SEISMIC ANALYSIS OF A DEWATERING SYSTEM

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SUMMARY

A dewatering system, consisting of a flexible diaphragm wall and a series of deep wells, has been analysed to verify its ability to withstand earthquake loadings. In order to check that the diaphragm wall would perform satisfactorily, a stress analysis was performed. This analysis showed that the strains in the diaphragm would be equal to the free-field strains of the surrounding soil and that the diaphragm stresses could be calculated using the elastic properties of the wall material. The design of the deep wells, including seismic analyses of the well casing, screen, and supports for the pump and discharge pipe are also described.

INTRODUCTION

A dewatering system has been designed for and installed at a nuclear power plant in southern Europe. The components of the system were designed on the basis of construction, geotechnical and hydraulic considerations. In addition, the dewatering scheme was analysed for seismic loading conditions to ensure that it would remain functional during and after the 0.24 g Safe Shutdown Earthquake (SSE) postulated for the plant site. The following sections describe the site conditions as well as the design and seismic analyses of the dewatering system's various components.

SITE DESCRIPTION

The plant is located within a river floodplain and is founded on deep alluvial soils. Extensive subsurface investigations identified four distinct soil horizons at the site (Fig. 1):

- A three to four-meter thick layer of topsoil and silty clay over,
- An 18-meter thick horizon of sand and gravel, within which the main plant buildings are founded. This horizon is underlain by,
- A five to seven meter thick layer of silty clay which overlies,
- A formation of sand and gravel which has a thickness of at least 125 meters and which contains lenses of clay.

The upper layer of silty clay was completely removed in the main plant area and was replaced by ten meters of sand in order to bring the plant grade 2.5 meters above the 100-year flood level of the nearby river (600 meters distant). The main plant buildings are founded about 15 meters below plant grade. Since the building foundations are below the normal groundwater level within a previous layer of sand and gravel, the dewatering system was installed at the start of construction and consisted of two main components:

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- A 23-meter deep, 45-centimeter thick, plastic cement/bentonite diaphragm wall, and
- A series of 21-meters deep, one-meter diameter wells within the area enclosed by the diaphragm wall.

The diaphragm wall, constructed from the original ground surface, extends two meters into the layer of silty clay beneath the upper sand and gravel formation. In this way, the lateral and upward flows of groundwater into the plant excavation were reduced by the diaphragm wall and the silty clay layer, respectively. The wells, which extend to the bottom of the sand and gravel horizon, served to remove the pore water from this horizon and any seepage which came through the diaphragm wall and the silty clay. While these deep wells had satisfactorily dewatered the plant excavation during construction, they are built of carbon steel and have been subjected to corrosion and encrustation due to the chemical constituents of the groundwater. As a result, it was not possible to determine, or to have confidence in, the continued performance of the well casing and screens and, therefore, new wells were designed and will be installed.

SEISMIC ANALYSIS OF EXISTING DIAPHRAGM

The frist step in the seismic analysis of the dewatering system was to check if the diaphragm could withstand the postulated seismic forces, i.e., to perform a stress analysis of the diaphragm under SSE loading conditions. To this end, four analyses were performed. The first three of these analyses indicated that strain compatibility between the soil and the diaphragm exists, i.e., the diaphragm can be considered as completely flexible, since soil-diaphragm interaction effects are negligible during an SSE and the wall will move with the deformations experienced in the free-field. Under these conditions, the free-field soil strains can be used to compute stresses in the diaphragm wall by employing the appropriate elastic properties.

Evaluation of Soil-Diaphragm Interaction

The first analysis of the diaphragm wall was performed by considering it as a structural member surrounded by soil. The most unfavorable loading condition on the wall would be induced by that portion of the seismic time history which includes the maximum acceleration. The analysis was carried out by considering an acceleration pulse with a parabolic shape, to represent the shape of the peak in the seismic time history, with a maximum acceleration value of 0.24 g (Fig. 2A). By integrating the equation of this acceleration pulse twice, the soil displacements are obtained from the following equation:

$$y_{soil} = -\ddot{y}_{max}\ \Delta t^2 \quad \frac{5}{48} - \frac{1}{2}\left(\frac{t_s}{\Delta t}\right)^2 + \frac{1}{3}\left(\frac{t_s}{\Delta t}\right)^4 \qquad (1)$$

where:

$\ddot{y}_{max}$ = the SSE peak horizontal ground acceleration

Δt = the time interval over which the acceleration pulse occurs, and

t_s = time measured from occurrence of $\ddot{y}_{max}$

Predominant earthquake frequencies are generally between one and five Hertz; as shown by studies of recorded earthquake accelerograms the maximum amplification occurs in this frequency range (Newmark et al., 1973). Hence, the time increment in the computation of displacements using Eq. 1 was taken as one-half of the period corresponding to the predominant earthquake frequencies, or 0.1 to 0.5 seconds. The free-field displacement of the soil thus obtained, varied between zero and six centimeters.

