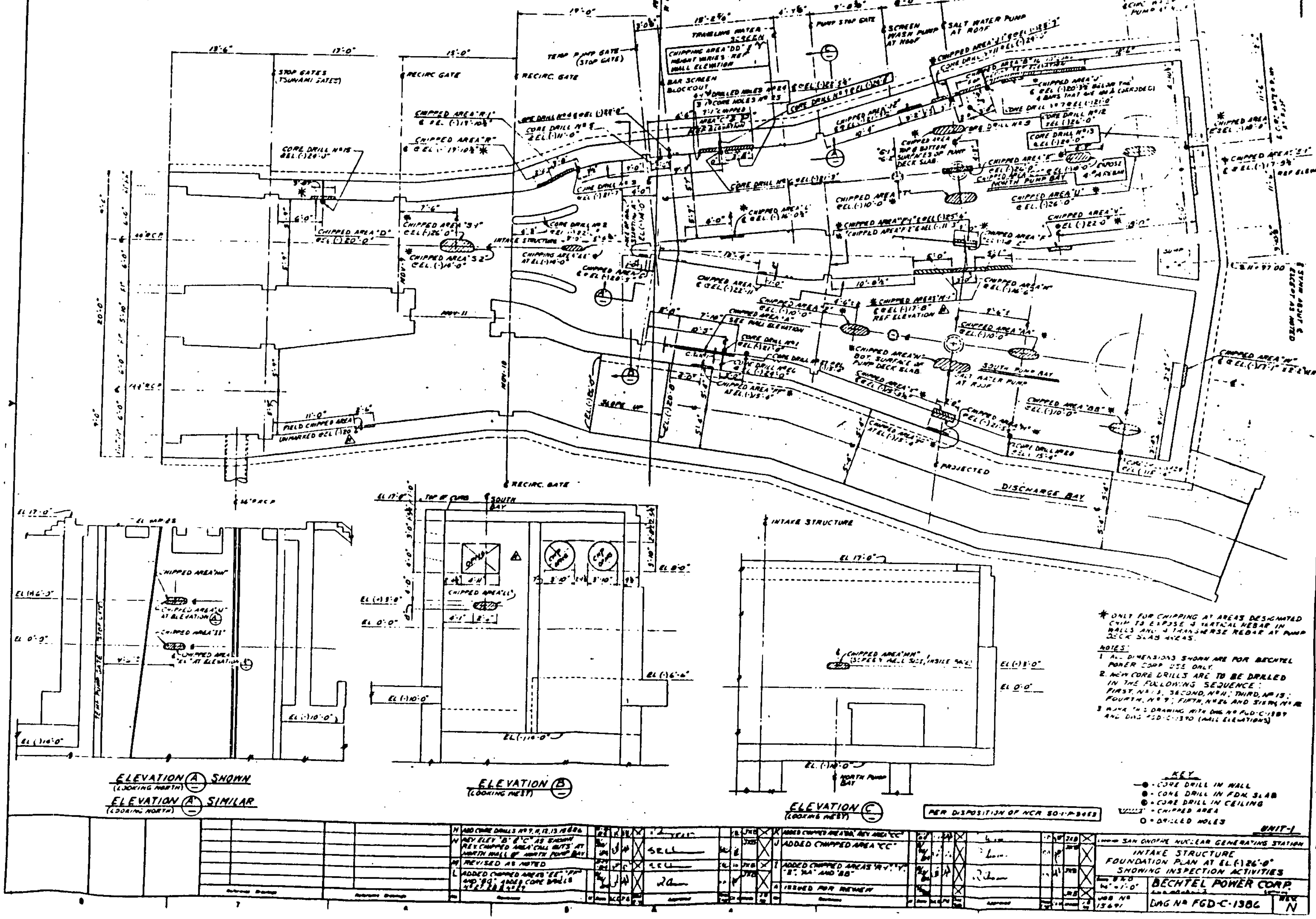
FIG. 1 CORE DRILLING AND CHIPPING AREA LOCATIONS - PLAN



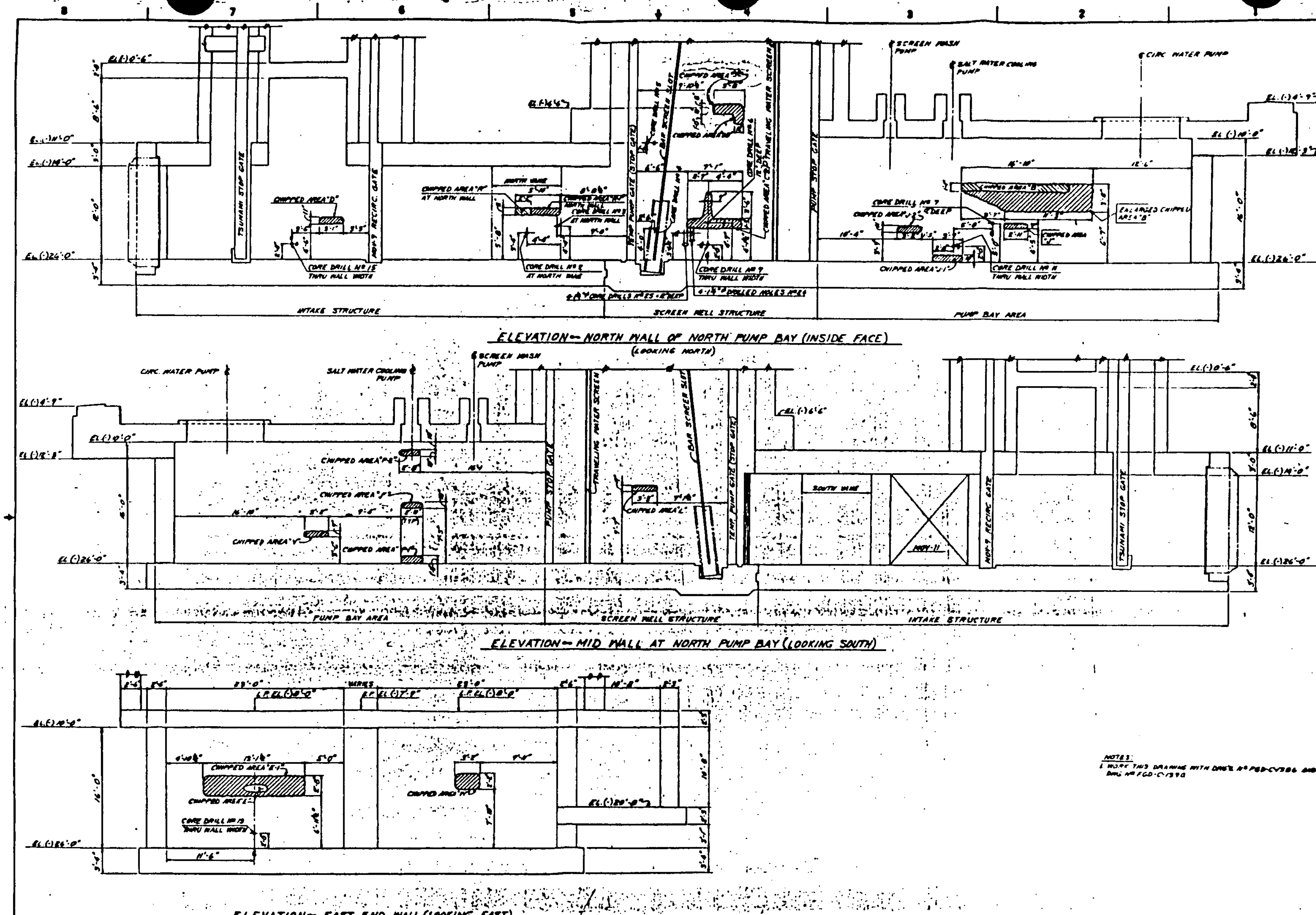


FIG. 2 CORE DRILLING AND CHIPPING AREA LOCATIONS — ELEVATIONS

