The Antiseismic Design of a Water Refuelling Tank for a Nuclear Power Plant

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SUMMARY

As part of the reevaluation of the seismic safety of the Beznau plant the water refuelling tank, which was reclassed in the category I, had to be checked for the SSE event. The dimensions of the tank -23 m high founded on a slab 10.3 m diameter and 1 m thick at depth 0.9 m in a gravel layer - was such that soil-structure interaction effects had to be considered in evaluating the overturning moments, as the fixed base fundamental period of the structure was increased by about 30%. The problem presented various complexities: influence of adjacent heavy reactor building; nonlinear behaviour of soil affecting shear moduli for the soil/rock layers; possible uplift of tank held down by gravity forces only, modelling of structure-soil system for dynamic analysis and seismic risk/input study.

Various methods of analysis are possible with available computer codes: linear, equivalent-linear, nonlinear, axisymmetric, quasi 3-D, 3-D (structure attached via rigid disc to half-space), etc. Special attention must be given to the interface base mat/ground, since uplift acts as a kind of safety valve with respect to overturning and horizontal acceleration response behaviour. Its inclusion, however, makes the problem nonlinear necessitating a solution in the time domain. The beneficial effects of uplift are exemplified in this study. Uplift and tilting of the base, however, raise the further problem of the existence of local plastic deformations and exceeding the bearing capacity of the ground, which is also discussed.

1. Introduction

This paper describes briefly the seismic safety analysis of the water refuelling tank for the Beznau nuclear power plant in Switzerland. When the tank structure was designed to resist earthquake loading back in the mid-sixties a pseudo-static type of analysis was carried out. With the subsequent development of safety philosophy the need for a reevaluation of the seismic safety of the various parts of the plant has arisen. The refuelling tank, due to its emergency cooling function, has now been placed in the safety category I corresponding to the SSE.

According to general practice the structure must be safe with respect to sliding, overturning, excessively high stresses in the tank walls and in the anchor bolts. Here, however, only the aspect of overturning is treated. As this condition is approached and substantial separation between the base mat and the ground occurs the question of large settlements associated with local bearing capacity failure arises. In regard to the latter, safety regulations tend to be somewhat vague, but in this case the question was given consideration.

1.1 Local site conditions

The plant is situated close by the river Aare. The bedrock lies at a depth of 13-16 m below the ground surface. It consists of an opalinus mudstone, the upper 4 m of which are weathered. The overlying material is a well-graded river gravel. The foundation of the reactor building lies at a depth of 16 m, and the adjacent water tank is founded on the recompacted gravel. The ground water table lies at a depth of about 5.5 m. To determine the elastic modulus values to a depth of 30 m a cross-hole seismic campaign was carried out. Samples of rock and of compacted gravel were also tested in the laboratory. The field test provides the value of the shear modulus (G_{max}) at low strains ($\gamma < 10^{-4}\%$), while the degradation of shear modulus with strain was investigated with the cyclic triaxial apparatus.

1.2 Description of Tank

The tank consists of a steel cylindrical shell 22.9 m high and 9.2 m diameter. The wall thickness is variable as shown in Fig. 1. The roof thickness is 5 mm and it is stiffened with longerons etc. with a central vent 0.56 m dia. The presence of the roof is insignificant for overturning moments. The base mat is 10.3 m dia. and 1.0 m thick embedded 0.9 m in the ground.

1.3 Seismic Input

Based on seismic risk studies the site intensity was determined as I = VII.6 on the MSK-64 scale. The corresponding horizontal and vertical peak acceleration components were estimated as 0.15 g and 0.10 g respectively. These values were increased by a factor of one-third to account for the influence of the heavy reactor building adjacent to the tanks. At a later stage this influence was investigated using the computer program FLUSH /1/. The main problem here was to simulate the 3-D nature of the system, representing the axisymmetric reactor structure with equivalent beams. In the first analysis below 3 m depth of rock the ground was assumed rigid. This model showed only a slight difference in the free field peak ground motion. This was in retrospect to be expected as the mass of the base mat (3 m thick) and internal components was placed almost directly on the rigid base, and the wall of the reactor building was comparatively rigid. A further analysis was carried out with a rigid base at 47 m depth. In this case the surface acceleration at the location of the tank was 0.16g. Thus the assumed increase of peak acceleration (1/3) was found to be on the conservative side.

Artificial seismograms of 30 s duration constructed to comply with the NRC design spectra were employed in these and the subsequent analyses. The horizontal acceleration spectra are shown in Fig. 2.

