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PROBABILISTIC SAFETY VERIFICATION FOR THE OVERTURNING OF A SERVICE WATER PUMP HOUSE AGAINST "OVER PRESSURE WAVE"

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1. INTRODUCTION

The Atucha II Nuclear Power Plant is being built in a riverside site located about 100 km north of the city of Buenos Aires. When completed it will become Argentina's third operating station. The Service Water Pump House of the 740 MW Natural Uranium NPP is supplied directly by the Parana de las Palmas River, which is a navigable waterway with heavy fluvial traffic that links the River Plate Estuary with ports along the Parana River, including Rosario, Argentina's third city.

Although dolphins were built to protect the water intake structures from direct impact from ships or barges, an accidental escape of gas or flammable material outside the protected area that may result in an explosion with a subsequent pressure wave, could not be precluded. Thus, the water intake structures had to be designed against this event. In addition, concern with the margin of safety against foundation instability due to the overturning moment associated to the induced pressures, made an evaluation of the conditional probability of failure for this loading condition, desirable.

The task placed still uncommon demands on the Project Engineers, who were required to provide estimates of the Pump House reliability. In the process, it became necessary to introduce assumptions concerning the unspecified variability of the loads which, according to current international practice, were defined in the form of "deterministic" design criteria. It seems appropriate to underline these deficiencies in current standards, as well as in aspects of the dynamic of soils that are responsible for a quite large model uncertainty.

2. DESCRIPTION OF THE PUMP HOUSE

The Service Water Pump House consists of a prismatic reinforced concrete structure, with length L = 35.00 m, width B = 12.00 m and total height h = 20.55 m, of which D = 10.25 m are below ground level, as shown in Fig. 2.1. Typical wall thickness is t = 0.60 m.

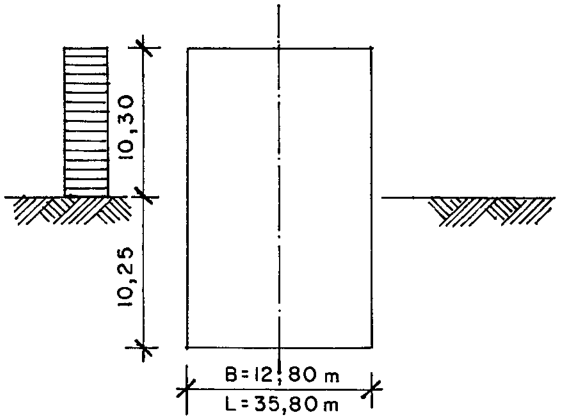

FIG. 2.1 PUMP HOUSE - TIPICAL SECTION

3. ACTIONS

The over-turning wave action was taken into the consideration as indicated in the Instructions established for the project which were in turn based on the Guidelines given in Ref. 2.

For the loading case under consideration, previous studies led to the design pressure vs. time curves presented in Fig. 3.1 for side walls and in Fig. 3.2 for the roof.