INTAKE STRUCTURE WALL ELEVATIONS - NORTH PUMP BAY - SHOWING INSPECTION ACTIVITY

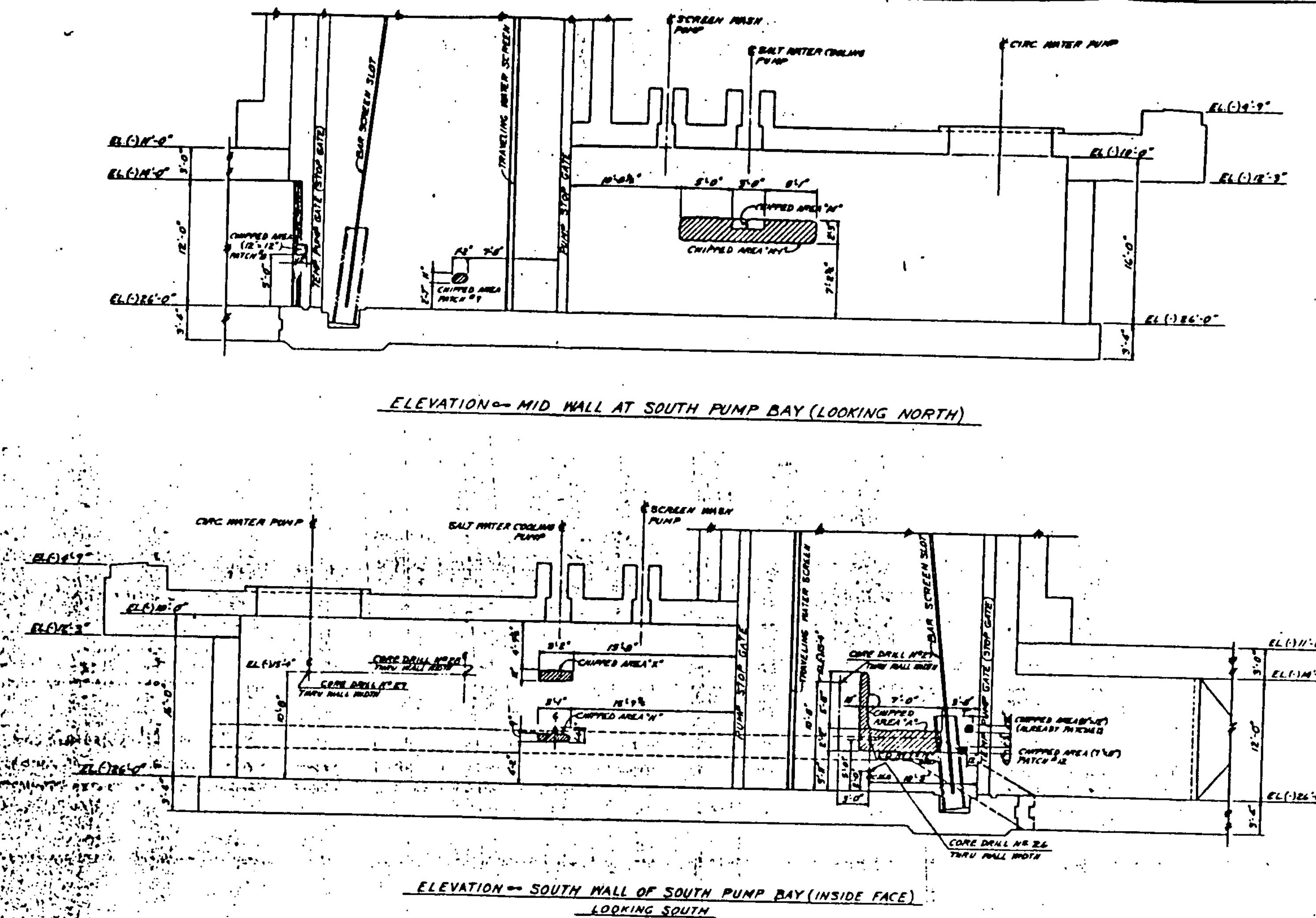


FIG. 3 CORE DRILLING AND CHIPPING AREA LOCATIONS_ ELEVATIONS

NOTES:
1 WORK THIS DRAWING WITH DWG NO F6D-C-1306 AND DWG NO F6D-C-1307.

