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April 20, 1981

Director, Office of Nuclear Reactor Regulation
Attention: Mr. Frank Miraglia, Branch Chief
Licensing Branch No. 3
U. S. Nuclear Regulatory Commission
Washington, D.C. 20555

Gentlemen:

Subject: Docket Nos. 50-361 and 50-362
San Onofre Nuclear Generating Station
Units 2 and 3

Enclosed as requested by the Structural Engineering Branch are seven (7) copies of revised structural calculations which confirm the capability of plant structures to withstand possible offsite explosions. (Mail Code B025)

If you have any questions or comments concerning this information, please contact me.

Very truly yours,

K P Baskin

Enclosures

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B001S 1/7
DRWGS
B025 s1/7
APERTURE
DIST

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PRESSURE LOAD CAPACITY OF SAFETY RELATED STRUCTURES

The probability of occurrence of potential explosion accidents characterized by overpressures in excess of the pressure load capacity of the safety-related structures has been calculated and established to be within the limits per 10CFR 100 exposure guidelines.

The explosion events potentially result in a transient impulsive pressure load which is considered in combination with the permanent gravity load postulated for the structures. The structural response to the impulsive load is determined by energy balance techniques utilizing flexural strain energy capacities which are limited by specific ductility criteria and are calculated based on ultimate strength methodology per applicable design codes of practice. The ultimate strength calculations incorporate load factors and/or allowable stress factors corresponding to extreme environmental loading combinations as defined in SRP[1] Sections 3.8.4.II.3b (4) and 3.8.4.II.5, respectively.

The pressure load capacity for each safety-related structure was evaluated, and the resulting minimum capacity was adopted as the pressure intensity to define the governing explosion events. The pressure load capacities typically range from 8 to 12 psi, with a minimum value of 7.1 psi dictated by one type of steel girder supporting the roof of the control area in the auxiliary building. Accordingly, explosion events yielding pressures of 7 psi and higher were considered for the evaluation of the probability of occurrence of damage-producing explosions. The 7 psi threshold level used to define explosion events as dictated by the structural design is conservative based on the following considerations:

1. Events resulting in peak reflected pressures of 7 psi rather than directly applied pressures were considered throughout in the probability evaluation. Peak reflected pressures as used in the evaluation are mainly applicable to vertical exposed surfaces. Normally such reflected pressures are over-conservative for horizontal exposed surfaces such as roof slabs which are not subject to reflection effects. Therefore, the generic consideration of peak reflected pressures for all explosion events without any scrutiny for vertical or horizontal surfaces is a definite conservatism for the cases under consideration where pressure load capacities are actually governed by roof systems.

2. The safety-related structures are located at the base of a 2:1 embankment, 95 feet below the flat terrain where the railway and interstate highway run parallel to the site at a minimum distance of 467 feet, as illustrated in Figures 1 and 2. However, comprehensive definition of the explosion events imposes consideration of vapor cloud migration effects for which the shielding afforded by the site topography is not necessarily effective. Therefore the topographic shielding, which was not quantified nor relied upon because of the strict inclusion of vapor cloud effects, remains as a favorable shielding feature applicable only to the main explosion events originating at the railway or highway locations.

3. The energy balance technique utilized for the structural analyses involves the following conservative simplifications:

   3.1 The impulsive load is implicitly treated as a sustained load subject to instantaneous build-up. The benefit of reduced response derived from more rigorous time-history analysis recognizing the transient, short-duration loading was not utilized.

   3.2 Energy absorption is solely accounted by flexural strain energy of slab or girders considered separately, without accounting for other energy losses by local deformation, shear deformation and momentum imparted to the structural elements.

   3.3 The load resistances are calculated based on specified minimum yield strength of materials without introducing the actual material strengths tested at about 5 to 15 percent higher, and without using the dynamic strength increase factors of 1.1 to 1.2 permitted by design codes. In addition, the capacity reduction factors (Ø = .90, .85) prescribed for reinforced concrete design were preserved in the calculated resistances.

4. A more extensive study of the explosion events, involving specific evaluation of the configuration and arrangement of each structure, and yielding specific pressure-time loading functions, would result in loads and responses lower than those derived by the current generic approach used.