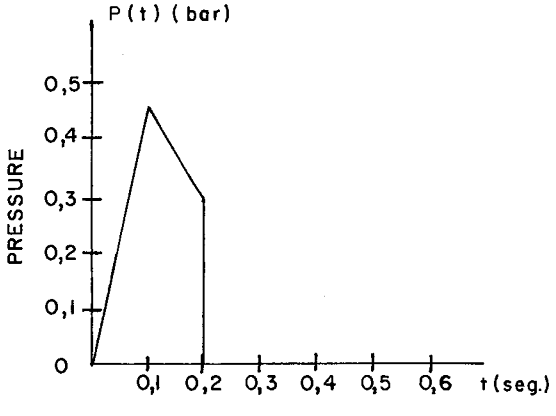

FIG. 3,1 WALL - PRESSURE VS. TIME

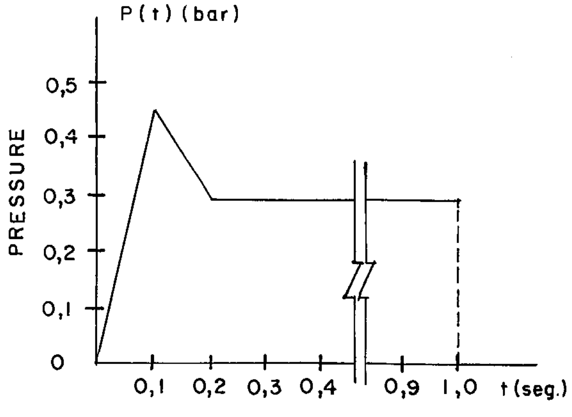

FIG. 3,2 ROOF - PRESSURE VS. TIME

On account of the dynamic nature of the excitation, the ensuing instability problem was approached using equivalent static pressures determined by multiplying the peak values in the pressure-time diagrams by the corresponding dynamic amplification factor. In order to evaluate the latter, the fundamental frequency of the Water Pump House was determined taking into consideration the influence of the surrounding soil. A value f = 3.4 Hz was finally obtained for a coupled horizontal-rocking mode, which led to an amplification factor d = 1.5.

4. DETERMINISTIC VERIFICATIONS

The foundation safety under the pseudo -static actions described in Section 3, was first evaluated following the conventional deterministic approach. The stress distribution in the soil was determined by means of a plane finite element model, which led to a safety factor of 1.38.

5. RESISTANCE AGAINST THE OVERTURNING MOMENT "H"

The overturning moment resistance has been evaluated adopting as principal variable the horizontal force "H" that, acting over the lateral wall of the building, is in equilibrium with the last resistance of the foundation soil.

The loading capacity of a plane strip of width b, founded at depth d, subjected to a vertical load acting eccentrically simultaneously with an horizontal load H, may be calculated by means of Brinch Hansen formulations (Ref. 1):

$$\frac{Q}{BL} = \frac{1}{2}.\bar{\tau}.B.N\tau.S\tau.d\tau.i\tau \;+\; \bar{q}.Nq.Sq.dq.iq \;+\; Cu.Nc.Sc.dc.ic$$

$$\left.\begin{array}{l} Nq = e^{\pi.\tan\phi u}.\tan^{2}(45^{\circ} + \frac{1}{2}\phi u) \\ Nc = (Nq - 1).\mathrm{ctg}\,\phi u \\ N\tau = 1.8\,(Nq - 1).\tan^{2}\phi u \end{array}\right\} \text{Bearing capacity factors capacity of the soil}$$

$$\left.\begin{array}{l} iq = \left(1 - \dfrac{H}{V+A.Cu.\mathrm{ctg}\,\phi u}\right)^{2} \\ ic = iq - \dfrac{1-iq}{Nq-1} \\ i\tau = \left(1 - \dfrac{H}{V+A.Cu.\mathrm{ctg}\,\phi u}\right)^{4} \end{array}\right\} \text{Inclination factors}$$

$$\left.\begin{array}{l} dq = dc - \dfrac{dc-1}{Nq} \\ dc = 1 + 0.35.D/B \quad (\text{para } D > B) \\ d\tau = 1 \end{array}\right\} \text{Depth factors}$$

$$\left.\begin{array}{l} Sq = Sc \quad (\text{ para } \phi u >= 25^{\circ}) \\ Sq = Sc - \dfrac{Sc-1}{Nq} \quad (\text{ para } 0 < \phi u <= 25^{\circ}) \\ Sq = 1 \quad (\text{ para } \phi u = 0^{\circ}) \\ Sc = 1 + (0.2 + \tan^{6}\phi u).B/L \\ S\tau = 1 + \frac{1}{2}(0.2 + \tan^{6}\phi u).\,B/L \end{array}\right\} \text{Shape factors}$$

$\bar{q}$ = 121.7 kN/m2; effective overburden pressure at foundation level

A = B.L; effective area at foundation level

The solution of these equations requires the knowledge of basic soil properties, namely, the cohesion and the angle of internal friction.