INTAKE STRUCTION
WALL ELEVATIONS - SOUTH PUMP BAY-SHOWING INSPECTION ACTIVITY

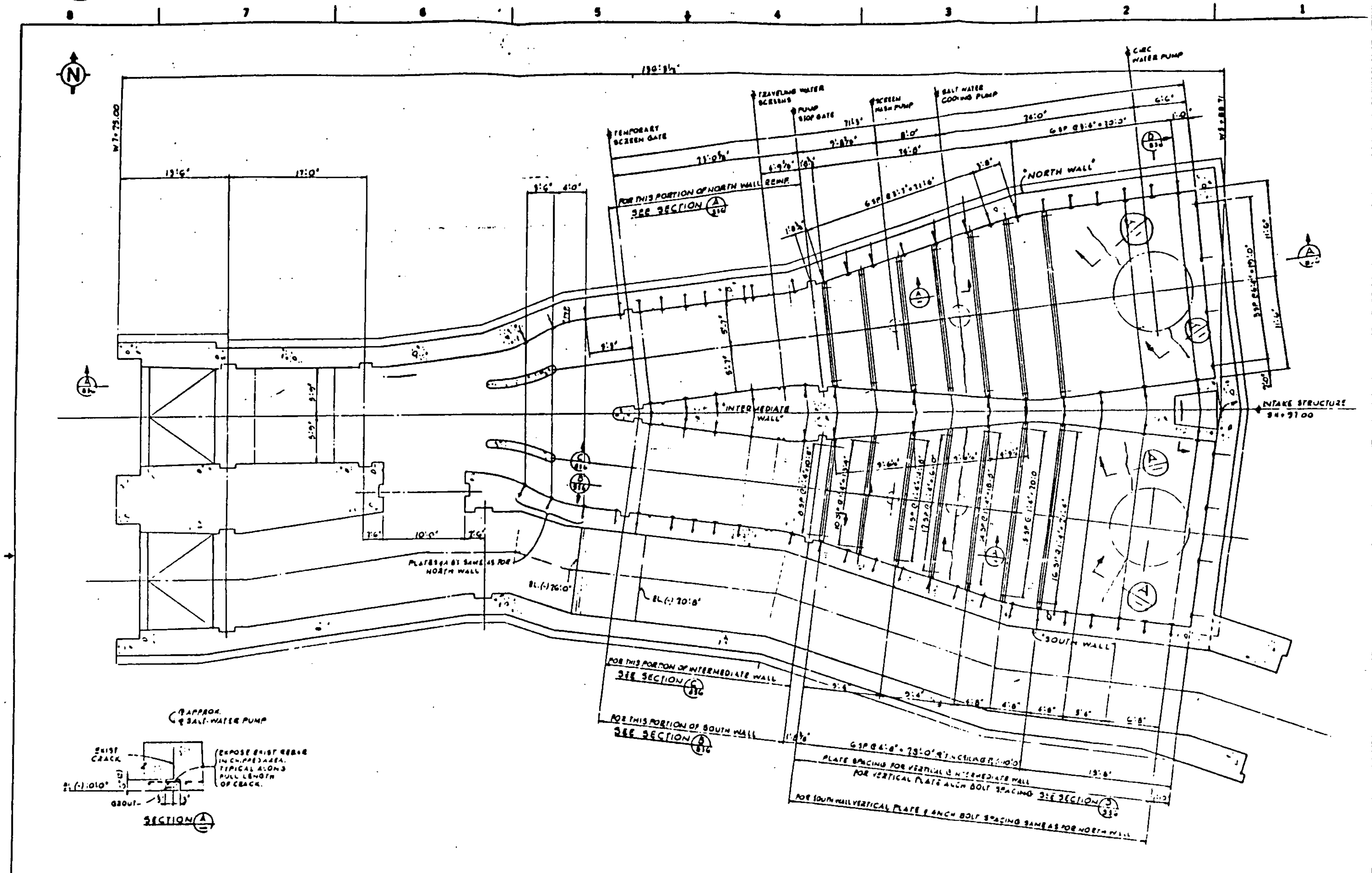


FIG. 9 REPAIR DETAILS - PLAN