2. Methods of Analysis

In the given problem soil-structure interaction effects are not negligible. The relative soil stiffness parameter, Veletsos /2/, i.e. $c_s\ T/h\ (r/h)^{1/4}$ where c_s = shear wave velocity in soil, T = natural period of fixed base structure, h = effective height of structure and r = radius of rigid base mat, equals approx. 4.5 < 8, the limit proposed by Veletsos. The structure not only modifies the free field motion in its vicinity but its own vibrational behaviour is affected, i.e. (i) modification (reduction, /2/) of the "eigenfrequencies", (ii) radiation of energy from structure to ground (usually net increase of effective damping, /2/), (iii) appearance of rotational components of motion at base of structure. A further consideration is a partial lift-off of the base mat. For a rigid base solutions for tilting are easily obtained, Meek /3/. This phenomenon is important, especially in the consideration of overturning behaviour. Putting aside the question of soil bearing pressure the influence is generally beneficial. In the context of soil-structure interaction, however, uplift renders the problem non-linear.

2.1 Linear Analysis

Linear elastic analysis is possible if the material laws for the structural members and the ground are linear and provided the base mat is contiguous with the ground, i.e. no lift-off occurs. Here the dynamic analysis was carried out using the axisymmetric program ASHSD 2 /4/. A free field deconvolution analysis using the program FLUSH, for 0.2 g peak acceleration at the surface, was carried out to find the bedrock acceleration at 16 m depth with iterations on the soil properties to obtain approximate shear moduli in the nonlinear range for the soil/rock divided up into 5 layers (see Table I), together with an compatible time history of acceleration at the interface weathered rock/sound rock. The whole system (structure-soil layers) was then discretized and a modal analysis was carried out. The first 5 mode shapes for horizontal excitation are shown in Fig. 3. The distribution of acceleration is given in Table II. To account for the impulsive force generated by the interaction of the compressible fluid with the flexible tank Fischer /5/ was followed, though in this case practically the total water mass was lumped to the shell structure. The first 5 eigenfrequencies for the fixed base were 4.16 , 8.92 , 12.3 , 16.6 and 23.1 Hz. For the soil-structure system they are 3.08 , 5.74 , 6.33 , 8.06 and 8.74 Hz. The fundamental frequency is reduced. The second mode (see Fig. 3) corresponds to the motion of the ground, i.e. for a layer $f_o = c_s/4D$ where D = thickness of layer. Mode 5 is mainly a motion of the roof, as also mode 6 (not shown).

The maximum longitudinal stress in the shell was 96,500 kN/m^2 at the base due to the horizontal excitation only. The safety factor against overturning based on the corresponding maximum moment is 0.72. The simultaneous consideration of vertical ground excitation would reduce this value even further. It must be taken into consideration that the time history of horizontal acceleration and overturning moments are of an extremely fluctuating nature. Further, the results of the analysis are not altogether conclusive because of the "welded" contact assumption.

2.2 Nonlinear Analysis

In this phase of the investiation only the nonlinear behaviour due to the separation of base mat and ground (so-called tensionless half-space) was considered. A program developed by Wolf and Skrikerud /6/ was used for the purpose. The tank, modelled as a 1-D lumped mass (stick) system, is attached to base springs and dashpots derived from 2 layer half-space impedance function theory, Luco /7/. The model is shown in Fig. 4. The spring/dashpot constants are assumed to be frequency-independent, chosen at the fundamental frequency of the structure-soil system, Rönnberg /8/. In this case the spring constants are very nearly equal to the static values. When the base mat lifts off an equivalent reduced contact area is assumed, enabling the corresponding spring/dashpot constants to be reevaluated. For the gravel layer an average G-modulus is assumed. Horizontal and vertical motions were input simultaneously. The results of the analysis are shown in Fig. 5 for horizontal acceleration and overturning moment. Fig. 6 shows the time history of the eccentricity e of instantaneous vertical load. The ratio e/r is just below the limiting value of 0.91 (F.S. = 1.1). The maximum soil pressure corresponded to a value of e = 4.62 m (cf. e_{max} = 4.64 m), namely q_{max} = 4,960 kN/m^2, with an effective disc radius of 1.02 m. Generally, even in the extreme case of tipping about the edge, the angle of rotation is small, Dalal et al /9/. It is then appropriate, however, to investigate the bearing capacity problem for foundation instability.

3. Soil Bearing Capacity Considerations

Several investigators (e.g. Taylor et al /10/) have considered a Winkler-type foundation with elastoplastic springs to represent plastic behaviour in the ground under rocking action. The difficulty here is assigning realistic parameters to the individual springs, cf. Dawson /11/. An alternative approach is to apply limit equilibrium analysis using the dynamic bearing capacity equation in which the inertia of the sliding block is included. The system has a single degree of freedom with Coulomb damping. Solutions exist for centrally applied pulse-type loading, e.g. for cohesionless material Wallace /12/. For rocking motion, however, the problem seems to be intractable due to the continually changing eccentricity e. The only feasible approach would be to resort to FE/finite difference methods (cf. Hoeg and Rao /13/). Both experimental /11/ and theoretical /13/ studies would indicate that for rapid (impact) loading punching shear is the preferred mode of failure. For rocking motion shakedown is also likely to characterize the actual behaviour /10/.