The purpose of the second analysis was to compute the load on the diaphragm wall due to soil-diaphragm interaction by determining the deflections of the wall as a result of the deflection of the soil (Fig. 2B). The load on the diaphragm is obtained from:

$$\text{Diaphragm load} = (y_{soil} - y_{diaph})K_h \tag{2}$$

where:

y_{diaph} = the wall deflection which is computed using an iterative procedure (Newmark, 1943)
K_h = the coefficient of subgrade reaction of the soil

A solution for stub piles presented by Richart et al. (1970), shows that the coefficient of subgrade reaction (K_h) is approximately equal to four times the shear modulus of the soil divided by the width of the pile. This solution has been adopted herein by considering one meter wide segments of the diaphragm wall. The shear moduli of the site soils were determined using the Hardin-Drnevich (1972) equation.

The results of the second analysis have shown that essentially zero loads are induced in the diaphragm wall under SSE conditions. This is explained by the fact that the wall has essentially the same stiffness as the surrounding soil. Thus, it is concluded that the diaphragm wall displacements are equal to the free-field soil displacements and therefore, the maximum deformation of the diaphragm during an SSE will be on the order of six centimeters.

The maximum bending moment in the diaphragm was then computed by a third analysis. Newmark and Rosenblueth (1971) have shown that for the condition of strain compatibility, the maximum curvature experienced by flexible buried structures is proportional to the maximum ground acceleration and inversely proportional to the square of the soil wave velocity, Therefore:

$$\text{Maximum Moment} = \frac{\ddot{y}_{max}(EI)}{V^2} \tag{3}$$

where:

$\ddot{y}_{max}$ = the SSE peak horizontal ground acceleration
E = Young's modulus of the diaphragm material
I = the moment of inertia of the section
V = the seismic wave velocity

Using this equation and the shear wave velocity of the soil derived from the shear modulus, the moments induced in the flexible diaphragm wall were found to be negligible. Indeed, using a Young's modulus of 1,500 kilograms per square centimeter for the diaphragm, a maximum moment of 0.002 ton-meters per meter length of wall, was computed. This Young's modulus is an upper bound value obtained from laboratory unconfined compression tests performed during construction of the diaphragm.

The above analyses showed that full strain compatibility exists between the diaphragm and the surrounding soil and, therefore, the following analysis, considering free-field soil strains, was used to calculate the stresses in the diaphragm wall.

Free-Field Analysis

The free-field shear stress and shear strain time histories for the soil profile were computed from a solution of the one-dimensional wave equation, using an artificial time history matching the USNRC Regulatory Guide 1.60 Response Spectra for a peak acceleration of 0.24 g. The computer code SHAKE (Schnabel et al., 1972) was used for this computation, together with the modulus and damping versus shear strain relationships obtained by using the equations presented by Hardin and Drnevich (1972).

As strain compatibility applies, the strains calculated by the SHAKE analysis were used to compute the shear stresses in the wall by multiplying the strains by the shear modulus of the cement/bentonite diaphragm. These shear stresses were compared with the shear strength of the diaphragm wall material to determine the margin of safety against shear stress failure. Using long term properties based on the unconfined compression test results, the shear strength of the diaphragm was estimated to be 2.5 kilograms per square centimeter. The computed shear stress level in the plastic diaphragm under SSE conditions is less than 15 percent of the maximum shear strength, and can be regarded as insignificant.

As a final consideration, the maximum tensile stress which could be induced by compression waves traveling through the site soils is about 0.7 kilograms per square centimeter. However, below a depth of 3.5 meters, the existing overburden pressure is greater than this value and, therefore, no tensile stresses can be developed below this depth. The top of the diaphragm is located 6.5 meters below final plant grade and thus, the diaphragm cannot be subjected to tensile stresses.

