ACCESSION NBR:7903120311   DOC.DATE: 79/02/28   NOTARIZED: NO   DOCKET #
 FACIL:50-361 SAN ONOFRE NUCLEAR STATION, UNIT 2, SOUTHERN CALIFORN 05000361
       50-362 SAN ONOFRE NUCLEAR STATION, UNIT 3, SOUTHERN CALIFORN 05000362
  AUTH.NAME   AUTHOR AFFILIATION
BARBIN,K.P.   SOUTHERN CALIFORNIA EDISON CO.
  RECIP.NAME   RECIPIENT AFFILIATION
VASSALLO,D.B.   ASSISTANT DIRECTOR FOR LIGHT WATER REACTORS

SUBJECT: IN RESPONSE TO 790112 LTR,FORWARDS PARTIAL RESPONSE TO AUDIT
         REQUEST ITEMS & AUDIT REQUEST ITEM SUBMITTAL SCHEDULE.W/ENCL
         TEST REPTS RE TRIAL BATCHES W/PRELIMINARY CYLINDERS.

DISTRIBUTION CODE: ZZZZS COPIES RECEIVED:LTR _1_ ENCL _1_ SIZE:_55_____
            TITLE: * * * * * * * * * * SPECIAL DISTRIBUTI

NOTES:_____

RECIPIENT   COPIES   RECIPIENT   COPIES
ID CODE/NAME   LTTR ENCL   ID CODE/NAME   LTTR ENCL

LTR + encl   LTR
Reg File   J. Lee
NRC PDR   OELD
LPDR   F. Schave
H. Rood   J. Knight
R. Lipinski
S. Chan
D. Jeng

MAR 13 1979

MA 4
GD

TOTAL NUMBER OF COPIES REQUIRED: LTTR   0   ENCL   0

MEMORANDUM FOR: TERA Corp.

FROM: US NRC/TIDC/Distribution Services Branch

SUBJECT: Special Document Handling Requirements

☑ 1. Please use the following special distribution list for the attached document.

LTR & ENCL

REG FILE
. NRC PDR
LPDR
H ROOD
R LIPINK (struck through)
R LIPINSKI
S CHAN
D JENG

LTR

J LEE
OELD
F SCHAUE
J KNIGHT

☐ 2. The attached document requires the following special considerations:

☐ Do not send oversize enclosure to the NRC PDR.

☐ Only one oversize enclosure was received - please return for Regulatory File storage.

☐ Proprietary information - send affidavit only to the NRC PDR

☑ Other: (specify)

NO TERA CY

cc: DSB Files

Mike Collins
TIDC/DSB Authorized Signature

Southern California Edison Company

K. P. BASKIN
MANAGER, GENERATION ENGINEERING

February 28, 1979

TELEPHONE
213-572-1401

Director of Nuclear Reactor Regulation
Attention: Mr. D. B. Vassallo, Assistant Director
           Light Water Reactors, DPM
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Gentlemen:

Subject: Seismic and Structural Design Analysis Audit
          Docket Nos. 50-361 and 50-362
          San Onofre Nuclear Generating Station
          Units 2 and 3

Reference: (A) NRC Letter from H. Rood to the Southern California
                Edison Company of January 12, 1979

Enclosures: (1) Partial Response to Audit Request Items
            (2) Audit Request Item Submittal Schedule

      Enclosure (1) contains a partial response to the requests of
Reference (A) which was generated as a result of the Structural Audit of
December 4 through 8, 1978. Enclosure (2) contains a listing of the requests
of Reference (A) and the manner in which they will be dispositioned. Those
request items which involve calculations will be presented in meetings tenta-
tively scheduled for early March and mid-April. Engineers who are familiar
with the requested calculations and appropriate reference material will be
present at these meetings to respond to Staff questions and facilitate their
review.

      If you have any questions, please contact me.

Very truly yours,

Enclosures

7903120311

AUDIT REQUEST ITEM
SUBMITTAL SCHEDULE

Structure	Request Items	Disposition	
1. Offshore Circulation Water System	1, 2, 3, 6, 7 4, 5, 8, 9	Letter, Meeting,	2/79 3/79
2. Intake Structure and Box Conduit	2 1, 3, 4, 5, 6, 7	Letter, Meeting,	2/79 4/79
3. Containment	2, 3, 6, 10 5, 7, 11, 12, 14 15, 16, 17, 18 8, 9, 13	Letter Meeting, Meeting,	2/79 3/79 4/79
4. Polar Crane Supports	2 1	Meeting, Meeting,	3/79 4/79
5. Auxiliary Building	2, 3, 4 6 5	Letter, Meeting, Meeting,	2/79 3/79 4/79
6. Fuel Handling Building	2, 4 5 3, 6	Letter, Meeting, Meeting,	2/79 3/79 4/79
7. Spent Fuel Crane Supports	1, 2	Meeting,	3/79
8. Safety Equipment Building	1, 2, 4 5, 6	Letter, Meeting,	2/79 4/79

*The information requested in Item #1 for Containment, Item #1 for Auxiliary
Building, Item #1 for Fuel Handling Building, Item #3 for Safety Equipment
Building will be provided in the response to question 131.31 to be submitted
in Amendment 14 to the FSAR (early March).

NRC STRUCTURAL AUDIT REQUEST ITEMS

Item   Provide the basis for developing allowable bearing values for

each structure as presented in Table 3.8-15.

Response

Allowable bearing capacities for various structures were calculated

considering local shear as the failure criterion together with a factor

of safety of 3 using the following equation:

$$q_{all} = 1/3 \ [1.3CN_c\delta_c + 1/2BN_\gamma\delta_\gamma + \gamma \ D_fN_q\delta_q]$$

where N_c, N_γ and N_q represent bearing capacity factors for local shear, dependent

on the angle of internal friction, ϕ , for the supporting soil δ_c, γ_δ and δ_q

are shape factors dependent on the shape of the footing, C is the cohesive

strength of the soil, D_f is the depth of embedment and γ is the effective

unit weight of the soil. Values of the bearing capacity and shape factors

were based on a value for ϕ of 40° and on the representative shape of foun-

dations for various structures. Bearing capacity values thus calculated

are summarized in Table 1 (revised from Table 3.8-15 of the FSAR). The

values used for all loads including wind and seismic are shown in

parentheses and include a one-third increase over the static value.

Settlements due to structural dead and live loads were calculated using

vertical stiffnesses for each structure. These stiffnesses are presented

in Appendix 3.7C of the FSAR. Strain compatibility was maintained in the

selection of modulus values and calculation of vertical displacements.

These displacements represent elastic settlements below structures and

would take place during construction and are summarized in Table 1. As shown in the table, the elastic settlements are less than 1 inch for all structures. A maximum of 0.75 inches of elastic settlement is indicated for the Fuel Handling Building. For most other structures, the settlement is on the order of 1/4 inch or less. Dynamic displacements were not considered for this evaluation because they were calculated during dynamic soil-structure interaction analysis for each structure.

TABLE-1

SUMMARY OF ACTUAL AND ALLOWABLE FOUNDATION BEARING PRESSURES, SETTLEMENTS,
AND FACTORS OF SAFETY AGAINST OVERTURNING FOR SEISMIC CATEGORY I STRUCTURES

Structure	Foundation Medium	Foundation Bearing Pressure (D+L+Seismic) (k/ft^2)	Foundation Allowable Bearing Pressure (k/ft^2)	Estimated Short-Term Settlements of Structure (in.)	Estimated Long-Term Settlements of Structure (in.)	Minimum Factor of Safety (1.5) Against Overturning	Remarks
Containment	Undisturbed natural San Mateo sand	18	54(72)	0.25	0.3	40	
Auxiliary Building	Undisturbed natural San Mateo sand	15	68(88)	0.30	See paragraph 2.5.4.10, FSAR	150	Foundation allowable value is lower bound dictated by shear capacity of soil, neglecting actual horizontal extent of basemats.
Fuel Handling Building	Undisturbed natural San Mateo sand	21	33(44)	0.75	See paragraph 2.5.4.10, FSAR	10	Foundation allowable value is lower bound dictated by shear capacity of soil, neglecting actual horizontal extent of basemats.
Safety Equipment Building	Undisturbed natural San Mateo sand	(DBE) 9.022 (OBE) 7.326	36(48)	< 0.25	0.40	Not applicable (See Remarks)	More than 2/3 of structure is embedded in soil
Intake Structure	Undisturbed natural San Mateo sand	–	-	–	–	Not applicable	Will be provided later
Electrical and Piping Gallery Structure	Undisturbed natural San Mateo sand	(DBE) 11.154 (OBE) 10.497	36(48)	< 0.25	0.40	Not applicable (See Remarks)	Approximately 2/3 of structure is embedded

NOTE--Bearing capacity values in parenthesis are for seismic loading and
include 1/3 increase over bearing capacity for static loading.

Item   Provide justification for the value of correction factor C_2 used

for calculation of horizontal spring constants.

Response

Stiffness of footings in various modes of motion (i.e., vertical and hori-

zontal translations and rotations about horizontal and vertical axes)

increase when the footings are embedded in soil as compared to stiffness

with no embedment. This increase is usually expressed as a ratio of stiff-

ness of embedded footing to that of footing with no embedment and has

been designated here as the C_2 factor, i.e.,

$$C_2 = \frac{k_{embedded}}{k_{unembedded}}$$

Determination of C_2 factor has been made by various investigators using

different procedures, both theoretical analyses and experimental tests.

Kaldjian (1968) used axisymmetric finite-element analysis to develop C_2

factors for various embedments, expressed in terms of the ratio of depth

embedment, h, to radius of footing, r. Stokoe (1972) conducted small scale

model tests on circular footings with different embedments and developed

relationships between C_2 and d/r_o, where d was the depth of embedment and

r_o was the radius of the footing. Footings were excited in horizontal and

vertical translation and rotation modes. Novak and Beredugo (1972)

utilized an analytical approach to investigate embedment effects on stiffness values. They extended the analysis for vibration of footings resting on elastic half space by adding the resistance developed at the sides of the embedded footing to that at the base of the footing. Dominguez (1978) used the Boundary Element Method to determine stiffnesses of footings with and without embedment. His analysis included rectangular footings with different length to width ratios and embedments.