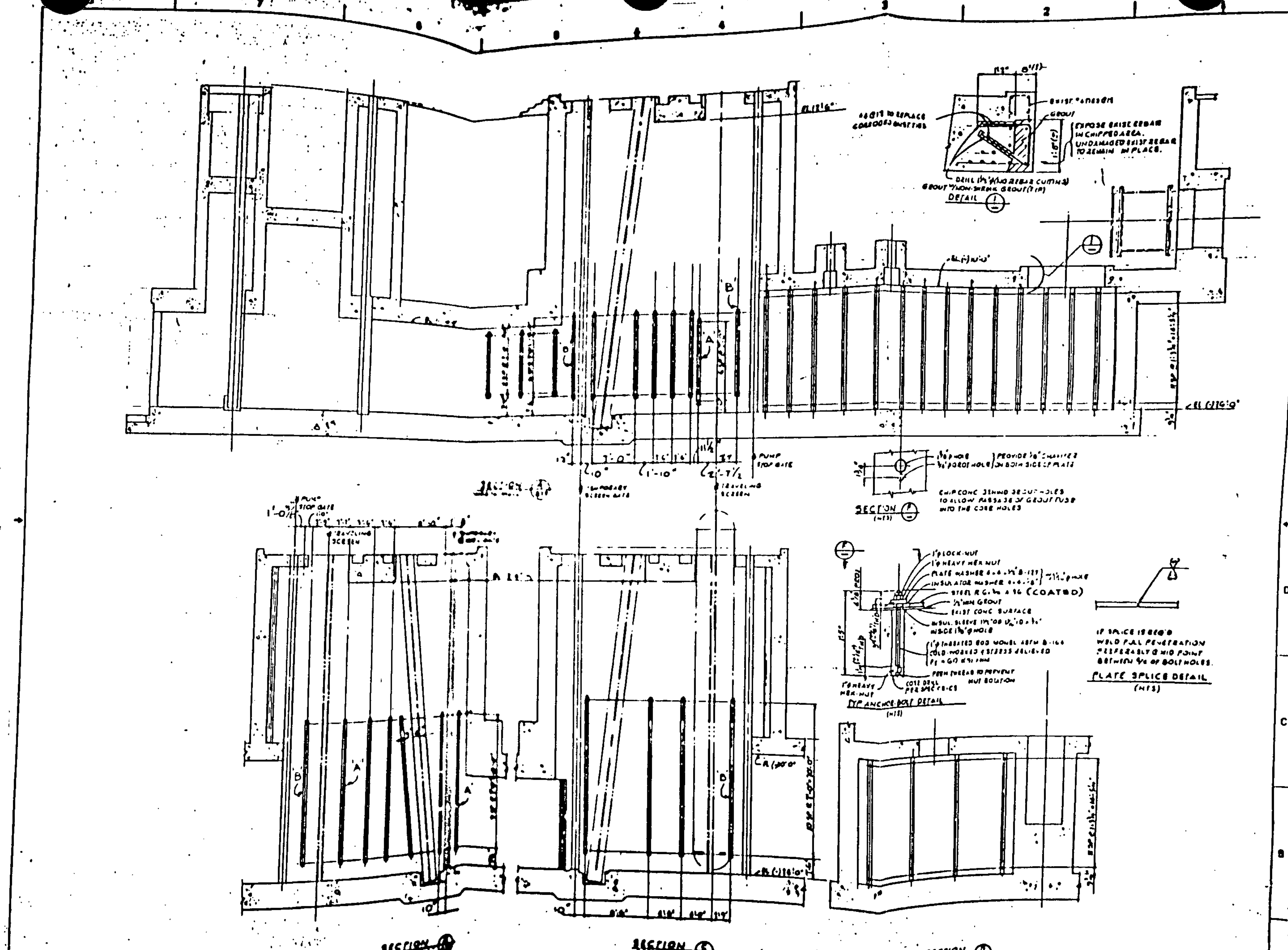
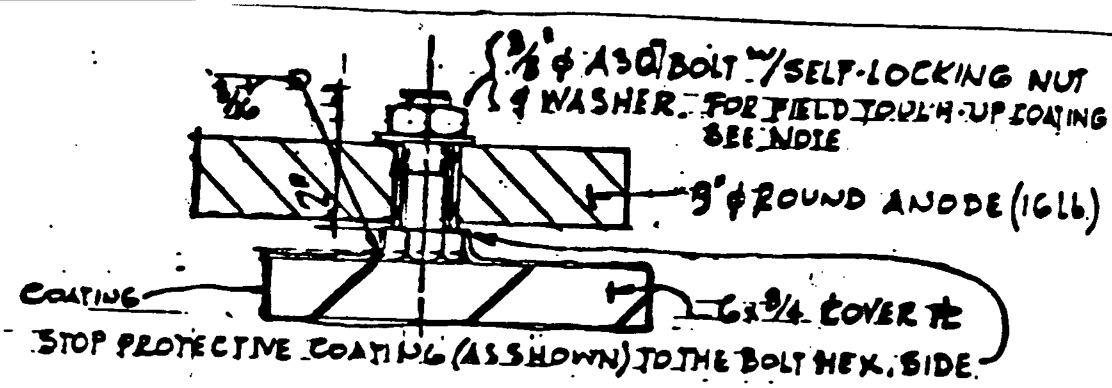
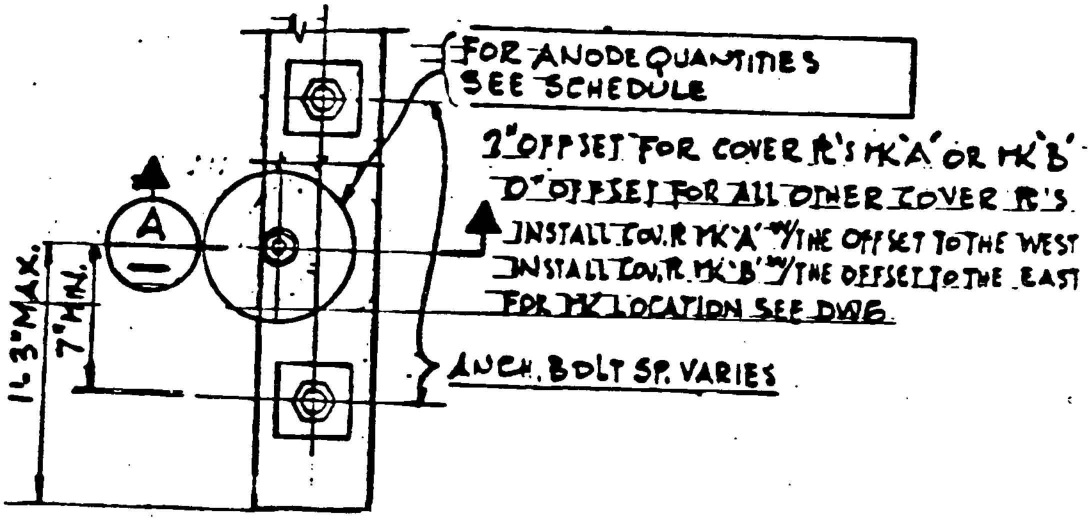


