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DESIGN OF A PILED FOUNDATION IN A SEISMIC AREA AND SOIL PRONE TO LIQUEFACTION

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SUMMARY

The objective of the paper is to present the design of a piled foundation in a highly seismic area with poor soil conditions that are susceptible to liquefaction. The piled structure is a cooling water intake pit and is part of the new 390MW power plant designed and constructed by VA TECH Hydro on behalf of Hellenic Petroleum close to the city of Thessaloniki in Greece. The reinforced concrete pit has overall dimensions of 30m length, 7m width and 8.50m depth with a top elevation just above sea water level. It accommodates large pumps and auxiliary facilities that circulate the cooling water system of the power plant. The pit is located at the shore of the Gulf of Thessaloniki in a flat area with alluvial materials that are deposited there by several rivers discharging in the sea. Down to a depth of 25-30m below ground level, the soil consists of loose to medium dense soft clayey and silty-clayey materials. Layers of stiff or dense soil are only encountered at higher depths. Additionally, the lithological composition of the soil suggests that a mass liquefaction is possible during an earthquake. To avoid structural problems under normal and seismic conditions, the pit was designed with piled foundations. Although piled foundations provide a stiff and robust solution they raise the problem of how to design them in an area prone to liquefaction. The paper presents the major steps in the design of this structure, such as the seismic conditions in the area, the soil parameters, the requirements of the Greek seismic code and the various scenarios investigated to design the pit and its piled foundations.

1 INTRODUCTION

A new 390MW Combined Cycle Power Plant was built in the city of Thessaloniki by VA TECH Hydro on behalf of Hellenic Petroleum. The cooling water required to operate the power plant is collected from the sea by large pumps that are placed in the cooling water intake pit, a reinforced concrete box type structure. The soil conditions at the construction site of the pit at the shore of the Gulf of Thessaloniki are very poor. Hence, it was required to take special measures for both the temporary construction and final permanent situation.
At the construction site, two structures, the cooling water intake pit and the electrical building are placed besides each other (see Fig. 1). During construction of the pit, problems with the temporary cofferdam led to a further investigation of the soil characteristics. At this stage, it was found out that the soil might liquefy during an earthquake and it was decided to change the foundations of both structures from raft to piled foundations. The re-design was reviewed resp. carried out by the authors of this paper, and supervised and checked by E. Sotiropoulos and Prof. Dr. George Gazetas. As both structures are re-designed in the same way, further on only the structural design of the pit is discussed in detail.

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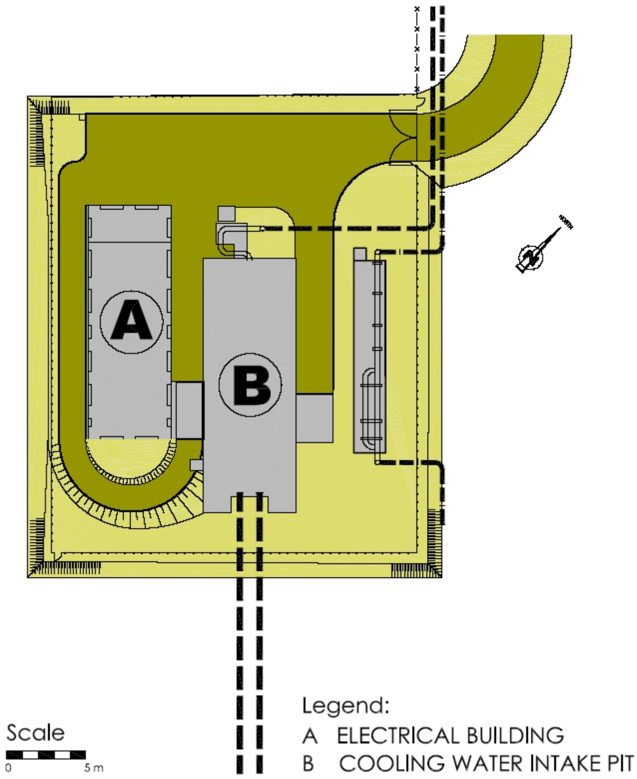