For the SONGS site large scale model tests were conducted by WCC (Woodward-Clyde Consultants) by setting five concrete slabs of different size, shape and embedment configurations into transient motion and measuring the response of these slabs. The slabs ranged from 4 to 10 feet in diameter and 2 to 5 feet in thickness. Details of these tests and their results are described in Appendix 3.7C of the FSAR. Most of the results obtained from these tests were for an embedment ratio of approximately 1. C_2 factors for other embedment ratios were developed by utilizing the form of C_2 versus h/r relationship developed by Kaldjian by normalizing that form to the result of the field tests described above. C_2 factors thus developed for vertical translation are presented in Figure 1. The figure also shows C_2 factors developed by Stokoe, Novak and Beredugo, and Dominguez, also normalized to the results of the field tests described above. In addition, the figure also shows the ±30% variation in soil stiffness used in the actual analysis for soil-foundation interaction studies for the site.

It is noted that the C_2 factors developed by the various investigators as described above, fall within the range used for the actual analysis.

Similar results for other modes of vibration are summarized in Figure 2. Based on the information presented above, the C_2 factors used in the development of stiffness parameters for various structures at the site represent reasonable values compatible with results obtained by various investigators.

VERTICAL MODE

NOTE: Shape of curve estimated from previous work by Kaldjian (1968)

EMBEDMENT 360°

C_2

$\left(\dfrac{K'_v}{K_v}\right)$

h/r
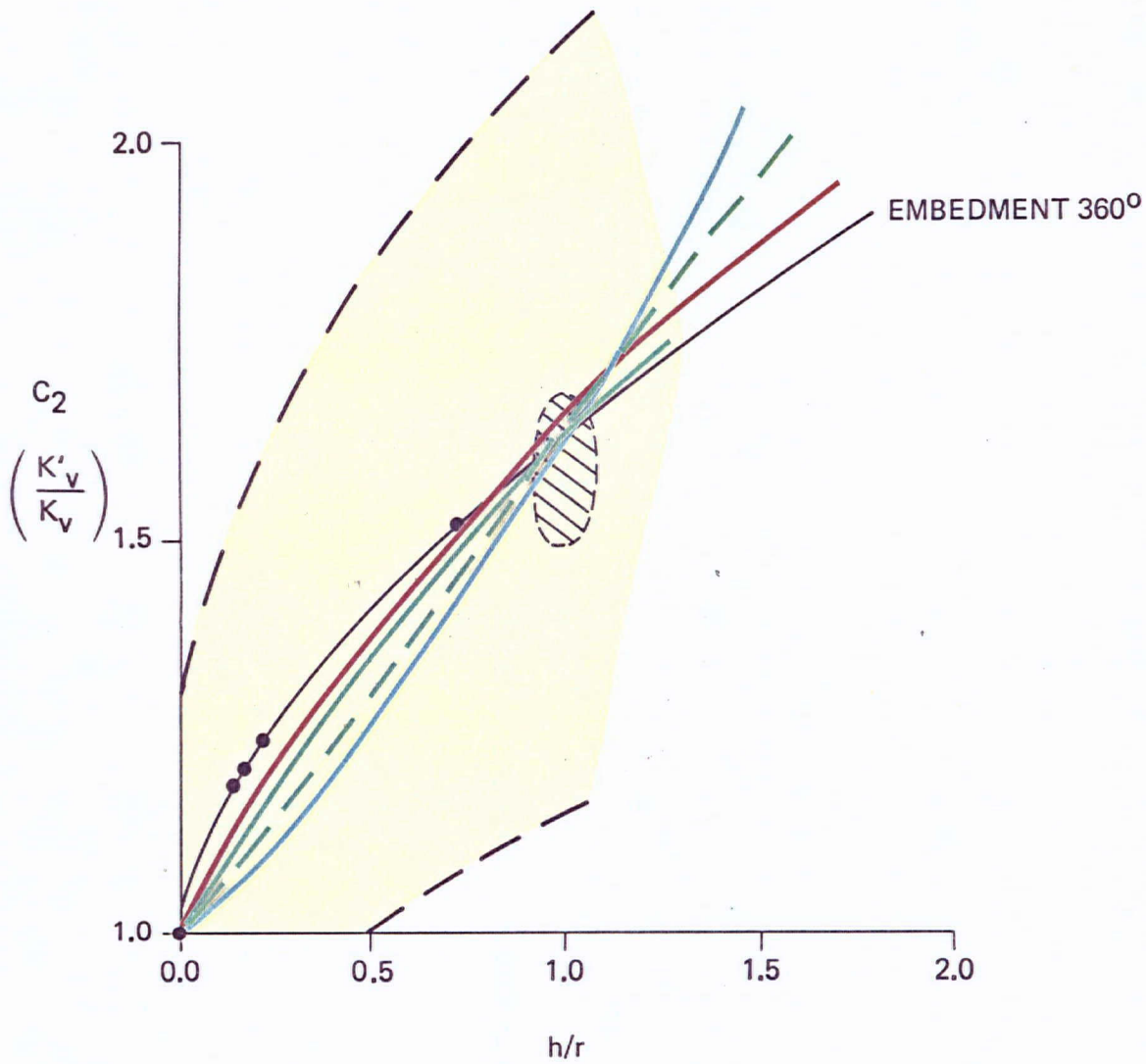
K' = EMBEDDED CASE
K = NON-EMBEDDED CASE
C_2 = EMBEDMENT CORRECTION FACTOR

= RANGE OF DATA POINTS

STOKOE 1972
NOVAK & BEREDUGO 1972

SONGS TESTS ◄── BEST EST.

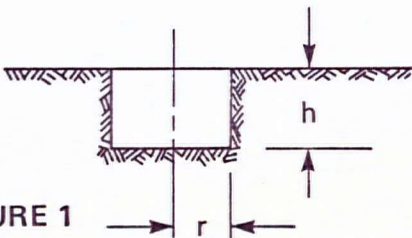VALUES USED ●
±30%

DOMINGUEZ 1978
SQUARE
RECTANGULAR

FIGURE 1
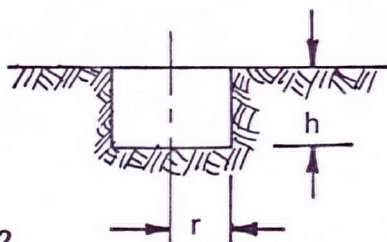C_2 FACTORS FOR VERTICAL TRANSLATION MODE
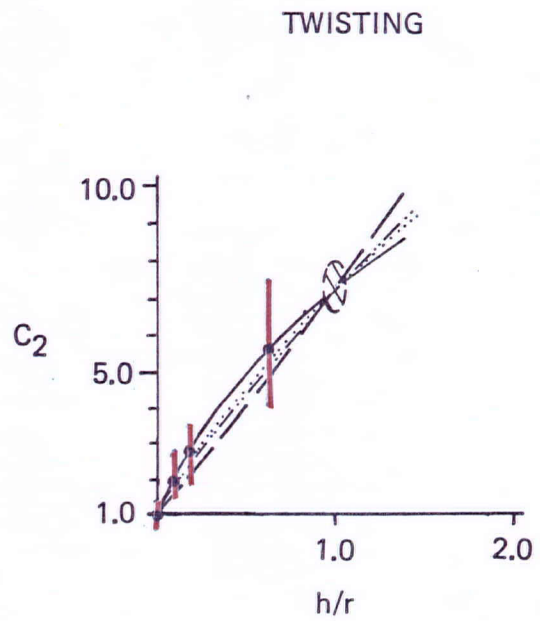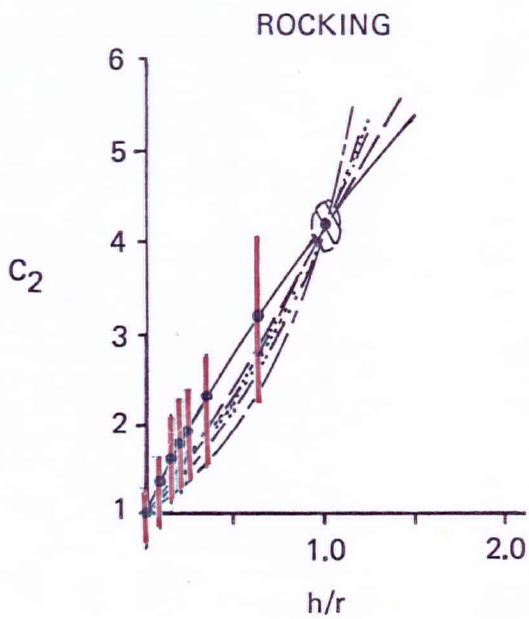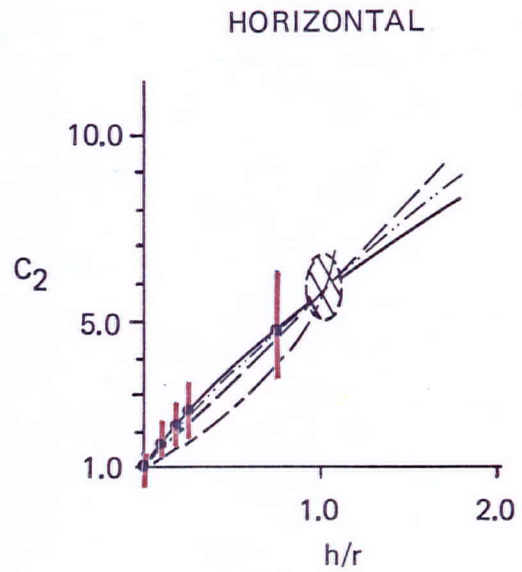
VERTICAL

HORIZONTAL

ROCKING
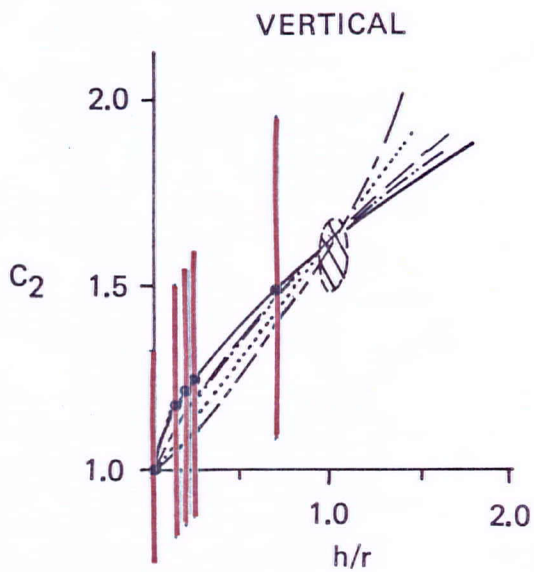
TWISTING

FIGURE 2

C_2 FACTORS FOR TRANSLATION
AND ROCKING MODES

RANGE OF DATA POINTS

VALUES AND RANGE

STOKOE 1972 — — —

NOVAK & BEREDUGO 1972 — – —

DOMINGUEZ 1978

SQUARE ....

