UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

SAFETY EVALUATION BY THE OFFICE OF NUCLEAR REACTOR REGULATION

EVALUATION OF DEGRADED INTAKE STRUCTURE

SOUTHERN CALIFORNIA EDISON COMPANY

SAN ONOFRE NUCLEAR GENERATING STATION, UNIT 1

DOCKET NO. 50-206

1.0 BACKGROUND

In June 1984, exposed reinforcing steel bars (rebars) and concrete spalls were found by the licensee at the north stop gate slot (Figures 1 and 2) of the Intake Structure. Subsequent dewatering and inspection of the Intake Structure showed that the reinforcing steel at the inside concrete surfaces of the pump wells and screen wells was corroded in many areas. In order to define the extent of corrosion and its effect on the structure, a coring and chipping of the concrete and testing was undertaken. Additional tests and examinations included halfcell potential measurements, petrographic examinations, and chemical testing for chloride content. Reference 1 describes the licensee evaluation of the degradation and corrective actions taken. Reference 2 is the NRC staff's Safety Evaluation related to the 1984 degradation of the Intake Structure. The evaluation was conditioned upon the licensee commitment to develop a surveillance program that would be implemented during the subsequent outages. Reference 3 describes the surveillance program.

The first and the second surveillances were performed during the Cycles 9 and 10 refueling outages in 1986 and 1988, respectively. The results of these surveillances were reported to NRC in References 4 and 5. A review of these reports indicated the continuing nature of the Intake Structure degradation. Because of these indications, the staff decided to perform an in-depth review of the degradation and corrective actions.

The following evaluation addresses the ability of the Intake Structure, together with its degradation and the corrective actions, to perform its safety function under the postulated seismic loadings. The NRC staff has reviewed the above information with its consultant, Atkinson-Noland and Associates, Inc.

2.0 EVALUATION

The San Onofre, Unit 1 (SONGS-1) Intake Structure is a reinforced concrete structure which provides transition between the pipes that collect and discharge seawater required for condenser cooling and the safety related salt water cooling system. The top slab of the structure also supports the salt water cooling pumps. The safety function of the structure is to allow adequate flow of water and to support the safety related pumps under the postulated loadings.

2.1 Design Basis

The Intake Structure is a completely embedded structure. During the Seismic Reevaluation Program (Reference 7), the structure was reevaluated for a Design Basis Earthquake (DBE) horizontal ground acceleration of 0.67g and a vertical ground acceleration of 0.44g using Housner Spectra. As it is a low height, long embedded structure, it was presumed to closely track the free-field ground motion. The controlling loading conditions applied to the peripheral walls and the base slab are due to normal loadings associated with a DBE event including corresponding dynamic earth pressures and hydrodynamic loads. The staff finds the design basis used by the licensee acceptable.

2.2 Current Condition of the Structure

A regular surveillance program has been established to monitor the condition of the structure and provide a basis for future repair recommendations. Inspection of the seawater intake structure would take place during refueling outages, or once every 18 months. Results from the surveillance program indicate that corrosion of the reinforcing steel is a continuing process, and is resulting in ongoing degradation of structural capacity. The scheduled inspection in 1986 [4] discovered continued staining of the concrete surface, but little severe deterioration. Additional corrosion and delaminations were discovered in 1988 [5] in a crossover section of the north wall of the structure, resulting in repairs being made to this area.

The third surveillance in 1990 [6] noted random corrosion and little change from past inspections, with a general trend towards an increase in the number and size of discolorations. The general conclusion is that corrosion is continuing, but there was one area in particular which exhibited a marked increase in corrosion related deterioration. Damage to an area of the MOV 11 gate structure in the form of extensive rust spotting and concrete spalls was observed, and repairs to these areas may be required in the future. Observation notes from this surveillance indicate that there are large amounts of corrosion byproducts in some areas, which form "protrusions" on the surface of the concrete. Some of these protrusions have been described as being as large as 3 inches in diameter and 1 inch in height, suggesting that even though damage to the concrete is not visible, a significant portion of the steel has corroded. During future inspections, areas where large protrusions are noted need to be investigated further to determine the condition of the remaining steel and also if repairs are needed in these areas.

2.3 Assessment of Damaged Areas and Corrective Actions

The Intake Structure was reanalyzed to determine the steel percentage necessary to provide the required moment capacity, and the amount of damage which could be tolerated before repairs could be needed. The reanalysis considered the design basis earthquake, concurrent soil pressures (active static and dynamic and passive dynamic and surcharge) and hydraulic pressures (static and dynamic). The acceptance criteria used were the same as those used during the seismic reevaluation program. The analysis determined that in some areas, the unreinforced moment capacity of concrete was sufficient to resist the design basis loads. Other areas were identified where a certain percentage (e.g., 50%, 75%) of the original areas of reinforcement was required to resist the design basis loads.

The licensee's method of assessing the damaged areas is based on the following assumptions.

1. The outside surfaces (backfill side) of the exterior walls are not damaged.

2. Any areas of corroded rebars and/or delaminated concrete are considered as not available to resist the loads.

These assumptions are acceptable for this evaluation. Assumption 1 is discussed further in Sections 2.4 and 2.5. The methods of identifying the damaged areas and computing the load carrying capacities are acceptable.