Figure 1: Layout plan of the cooling water intake area

2 DESCRIPTION OF THE STRUCTURE

The reinforced concrete pit is a box type structure with thick perimeter and internal partition walls. It accommodates large pumps and auxiliary facilities that circulate the cooling water system of the power plant. Overall dimensions of the pit are 30m length, 7m width, and a varied depth from 6.85m to 8.8m. The pit is at a distance of a few hundred meters from the shore of the Gulf of Thessaloniki with top elevation of +1.70m above sea water level. The pit has a complex geometry with miscellaneous openings and local extensions in order to serve its purpose. It has an 80cm thick bottom slab, partially a 30cm thick top slab and 50cm thick perimeter walls. All longitudinal and transverse internal partition walls are also minimum 50cm thick. It is designed as a water-tight structure, crack widths are reduced by providing sufficient reinforcement.

The pit was originally designed to rest on its bottom slab as a raft foundation without piles. Construction works began based upon the assumption that the raft foundation would be adequate. Problems with the temporary cofferdam led to further geotechnical investigations and studies that raised the problem of possible soil liquefaction during a seismic event. At this stage it was decided that piled foundations would be the most reliable and robust solution for the pit in order to satisfy both the client and the certifying authorities with respect to problems that may arise from liquefaction induced displacements.

12pcs Ø 1200mm piles with a length of 34.5m were provided on each side of the pit without modifying the original design. These piles with an average spacing of 2.75m were rigidly connected to the pit by means of a stiff ring beam with 1.50m height at 2.50m distance from the perimeter walls of the pit (centreline pile to centreline wall). It was decided that this was the most effective solution at the time being in order to avoid disturbing the construction programme of the pit. Introducing piles at the bottom of the raft foundation at this stage was practically impossible since construction of the pit was already under way. A longitudinal and cross-section of the pit are shown in Figure 2.