RECTANGULAR — ·· —

NRC STRUCTURAL AUDIT REQUEST ITEMS

OFFSHORE CIRCULATING WATER SYSTEM

Item #1   Provide more details of modeling techniques used in the CIDP

analysis including boundary condition assumptions. Describe

how the method accounts for varying stiffnesses of different

structures.

Response

Critical Instantaneous Displacement Profile (CIDP) represents the most

severe distortion of the structure required to accommodate horizontally

propagating earthquake waves. The severity is related to the maximum

bending stresses induced due to relative displacements or rotations

between adjacent segments of the structure.

The following steps were involved in determining the Critical Instantaneous

Displacement Profile (CIDP).

1.   A section representative of the overall stiffness of the structure

was selected and cast into a finite-element model representing

soil-structure system. A two-dimensional plane strain dynamic

finite-element program developed at the University of California,

Berkeley, was used for this analysis (See FSAR App. 3.7C App. H).

Because the computer program utilizes rigid boundaries for the

model, these boundaries were located at significant distance away

from the structural elements to minimize the effects of boundary-

reflections. Dynamic properties of various elements were assigned

on the basis of available information for the soil and structural elements. Dynamic properties of soil elements were based on results of field and laboratory tests presented in Appendix 3.7C of the FSAR and accommodated strain dependent behavior. Dynamic properties of structural elements represent an approximation of the actual structural geometry when represented by plane-strain finite elements. The elastic modulus of the solid model element is determined such that its overall stiffness approximates that of the box-shaped concrete element it represents. These values of modulus were developed based on joint input from Bechtel, Edison and WCC regarding the overall size and thicknesses of structural elements and their dynamic behavior in the finite element model. A schematic finite-element model of soil structure system is shown in Figure 1a.

2. The DBE input earthquake motion was assumed to be a traveling wave at the base of the finite-element model. The velocity of the traveling wave was conservatively based on estimated shear wave velocity compatible with the average DBE induced strains in the free field. On that basis a velocity of 700 fps was used for the traveling wave input motion. Subsequent to the analysis, it was found that a major portion of the earthquake energy arrives at the site as vertically traveling shear waves, whose front travels at an apparent wave velocity of approximately 2000 fps due to a steep angle of incidence (approximately 15 degrees from vertical based on calculation by Del Mar Associates). The assumption of 700 fps for velocity of input

motion is very conservative, compared to the more likely velocity of 2000 fps, for computation of induced displacements in the structure.

3. Horizontal and vertical displacement time histories at selected points in the structure were obtained from the finite-element analysis described above.

4. Displacement time histories, obtained in Step 3, were examined to observe changes in phase and amplitudes between time histories at various locations. However, due to the short distances (approx. 25 ft.) between these locations, the results of the computer analysis indicated no phase shift or amplitude differences. To conservatively evaluate relative displacements, the displacement time histories were then physically shifted as shown in Figure 1b by a time equal to that required by the input motion to travel between two adjacent points. The displacements at each point were then calculated at each instant in time to develop instantaneous displacement profiles as indicated on Figure 1c. Displacements between adjacent points were used to calculate relative rotations, which are a measure of bending stresses between adjacent parts of the structure, as shown schematically on Figure 2. The displacement profile at an instant, when a maximum relative rotation between any two adjacent parts is obtained, was considered to be the critical instantaneous displacement profile which was used as a deflected shape to calculate structural stresses.

The steps described above were applied specifically to the onshore intake structure with reinforced box conduit, and the offshore conduits composed of a number of short lengths (approximately 25 feet) of circular concrete conduits.

For the onshore intake structure, the finite-element model included intake structure, reinforced concrete box conduit and auxiliary building. Stiffness properties of structural elements were determined through consultations with Bechtel and Edison. Stiffness properties of the reinforced box conduit, in particular, were based on actual/solid ratios of section modulus and area of the conduit. For the intake structure and box conduit the specific values of modulus used were one-third and one-half of the modulus of concrete, respectively. The intake structure is highly complex, but, in general, is more open than the box conduits and presents less overall rigidity in proportion to its size than the box conduits. The instantaneous displacement profile developed from this model was then used to distort a model of the structure from which the resulting stresses could be calculated and designed for.

For the offshore conduit it was considered that the conduit would track motions in surrounding soil during an earthquake because:

a. these are made of small sections of pipes (length small relative to the wave length) flexibly connected;

b. they mobilize similar inertia load compared to that of the soil they displace.

Thus, in the finite-element model, no structural elements were included and the model consisted of soil elements only. Horizontal and vertical displacement time histories were physically shifted by a time required for the input motion to travel the length of a single conduit (approximately 25 feet). Thus, horizontal and vertical displacement profiles, providing displacements at all conduit joints at any given instant were obtained. These free-field displacements were then conservatively used to calculate the maximum rotation between any two adjacent conduits to obtain the critical instantaneous displacement profiles in horizontal and vertical directions. In addition, the horizontal displacement profiles were used to calculate the maximum joint separation at all instants of time. These maximum values of joint rotations (in horizontal and vertical directions) and joint separation were used in designing the bell and spigot joints between individual conduits.

SHADED ELEMENTS REPRESENT CONCRETE

OTHER ELEMENTS REPRESENT SOIL

x_1 x_2 x_3 x_4

DIRECTION OF WAVE PROPAGATION

(a) SCHEMATIC DIAGRAM OF F.E. MODEL

t_a

5 INSTANTANEOUS DISPLACEMENTS CORRESPONDING TO MAX. BENDING

① TIME (t)

② t

③ t

④ t

⑤ t

DISPLACEMENT

t_a

t_i

(b) DISPLACEMENT — TIME HISTORIES

DIFFERENTIAL DISPLACEMENT AT TIME t_i

INSTANTANEOUS DISPLACEMENT PROFILE AT TIME t_2

① ② ③ ④ ⑤

(c) TYPICAL INSTANTANEOUS DISPLACEMENT PROFILE

FIGURE 1 — SUMMARY OF PROCEDURE FOR DETERMINATION OF INSTANTANEOUS DISPLACEMENT PROFILE

Instantaneous Displacement Profile at Time t (e.g. from Fig. 1-c)
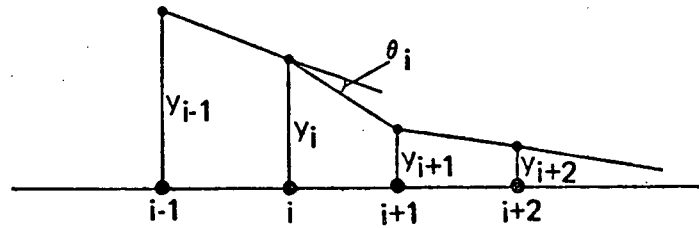
i — Nodal points in the F.E. Model

y_i — Displacement at nodal point i

θ_i — Rotation at nodal point i

The CIDP is determined by examining θ_i at all nodal points
and all times t. The displacement profile with maximum θ_i
is considered to be the CIDP.

FIGURE 2 — SCHEMATIC FOR DETERMINING CIDP FROM DISPLACEMENTS

NRC STRUCTURAL AUDIT REQUEST ITEMS

OFFSHORE CIRCULATING WATER SYSTEM

<u>Item #2</u>: Provide available data concerning longevity of the rubber joint gasket and joint wrapping.

<u>Response</u>

The rubber joint gaskets for the intake conduits are fabricated from a high grade rubber compound. The basic polymer is first grade natural rubber. The material has the following properties:

o   Hardness – Shore Durometer, Type A 55 ±5

o   Elongation at rupture – minimum 500%

o   Tensile strength – 3000 psi, minimum

o   Tensile strength after accelerated aging per ASTM D-572 (48 hours @ 70°C, 300 psi) – 80% of minimum strength before aging

o   Specific gravity – 1.15 ±0.05

o   Serviceability – Goodyear Bulletin 821-947-79 indicates that for natural rubber long service in seawater may be expected with little reduction in properties due to exposure. It is suitable for continuous service.

The joint wrapping material is 1/2 inch thick 3 feet wide Neoprene (a DuPont trademark for synthetic rubber). DuPont publication Bulletin E-02141 indicates sodium chloride solutions (saltwater) has little or no effect on Neoprene.

DuPont Bulletin (DuPont Neoprene - the Proven Performer) No. E-19837

records long duration use of Neoprene under actual and test conditions.

From all available documentation, Neoprene shows excellent service in

applications exposed to weathering.

NRC STRUCTURAL AUDIT REQUEST ITEMS

OFFSHORE CIRCULATING WATER SYSTEM

Item #3: Document the capacities of the offshore conduits when compared
to current SRP criteria as previously requested by the NRC staff.
In particular, address simultaneous load application in three
principal directions by SRSS combination and use of the 1.5
multiplier on the peak amplified responses.

Response:

Per our remarks at the audit and in the previously transmitted letter
report (summary of 3/10/78 meeting with NRC in Bethesda), it is our posi-
tion that application of the 1.5 times the peak response is unnecessarily
conservative for completely buried structures with similar mass and inertial
characteristics as the displaced soil. It is our judgment that the design
methodology described in FSAR sections 3.7.2.1.1.3 and 3.7.2.1.10.2 is
appropriate for these structures. However, to comply with the NRC request,
a comparison of loads calculated using the SRP criteria versus capacity is
provided on the attached Table 1.