For those areas where the unreinforced moment capacity was insufficient to resist the postulated loads, the structural capacity was increased by bolting a series of plates (Figure 3) to the surface of the concrete. The strap plates were conservatively designed to carry the tensile stresses that would otherwise be carried by the corroded reinforcing bars in areas in positive moment (tensile stresses on the inside face). In the areas of negative moment, the outer reinforcing steel was assumed to be totally intact to carry tensile stresses at the exterior face (backfill side). Design of the strap plates and anchor bolts followed AISC and ACI method for composite steel-concrete design, which relies on shear-friction to transfer stresses from the concrete to the strap plates. During the review process, the staff had questioned the licensee practice of not monitoring the bolt-tension which is critical in ensuring shear friction mechanism in transferring stresses. The licensee response included the AISC-ACI practice as well as the fact that failure strength of the connection would be limited by ultimate shear capacity of the anchor bolts or failure of concrete in front of the anchor bolt if slip of the plates were to occur. The staff accepts the licensee explanation, but notes that any further delamination (i.e., from presently assumed 3 inches) of concrete will require reevaluation.

2.4 Prediction of Chloride Intrusion

As indicated in Section 2.3, the licensee has assumed that the outside faces (backfill side) of exterior walls are undamaged. To support the assumption, the licensee provided the results of chloride content found from the core samples taken in 1984 as shown in Figure 4. The figure indicates an average of 0.3% chloride content by weight of cement at the outside surface and about 0.08% at the outside rebar depth. The chloride content of 0.08% can be considered as lower than the threshold values established by ACI 201 (Ref. 9) and ACI (Ref. 10) 349 which are 0.1% and 0.15% respectively.

The figure indicates that in 1991, the chloride content at the outside rebar depth can be as high as 0.16% which is higher than the threshold limits established in References 9 and 10. In year 2001, the chloride content can reach as much as 0.19% of the weight of the cement. At the chloride content level predicted for year 1991, the passivating layer of iron oxide has a likelihood of being disrupted. The disruption will lower the pH value of the concrete pore water which could, with time, dissolve the passivating layer. Areas where passivating layer is thus broken are the areas where the electrochemical potential becomes more negative and act as anode supplying electrons to the electrochemical cell. These are very slow processes, particularly when the availability of oxygen (in backfill soil) is limited. For this reason, the staff considers the structure acceptable at present.

2.5 Inservice Surveillance Progam

The staff's safety evaluation dated April 24, 1985 (Ref. 2) required that the licensee develop a surveillance program (SP). The purpose of the SP would be to detect the areas of degradation, reevaluate the condition of the structure, and ensure that the structure after the appropriate corrective measures could withstand the postulated loads. The following is an outline of the SP (Ref. 3).

1. Surveillance of the Intake Structure shall be performed at each refueling outage. The core drilling (to detect indepth condition) shall be done at every fourth refueling outage. The interval may be revised as a result of the evaluation of the prior surveillance.

2. Divers are to be used for the underwater visual examination of the Intake Structure. They will videotape and photograph the areas of suspected degradations.

3. The areas to be examined will consist of the walls, ceilings, floors, internal concrete beams and gate slots. The areas to be examined in detail may be changed based on engineering reviews of the prior surveillances.

4. A minimum of three core samples shall be taken during every fourth outage (cycle 12 outage in 1992 will be the first such outage). Samples shall be taken from the unrepaired portions of the Structure. They will be examined for internal delaminations and rebar corrosion, and will be tested for indepth chloride content.

5. All the data accumulated from visual examinations and core sampling shall be assessed. The areas of degradations shall be subjected to engineering evaluations, and determination shall be made as to the required corrective actions.

6. Implementation of the required corrective actions.

2.6 Future Surveillances

As described in 2.5, the licensee has committed (Reference 3) to perform the surveillances at each outage or approximately at 18 months time interval. During the meeting at site and subsequent teleconferences the staff has made several recommendations to the licensee regarding future surveillances. The licensee has committed to incorporate these recommendations in the future surveillances. However, the following is a one place summary of such recommendations and corresponding explanations.

1. Cores will be taken during the next surveillance to investigate the internal condition of the concrete. Locations where cores are taken should be chosen carefully to provide a representative sample of damage and corrosion. Nearly all of the cores drilled in 1984 were through the walls near the base slab, presumably because of easy access to these areas. Corrosion damage is more prevalent higher on the walls, closer to the roof slab, and it is recommended that cores be taken in these areas as well. Visual and petrographic examination of cores should concentrate on determining the maximum depth of cracks and delaminations because, as shown previously, the maximum crack depth will affect the unreinforced moment capacity of the structure.

2. The condition of the external reinforcement should be checked with core samples. Chloride measurements should continue to be taken to determine the possibility for corrosion of the outside reinforcing bars. Core sampling is currently the only viable means to investigate the condition of the outer reinforcement, and the current sample interval of 6 years is insufficient to adequately evaluate the condition, particularly when the chloride content near outside rebars are indicated to be near the established threshold. It is recommended that core samples be taken every other surveillance, or once every 3 years.

3. Other factors which affect the rate of corrosion need to be investigated as well. The chloride concentration required for corrosion initiation is highly sensitive to pH of the environment surrounding the reinforcement; this factor should be investigated, especially in the vicinity of the exterior reinforcing bars. The amount of dissolved oxygen generally controls the rate of corrosion, and it would be useful to know the quantity of oxygen available both in the seawater and the ground water. Determining the pH and dissolve oxygen are simple tests and could be incorporated into the existing surveillance program. The licensee should attempt to determine these parameters during core sampling.

4. Visual examinations have been useful and should be continued to identify changes in corrosion activity and concrete damage. However, the areas added during 1990 surveillance and the areas where large protrusions are noted (see Section 2.2) should be continued to be inspected during future surveillances.

