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Exhibit B 3

CONSOLIDATED EDISON COMPANY OF NEW YORK, INC.

INDIAN POINT NUCLEAR GENERATING UNIT NO. 3

THIRD SUPPLEMENT TO:

PRELIMINARY SAFETY ANALYSIS REPORT

4654

PREFACE

The following listing of material furnished as Supplement No. 3 will serve as a check list for entering new pages into Supplment No. 2 to the PSAR.

The listing shows the original pages and the replacement or addition pages to be inserted into the text.

pages to be replaced	pages to be inserted (10/68)
1) 2.4(a), page 1/blank	2.4(a), page 1/page 2
2) 2.6(a), page 1/blank	2.6(a), page 1/page 2, page 3/page 4, page 5/page 6

QUESTION 2.4

For the containment structure, provide:

(a) A preliminary design drawing of the containment presenting details of the base slab, dome-cylinder junction, cylinder-slab junction, showing reinforcing and liner features, including liner anchors.

ANSWER

The preliminary design drawings are given in Figures 2.4(a)-1 through 2.4(a)-4. The preliminary design drawings for the dome-cylinder junction will be given in a subsequent supplement to the PSAR.

The shear reinforcing is placed in the mat to accomodate redistribution of internal forces should a diagonal crack occur. The density of the reinforcing steel in the mat is including the wall dowels in 11 lb/cu. ft.

The shear stresses in the inclined stirrups in the mat are calculated for the 1.25P loading condition which is the most severe loading condition for shear considerations due to the uplift from the design earthquake with only 2% critical damping. Formula (17.5) of the ACI 318-63 Code is used to calculate the shear stresses. The anchorage bond stresses in the bars are calculated and compared with the allowable bond stress of $.8 \times .6\sqrt{f'_c} = 262$ psi recommended in Chapter 18 of the ACI-318 Code. In no case is the allowable bond stress exceeded. The following is a table of the shear and anchorage bond stresses for the stirrups:

Stirrups	Shear Stress	Bond Stress
#18S	$45^k/in^2$	240 psi *
#14S	$20.1^k/in^2$	87 psi **
#11	All shear can be taken by the concrete section	

* The anchored portion of the stirrup is in a compression zone, therefore, the problem of anchorage is greatly reduced.

** In the event that a crack opens and reduces the anchorage length, it can be seen that a margin of safety of approximately 3 exists between the actual stress and the allowable stress.

The length of the stirrup above or below mid-height of the mat is used as the anchorage length.[1] No credit is taken for the additional anchorage provided by the bend in the bar. It is felt that this assumption is conservative since standard hooks in tension may be considered to develop 10,000 psi working stress, or 19,000 psi ultimate stress in the bar, or may be considered as simple extensions of the bars at the appropriate bond stresses.[2]

The diagonal shear reinforcing will terminate at approximately 20'-0" above the base. At this point the radial shear from the discontinuity at the base mat cylinder junction has virtually dissipated and the steel has been carried past the point of requirement specified in the ACI-318 Code.

There are two types of radial shear reinforcing; the inclined stirrups and the secondary vertical bars which are bent across the wall. The inclined stirrups are mechanically anchored by hooking them around the vertical bars. Each secondary vertical bar is terminated further up the wall by splicing it to a #11 bar and mechanically anchoring this bar with a 180° hook.

1. "Reinforced Concrete Fundamentals" by Phil. M Ferguson - John Wiley & Sons, 1958.

2. "Design of Concrete Structures" by George Winter, L. C. Urquhart, C. E. O'Rourke, and Arthur H. Nilson - McGraw-Hill Book Company - 1964.

QUESTION 2.6

For the base slab, describe:

(a) The analytical procedures used to arrive at the forces, moments, and shears, considering the rigid support afforded by the ground. State whether transient thermal gradients have been considered.

ANSWER

The base slab was treated as a flat circular plate supported on a rigid non-yielding foundation. For loads applied uniformly around the slab, the analysis considers a 1 ft. wide beam fixed at a point where the vertical shear is equal to zero. This is the point where the downward pressure on the mat and the dead weight overcome the uplift, at the containment wall base mat juncture, from pressure and earthquake loadings. Radial and circumferential reinforcing is provided at the top and bottom of the mat to resist moments in the areas where uplift occurs. Temperature steel was added in other areas to meet requirements of the (ACl-318) Code. Diagonal tension reinforcement was added to meet requirements of ACl-318 Code.

Moments and shears were calculated by writing equations for moment and shear in terms of X using the containment wall--base slab juncture as the origin with X increasing as you proceed toward the center of the containment building. The point along the circumference of the containment wall chosen as the end of the beam is a point where the maximum tension from the earthquake will exist. Since the containment structure is considered a beam in all earthquake analysis, the maximum uplift for which the mat is designed will occur at only one point on the circumference and will represent the worst possible uplift on the mat.

All stresses were calculated using Part IV-B Structural Analysis and Proportioning of Members - Ultimate Strength Design of the Building Codes Requirements for Reinforced Concrete (ACl-318-63). No rebar stresses exceed $.90\ f_y$.