For reinforced concrete slabs and walls the ultimate flexural and shear strengths per ACI 318-71 [2] are used in conjunction with yield-line theory per Reference [3] in order to evaluate the pressure load resistance. The methodology for the impulsive load analysis is in accordance with ACI 349-76[4], with specific references summarized as follows:

Description	Section(s) of ACI 349-76, App.C	Remarks
Dynamic increase factors not utilized	C.2.1	Conservatively the increase factors were disregarded, if included could improve load capacity by 10%.
Limit design by yield-line theory	C.3.1	
Resistance for impulsive load exceeds by over 20% the peak load.	C.3.2	Conservatively, not taking advantage of short-duration nature of loading, and neglecting absorption of energy by kinetic energy, shear and local deformations.
Ductility ratio for elements that could affect the overall structural integrity (bearing walls) $\mu = 1$ and for structurally non-essential elements (roofs) $\mu = 3$	C.3.4	The limited ductibility ratio used represent conservative practice with respect to $\mu = 3$ allowed per C.3.4 and $\mu = 10$ allowed per C.3.3.
Rotational Capacity limitation satisfied	C.3.5	This criterion for rotational capacity, in conjunction with the limited ductility ratios used are extremely conservative per Reference [8].

Description	Section(s) of	Remarks
Shear capacity checked to exceed by at least 20% the flexural capacity.	C.3.6	
Beam-columns and slabs carrying axial compression loads are not involved; walls' compression load is within limit per C.3.8(b)	C.3.8	Structural steel columns have very low slenderness ratio and develop the capacity of roofs.
Materials, reinforcing proportion and arrangement, (no concrete columns involved) satisfy ductility requirements	C.4	

For composite action steel girders and beams supporting the reinforced concrete roof slabs the ultimate moment capacity calculation is based on rectangular distribution of stress imposed on the composite flexural section. This strength design method is equivalent to the plastic design method indicated in SRP [1] Sections 3.8.4 II (b) (ii) (4) and 3.8.4 II 5 (4), and it is the basis for the AISC Specification [5] provisions for working stress design of composite beams as stated in page 2-137 of the AISC Manual[5]. It is reiterated that in accordance with the fundamental structural analysis method being used, whereby evaluation of ultimate load capacity is relevant, the strength design method must be used in lieu of the AISC working stress design method which is directly applicable only for service load conditions. The strength design method for composite beams, even though not directly incorporated in the AISC Manual, is well established based on ASCE publication per Reference [6], and is fully incorporated in other design codes such as the AASHTO Specification [7].

Other relevant ultimate load capacities dictated by web shear, web crippling and member connections are evaluated based on AISC allowable stresses subject to SRP stress factors per Section 3.8.4 II 5 and allowing sufficient margins to justify the flexural ductility ratio of 3.0.

References:

(1) USAEC Regulatory Standard Review Plan (SRP) Section 3.8.4

(2) Building Code Requirements for Reinforced Concrete (ACI 318-71)

(3) Handbook of Concrete Engineering by M. Fintel, Section 3.10

(4) Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-76), Appendix C

(5) AISC Manual of Steel Construction, 7th Edition

(6) "Flexural Strength of Steel-Concrete Composite Beams" by R. G. Slutter and G. C. Driscoll; Journal of the Structural Division, ASCE Proceedings, April 1965

(7) Standard Specification for Highway Bridges, 12th Edition American Association of State Highway and Transportation Officials; Sections 1.7.52 thru 62

(8) "Ductility Ratio for Slabs" by E. G. Burdette and D. Bernal; Journal of the Structural Division, ASCE Proceedings, November 1978