The seismic loads were determined by applying equivalent inertia loads
calculated using 1.5 times the amplified response from the appropriate
project response spectra. The seismic loads from the three principal
directions were then combined using SRSS. The resultant seismic load was
then combined with the dead load, live load and operating loads using
load factor equations consistent with the Standard Review Plan (refer to
FSAR section 3.8.4).

For the cases where the SRP OBE equations governed the design, the SRP DBE loads are also provided. The values in the table demonstrate the conduits have sufficient capacity to meet the current SRP criteria.

TABLE 1

Offshore Circular Conduits
Summary of Governing Load Interactions

Description of Member	Governing Load Combination	Calculated Axial Load Pu* & Flexural Load Mu		Maximum Flexural Capacity Mu Given Axial Load Pu(ft-Kips)	DBE Loading* if Different Mu than Governing Combination	Remarks
		Pu(Kips)	Mu(ft-Kips)			
Prestressed Conduit Sections	10a	33.0	50.2	58.0	38.0	
Prestressed Conduit Section Adjacent to Box Conduit	10a	35.9	57.1	58.0	45.6	
Conventionally Rein-forced Manhole Conduit	10a	29.2	48.2	52.0	38.5	
First Conduit Section Longi-tudinal Bending	11	–	975.4	1364	–	
Joint Shear-First Conduit Section	11	–	–	–	–	Actual Shear = 216.6^k Capacity = 283^k

*Loads are calcualted using E = 1.5X Peak Response from the response spectrum.

Item #6: Provide available data for Ec and fc' from Auxiliary Intake

Structure Mix Design.

Response

Reference the attached letter from Smith-Emery Company to Southern California

Edison. Mix Design #78-SE-559 with appropriate test result data for the

4000 psi mix was submitted to SCE for approval. The mix was tested using

two types of curing cycles. Method A, the steam cure cycle, was used because

the pipe section and riser section of the structure were steam cured.

Method B, the standard ASTM C192 cure, was also performed because the

buttresses were air cured following application of liquid membrane to all

exposed concrete surfaces. The test results demonstrate the mix design

meets the design requirements for fc' = 4000 psi @ 28 days.

Ec used in the design was not obtained by testing. It was calculated using

the equation specified in ACI 318-71 Section 8.3.1.

SMITH-EMERY COMPANY

CHEMISTS · TESTING · INSPECTION · ENGINEERS

781 EAST WASHINGTON BOULEVARD · LOS ANGELES, CALIFORNIA 90021 · (213) 749-3411
3148-Q LA PALMA AVENUE · ANAHEIM, CALIFORNIA 93806 · (714) 630-4910

FILE NO: 145B
LABORATORY NO.: 0-78-357

DATE: JUNE 12, 1978

SOUTHERN CALIFORNIA EDISON COMPANY
P.O. BOX 800
ROSEMEAD, CA 91770

ATTENTION: MR. STANLEY R. WRIGHT

RE: CONCRETE MIX DESIGNS FOR
PRECAST INTAKE STRUCTURE
SAN ONOFRE N.G.S.
P.O. #SO 598901

REPORT OF TESTS

IN COMPLIANCE WITH YOUR REQUEST WE HAVE DETERMINED THE PROPORTIONS
AND CONDUCTED TRIAL BATCHES WITH PRELIMINARY CYLINDERS FOR THE
LIGHTWEIGHT CONCRETE MIX DESIGN TO BE USED ON THE REFERENCED
PROJECT. SEE ATTACHED MIX DESIGN NO. 78-SE-529.

THE WASHED CONCRETE SAND WAS SAMPLED AT THE CONROCK COMPANY
READY MIX PLANT IN SAN JUAN CAPISTRANO. THE CEMENT WAS
OBTAINED FROM THE CALIFORNIA PORTLAND CEMENT COMPANY. THE
LIGHTWEIGHT AGGREGATES WERE SAMPLED AT THE CRESTLITE PRODUCTS
PLANT IN SAN CLEMENTE, AND THE ADMIXTURE WAS OBTAINED FROM
MASTER BUILDERS.

A TRIAL BATCH OF THE MIX DESIGN WAS MADE ON MAY 11, 1978 AND
THE FOLLOWING CURING PROCEDURES AS OUTLINED FOR US WERE
UTILIZED:

SMITH-EMERY COMPANY

METHOD A

ONE-DAY TEST (12 HR. STEAM @ 110°F)

1. MAKE CYLINDERS - 11:00 A.M.
2. PLACE IN STEAM CELL - START STEAM - 3:00 P.M.
3. STEAM FOR 12 HOURS @ 110°F, SLOW UP AND COOL DOWN, NOT MORE THAN 40° PER HOUR:
4. REMOVE ALL CYLINDERS FROM STEAM CELL AND STRIP ALL AT 7:00 A.M.
5. CAP AND TEST BY 8:00 A.M. (2 CYLINDERS).

TWO-DAY TEST (18 HR. STEAM @ 140°F)

6. PLACE REMAINING CYLINDERS IN SECONDARY CURE CELL.
7. TURN ON SECONDARY STEAM - 10:00 A.M. (40°F RISE/HR.)
8. STEAM FOR ± 18 HOURS @ 140°F.
9. REMOVE CYLINDERS FROM STEAM CELL - 7:00 A.M. (40°F DROP/HR.)
10. CAP AND TEST BY 8:00 A.M. (2 CYLINDERS).

THREE-DAY TEST

11. REMOVE 6 CYLINDERS FOR AIR CURE (7, 14, 28 DAY TESTS).
12. REMAINING 8 CYLINDERS TO RECEIVE SECONDARY STEAM @ 140°F. TURN ON AS SOON AS POSSIBLE.
13. STEAM FOR ± 18 HOURS (40°F RISE AND DROP/HR.)
14. REMOVE 8 CYLINDERS FROM STEAM CELL - 7:00 A.M.
15. CAP AND TEST BY 8:00 A.M. (2 CYLINDERS).

SEVEN-DAY TEST

16. TEST 2 CYLINDERS W/30 HR. STEAM CURE.
17. TEST 2 CYLINDERS W/54 HR. STEAM CURE.

FOURTEEN-DAY TEST

18. TEST 2 CYLINDERS W/30 HR. STEAM CURE.
19. TEST 2 CYLINDERS W/54 HR. STEAM CURE.

TWENTY EIGHT -DAY TEST

20. TEST 2 CYLINDERS W/30 HR. STEAM CURE.
21. TEST 2 CYLINDERS W/54 HR. STEAM CURE.

METHOD B

1. AFTER 24 HOURS AIR CURE 6 CYLINDERS WERE STRIPPED OF THE MOLDS AND PLACED IN THE MOIST CLOSET FOR STANDARD ASTM C192 CURE. 1 CYLINDER WAS TESTED AT 2 DAYS, 1 CYLINDER WAS TESTED AT 3 DAYS. 2 CYLINDERS WERE TESTED AT 7 DAYS AND 2 AT 28 DAYS.

SMITH-EMERY COMPANY

RESULTS OF COMPRESSIVE STRENGTH TESTS ON CYLINDERS CAST IN THE TRIAL BATCH ARE AS FOLLOWS:

DATE TESTED	AGE, DAYS	COMPRESSIVE STRENGTH, PSI		
		CYLINDER 1	CYLINDER 2	AVERAGE
METHOD A CYLINDERS				
5/12	1	3360	3060	3210
CYLINDERS RECEIVING 30-HRS. STEAM CURE				
5/13	2	3500	3590	3545
5/18	7	4740	4510	4625
5/25	14	4705	4755	4730
6/8	28	4845	5285	5065
CYLINDERS RECEIVING 54-HRS. STEAM CURE				
5/14	3	4455	4280	4365
5/18	7	4775	4985	4880
5/25	14	5040	5005	5020
6/8	28	5095	5200	5145
METHOD B CYLINDERS				
5/13	2	3290		
5/14	3	3910		
5/18	7	4775	4705	4740
6/8	28	5075	5340	5205

RESPECTFULLY SUBMITTED,

SMITH-EMERY COMPANY

Paul Linstrom

PAUL LINSTROM
CIVIL ENGINEER

1-Addressee
4-Edison Co. CMI
   Attn: John Garbellini
1-Edison Co.
   Attn: Don Schone

PL:BL

746-3411

SMITH-EMERY COMPANY
CHEMISTS · TESTING · INSPECTION · ENGINEERS

FILE NO. 1458   LAB. NO. 0-78-357   **CONCRETE MIX**   DATE: APRIL 27, 1978

SOURCE OF MATERIAL: L/W: CRESTLITE SAN CLEMENTE; WCS: CONROCK SAN JUAN CAPISTRANO
BATCH PLANT: WCS: CONROCK, SAN JUAN CAPISTRANO
SAMPLED BY: SMITH-EMERY COMPANY
DATE: 2/22/78
WASHED CONCRETE SAND ······ NO. 200 WASH 2.6   ORGANIC MATTER: OK

SIEVE ANALYSIS:
PER CENT PASSING U.S. STANDARD SIEVE

MATERIAL	1½	1	3/4	3/8	NO. 4	NO. 8	NO. 16	NO. 30	NO. 50	NO. 100	WT./CU. FT.
FA1 WCSand				100	99	87	73	48	20	6	91.4
FA2											
CA1 ½" CRESTLITE			100	51	2	1					44.3
CA2											
CA3											
COMBINED			100	82	64	56	47	31	13	4	

		1½	1	3/4	3/8	NO. 4	NO. 8	NO. 16	NO. 30	NO. 50	NO. 100	
COMBINED	MAX											
LIMITS	MIN											

DESIGN FOR ONE CUBIC YARD OF CONCRETE
(SATURATED SURFACE DRY)

NUMBER OF SACKS OF CEMENT PER CUBIC YARD OF CONCRETE: 8.0
GALLONS OF WATER PER CUBIC YARD OF CONCRETE: 37.0
GALLONS OF WATER PER SACK OF CEMENT ·· DESIGNED: 4.6
GALLONS OF WATER PER SACK OF CEMENT ··· ALLOWABLE: 4.6

MATERIAL	BATCH LBS.	PER CENT	SP. GR.	ABSOLUTE VOLUME CU. FT.
CEMENT	752.0			3.84
WATER	308.0			4.93
ESTIMATED AIR		3.0%		0.81
FA1 WCSand	1270	64	2.60	7.81
FA2				
CA1 ½" CRESTLITE	709	36	1.18	9.61
CA2				
CA3				
			TOTAL	27.00

RESPECTFULLY SUBMITTED
SMITH-EMERY COMPANY

BY *Paul Lundstrom*

R. E. NO. 13624

CIVIL ENGINEER

JOB: SOUTHERN CALIFORNIA EDISON COMPANY

ADDRESS: SAN ONOFRE N.G.S., SAN ONOFRE, CALIFORNIA
CONTRACTOR: AMERON

NOTE:
LIGHTWEIGHT CONCRETE FOR
PRECAST INTAKE STRUCTURE
THIS MIX TO BE USED ONLY WITH
3.0 FL. OZ. POZZOLITH 300N+0.4
ADMIXTURE PER 100 # OF CEMENT. FL. OZ.
MBVR

THIS MIX DESIGNED FOR 4000
P.S.I. AT 28 DAYS IN ACCORDANCE
WITH PROJECT REQUIREMENTS.
ALLOW UP TO 10.0 GALS. OF WATER
PER CUBIC YARD FOR ABSORPTION
OF THE LIGHTWEIGHT AGGREGATE.
BATCH PLANT INSPECTION REQUIRED
WHEN USING THIS MIX.