FIG.10 REPAIR DETAILS _ SECTIONS

FIG. 10



SECTION A
(N.T.S.)



PARTIAL ELEVATION
OR PLAN OF COVER PLATE

SACRIFICIAL ANODE SCHEDULE

PLATE LENGTH	NO. OF ANODES PER PLATE
More than 9'-0" but less than and including 13'-0"	3
More than 13'-0" but less than and including 17'-0"	4
More than 17' -0"	5

Notes For Anode Installation

1. Maximum anode spacing shall be 60" ± 6".
2. After completion of bolting, bolt/nut/washer assembly on top of anode shall be coated
3. Zinc anode shall be 1" X 9" ϕ, 16 pounds, federated metals, type CZ-9 W/ 1/2" I.D. embeded galvanized sleeve or approved equal.

FIGURE-11 CATHODIC PROTECTION DETAILS

TABLE 1: LOG OF CORE SAMPLES TAKEN AND OBSERVATIONS

CORE NO.	LOCATION	OBSERVATIONS ON CORE
1 (3" dia. x 6")	10'-5" east of stop gate slot on south wall of south pumpwell EL (-)21'0" Area had a rust spot of approximately 2'3" diameter with no visible signs of cracking.	First 1/2"-1" appeared relatively sound, next 2" up to the rebar had coarse aggregate only. Remaining section of core appeared sound. Core hole displayed signs of both rebar and concrete in late stage of deterioration. The extent of damage is inward to first layer of rebar. Corrosion and deterioration followed layer of rebar along the wall.
2 (3" dia. x 9")	4'-3" from upstream nose on south side of north pumpwell vane EL (-)22'0" Area of sample was sound concrete with no signs of rust etc.	Core and core hole appears sound. Rebar is relatively good with signs of minor rusting.
3 (3" dia. x 9")	9' upstream of stop gate on north transition of pumpwell EL (-)21'-7" Area has typical rust spots similar to core #1	Core was removed in two pieces separated by rust and exudence. Separation of core was at 3" to 4" from concrete surface. No rebars visible. Core hole shows circumferential crack approximately 3" to 4" from concrete surface with rust bleeding.
4 (3" dia. x 9")	3' downstream of stop gate slot on north wall of north pumpwell EL (-)22'-0" Area cored showed no signs of distress in the immediate area.	Core was removed in two pieces. The core separated at the rebar interface, interface filled with rust and exudence.
5	EL (-)11'-0" directly above repair area of north wall screen well gate slot.	A zone of approximately 2" wide beginning 4" from the surface was discolored to varying degree with two planes about 1" apart where core fractured. Fractured plane is discolored and the fractures passed through aggregate. Core hole has circumferential cracks separating the layers of concrete. No rebar was cut.
6	1st attempt halfway between the two screen slots in the north wall of north screen well.	Hit rebar. Face of hole at the rebar was badly discolored with rust and rebar was corroded on its surface
	2nd attempt 4" to east. EL (-)21'-3"	The core has zone of discoloration about 2" wide starting about 3-1/2" in from the surface with 2 major cracks running through core. Core hole is normal except for rust stained circumferential crack about 3" from the surface of concrete.