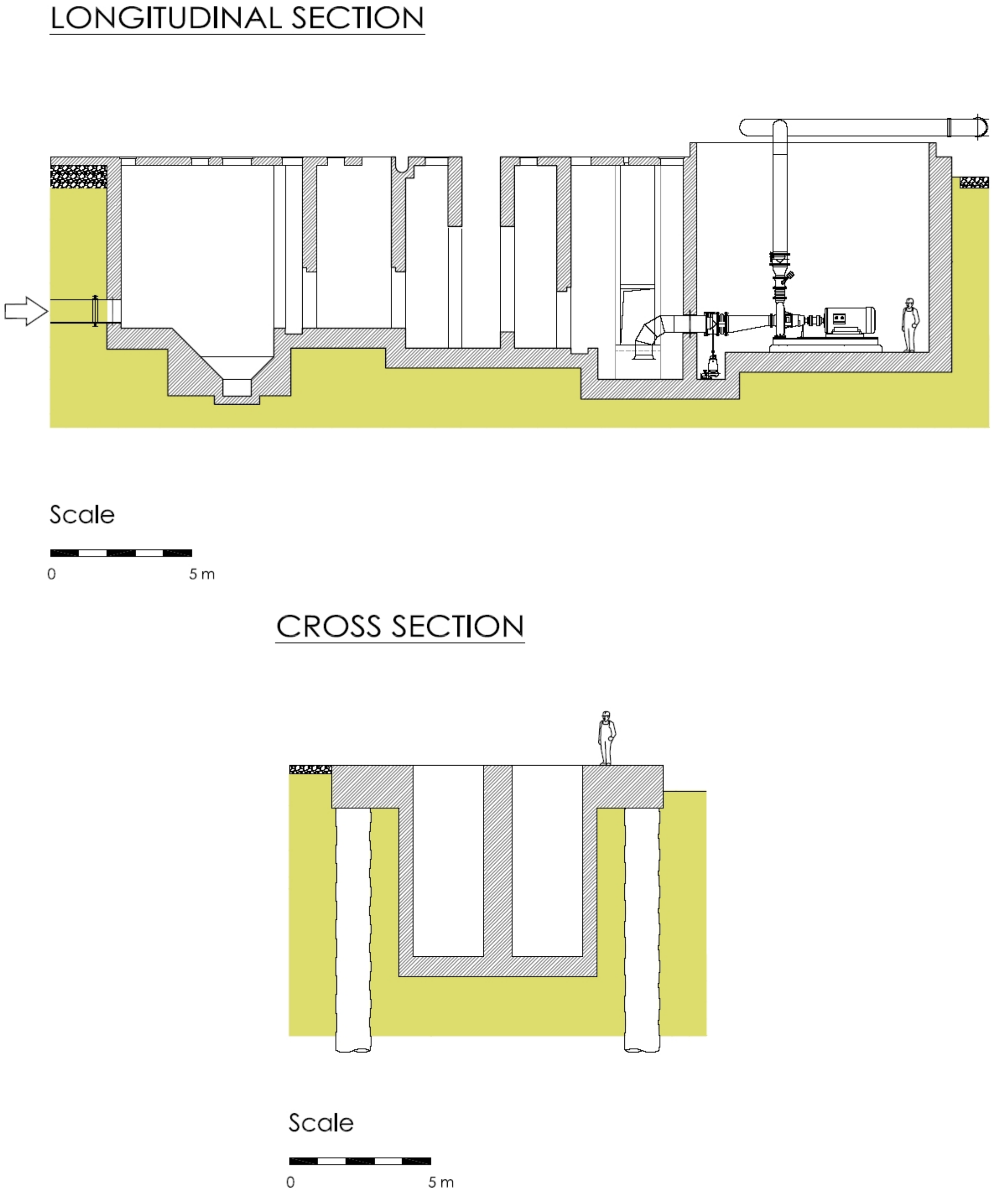

Figure 2: Longitudinal and cross-section of the pit

3 CODES AND STANDARDS

The design of the pit was performed in accordance with the contract following local codes, i.e. the Greek seismic code [EAK 2000] and the Greek regulation for reinforced concrete [EKOS 2000]. In principle, the Greek codes follow the guidelines of Eurocode 2 and Eurocode 8 with suitable additions and modifications to match the site conditions encountered in Greece. Where these codes were not sufficient, international codes and regulations were used. In the following, design criteria and rules of the Greek seismic code EAK 2000, relevant for the design of the pit are summarized.
EAK 2000 is a modern seismic code for the design of earthquake resistant reinforced concrete structures and focuses on the ability of structures to dissipate energy through large inelastic cyclic deformation without

substantial reduction of resistance. In order for this to be achieved, the whole structure needs to exhibit a ductile behaviour. The capacity design is one of the special demands of EAK 2000 by which the ductile behaviour is implemented. It intends to ensure the controlled damage of the structure. This is achieved by preventing the formation of a story mechanism, requiring beam failure before column failure and designing to avoid brittle shear failure of the building elements.

Regarding the seismic analysis, EAK 2000 specifies two methods of analysis: the Response Spectrum Method (dynamic) and the Simplified Spectrum Method (equivalent static method). The design seismic ground acceleration is defined at the free surface of the ground. According to EAK 2000, the area of Thessaloniki is classified in the seismic risk zone I of high seismicity and the design ground seismic acceleration is given to be A=0.16g. The design seismic forces depend upon the design seismic ground acceleration, the importance factor of the structure, the behaviour factor, the soil classification, and the foundation factor. The behaviour factor q controls the amount of non-linearity the structure is designed to accept. High values of q reduce seismic design forces but increase the capacity design requirements. The value of q=1 corresponds to an elastic response and the suppression of the capacity design requirements. For pile foundations, EAK 2000 proposes to remain within the elastic range and take q=1. This is the methodology adopted in the present design.