CU. FT./CU. YD. DRY LOOSE
WC SAND   13.9 @ 91.4
½" CRESTLITE 16.0 @ 44.3

SLUMP: 3½"

NOTE: THIS MIX WILL PROVIDE A
AIR DRY UNIT WT. BETWEEN 107 &
112 LBS./CU. FT. IN 28 DAYS.

DESIGN NO.   78SE-529

OFFSHORE CIRCULATING WATER SYSTEM

Item #7   Provide results of hand calculations showing effects of
           assumed minimum torsion for the Auxiliary Intake Structure.

Response

An assessment has been made of the expected amount of accidental geometric
torsion which would be expected for this structure. The structure is
composed of precast concrete elements which are fabricated under shop
conditions. The forms used for most elements are steel forms which are
provided for the casting of more than 50 similar elements. The conduit
and riser base block forms are used to manufacture the San Onofre discharge
diffuser elements and the riser was formed by Ameron standard pipe forms
for the specified inside and outside diameters. Because of the reuse
requirements for these forms, and the fact that significant dimensional
variation could lead to repetitive problems in handling and assembling
these conduits, great care is taken to fabricate these forms to careful
tolerances. Significant configuration variation is thus not expected for
this structure.

In its final location, the structure is entirely submerged in the ocean
with all portions buried beneath the ocean floor except for the intake
riser. Because of this, conventional live loads are not imposed on the
structure and its inertia load results solely from the inertia of the
structural elements. Hydrodynamic and soil loadings, by their nature

are imposed at the geometrical center of the structure and are independent of variations in thicknesses of individual elements.

Sample calculations have been prepared assuming a variation of 20 percent in structure mass from one half of the structure to the other. This variation would be considered an extreme variation for the reasons outlined above. This case resulted in a eccentricity of less than 2 percent of the overall length of the structure. An eccentricity of this amount will have a negligible effect on design of the structure and detailed consideration of the results is not necessary. Approximately 70 percent variation in mass would be required to produce an eccentricity of 5 percent.

NRC STRUCTURAL AUDIT REQUEST ITEMS

Item #1, Containment Structure, Item #1, Auxiliary Building; Item #1,

Fuel Handling Building; Item #3, Safety Equipment Building.

Provide a comparison of the effects of simultaneous consideration of

3 components of seismic response by SRSS combination in both horizontal

directions and the vertical direction for representative elements of the

buildings

Response

The requested information will be provided in the response to Question 131.31.

NRC STRUCTURAL AUDIT REQUEST ITEMS

Item 6, Containment Structure; Item 4, Auxiliary Building;

Item 4, Fuel Handling Building; Item 4, Safety Equipment Building.

Provide results of a conventional analysis of safety factors for sliding

and overturning.

Response

Within the general Engineering profession, the typical design of structures

to resist seismic loadings utilize simplified static equivalent techniques.

These techniques were developed in an attempt to simplify the analytical

procedures while at the same time maintain a degree of conservatism and

margin in design.   The method of utilizing a static force balance procedure

in determining overturning moments within a structure is a prime example of such

a technique (Reference 1, pgs. 500-507).   Although this technique tends to produce

conservative results, such a method is not a realistic approach for use in deter-

mining the overturning stability of buildings or other large structures.   The

method totally ignores the dynamic nature of the response of a complex structure

during seismic excitation by its inherent assumptions that: all modal responses

act in phase, and all maximums are additive, (peak acceleration at all

locations acting simultaneously in the same direction), and that the

resulting peak loading is a sustained loading rather than an instantaneous

occurrence.   Application of such a technique to the evaluation of the overturning

tendencies of a structure can lead to totally unrealistic conclusions on the

stability of a structure.   As evidence of this condition, licensing authorities

for buildings in seismically active areas are currently requesting a dynamic

treatment for the overturning stability analysis of multistory structures.

Although structural failures have occurred within structures due to a lack of proper consideration of the effects of the overturning moment, actual failures of structures by tipping over in response to a seismic excitation have not occurred.[2] The few isolated instances of structural overturning have all been related to soil failures (Reference 1, p. 434) and not structural instability.

Therefore, it becomes apparent that a more realistic approach needs to be applied in establishing the overturning tendencies of a structure. A more fundamental approach would be to consider whether sufficient energy (or momentum) has been imparted to the structure to raise the center of gravity past the point of impending instability or whether the structure possesses enough potential energy to right itself. This method is a direct application of the basic law of physics dealing with the conservation of energy. This procedure is discussed in more detail in Section 4.4 of Bechtel Topical Report BC-TOP-4A and has been reviewed and approved by the NRC for application in the design of nuclear power plants.

The values for overturning stability listed in the FSAR and discussed in the NRC structural audit of the SONGS Project are based on this procedure. They show ample margin against overturning and we feel that any evaluation by the simplified static force balance procedure is unwarranted. It should be pointed out that the analytic techniques and procedures employed in the design of all Category I structures adequately accounted for the effects of overturning moments on the resultant member forces of the various elements within the structures and thereby preclude any direct structural failure.

As a final check on the overturning stability of all seismic Category I structures we have also evaluated the maximum soil pressures developed under the toe of foundation and compared these values to the allowable dynamic soil pressure of the supporting media. Results of this evaluation were reported to the NRC Structural Audit team. In all cases ample margins exist to preclude bearing failures which might result in overturning of the structures.

The sliding stability of Category I structures was addressed in the response to NRC round two question 131.28.

References

(1) Fundamentals of Earthquake Engineering by Newmark & Rosenblueth.

(2) Design of Multistory Reinforced Concrete Buildings for Earthquake Motions by Blume, Newmark and Corning, pg. 62.

NRC STRUCTURAL AUDIT REQUEST ITEMS

INTAKE STRUCTURE AND BOX CONDUIT

Item #2: Provide basis for establishing the maximum ground water level
used in determining hydrostatic pressures on structural walls.

Response

A number of observations for ground water level has been made at the sites
for Unit 1 and Units 2 and 3 over the past 10 years. These observations
were made during various investigations such as soil explorations, pumping
tests for evaluating field permeability of native soils, installation of
the Units 2 and 3 dewatering system and investigations during demobiliza-
tion of the Units 2 and 3 dewatering system. A summary of these observa-
tions is presented in Table 1. Figure 1 presents the locations of various
observations points. The data presented in Table 1 indicate a maximum
value of +5.8 feet. Only a few of the measurements indicated a level
above +5 feet and those were made on the dewatering wells or test piezo-
meters and may reflect transient conditions related to a rapid inflow of
water occurring when dewatering was terminated. For this reason, and based
on the majority of the data, it is concluded that elevation +5 feet
represents a reasonable maximum for the ground water level at the site.
This conclusion is also consistent with areal observations as follows:
(1) ground water contours for typical high and low ground water conditions
are presented in Figures 2.4-27 and 2.4-28 of the FSAR (presented as
Figures 2 and 3 here for completeness) indicate a maximum ground water
elevation of +5 feet in the immediate vicinity of the site; and (2) a

report by California Department of Water Resources (Reference 1) provides regional data indicating an average ground water elevation in the vicinity of the site is +5 feet.

Ground water sources for the general area of the site are described in Section 2.4.6.13 of the FSAR. Because the foundation soil at the site (San Mateo Sand) is free draining and because the site is adjacent to the ocean, the fluctuations in the ground water are primarily controlled by tidal fluctuations. Further, because of the free draining nature of the foundation soils and the surface drainage facilities at the site, direct rainfall on the site is not expected to have an influence on the ground water level. In view of the above information and the observed ground water levels in the various wells summarized in Table 1, it is concluded that an elevation of +5 feet represents a reasonable maximum level for the ground water at the plant site.

References

1. California Department of Water Resources, 1968, Reclamation of water from wastes - central San Diego County: Bulletin 80-2.