From the preceeding results, it can be seen that the diaphragm will be subjected to only negligible stresses and, therefore, it is expected to behave satisfactorily under SSE conditions.

DESIGN OF DEEP WELLS

In order to control the groundwater table within the diaphragm, six deep wells will be installed down to the bottom of the upper sand and gravel layer. Each 29-meter deep well will have a diameter of one meter within which a well screen and casing with a diameter of 40 centimeters will be centered and surrounded by a filter pack. To prevent corrosion, the casings and well screens will be of stainless steel. Furthermore, thin PVC tubes will be installed around the perimeter of the filter pack to allow chemical treatment of the well screens.

Seismic Analysis of Deep Wells

In order to assure proper operation of the deep wells during or subsequent to the postulated occurrence of an SSE, analyses of the well casing and screen were performed to determine a suitable casing wall thickness. The basic input to the analyses consisted of the soil profile and shear wave velocities derived from the Hardin-Drnevich (1972) equation. The dynamic Poisson's ratio was taken as 0.45 below the groundwater table and as 0.35 above. The unit weight of the soil is 2.0 tons per cubic meter, while a coefficient of earth pressure at rest equal to 0.7 was utilized. The stainless steel well casing and screen have a diameter of 40 centimeters and lengths of 23.0 and 6.0 meters, respectively.

The method of analysis used (Nazarian, 1973) accounts for both the effects of seismic shear and compression waves which produce stresses in the casing and well screen. Seismic waves traveling through the soil and striking the surface of the well induce shear, axial and bending stresses. The maximum stress occurs in the longitudinal direction as shown by Sakurai and Takahashi (1969). The first consideration in this analysis is the computation of the maximum hoop stresses in the casing due to lateral at rest earth pressures, considering also buoyancy and hydrostatic effects of the groundwater. The basic premise of the approach is that dynamic stresses are superposed on the at rest earth pressures during a seismic event.

Next, the maximum axial strain in the casing resulting from seismic compression wave propagation is computed. The axial strain relation due to compression waves assumes that the surrounding soil holds the casing rigid by friction. The equation given by Sakurai and Takahashi (1969) computes an axial strain based on the critical period of ground movement, compression wave velocity of the soil, and maximum ground acceleration. Accordingly, the strain computed for the well casing is a conservative value. If slippage occurs between the soil and casing, the actual maximum strain in the casing will be due to the total frictional force between the soil and the casing and, therefore, it will be less than that given by the Sakurai and Takahashi (1969) equation.

Seismic waves also produce axial strain in the casing due to stretching of the casing as outlined by Kuesel (1969), who presented equations for computing maximum strains due to distortion of the ground. This analysis conservatively assumes that all strains produced in the well casing act simultaneously. The maximum possible shear stress is computed including axial, hoop, and shear stress in the casing due to distortion of the soil. Principal compressive and tensile stresses were evaluated and factors of safety, based on a yield strength of 2,800 kilograms per square centimeter for the six millimeter thick stainless steel casing, were computed. These factors of safety had minimum values of three both in tension and in compression. A similar analysis for the well screens, using the dimensions and material properties provided by the manufacturer, also yielded a factor of safety of three.

Seismic Analysis of Supports for Pump and Discharge Pipe

Inside the well casing and screen, the submersible electric pump and the vertical discharge pipe to the ground surface will be held in place by six sets of lateral springs. Each set of springs consists of four stainless steel leaf springs arranged on two mutually perpendicular axes. The spring sets will be fabricated to fit accurately within the casing. This support system is designed to keep the pump and discharge pipe centered within the casing and to prevent damage to the pump and the well screen which might occur if the pump and discharge pipe were free to move during the postulated SSE. The locations of the springs and the lumped parameter model used in the seismic analysis of the pump-pipe-spring system are shown in Fig. 3. The two lowest sets of springs are attached to the top and bottom of the pump while the remaining four sets are connected to the discharge pipe. The large gap between the second and third sets of springs from the bottom is to span the six-meter long well screen.

The seismic analysis of the well casing and screen has shown that interaction effects between the wells and the surrounding soil can be conservatively ignored. Therefore, the input to the dynamic analysis of the pipe and pump support system consisted of the free-field soil motions under the postulated SSE. Waves traveling vertically with the shear wave velocity of the soil were considered in the analysis, since this is the situation which provides the highest response in the system and better simulates the actual situation in case of an earthquake.