5. Repairs made should be thoroughly inspected to ascertain that the concrete on which the strap plates are attached is sound and able to transfer loads to the plates.

6. The NDE method (Electrochemical Corrosion Detection Method) that the licensee has found as a workable method should be further evaluated to assess if it could be effectively used to determine the in-depth condition of the concrete.

3.0 CONCLUSION

Based on the review of the degraded intake structure as evidenced by the surveillances in 1986, 1988 and 1990, examination of the accessible portions of the structure, review of the corrective actions taken by the licensee and pertinent design calculations; the staff concludes that the structure is able to meet its safety function under the postulated loadings, at least, until the next surveillance scheduled during the cycle 12 outage. Also, the staff review of the licensee's surveillance procedure provides an adequate assurance that conditions adverse to safety will be identified and the required corrective measures will be implemented prior to plant start-up following a surveillance.

However, as the acceptance criteria for a structure, that has been repaired following the identification of degradation (e.g., rebar corrosion, spalling and cracking of concrete) have to rely on the effectiveness of the repairs performed, the evaluation of the results of each of the future surveillances will set its own benchmark for acceptability and safety of the structure. In order to make the future surveillances effective in establishing the serviceability of the structure, the licensee has committed to incorporate the staff recommendations (as stated in Section 2.6) in the future surveillance procedures.

Principal Contributor: H. Ashar

Date:

Attachments:
1. Figures 1-5
2. Atkinson-Noland Report entitled "Chloride Concentration Diffusion Analysis"

4.0 REFERENCES

1. A letter from M. O. Medford (SCE) to W. A. Paulson (NRC), "Intake Structure Degradation and Repair," dated October 18, 1984.

2. A letter from J. A. Zwolinski (NRC) to K. A. Perkins (SCE), "Staff's Safety Evaluation," dated April 24, 1985.

3. A letter from M. O. Medford (SCE) to J. A. Zwolinski (NRC), "Surveillance Guidelines," dated October 4, 1985.

4. "Report of First Surveillance for SONGS-1 Intake Structure (Cycle IX)," prepared for Southern California Edison by Bechtel Power Corporation, May 1986.

5. "Second Surveillance Report for SONGS-1 Intake Structure (Cycle X Refueling Outage)," by Southern California Edison Company, March 1989.

6. "Third Surveillance Report for SONGS Unit 1 Intake Structure (Cycle XI Refueling Outage), by Southern California Edison, October 1990.

7. Safety Evaluation Report on Long Term Seismic Reevaluation Program attached to the letter from T. M. Novak (NRC) to K. P. Baskin (SCE) dated July 11, 1986.

West, R., Hime, W., "Chloride Profiles in Salty Concrete," Materials Performance, National Association of Corrosion Engineers, July 1985.

9. ACI Committee 201, "Guide to Durable Concrete," (ACI 201-2R-77) (Reapproved 1982), American Concrete Institute, Detroit.

10. ACI Committee 349, "Code Requirements for Nuclear Safety Related Concrete Structures," (ACI 349-85), American Concrete Institute, Detroit.