As a first approximation the static bearing capacity equation, which is conservative due to the neglect of the inertia term, may be used. Meyerhof's reduced area concept /14/ appears to give satisfactory (slightly conservative) results from the experimental viewpoint. For a non-cohesive soil the bearing capacity is

$$q_D = \gamma D N_q \lambda_q d_q i_q + 0.5 B' N_\gamma \lambda_\gamma d_\gamma i_\gamma$$

where $B' = 2(r-e)$; γ = unit weight of soil (slip zone assumed above ground water); d, i, λ correction factor for depth, inclination of load, shape of footing; N = bearing capacity coefficients. For ultimate soil friction $\phi' = 43^o$ (for gravel) N_q and N_γ (circular footing) are 150 and 260 respectively. Under extreme loading the contact area becomes more strip-like and so the lower values of 90 and 190 were adopted for large e. In the range $1.1 \text{ m} < e < 4.8 \text{ m}$ λ_q and d_q ($= \lambda_\gamma$ and d_γ resp.) vary from 1.42 to 1.03 and 1.03 to 1.31 respectively. The calculated value of $q_D = 5050 \text{ kN/m}^2$ giving a safety factor of 1.02. Considering that conservatism has been introduced at a number of places in the seismic evaluation it may be regarded as a satisfactory value.

4. Conclusions

The parameters of the problem (geometry, flexibility of ground, loading conditions) necessitated the consideration of soil-structure interaction effects. The results of the seismic safety evaluation illustrate the well-known fact that if lift-off of the base slab is not allowed (assumption of tensile stresses at the ground/slab interface) the analysis approach is too conservative. The question of plastic deformation in the ground due to local bearing capacity failure as the base slab rests on its edge, punching failure or shakedown requires further investigation, preferably with a fully nonlinear material model for the soil - perhaps at the research level, but for design purposes (pseudo-static) limit equilibrium analysis should suffice. For large mat foundations the flexibility of the mat may have a significant influence on the results and this aspect should also be investigated.

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Table I Material properties from deconvolution analysis (FLUSH)

Depth (m)	Poisson's ratio	G_{max} (x 1000	G_{used} kN/m^2)	Damping %
0 - 3	.35	179.6	148.5	4.7
3 - 6	.35	314.8	245.1	5.4
6 - 9	.35	362.6	271.2	5.9
9 - 13	.35	427.0	310.8	6.2
13 - 16	.40	501.8	389.0	3.7
16 - 20	.40	928.6	750.0	3.4
.	.	.	.	.
.	.	.	.	.
49.5 - 58	.40	2009.	1605.	3.4

Table II Horizontal accelerations from ASHSD2 analysis (maxima occur at different times)

Height (m)	Absolute Max. Acceleration (m/s^2) Total	Relative
22.9	- 14.9	- 14.1
16.0	- 10.4	- 9.57
12.0	- 7.70	- 6.90
6.0	- 4.23	- 3.46
0.	+ 1.90	+ 1.12
free field	+ 1.97	+ 0.89
Bedrock	+ 1.63	0.

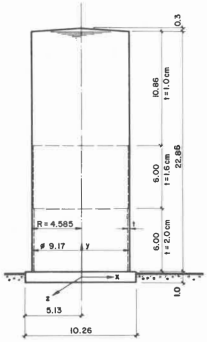


Fig. 1 Tank

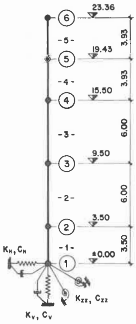
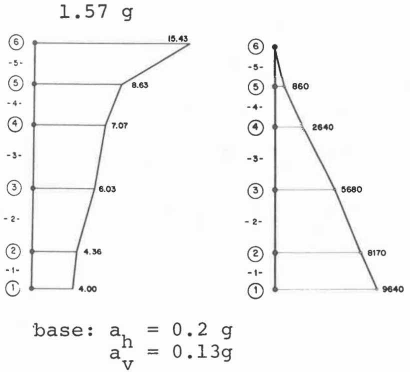


Fig. 4 Model



(a) Acc. (m/s^2) (b) OM (t.m)