## TABLE 1

SUMMARY OF GROUND WATER LEVEL OBSERVATIONS

Well/Boring Designation	Date of Observation	Elev. of Observed Water Level	Remarks
Water Well #1	May 1967	+3.9	Well #1 through 5 are
	June 1967	+4.0	located in area of
	March 1970	+4.1	Unit 1 (see Figure 1)
	Dec 1970	+3.9	
Water Well #2	April 1967	+2.6	
	May 1967	+3.1	
	June 1967	+3.3	
	March 1970	+3.3	
	Dec 1970	+3.5	
Water Well #3	April 1967	+3.0	
	May 1967	+3.3	
	June 1967	+3.5	
	March 1970	+3.7	
	Dec 1970	+3.7	
Water Well #4	April 1967	+1.2	
	May 1967	+2.1	
	June 1967	+1.7	
	March 1970	+2.5	
	Dec 1970	+2.2	
Water Well #5	April 1967	+0.8	
D-M Boring #2	March 1970	+5.0	Soil boring.
Piezometer Well #1	May 1974	+5.60*	Observation wells #1 through 7 installed for pump tests. The observations represent initial readings before pump tests.
Piezometer Well #2	May 1974	+3.09	
Piezometer Well #3	May 1974	+5.80*	
Piezometer Well #3A	May 1974	+5.24*	Test well.
Piezometer Well #4	May 1974	+5.60*	

Table 1
Page 2

| Well/Boring Designation	Date of Observation	Elev. of Observed Water Level	Remarks
Piezometer Well #5	May 1974	+5.76*	
Piezometer Well #6	May 1974	+5.34*	
Piezometer Well #7	May 1974	+5.47*	
Deep Well #1	July 1977	+4.5	Deep wells installed for dewatering system.
Deep Well #2	Aug 1977	+4.5	
Deep Well #3	Nov 1977	+5.5	
Deep Well #3A	Sept 1977	+5.0	
Deep Well #4	May 1976	-0.50**	
Deep Well #5	April 1976	0.0**	
Deep Well #6	July 1976	-1.5**	
Deep Well #7	June 1976	-3.0**	
Deep Well #8	June 1976	-3.0**	
Deep Well #9	Nov 1976	-3.5**	
Deep Well #10	July 1977	+4.5	
Deep Well #11	July 1977	+2.5	
Deep Well #12	July 1977	+2.5	

* Values above Elev. +5 feet may be affected by testing operation (i.e., rapid water inflow after test covering a local high)

** Readings represent maximum values monitored over a period from 1974 to 1977. Low values may be related to dewatering from adjacent wells.
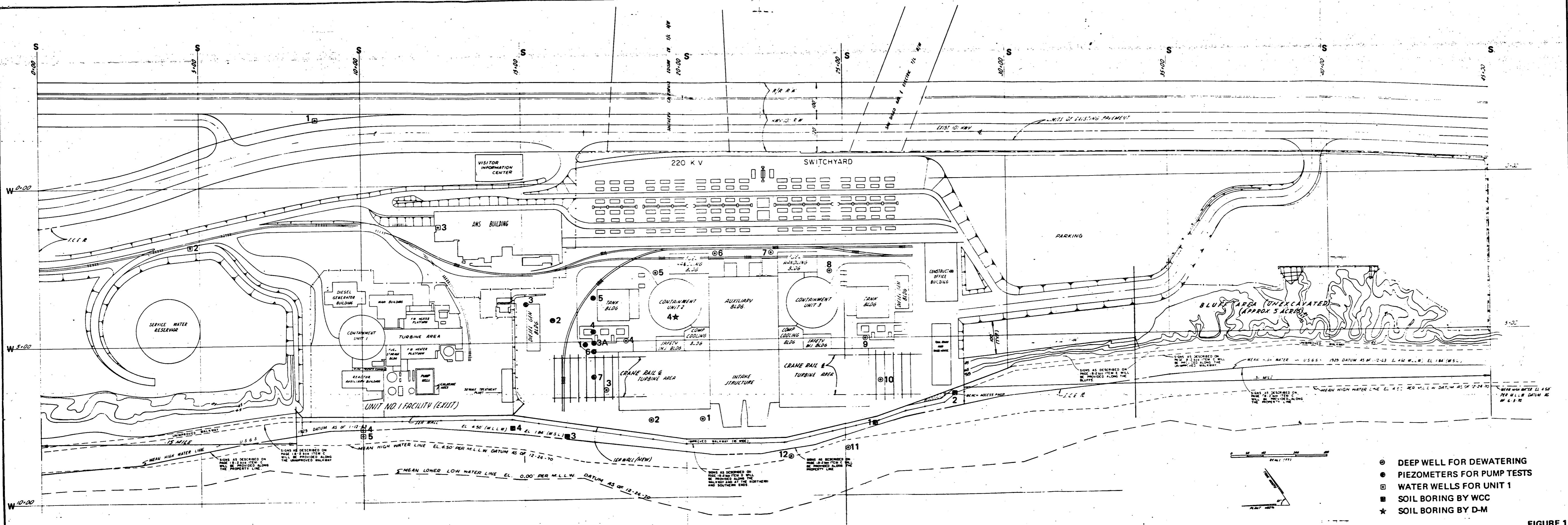
FIGURE 1

Legend:
- ⊙ DEEP WELL FOR DEWATERING
- ● PIEZOMETERS FOR PUMP TESTS
- ⊡ WATER WELLS FOR UNIT 1
- ▪ SOIL BORING BY WCC
- ★ SOIL BORING BY D-M
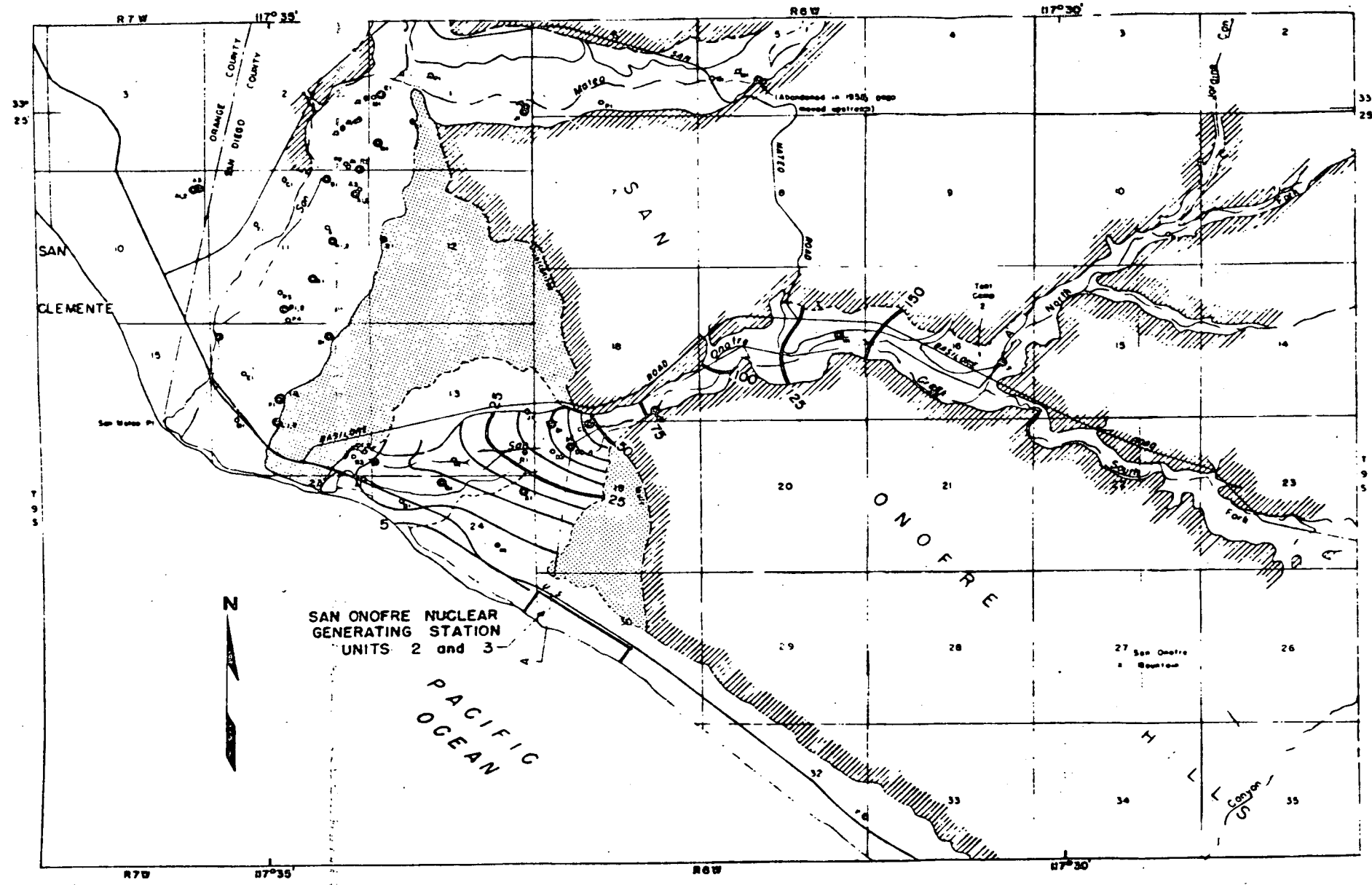
EXPLANATION

Permeable formation
Semi-impermeable formation
Boundary of impermeable formation

Camp supply or irrigation well

Domestic, stock, or unused well

Destroyed or dry well

Stream-gaging station

Ground-water contour March 1952
(5 foot interval)