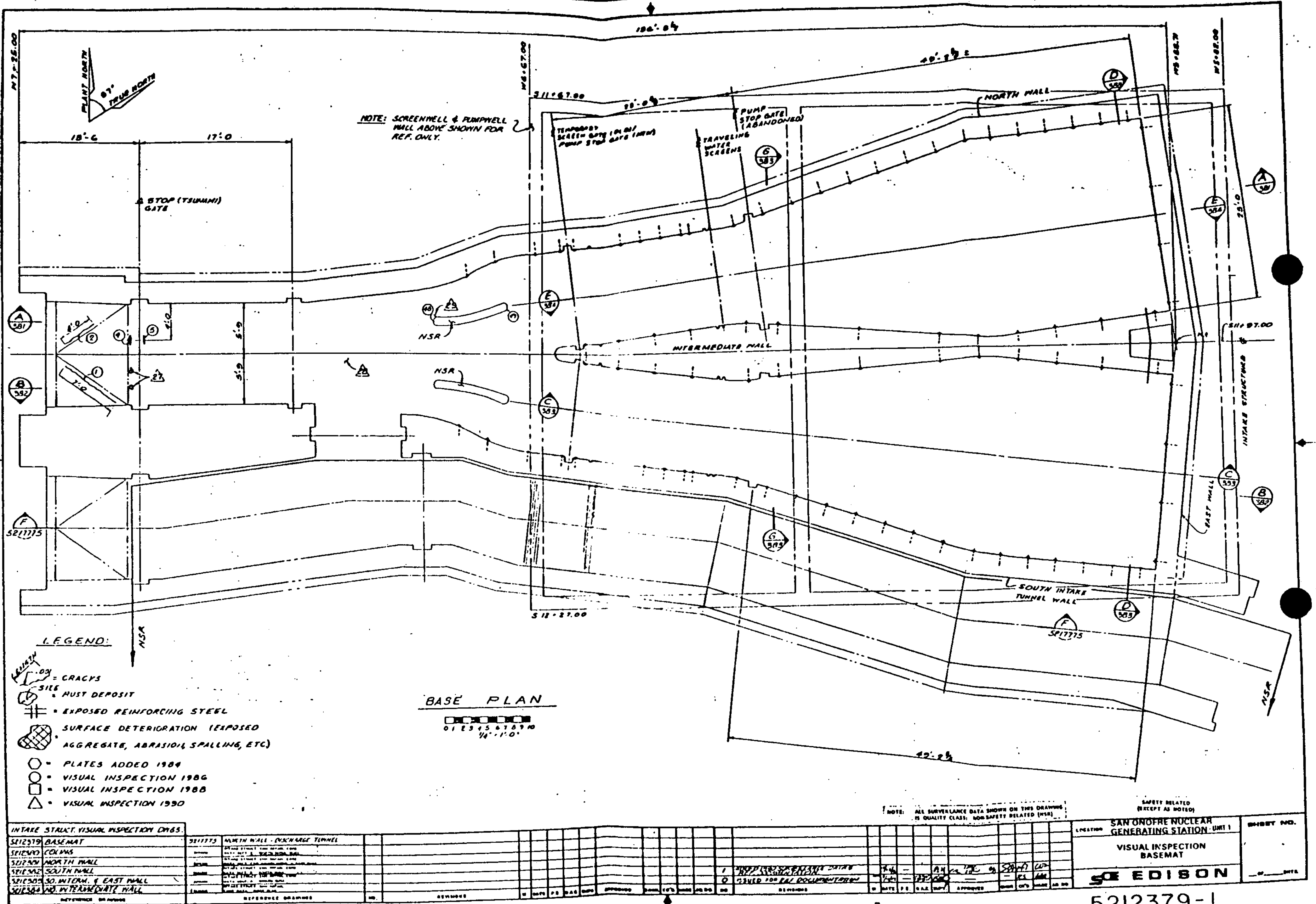

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FIGURE 1

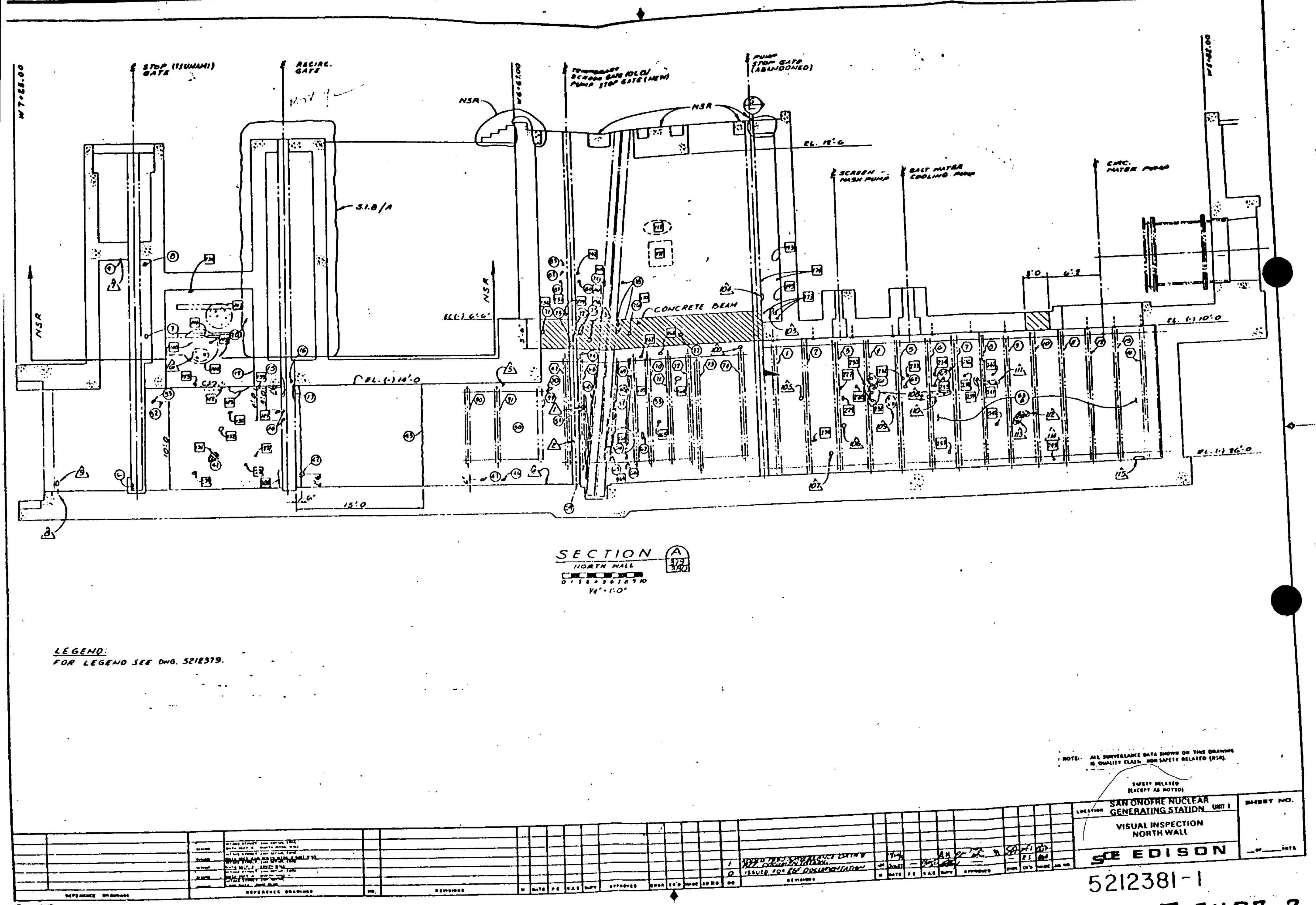
STOP (TSUNAMI) GATE
RECIRC. GATE
PUMP STOP GATE (ABANDONED)
NSR
S.I.B/A
EL. 19'-6
SCREEN WASH PUMP
SALT WATER COOLING PUMP
CIRC. WATER PUMP
CONCRETE BEAM
EL. (-) 6'-6"
EL. (-) 10'-0
EL. (-) 14'-0
EL. (-) 26'-0
15'-0
SECTION A
NORTH WALL
1/4" = 1'-0"
LEGEND:
FOR LEGEND SEE DWG. 5212379.
NOTE: ALL SURVEILLANCE DATA SHOWN ON THIS DRAWING IS QUALITY CLASS NON SAFETY RELATED (NSR).
SAFETY RELATED (EXCEPT AS NOTED)
SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1
VISUAL INSPECTION NORTH WALL
SCE EDISON
ISSUED FOR REF DOCUMENTATION
REVISIONS
REFERENCE DRAWINGS
5212381-1