TABLE 1: LOG OF CORE SAMPLES TAKEN AND OBSERVATIONS (CONTINUED)

CORE NO.	LOCATION	OBSERVATIONS ON CORE
7	North wall of north pumpwell north east of salt water pump EL (-)18'-0"	Core had 2 cracks about 1-1/2" apart, the first one about 2-1/2" from surface. The crack planes had rust stains on their surfaces. Core hole had circumferential crack which was rust stained about 3" from surface. No rebars were cut.
8	4'-3" north of salt water pump opening intercepting a crack that runs from pump opening to the north wall of north pumpwell ceiling.	The crack runs through the core at a slight angle about 9" where it angles out of the core. Crack is rust stained and of significant size. There are two laminar cracks about 3" and 4-1/2", respectively from concrete face. The crack surfaces are rust stained, crystalline cover of salt, crack passes through some of the aggregate.
9 (1-3/4" dia through core)	North wall of north screenwell EL (-)24'-0"	A piece of bar with 3-3/4" cover was cut, slightly rusted. Delamination at innermost layer of rebar is severe and badly rusted stained. Remainder of core in excellent condition. Backfill side rebar has no rust, concrete is neither discolored nor delaminated.
11 (1-3/4" dia through core)	North wall of north pumpwell at EL (-)24'-0" at salt water pump centerline	Three inches from inner surface typical laminar crack with rust discoloration. Remainder of sample is completely normal in all respects. No rebar were cut in this sample.
12 (1-3/4" dia through core)	Floor of north pumpwell 2 feet south of north wall	From top surface inward a delamination appears at 4-3/4 inches from the top surface which is severely rust colored. No rebars were cut in this sample. Remainder of the sample appears excellent.
13 (1-3/4" dia through core)	East wall of north pumpwell on pump centerline EL (-)24'-0"	Innermost layer of rebar rusted. Rust stained laminar crack about 3" into the wall separates the core. Both surfaces of the crack rust stained. One vertical and one horizontal bar were cut. Both bars rusted on the surface. Remainder of the core is in good condition.
15 (1-3/4" dia through core)	North wall of intake 6 feet east of tsunami gate EL (-)24'-0"	3 rebars were cut (1 horizontal in the inner layer and 2 bars in outer layer). Concrete and rebars in this sample are in excellent condition. No rust or delamination.
24	Adjacent to core #25	Series of four 1" diameter holes for chloride analysis.

CORE NO.	LOCATION	OBSERVATIONS ON CORE
25	North wall north screen well	Core #25-1, -2, and -3 1-3/4" diameter. Sent for chloride analysis. These cores had wide laminar zone about 3-1/2" from surface. Laminar separation severely rust stained. No rebars were cut. Concrete beyond laminar zone is in good condition.
26	South wall south screen well EL (-)24'-0"	Partial core. Core barrel broke off.
26A (1-3/4 " dia. through core)	South wall of south screen well EL (-)24'-0"	Inner layer of rebar with 3 inches cover was cut. Rebar is severely rusted. Concrete at first layer of rebar is delaminated and badly stained with rust. Remainder of sample is in good condition. One outer layer rebar was cut with 4-1/2 in. cover remaining. Backfill side concrete and rebar was neither rusted nor delaminated. When core was removed, water came rushing at the rate of 70 gpm for 3-1/2 hours.
27 (1-3/4 " dia. through core)	South wall of south screen well EL (-)15'-4"	North (screenwell) side concrete is delaminated and discolored at 3" depth. No rebar was cut. South (discharge box) side - no signs of concrete delamination One rebar cut shows no sign of rust.
28 (1-3/4 " dia. through core)	South wall of south pump well half way between circ. water and salt water pumps EL (-)15'-4"	North (pump well) side - concrete delaminated and severely rust stained. No rebar cut. South side - core damaged 2-1/2 in. from discharge face showed appearance of products of corrosion, but could not be verified as delamination, since core faces were severely ground against each other in the removal process. No rebar cut.
29 (1-3/4 " dia. through core)	South wall of south pump well on circ. water pump centerline EL (-)15'-4"	North (pump well) side - concrete delaminated and rusted. No rebar cut. South (discharge box) side - No signs of delamination and rust.