Location of cross section

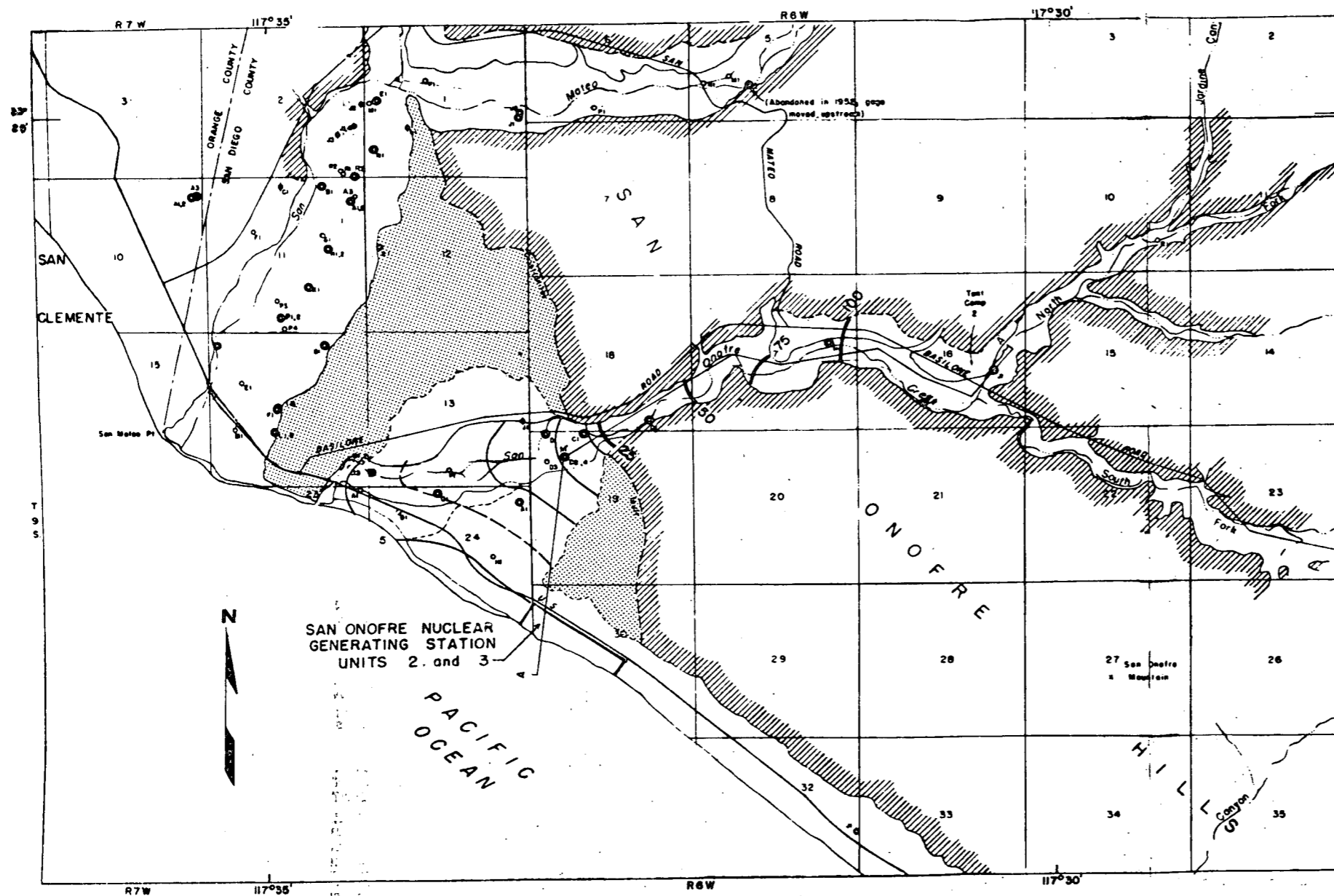SCALE: 1 = 36000
DATUM IS MEAN SEA LEVEL

COMPILED FROM: G. F. WORTS ET AL., 1953
R. F. BOSS ET AL., 1958

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

GROUNDWATER CONTOUR MAP
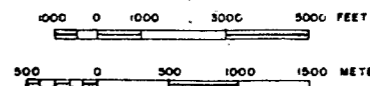SAN ONOFRE VALLEY BASIN
TYPICAL HIGH-WATER CONDITION

Figure 2.4-27

FIGURE 2

EXPLANATION

☐ Permeable formation

▨ Semi-impermeable formation

▨ Boundary of impermeable formation

○ R1
Camp supply or irrigation well

○ R2
Domestic, stock, or unused well

● R2
Destroyed or dry well

Stream-gaging station

Ground-water contour, August 1951
(5 foot interval)

A
Location of cross section

SCALE 1 = 36,000
DATUM IS MEAN SEA LEVEL

1000 0 1000 3000 5000 FEET

500 0 500 1000 1500 METERS

COMPILED FROM: G. F. WORTS ET AL, 1953
R. F. bOSS ET AL, 1958

| SAN ONOFRE |
| NUCLEAR GENERATING STATION |
| Units 2 & 3 |
| GROUNDWATER CONTOUR MAP |
| SAN ONOFRE VALLEY BASIN |
| TYPICAL LOW-WATER CONDITION |
| Figure 2.4-28 |

FIGURE 3

SAN ONOFRE NUCLEAR
GENERATING STATION
UNITS 2 and 3

NRC STRUCTURAL AUDIT REQUEST ITEMS

CONTAINMENT STRUCTURE

Item #2   Provide results of hand calculations of shear stresses at the

base of the containment shell and secondary shield walls

assuming an eccentricity of 5 percent of the base width of the

structure.

Response

The containment shell is axisymmetric, therefore there is no eccentricity

between center of mass and center of resistance.

In the case of containment internal structure, structural analysis was

performed using a 3-D finite element model.   Therefore, the effect of

actual geometric eccentricity is inherent in the computer solution.

Refer to Item #3 of Auxiliary Building on the subject of accidental

geometric torsion.

NRC STRUCTURAL AUDIT REQUEST ITEMS

CONTAINMENT STRUCTURE

Item #3:   Provide available information concerning longevity of PVC

electrical conduit material.

Response

The electrical ducts for buried conduits are Carlon Power and Communications
Duct,[1] Type DB and conform to NEMA Standard TC-6, 1974. The duct is manufactured
from PVC compound consisting of at least 75% PVC homopolymer and 20% inert
additives for increasing the heat distortion temperature. It has a tensile
strength of 6200 psi. The ducts have high resistance to wide range of
chemicals resulting in long product lifes, have high resistance to heat,
good aging and weathering characteristics and high structural strength to
resist imposed traffic loads. These ducts house electric conduits which have
similar characteristics and do not depend on the ducts to provide protection
against applied forces or moisture.

Reference

1.   Carlon, Manufacturer's Catalogue for Plastic Conduit Fittings and
Accessories, An Indian Head Company located at 23200 Chagrin Blvd.,
Cleveland, Ohio 44122.

CONTAINMENT STRUCTURE

Item #10: Provide material specifications for the waterstop materials.

Response

The waterstop material used for SONGS 2&3 is styrene-butadiene synthetic rubber (SBR), designated by W. R. Grace and Company, the supplier, as serviced rubber. The material is covered by the Army Corps of Engineers Specification for Rubber Waterstops, CRD C-513. Four main types of waterstops are used: six-inch flat dumbbell, nine-inch flat dumbbell, twelve-inch flat dumbell, and nine-inch with centerbulb. The material has the following properties:

1. Hardness - Shore A durometer hardness of 60 to 70

2. Elongation - Minimum 450%

3. Tensile strength - 2500 psi, minimum

4. Average specific gravity - 1.17

5. Tensile strength after aging - Required to retain 80% of original tensile strength after accelerated aging test of 7 days at 158°F.

6. Serviceability - According to Goodyear Bulletin 821-947-79, SBR material exhibits excellent service in the sea water environment.

Rubber waterstops (SBR) have been used extensively in nuclear installations as well as many other industrial facilities. Some of the nuclear power plants which have used rubber waterstops include Calvert Cliffs 1 and 2, Zion 1 and 2, Beaver Valley 1, Hatch 1 and Farley 1 and 2. Past experience with this material indicates that the material has successfully performed under similar conditions. Additional tests have also been successfully performed to verify the waterstops' ability to accommodate large displacement simulating earthquake conditions.[1,2] Other tests have shown good capability

to withstand nuclear radiations in the form of fastneutron flux, integrated
thermal neutron flux and gamma flux.[1,3]

References

1. Personal communication to Mr. J. Patel, dated August 16, 1973, from Robert W. Faid, Manager of Engineering Services, W. R. Grace & Co., Construction Products Division.

2. Report of Test on "Seismic Vibration and Water-Leakage Test on Rubber Waterstops for Use in Concrete Joint Systems" for W. R. Grace and Co. by Acton, Environmental Testing Corporation, February 11, 1974.

3. Radiation Resistance Study of Grace Materials used in Construction of Nuclear Power Plants, Construction Products Division, W. R. Grace and Co., Cambridge, Mass., March 23, 1976.

NRC STRUCTURAL AUDIT REQUEST ITEMS

AUXILIARY BUILDING

Item #2: Provide description of procedure used in design of the auxiliary
building truss utilizing SMIS computer program considering
multiple point response spectral input.

RESPONSE:

The dynamic analysis of the control room roof truss was performed by modal
response spectra analysis, using, as inputs, motions defined by vertical
in-structure response spectra calculated at the two support points of the
truss. This analysis considered only vertical response since the truss is
fully supported horizontally at the top and bottom by floor diaphragms and
its horizontal response was evaluated in the overall analysis of the con-
crete structural system. For vertical response, the support points were
defined by the top of the concrete wall on one side, and the bottom of the
column on the other side. The two-dimensional model for the truss was
developed, using beam elements which accounted for the axial loads and
bending moments that developed in the heavily welded members of the truss.
The steel column was included in the truss model. The dynamic analysis and
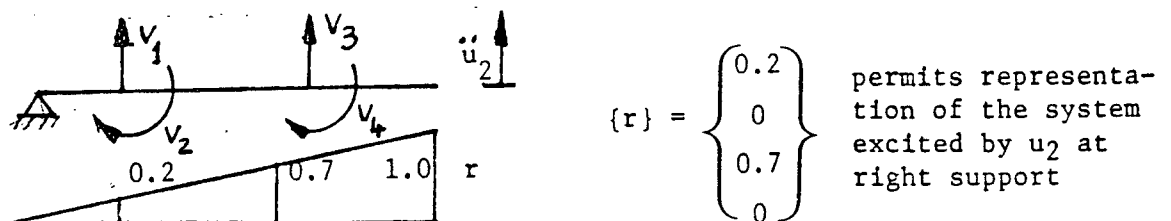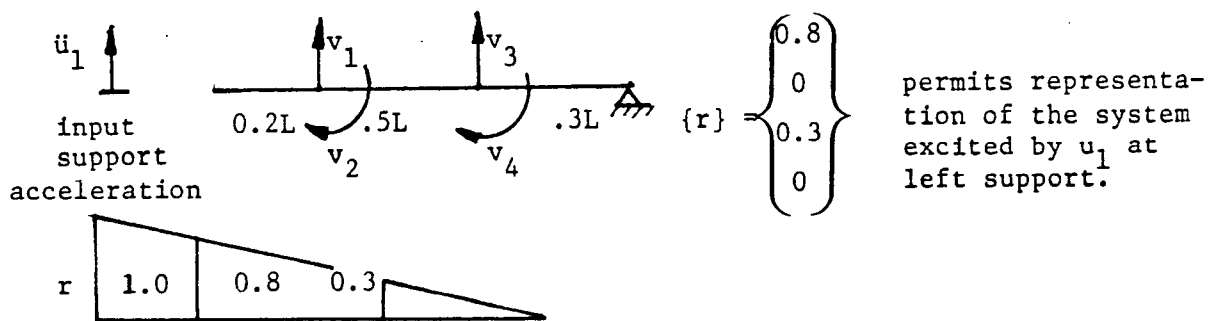the matrix formulation were both performed using the SMIS computer program.