FIGURE 2

Southern California Edison Company
Songs 1, 2 & 3

INTERIM DESIGN CHANGE NOTICE (IDCN)/DESIGN CHANGE NOTICE (DCN)
SUPPLEMENTAL PAGE

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THIS CHANGE IS AN ADDITION ONLY. 'BEFORE' CONDITION DOES NOT EXIST.

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Date 6-15-88 Page 5 of 9

By D SCHMIDT

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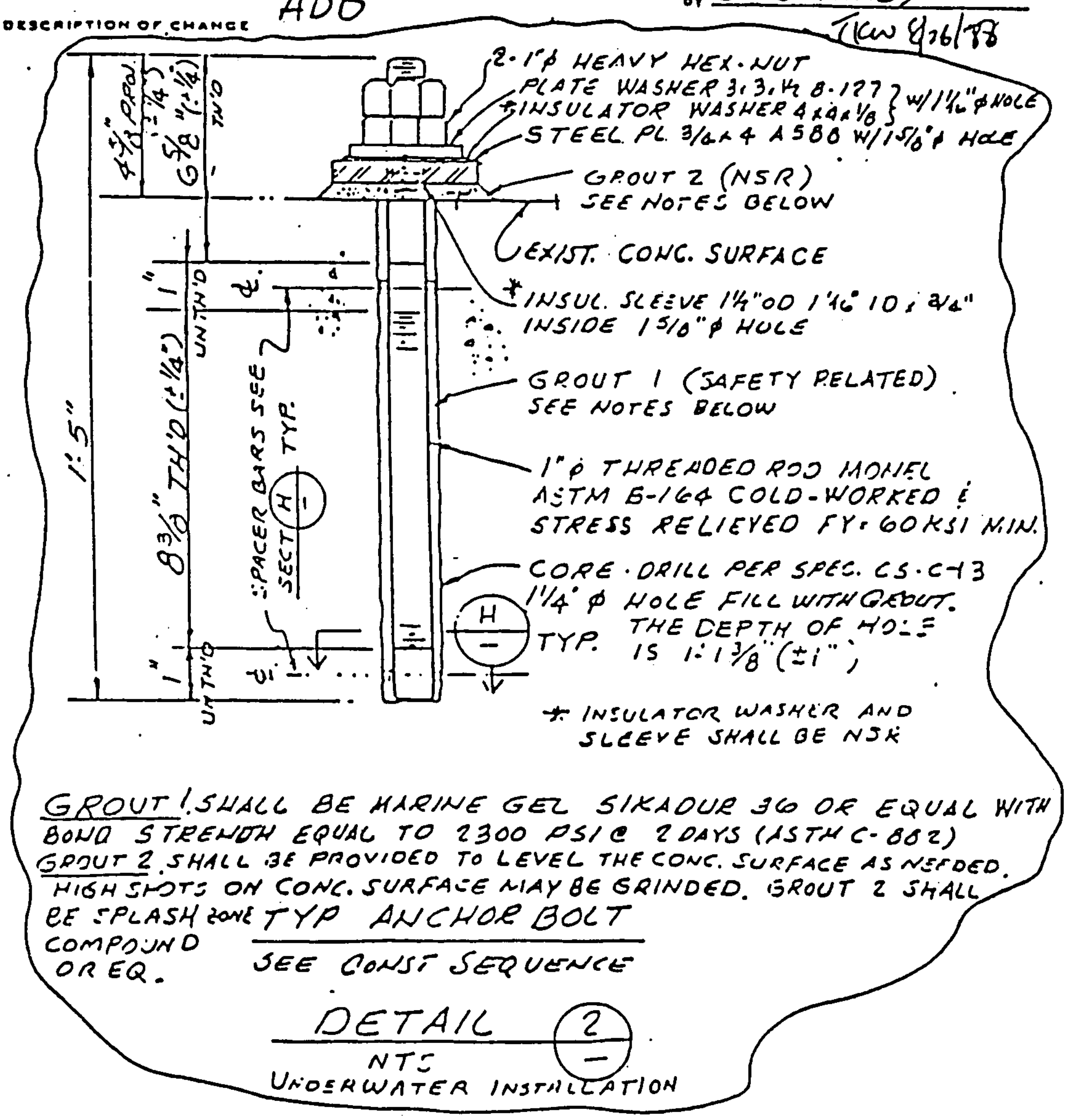

Figure 3. Anchor bolt detail.