A gradient with an operating temperature of 120°F inside the containment and a 50°F temperature at the mat-rock interface was considered and stresses are negligible. Accident temperatures have no appreciable effect on the base slab.

It is not possible to show the design on non-yielding rock is more conservative than assuming the rock to be elastic. However, due to the installation of temperature reinforcing, the design is conservative. Reinforcing and concrete stresses are very low when considering the rock to be elastic.

To substantiate the above statement the following studies were performed:

1) The foundation modulus was determined utilizing the expression[(1)]:

$$k_z = \frac{4\,Gr_o}{1 - \mu}$$

where k_z = The vertical spring constant of a circular base supported on an elastic foundation

$$G = \frac{E}{2\,(1 + \mu)}$$

r_o = Radius of Foundation

μ = Poisson's Ratio

To obtain the foundation modulus, k_z is divided by the area of the circular base to yield

$$k_o = \frac{k_z}{A} = \frac{4\,G}{\pi r_o\,(1 - \mu)}$$

Substituting for G

$$k_o = \frac{2\,E}{\pi r_o\,(1 - \mu^2)}$$

2) The first case examined was that of a rectangular strip loaded with 1.5 times design accident pressure plus dead load using conservative properties for the Dolomitic limestone[(2),(4)]:

$E = 6.0 \times 10^6$ psi

$\mu = 0$

Applying these values

$k_o = 4370\#/in.^3$

The "characteristic" λ is defined as:[3]

$$\lambda = \sqrt[4]{\frac{k}{4\ EI}}$$

Where E is the modulus of elasticity of the structural base (concrete), I is the moment of Inertia of the structural base,

and $k = k_o\ b$, (b = width of base)

using base properties

$\lambda = 7.56 \times 10^3\ in.^{-1}$

Where $\lambda \ell > \pi$ beams may be considered as infinite in length.[3]

Taking the length of beam as being the base diameter

$\lambda \ell = 13.1 \quad > \pi$

The beam was then analyzed as a beam of unlimited length loaded over an area equal to the base diameter with an 80 psi uniform load.

The solution to this problem gives

$$y_c = \frac{q}{2k} \quad (2 - D_{\lambda a} - D_{\lambda b})$$

$$M_c = \frac{q}{4\lambda^2} \quad (B_{\lambda a} + B_{\lambda b})$$

$$Q_z = \frac{q}{4\lambda} \quad (c_{\lambda a} - C_{\lambda b})$$

Where

y_c is deflection of point being considered

M_c is the moment at point being considered

Q_c is shear at point being considered

q is the uniform load

a is distance from point under consideration to end of load

and b is distance from point under consideration to other end of load.

$$B_{\lambda x} = e^{-\lambda x} \text{Sin } \lambda x$$

$$C_{\lambda x} = e^{-\lambda x} \text{ Cos } \lambda x - \text{Sin } \lambda x$$

$$D = e^{-\lambda x} \text{ Cos } \lambda x$$

Maximum moment occurs at mid-point of load and is equal to 352 in-#/in.

For the area of the mat where there is only temperature reinforcing, the maximum moment would cause a stress of 30 psi in the reinforcing.

The maximum shear would occur at the ends and is equal to 2.64 $^k/$ in. This shear would cause a shear stress in an unreinforced concrete section of 26.4 psi.

3) A second case examined was for the foundation material being less rigid than the concrete base. The model was the same for the first case:

Assumed $E_{rock} = 2.6 \times 10^6$ psi

$\mu = 0.$

For this case, the following were determined:

k_o = 1890lb/in.^{3}

λ = 6.2 x 10^{-3} in.$^{-1}$

M_{max} = 3.66 in-k/in.

Q_{max} = 3.23 ^{k}/in.

S_{rebar} = 312 psi

ν_{conc} = 32.3 psi

4) As a final study, the maximum deflection as calculated in first case was imposed as a settlement of the base mat for the outer portion and a section of the mat was analyzed for this settlement. A 30 foot section was used with fixity at the reactor pit, the remainder cantilevered from the pit.

The resulting moment and shear are as follows:

M = 142 in-k/in.,

q = 396#,

resulting in a rebar stress of 12.2 ksi and a shear stress of 4.0 psi.

From the above, it can be seen that the assumption that a foundation on rock is a rigid unyielding foundation is a valid one and that temperature reinforcing provides much greater resistance than required to accommodate the effects of any elastic deformation of the sub-grade.

References:

1. "Design Procedures for Dynamic Loaded Foundations" by R. V. Whitman and F. E. Richart, Jr. - A.S.C.E. - Journal of the Soil Mechanics and Foundations Division Vol. 93 No. SM6, No. 1967.

2. "Elements of Strength of Materials" by S. Timoshenko and D. H. Young - D. Van Nostrand Company, Inc. 1962.

3. "Beams on Elastic Foundation" by M. Hetenyi - Ann Arbor Press 1955.

4. "Formulas for Stress and Strain" by Raymond J. Roark - McGraw-Hill Book Co., Fourth Edition.