The walls of the pit are practically rigid walls embedded inside the soil. EAK 2000 proposes that the static at-rest pressures acting on such walls should be increased taking into account additional loading during an earthquake. These additional horizontal pressures are linearly distributed over the depth of the wall, with a maximum value at the ground surface equal of 1.5 $\alpha\ \gamma$ H and a minimum value at the bottom level of the wall equal to 0.5 $\alpha\ \gamma$ H. Hereby is α the normalized seismic ground acceleration, γ the unit weight of the soil and H the depth of the wall below the free surface, which need not to be taken larger than 10m.

Additionally, the code proposes that in very permeable soils (permeability $k > 0.50\ 10^{-3}$ m/sec) the seismic actions of the masses of soil and water are to be calculated independently and superposition of the results to be performed. In this case, the earth pressures calculated as above using the buoyant unit weight of the soil shall be increased by the hydrodynamic variation of the water pressure (see Equation 1):

$$p(z) = \pm\left(\frac{7}{8}\right)\alpha\psi_w\sqrt{Hz} \tag{1}$$

whereby z is the depth of the point under examination and γ_w the unit weight of the water.

In addition to the ultimate limit design EAK 2000 imposes a serviceability check to ensure that permanent displacements are compatible with the functional and aesthetic requirements of the structure. In the pit this ensures that the functionality of the pumps inside the pit will not be affected by displacements and rotations during a seismic event.

The previous summarized design criteria and rules are developed mainly for buildings and ordinary design conditions. For more complicated situations such in soils susceptible to liquefaction the code proposes more accurate methods for analysis and design following the consent and approval of the responsible Public Authority. The code requires the alternative methods of analysis to be based on well founded and recognized scientific principles in order to achieve the same level of safety as the one aimed by the code.

4 GROUND INVESTIGATION AND SOIL PARAMETERS

According to the geological map of the Hellenic Institute of Geology and Mineral Exploration, the subsoil in the vicinity of the project site consists mainly of alluvial material, which was deposited there by several rivers that are being discharged at the Gulf of Thessaloniki. These materials consist usually of soft to firm clay and silty-clay as well as loose to medium dense silty-sandy materials. Layers of very stiff or very dense soil are usually encountered at depths higher than 25-30m below ground surface.

A number of geotechnical investigations have been performed between the years of 2002 and 2004 (boreholes and cone penetration tests) indicating very poor geotechnical conditions in the project area [Geognosi, 2004]. Very loose to loose sandy to silty layers and very soft clay to silty-clayey layers are met in a depth around 24m. The groundwater table is very shallow due to the nearby presence of the sea. A typical stratigraphy of the area is summarized below (see Figure 3):

- From ground level down to 2m depth the subsoil consists of fill material.
- From 2m down to depth 8m below ground surface, the subsoil consists of very loose to loose grey to green clayey silty sand (layer SM).
- From 8m down to a depth of 11.4m below ground surface, the subsoil consists of grey-green organic silt to clayey silt of low to high plasticity with little sand content (layer M1).

- From 11.4m down to a depth of 24m below ground surface, the subsoil consists of grey-green organic silty clay to clayey silt of low to high plasticity, being in a very soft state with organics and shells (layer M2).
- From 24m to 27.85m below ground surface, the subsoil consists of light grey-green sandy clay of low plasticity, stiff with shells (layer CS).
- From 27.85m to 39.45m the subsoil consists of light brown sandy clay of low to medium plasticity, stiff to very stiff with oxidations and calcareous concentrations (layer CS1).
- Finally from depth 39.45m down to the end of soil investigation of 43.5m below ground surface, the subsoil consists of light brown-red clayey sand, dense to very dense with trace of gravel and oxidations (layer SC).