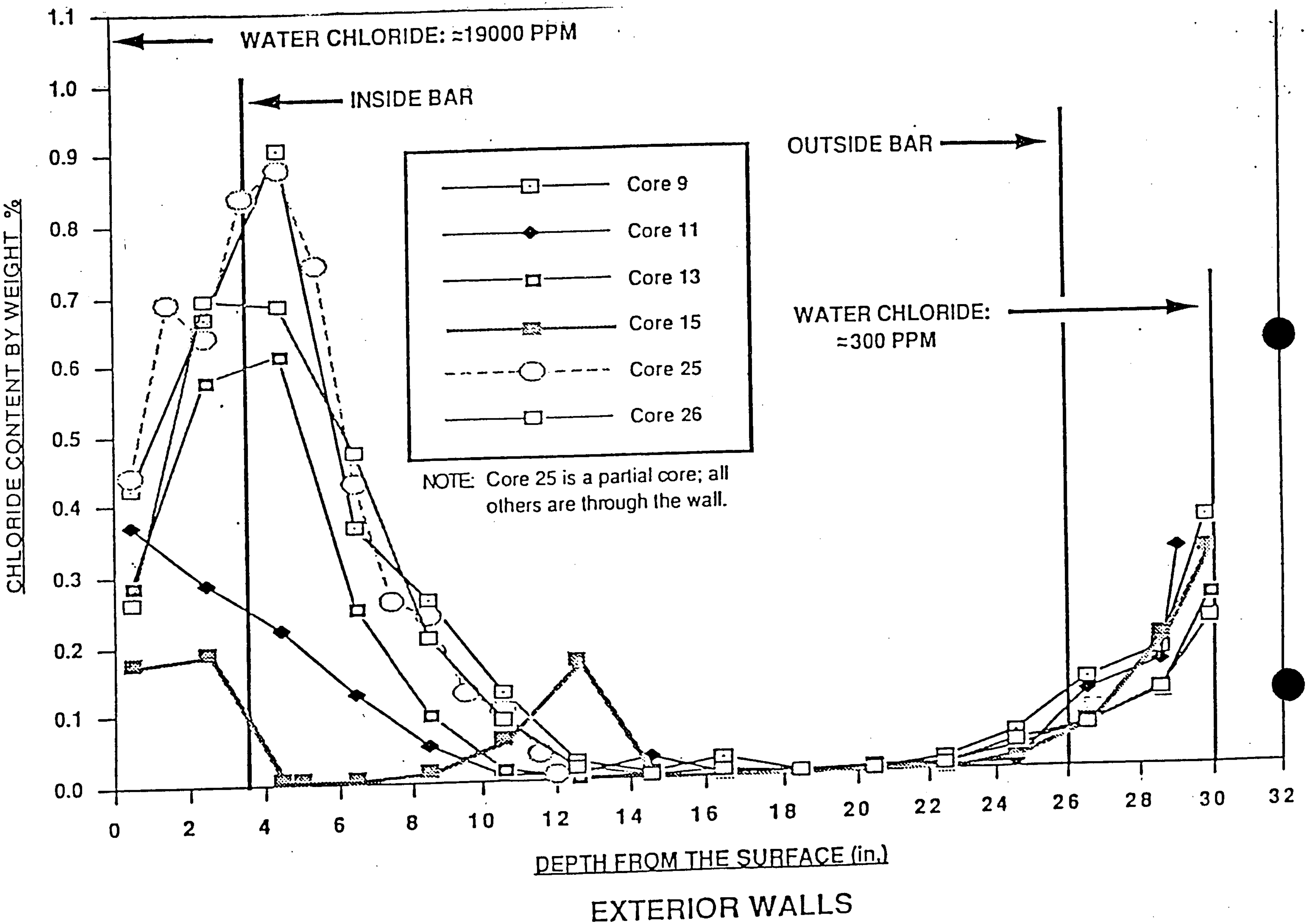

Figure 4. Chloride distribution curve for exterior wall.