At each of the support points of the truss, different in-structure response
spectra were used. These exhibited different response characteristics due
to the variable stiffness characteristics of the supports. On one end, the

steel column which was relatively flexible introduced dynamic amplifications,
and on the other end the thick concrete wall exhibited nearly a rigid body
behavior, tracking the base vertical response with minor amplifications.

The general procedure for the analysis can be described by using the
equation of motion and the transformation matrix for the input base
acceleration. The conventional equation of motion for input base accel-
eration excitation is given by:

$$[M]\ \{\ddot{v}\} + [C]\ \{\dot{v}\} + [K]\ \{v\} = -[M]\ \{r\}\ u\ (t)$$

Where, all the terms in this equation are per the usual definition, including the
{r} transformation matrix which according to its fundamental definition is
used to scale and to define the degrees of freedom that are parallel to
the input base motion {u}. The application of the transformation matrix
is illustrated as follows:

$$\{r\} = \begin{Bmatrix} 0.8 \\ 0 \\ 0.3 \\ 0 \end{Bmatrix}$$

permits representa-
tion of the system
excited by u_1 at
left support.

$$\{r\} = \begin{Bmatrix} 0.2 \\ 0 \\ 0.7 \\ 0 \end{Bmatrix}$$

permits representa-
tion of the system
excited by u_2 at
right support

As a comparison to methods used in normal analysis, this is analogous to the base excitation of a cantilever system representing a building.
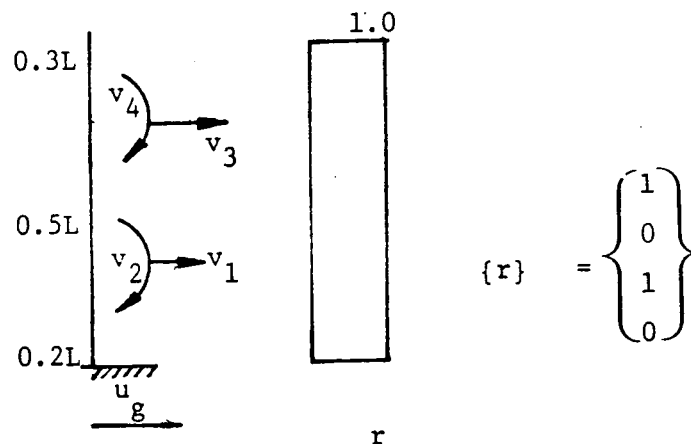
Based on the above general description, the procedure outlined below was used for multipoint response spectral input and combination of various response parameters in the truss elements. Two separate computer analyses were performed for each support point. The inputs at each support were the in-structure response spectra obtained from: 1) vertical base input to the overall building model and 2) vertical rocking response resulting from horizontal input to the overall building model. For each computer run, the transformation matrix, r, was used to define the linearly decreasing support accelerations at each point where accelerations were input into the truss model.

From each computer run, member forces consisting of axial loads, moments and shears were obtained through SRSS combination of modal responses. The next step was to account for responses resulting from the different

input accelerations applied at each support. These were combined by taking the absolute sum of the responses. Since there were basically two types of vertical support input motions generated from the overall building model, each was combined separately. Once the effect of multipoint spectral input was considered for each type of vertical base input motion (Vertical support response due to vertical input at the base of the building model and vertical support response [rocking] due to horizontal input at the base of the building model) the final member seismic design forces were obtained by taking the SRSS of the forces.

AUXILIARY BUILDING

ITEM #3   Provide justification for use of less than 5 percent accidental

torsion in addition to actual geometric torsion which is

considered.

RESPONSE

An evaluation of the accidental geometric torsion was made using the

auxiliary building. This building was chosen because of its inherent

eccentricity induced by unsymmetric building mass distributions.

San Onofre power plant buildings are characterized by large horizontal

dimensions and heavy permanent structural masses (concrete walls and slabs

which are well defined by the detailed structural analysis required for

this building). Few additional geometric eccentricities would be expected

from approximations used in computations, if any, and/or arbitrarily placed

floor live loads. In addition, because of high lateral force requirements

for these structures, they require continuous perimeter walls which provide

excellent resistance to torsional responses.

Table 1 lists the overall physical dimensions of the building, the main

dimensions and the characteristic masses associated with the major areas

of the auxiliary building. From the calculation of eccentricity at each

floor level and the mass centroid of the whole building it was found that

the mass of the combined radwaste and tank areas, located 43 feet from

the centroid of the building and representing 49 percent of the total

building mass, introduced an eccentricity of 3.4 percent on the building

which was included in the building design. This result is significant

since such a substantial shift in mass produced only a 3.4 percent eccen-

tricity when all building contributing masses and geometric eccentricities

were considered. Further computations were performed to assess the effect

of a redistribution of the floor live loadings to produce an accidental

geometric torsion of 5 percent on live load only. Because of the small

magnitude of these loads compared to the overall building weight, this

redistribution produced only a 0.2 percent eccentricity for the total

design loading. Therefore, it is not reasonable to impose on this design

an accidental geometric torsional eccentricity of 5 percent beyond the

calculated geometric eccentricity by an accidental redistribution of floor

loads and/or interior partition loads. The effect of accidental geometric

torsion is minor and may be neglected.

TABLE 1

AUXILIARY BUILDING STRUCTURAL CHARACTERISTICS
AND
ACCIDENTAL TORSION COMPUTATIONS

1. Building Size: height = 94 feet, length = 280 feet, width = 221 ft

2. Total Weight = 273,000 Kips

3. Mass Distribution

	Total Weight of Structural Components Kips	Floor Loads Subject to Variation Kips	Ratio of Variable Load to Total Load
Basemat	87,000	3,500	4%
Radwaste and Tankage Area Walls & Elevated Floors	133,000	8,500	6%
Control & Penetration Area Walls & Elevated Floors	53,000	5,000	9%
Total	273,000	17,000	6%

NRC STRUCTURAL AUDIT REQUEST ITEMS

FUEL HANDLING BUILDING

Item #2   Provide justification for use of less than 5 percent accidental
torsion in addition to actual geometric torsion which was
considered.

Response

Refer to the response of item 3 of auxiliary building for the justifi-
cation of using less than 5 percent accidental torsion.

NRC STRUCTURAL AUDIT REQUEST ITEMS

SAFETY EQUIPMENT BUILDING

Item #1: Provide a description of the parametric study used to determine
that 80 percent of DBE loads give conservative values for
OBE loads for the Safety Equipment Building.

Response

For the safety equipment building, it was determined that 80% of the
DBE response will conservatively represent the OBE response. The determina-
tion was based on a comparison of DBE and OBE responses, as calculated
from the dynamic analysis of the fuel handling building. For the dynamic
analysis of the fuel handling building, a lumped parameter model was used.
This building was chosen for comparison because it has similar characteris-
tics as the safety equipment building.

Table 1 lists mass, embedment and frequency characteristics of the
fuel handling building and the safety equipment building. The listed data
indicates reasonable similarity between the two buildings, particularly
with respect to the important parameter of natural frequency.

Table 2 lists the seismic acceleration response at two key nodes for
the fuel handling building. The salient feature is that the ratio of OBE
to DBE responses range from 0.55 to 0.65, which is substantially less
than the 0.80 ratio adopted for the safety equipment building. The 80 percent
of DBE loads were directly used for OBE conditions, using the appropriate load
factors in the load combination equations.

TABLE 1

Comparison of Building Characteristics for the Safety Equipment and Fuel Handling Buildings

Fuel Handling Building	Safety Equipment Building
Total weight = 51,000 k	Total weight = 37,000 k
20 ft embedment, on 3 sides	Embedment on 4 sides {11 ft, 14 ft, 20 ft, 45 ft}

	Natural frequencies of dominant modes		Natural frequencies of dominant modes	
MODE	DBE(cps)	OBE(cps)	MODE	DBE(cps)
8*	2.39	2.74	1	2.39
9	2.58	2.97	2	3.22
10	2.99	3.46	3	4.12
11	4.57	5.24	4	5.05
12	5.53	6.39	5	7.31
13	5.85	6.74	6	8.85

*Modes 1 through 7 correspond to response of liquid masses of pools

TABLE 2

Fuel Handling Building Response (ft/sec^2)

Seismic Acceleration Response of two key nodes

Response direction	Node 1 basement elev. 17' - 0"			Node 6 roof elev. 114' - 0"		
	X	Y	Z	X	Y	Z
DBE excitation:						
(horiz.) X	29.16	0.45	1.41	39.97	0.36	5.89
(horiz.) Y	0.55	29.42	0.57	0.37	49.69	2.37
(vert.) Z	3.85	2.43	19.87	4.90	1.87	19.98
(hor & vert) Asb. sum	33.01	31.85	21.28	44.87	51.56	25.87
OBE excitation:						
X	19.40	0.38	0.78	22.77	0.22	3.16
Y	0.50	18.86	0.32	0.29	29.02	1.27
Z	2.22	1.34	10.94	2.69	1.12	11.05
(hor & vert) Asb. sum	21.62	20.20	11.72	25.46	30.14	14.21
Ratio of OBE/DBE response, using absolute sums	0.65	0.63	0.55	0.57	0.58	0.55

SAFETY EQUIPMENT BUILDING

Item #2 Provide results of a hand calculation of shear stresses at the base of the exterior shear walls due to accidental torsion assuming an eccentricity of 5 percent of the base width of the structure.

Response

Safety equipment building was analyzed using a 3-D finite element model. Therefore, the effect of actual geometric eccentricity is inherent in the computer solution.

Refer to the response of Item 3 for the justification of using less than 5 percent accidental geometric torsion.