Fig. 5 Response Maxima for Nonlinear Analysis

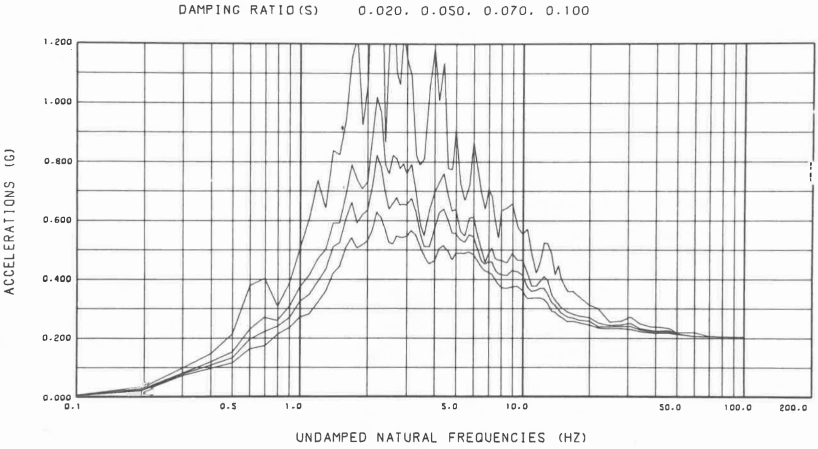


Fig. 2 Acceleration response spectra for the artificial horizontal accelerogram fulfilling the NRC criteria

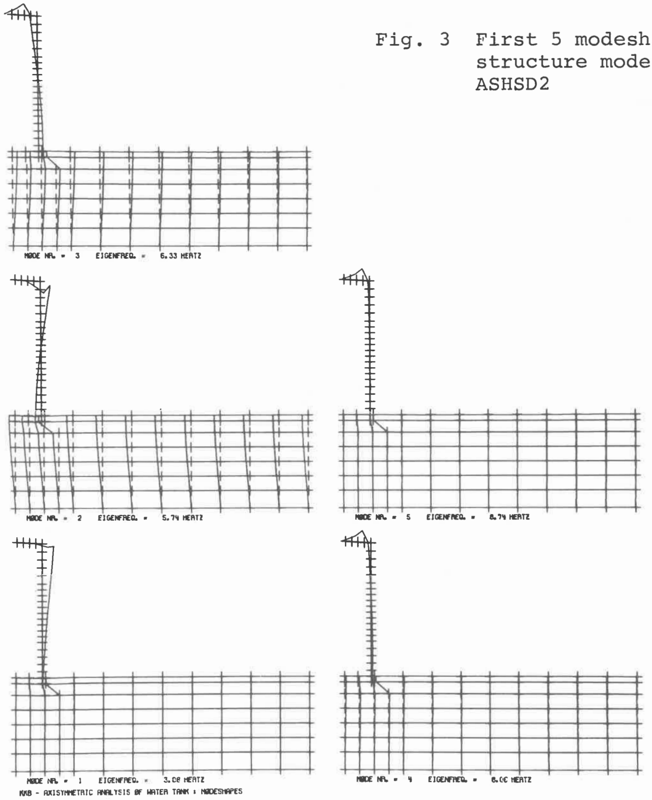


Fig. 3 First 5 modeshapes for soil-structure model using program ASHSD2

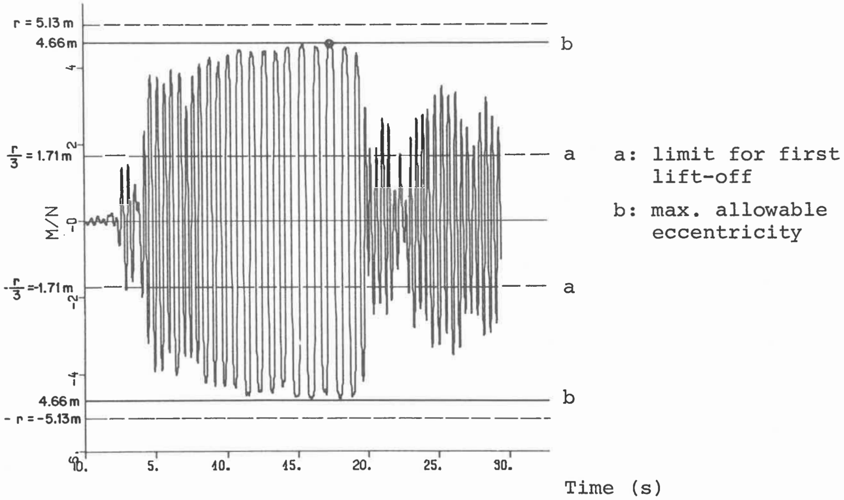


Fig. 6 Eccentricity of base mat (Moment/Vert. Force in Node 1, Fig. 5)