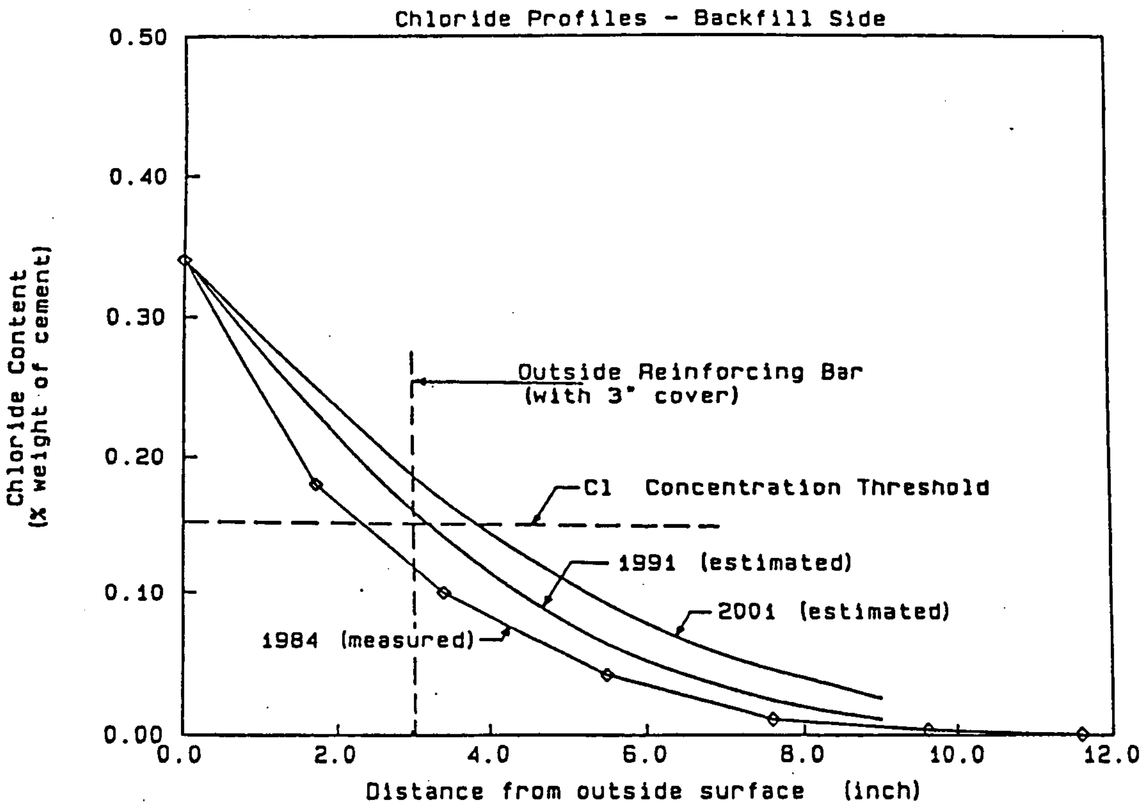

Figure 5. Estimated chloride ion concentrations, based upon diffusion analysis.

SAN ONOFRE NUCLEAR GENERATING STATION
UNIT 1 INTAKE STRUCTURE EVALUATION

CHLORIDE CONCENTRATION DIFFUSION ANALYSIS

April 1, 1991

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APPENDIX A

CHLORIDE CONCENTRATION DIFFUSION ANALYSIS

Chloride concentration profiles obtained from core samples taken in 1984 give important information about the amount of chloride available to the corrosion process in 1984. The condition of the outer reinforcing bars were examined with these core samples and no corrosion or deterioration at this location was noted at the time. Chloride concentrations will increase over time, however, and it is possible to estimate future chloride concentrations even if no additional core data is present.

There are several means by which chloride ions are able to penetrate into the concrete, including permeation into small voids, through an interconnected pore structure, or into cracks. In the absence of these pathways, chloride ions are still able to enter the concrete through the process of diffusion. This analysis models the ions' tendency to diffuse into the concrete, in an attempt to reach equilibrium with the surrounding environment. Transfer of ions from areas of high concentration (seawater) to low concentration (concrete) follows Fick's second law of diffusion, which is a second order partial differential equation of the parabolic type:

$$\frac{\delta [Cl^-]}{\delta t} = D_e \frac{\delta^2 [Cl^-]}{\delta x^2}$$

in which

- Cl = chloride concentration
- t = time (seconds)
- x = distance from surface (inches)
- D_e = diffusion coefficient (in^2/sec)

Solution of this equation and development of formulae describing the diffusion process is out of the scope of this paper, and the reader is referred to West [8], who provides an excellent description of the analytical method. The analysis was conducted using this method with the assistance of MathCad, a commercially available mathematical solver; screen printouts of the analysis are included on the following pages.

Several assumptions have been made in the process of this analysis; these assumptions attempted to model the average conditions and by no means represent the worst possible case which could be present. These assumptions are described in the following paragraphs.

An original chloride concentration curve is needed to estimate the average diffusion coefficient through the concrete. This curve was obtained by averaging the three curves with the highest chloride concentrations given for the backfill side of the structure: from cores #9, #11, and #15. The original curve is shown on page 3. These cores were taken in 1984, which represents approximately 16 years of service. Calculations shown on page 3 have determined the average diffusion coefficient to be 1.189×10^{-8} in^2/sec, which is within the typical range for concrete structures with sound concrete.

Once the average diffusion coefficient has been determined, it is possible to estimate chloride concentrations at any point within the structure, for any time interval. Future chloride concentrations have been estimated using the conservative assumption that the surface concentration remains constant, and that the concrete remains free of cracks. Calculations for current chloride concentrations (in the year 1991) and for future concentrations (for the year 2001) are shown on pages 4 and 5; estimated concentration curves have been plotted previously in Figure 5. If the reinforcement cover is assumed to be three inches, as indicated by cores taken in 1984, then chloride concentrations at this point can be estimated. Chloride ion concentration at the reinforcing bar in 1991 is estimated to be 0.16% and in 2001 is nearly 0.19%. Typical threshold concentrations are thought to be between 0.1% and 0.2%, hence this analysis shows there to be a definite possibility for corrosion of the outer reinforcing bars.

ıCAD [redacted] MathSoft Inc.