Since part of the discharge pipe and the pump will be surrounded by water, hydrodynamic effects were taken into account for the computation of the horizontal response. The presence of water can be included in the analysis in terms of added mass and damping (Chen et al., 1976) both of which vary with the ratio of the diameters of the external and internal cylinders, the frequency of excitation and the viscosity of the fluid. Total damping for the system was conservatively taken as five percent (Nazarian, 1973) for elements surrounded by water and two percent for elements not surrounded by water. Two percent damping was also considered in the calculation of the vertical response which is not influenced by the hydrodynamic effects. The computed displacements and stresses for the pipe, pump and supports considered the combined effects due to ground motions in two horizontal, mutually perpendicular directions (Hall et al., 1975).

The traveling wave dynamic analysis, using the computer code DAPSYS (D'Appolonia, 1977), yielded the maximum normal forces, shears and moments in the discharge pipe and forces in the springs due to the postulated SSE, as well as the maximum displacements of the pipe relative to the casing.

CONCLUSIONS

The design and seismic analysis of a dewatering system have been presented herein. The major difference between this system and a typical dewatering system is that all components were designed or verified and analysed to resist earthquake forces. In general, a flexible diaphragm wall and deep wells designed to meet static geotechnical and hydraulic requirements can be expected to withstand moderate earthquakes.

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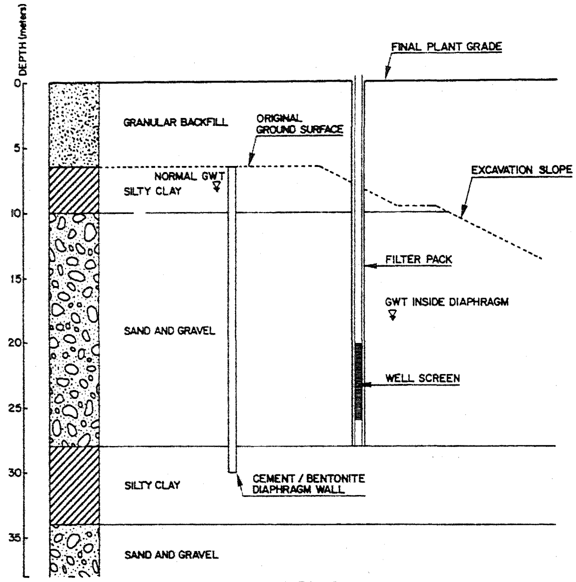

FIGURE 1

TYPICAL SECTION OF DIAPHRAGM WALL AND DEEP WELL

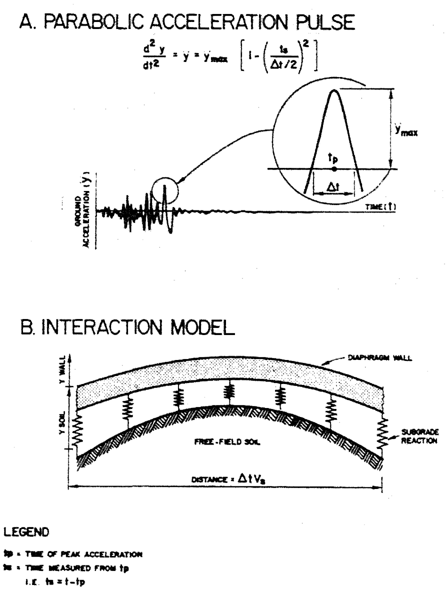

FIGURE 2

SOIL-STRUCTURE INTERACTION MODEL OF DIAPHRAGM WALL

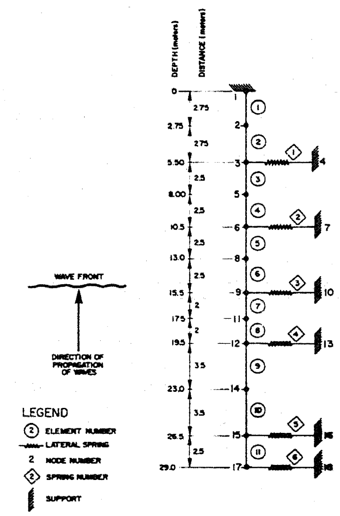

FIGURE 3

MODEL OF PUMP, CASING AND VERTICAL DISCHARGE PIPE