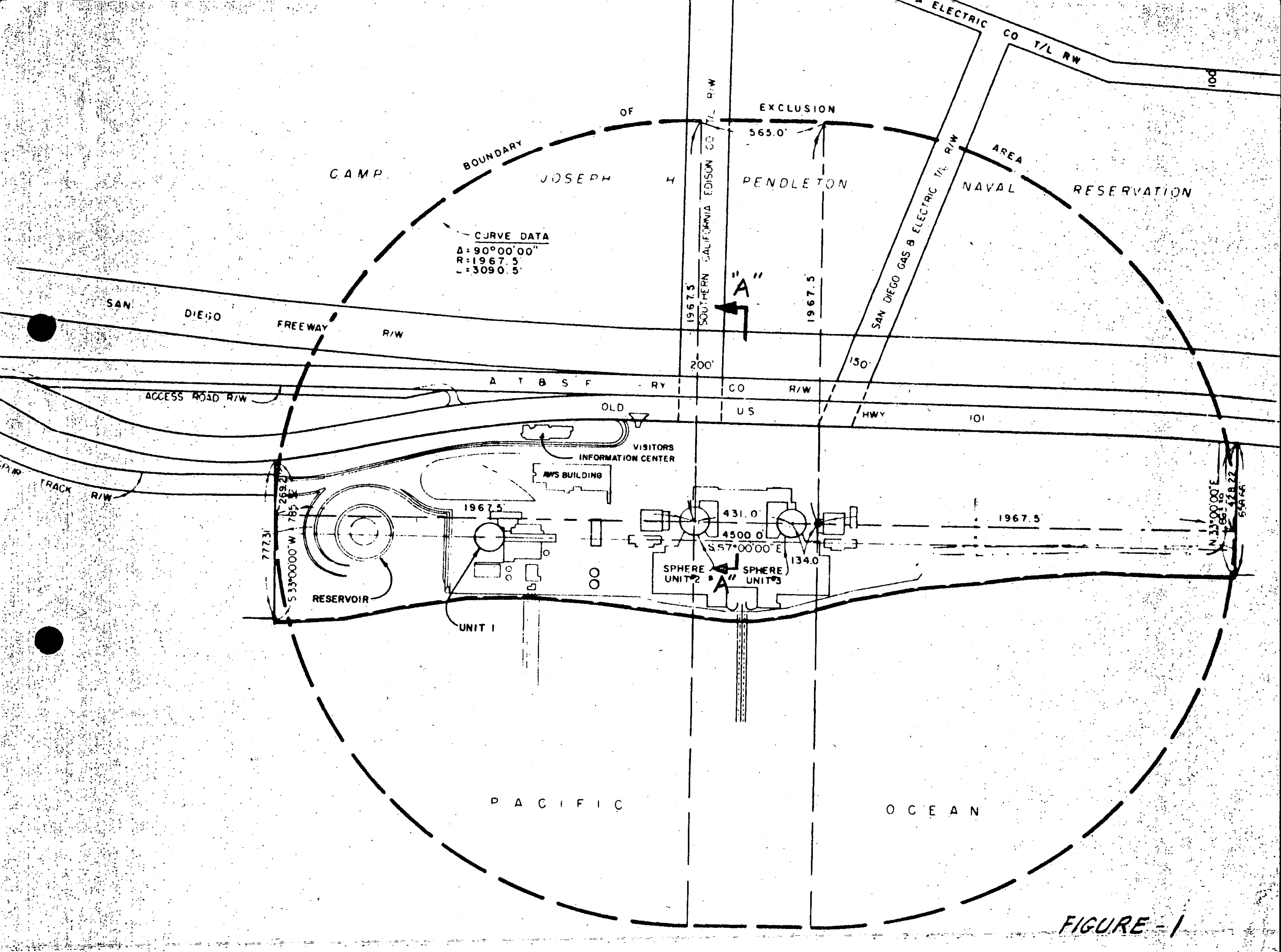
ELECTRIC CO T/L RW
EXCLUSION
OF
BOUNDARY
AREA
565.0'
CAMP JOSEPH H PENDLETON NAVAL RESERVATION
SOUTHERN CALIFORNIA EDISON CO T/L R/W
SAN DIEGO GAS & ELECTRIC T/L R/W
CURVE DATA
Δ=90°00'00"
R=1967.5
L=3090.5
"A"
1967.5
1967.5
SAN DIEGO FREEWAY R/W
200'
150'
A T & S F RY CO R/W
ACCESS ROAD R/W
OLD US HWY 101
VISITORS INFORMATION CENTER
AWS BUILDING
TRACK R/W
1967.5
431.0'
4500.0'
S.57°00'00"E
134.0
1967.5
SPHERE UNIT 2
SPHERE UNIT 3
"A"
RESERVOIR
UNIT 1
777.31
S.33°00'00"W
N.33°00'00"E
PACIFIC OCEAN

FIGURE-1