The scarce number of sample points, in conjunction with the inhomogeneity of the soil, rendered any effort to fit a probability density function to the data set unfeasible. For this reason, simple triangular or rectangular density functions were adopted (according to Ref. 4), limited by the judgement-supported minimum and maximum values. The basic soil parameters were finally characterized by an angle of internal friction with mean value equal to 4 degrees and VC of 9%, and a cohesion c with mean 100 kN/m2 and VC of 7% (See Fig. 5.1 and 5.2).

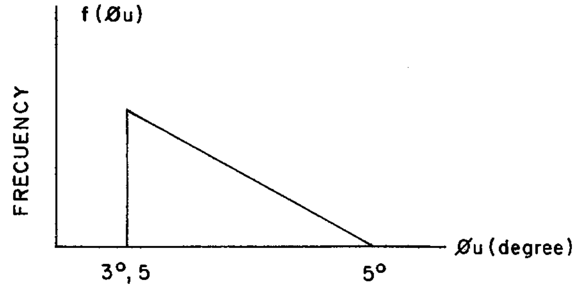

FIG. 5,1

FRICTION DENSITY FUNCTION

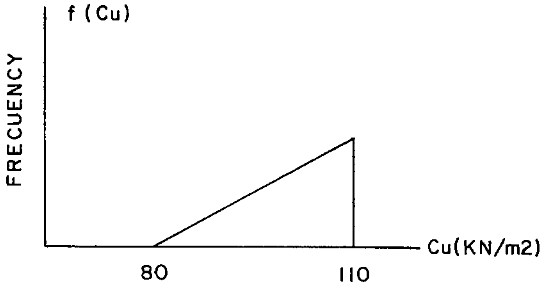

FIG. 5,2

COHESION DENSITY FUNCTION

The applicability of the above formulation for short duration, impulsive loading has no adequate experimental support. However, in view of the fact that no alternative approach was readily available, the impulsive nature of the excitation was taken into consideration simply by modifying the soil parameters. These changes, which will not be justified in this paper, implied a big reduction of the angle of internal friction as well as a 50% increase in the cohesion c.

Moreover, the formulations is based on the hypothesis of a homogeneous soil. Any possible layering or other inhomogeneities can only be handled in approximate manner through the use of modified soil properties.

In view of the complex nature of the above equations, the density function of the strength was evaluated, as outlined Ref. 4, by simulation. Series of random values of the governing variables were generated assuming no correlation between cohesion and friction, which led to the probability density function of the horizontal resistance shown in Fig. 5.3.

6. PROBABILITY DISTRIBUTION OF THE LATERAL FORCE "S"

The pressure vs. time curves referred to in Section 3 were specified as deterministic criteria, without any reference to their uncertainty or variability. A review of the available literature indicated that, under appropriate although unlikely conditions, larger peak pressures may occur (Ref. 6). Thus, the peak pressures were assumed to be clipped normal distributions, centered at the design values (Fig. 6.1).

The most unfavorable loading case, used for the analysis includes pressure on the lateral wall and dead load.

The pressure distribution was nevertheless assumed independent of the intensity level and also uniform. It should be pointed out that the peak values of the presure-time diagram at different locations on the structure should occur at slightly different times, resulting in a smaller total effect.