TABLE 2: LOG OF OBSERVATIONS ON CHIPPED AREAS

Chipping Designat.	Location	Remaining Reinforcing % (Note 1)	Remarks
A	South Screenwall	80-85	Delamination
B	North Wall, pumpwell El, -16'-0" ft.	15-20	Delamination. Loss of continuity of rebar
C	North Screenwall	80-85	Delamination. One rebar mostly gone.
D	North Wall, near tsunami gate	100	No delamination. Looks O.K.
E	East Wall, north side	50	Delamination
F	Center Wall, north side	80	Partial delamination
G-1	Pump Deck, top, near salt water pump	100	No rust except slight rusting at visible crack. No delamination.
G-2	Pump Deck, bottom, under G-1	100	No delamination. However, core #8 a few feet away shows delamination.
H	South Wall, pumpwell area El - 21'-0"	100	Further chipping vertically up shows delamination and rust, one bar at 50%
J	North Wall, below B	75	Delamination. Continuation of bars from B.
K	North Pumpwell, floor under SWP	100	No delamination. More than 4-inch cover. No apparent rust.
L	Center Wall, north side near screens	90	No delamination.
M	Center Wall, south side	40-50	Signs of delamination.
N	South Pumpwell, floor under SWP	100	Minor rust. No delamination.
P	North Wall of North Pump Well El - 21'-9 1/2"	80	Discoloration and delamination of concrete. No rust on the form tie.
R	North Wall, transition west of screenwell Elev. (-)20'-0"	100	No signs of discoloration or delamination.
S-1	Floor east of Tsunami gate	100	No rust nor delamination.
S-2	Roof east of Tsunami gate	100	No signs of rust discoloration or delamination
T	Underside of Pump Deck near the north salt water pump	95	No signs of discoloration and delamination.
U	Floor directly under north circ water pump EL (-)26'-0"	85	Signs of discoloration or delamination

Chipping Designat.	Location	Remaining Reinforcing % (Note 1)	Remarks
V	Center wall, north side Elev. (-)22'-0"	60	Discoloration and delamination of concrete.
W	East wall south pump well Elev. (-)17'-0"	50	Concrete is severely discolored and delaminated. Discolored moisture oozing from the chip.
X	South wall of south pumpwell Elev. (-)15'-0"	80	No discoloration or delamination.
B-Ext	Extension of Chip B described above	5	Concrete severely discolored and delaminated.
E-1	Extension of Chip E described above	70	Concrete severely discolored and delaminated.
F-1	South wall, north pump well Elev. (-)25'-0"	70	Concrete appears discolored and delaminated.
F-2	South wall, north pump well Elev (-)11'-0"	80	Concrete is discolored and delaminated
J-1	North wall, north pump well Elev (-)26'-0"	80	Concrete is discolored and delaminated.
J-2	North wall, pumpwell Elev (-)22'-0"	50	Concrete is discolored and delaminated.
M-1	Extension of Chip M center wall, south side Elev (-) 16'-6"	25	Concrete severely discolored and delaminated.
AA	Roof of South Pump well midway between circ. water and salt water pump centerline	95	Slight delamination and discoloration of concrete. 4 rebars exposed, show signs of slight rust.
BB	Roof of South Pump well on circ. water pump centerline 3-1/2 feet from south wall	100	No delamination or discoloration of concrete. 4 transverse and 1 longitudinal rebar exposed, show signs of slight rust.
CC	Roof of North Pump Well 6 feet west of circ. water pump centerline	100	No signs of delamination and discoloration of concrete. 10 rebars exposed, show no signs of rust.
DD and Y	Inner face of north wall of screen well between Elev (-)6'-0" and (-)10'-0" at grade beam	80	Concrete delaminated and discolored near inside rebar. Grade beam rebar was not visible after chipping into the wall 18" and no corrosion or discoloration was observed in the concrete.

TABLE 2: LOG OF OBSERVATIONS ON CHIPPED AREAS (CONTINUED)

Chipping Designat.	Location	Remaining Reinforcing % (Note 1)	Remarks
FF	North wall of discharge bay at EL (-)15'-4" opposite the screen well	95	Four vertical rebar exposed, signs of minor rusting.
GG	North wall of discharge bay at EL (-)15'-4" on the centerline of salt water pump	100	Four vertical and one horizontal rebar exposed, no signs of rust and the concrete neither delaminated nor discolored.
HH	North wall of the screen well EL (+)6'-0"	100	5 rebar exposed. No sign of rust, delamination or discoloration of rebar or concrete.
II	North wall of the screen well EL (+)0'-0"	100	2 rebar exposed. No signs of rust, delamination or discoloration
JJ	South face of the center wall EL (+)6'-0"	100	No signs of rust, delamination or discoloration
KK	South face of the center wall EL (+)0'-0"	100	4 vertical rebar exposed. No signs of rust, delamination or discoloration.
LL	East face of wall upstream of south screen well EL (+)3'-0"	100	2 rebar exposed. No sign of rust, delamination or discoloration.
MM	West face of north screen well wall that separates the screen well from the pump deck	100	No sign of rust, delamination or discoloration.
R and R-1	North wall of transition west of screen well Elev (-)22'-0"	100	No signs of delamination and discoloration of concrete. 7 vertical rebar exposed show slight rust.
Z	Roof of south pump well, 4 feet west of screen wash pump Elev (-)10'-0"	60	Concrete is severely delaminated and discolored. 4 transverse, 1 horizontal and 1 bent rebars exposed; all show severe rust.
Unmarked	South wall of discharge bay at EL (-)20'-6" and approximately 12'-0" east of Tsunami gate.	100	Four vertical and one horizontal rebar exposed. Minor rusting. No signs of concrete delamination or discoloration.