ORIDE PROFILES FOR BACKFILL SIDE, 1984 READINGS

$= 1 \,..7$

$- 2 \,..7$

$d_i :=$, $Cl_i :=$, $u_i := \dfrac{Cl_1 - Cl_i}{Cl_1 - Cl_7}$

d_i (Depth)	Cl_i (Measured Cl⁻ %)	u_i
0.0	0.34	0
1.7	0.18	0.471
3.4	0.10	0.706
5.5	0.04	0.882
7.6	0.01	0.971
9.6	0.003	0.991
11.6	0.0	1

$= 0.1$ $\quad q_1 := 0.0$

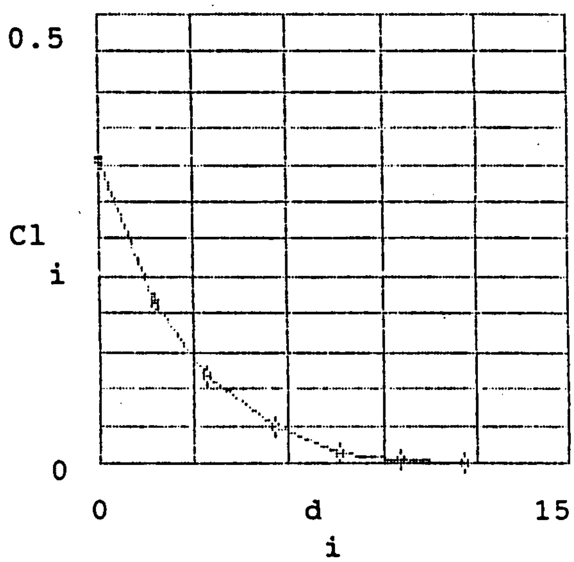
$:= \mathrm{root}\left[\mathrm{erf}(w) - u_k , w\right]$

error function

$arg_k := \dfrac{d_k}{q_k}$

$j := 3 \,..7$

q_i	arg_i
0	0
0.445	3.823
0.742	4.583
1.106	4.972
1.536	4.949
1.837	5.227
2.429	4.776



$an := \displaystyle\sum_j \frac{arg_j}{5}$ $\quad mean = 4.901$

$:= 5.05 \cdot 10^{8}$ ⇒ 1984
time to 1984 (seconds)

$Dc := \dfrac{mean^2}{4 \cdot t}$ $\quad$ $\underline{Dc = 1.189 \cdot 10^{-8}}$: in²/sec

Diffusion Coefficient

CAD [redacted] MathSoft Inc.
KFILL SIDE - CHLORIDE PROFILE AT 1991
E = 23 YEARS

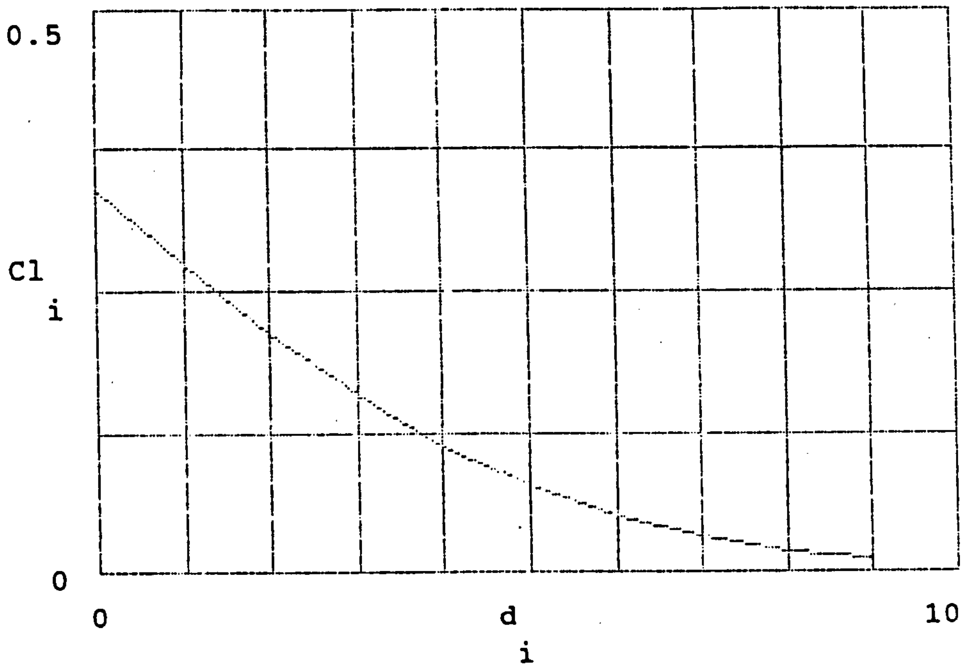
1 ..10

$Dc := 1.189 \cdot 10^{-8}$ $\quad t := 7.25 \cdot 10^{8}$ $\quad Co := 0.34$

Diffussion Coefficient Determined Previously (in²/sec) — time to 1991 (sec) — Cl⁻ Concentration at Surface (% by wt of Cement)

$$Cl_i := Co \cdot \left[1 - \operatorname{erf}\left[\frac{d_i}{2 \cdot \sqrt{Dc \cdot t}}\right]\right]$$

d_i	Cl_i
0	0.34
1	0.275
2	0.214
3	0.16
4	0.114
5	0.078
6	0.05
7	0.031
8	0.018
9	0.01
Depth (inch)	Calculated Cl⁻ %



CAD [redacted] MathSoft Inc.

KFILL SIDE - CHLORIDE PROFILE AT 2001

E = 33 YEARS

$1\,..10$

$Dc := 1.189\cdot 10^{-8}$ Diffusion Coefficient Determined Previously (in^2/sec)

$t := 10.41\cdot 10^{8}$ time to 2001 (sec)

$Co := 0.34$ Cl^- Concentration at Surface (% by wt of Cement)

$$Cl_i := Co\cdot\left(1-\mathrm{erf}\left(\frac{d_i}{2\cdot\sqrt{Dc\cdot t}}\right)\right)$$

d_i	Cl_i
0	0.34
1	0.286
2	0.234
3	0.186
4	0.143
5	0.107
6	0.077
7	0.054
8	0.037
9	0.024
Depth (inch)	Calculated Cl^- (%)