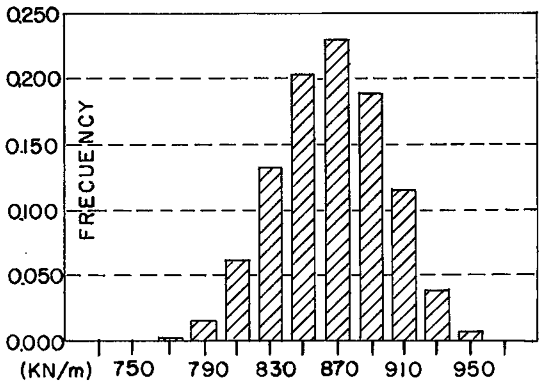

FIG. 5,3 RESISTANCE (H) - DISTRIBUTION

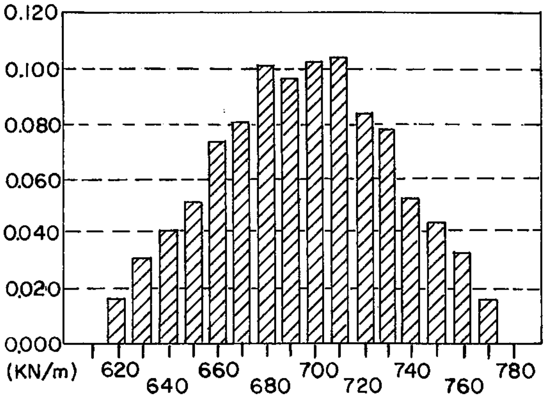

FIG. 6,1 LATERAL FORCE (S)-DISTRIBUTION

7. CONDITIONAL PROBABILITY OF FAILURE

Fig. 7.1 presents the probability density function of both the external excitation and the resistance of the foundation. The computed failure probability in terms of the number of simulations has been plotted in Fig. 7.2. It may be seen that above about 500.000 simulations, the computed value stabilizes in 1.5E-05.

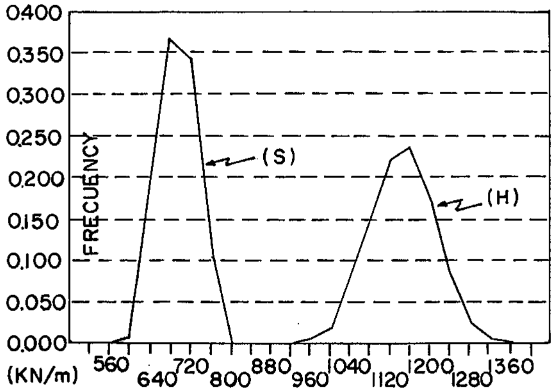

FIG.7,1 S and H DISTRIBUTIONS

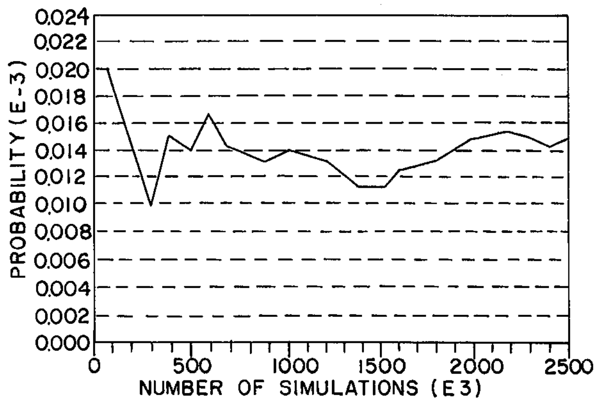

FIG.7,2 PROBABILITY VS. NUMBER OF SIMULATIONS(E3)

The conditional probability of failure obtained was 1.48 E-05.

8. CONCLUSIONS

On the basis of available information, it may be established that the probability of occurrence of a pressure wave at the site is less than 1E-03 per year (Ref. 3). Accordingly, the probability of failure of the foundation due to a pressure wave does not exceed 1.5E-08 per year, value that is considered admissible by current standards.

The use of triangular probability density functions for some of the variables, in particular soil material properties, proved valuable by allowing the introduction of engineering judgment in situations in which there was scarce instrumental data available.

It should be stressed that little attention was paid to computational aspects of the failure probability, which was accomplished by a very simple and flexible direct simulation process, on account of the large model-related uncertainties discussed in the text, such as the variability of the peak pressure and the dynamic (impulse) foundation response.

9. ACKNOWLEDGES

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