NOTES: 1. Remaining reinforcing steel is estimated as a percentage of the original cross-sectional area of all exposed bars by visual examination.

TABLE 3: STRUCTURAL EVALUATION

SECTION DESIGNATION	THICKNESS (T)	ORIGINAL REINFORCEMENT	REQUIRED REINFORCEMENT PERCENT	AVAILABLE REINFORCEMENT BEFORE REPAIR PERCENT	REFERENCE CHIPPED AREA OF TABLE 2	REMARKS
A and B	2'-6"	#10 @ 4-1/2" (V), outside	63	100	6.1	Section is O.K.
C	2'-6"	#11 @ 9" (V) outside	83	100	-	Section needs repair to provide positive moment reinforcement
		#6 @ 9" (V) inside	83	15-20	B, J, J-1, J-2, B-ext.	
D	4'-10"	#7 @ 9" (V)	25	25-90	V	Section needs repair on both sides
E	2'-6"	#8 @ 9" (V) inside	70	50-100	H, X	Section needs repair to provide positive moment reinforcement only
F	2'-6"	#10 @ 9" (V) inside	36	100	-	No rebar deterioration is expected
		#8 @ 9" (V) outside	34	100	-	
G	2'-6"	#11 @ 9" (T)	40	100	-	No rebar deterioration is expected
		#10 @ 9" (B)	59	100	-	
H	2'-3"	#8 @ 9" (T)	13	100	6.1	Core #8 and observation of pump opening show delamination. Section needs repair
		#6 @ 9" (B)	61	100	6.2, T, CC	
J	2'-6"	#8 @ 9" (V) outside	97	100	-	Observed inside face delamination and corrosion are in negative moment area (near the grade beam) and no repair is required. Inside reinforcement at higher elevation which is in the positive moment area is OK EL (-) 6'-6" to +15'-0"
		#8 @ 9" (V) inside	118	100	DD, Y HH, II	

TABLE 3: STRUCTURAL EVALUATION (CONTINUED)

SECTION DESIGNATION	THICKNESS (T)	ORIGINAL REINFORCEMENT	REQUIRED REINFORCEMENT PERCENT	AVAILABLE REINFORCEMENT BEFORE REPAIR PERCENT	REFERENCE CHIPPED AREA OF TABLE 2	REMARKS
J	2'-6"	#8 @ 9" (V) outside	102	100	A, C	Section needs repair to provide positive moment reinforcement EL (-) 10'-0" to (-) 26'-0"
		#9 @ 9" (V) inside	94	80-100		
L	2'-0"	#8 @ 12" (H)	89	100	-	No deterioration of rebar is expected
		#6 @ 12" (H)	96	100	-	
M	2'-0"	#10 @ 9" (V)	93	100	-	No rebar deterioration is expected
		#6 @ 9" (V)	87	100	-	
N	2'-6"	#8 @ 9" (V)	106	50		Section requires repair along interior face (Positive moment area)
		#7 @ 18" (H)	131		E, W, E-1	
R	3'-0"	#11 @ 9" (B)	42	100	S-2	Section has appropriate integrity
		#7 @ 18" (B)	62	100		
S	2'-6"	#9 @ 12" (H)	111	-	-	Delamination nearby. Assumed corrosion; needs repair.
		#7 @ 12" (H)	73	100	-	
T	2'-6"	#7 @ 9" (V)	101	100	D	Section has appropriate integrity
U	1'-6"	#8 @ 9" (H)	152	100	-	Section is O.K. Required ductility is less than 3.
V	Varies	-	-	-	F, F-1, F-2, M, M-1	Section needs repair
W	4"-10"	#8 @ 9" (V)	64	25-90	L	Section needs repair
		#8 @ 18" (H)	85			

TABLE 3: STRUCTURAL EVALUATION (CONTINUED)

SECTION DESIGNATION	THICKNESS (T)	ORIGINAL REINFORCEMENT	REQUIRED REINFORCEMENT PERCENT	AVAILABLE REINFORCEMENT BEFORE REPAIR PERCENT	REFERENCE CHIPPED AREA OF TABLE 2	REMARKS
X	1'-0"	#5 @ 18" (V)	60	100	-	Vanes are O.K. except for cracking and rusting at the tips which need local repairs
Beam K	-	10 - #18	-	100	DD, Y	No delamination or corrosion at beam reinforcement (more than 18" of cover)
Floor	3'-4"	varies	-	85-100	K, N, S-1, U	No signs of delamination or corrosion except beneath the circulating water pumps. Section is O.K. even if the inside rebar is assumed not to exist.

NOTES:

1. Required reinforcement, percent, is based on ignoring 3 in. concrete on the inside.
2. All sections, where required reinforcement percent exceeds 100% and where no repair was needed, are adequate with the use of ductility ratio $\leq$3.0.
3. Available reinforcement percentages are based on conditions before repair.
4. All repaired sections are designed to remain elastic under the design loading condition.