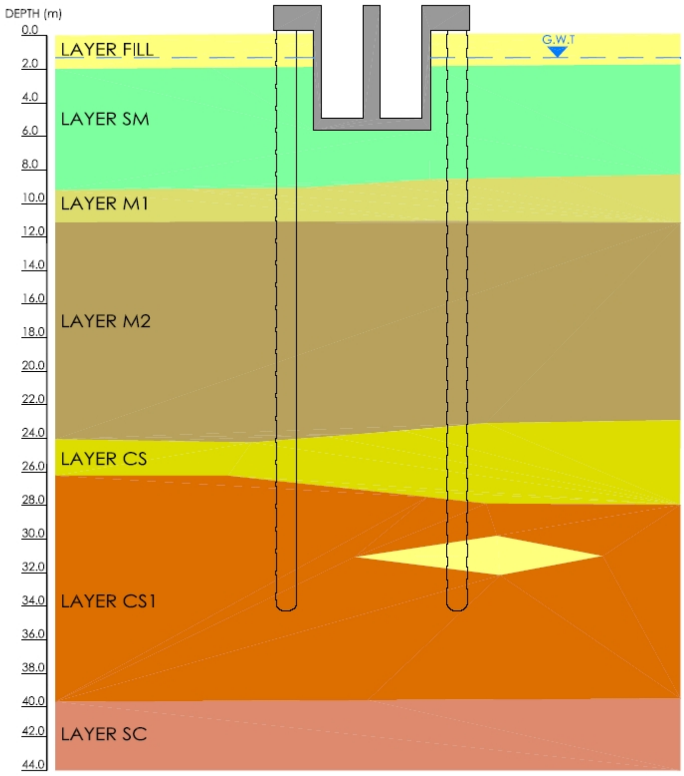

Figure 3: Typical soil profile

According to the findings of the geotechnical investigation, the soil type of the construction area is classified as soil category 'Γ' according to EAK 2000 (silty and clayey soils of low shear strength, having a thickness more than 5m). The geotechnical reports confirm a potential for liquefaction in both sand (layer SM) and silt (layer M) strata. Even if not liquefied, all sandy and silty materials of the subsoil being in a loose state are characterized as 'sensitive to earthquake' soil materials according to EAK 2000 and thus, they may undertake some loss of their initial shear strength during an earthquake due to the development of increased pore pressure. For this reason, EAK 2000 suggests to take into account a reduced value of the angle of internal friction φ_E for all seismic sensitive soils in the case of earthquake loading.

5 DESIGN ACTIVITIES

5.1 Geotechnical design issues

The problem of piles in lateral-spreading fields is very complex and it is very difficult to develop simple design guidelines to anticipate stresses and displacements for a structure inside a liquefied layer.
For weak seismic motions the induced shear strains in soils are low and the soil is treated as a conventional linear elastic material. This means the relationship of the stress and the strain in the soil is strain independent. In the initial weak motion phase, the surface soil responds linearly and the ground motion is dominated by high

frequency waves. In case of stronger seismic motions the soil performs nonlinearly. In particular, for saturated sandy soils subjected to strong cyclic loading under undrained conditions, the gradual built up of excess pore pressure will result in a significant degradation of soil stiffness. For the strongest part of the shaking and high pore water pressures the soil reaches the so-called liquefaction state and looses abruptly its stiffness. This results in a reduction of amplitude and in the lengthening of the predominant period of the surface ground motion.
The evolution of the soil dynamic response during an earthquake and the possibility of liquefaction depend upon the local soil conditions, the dynamic characteristics of the soil-structure ensemble, the frequency content and the amplitude of the seismic ground motion. Additionally, the distance from the seismic source and the magnitude of the event are important. Numerous studies concerning liquefaction have been published in the past and some of these findings are summarized next.
[Yang, Sato and Li, 2000]; [Popescu, 2002] and [Madabhushi and Schofield, 1993] examine the influence of local soil conditions on the amplification of seismic waves and the damages they cause to structures. All of them conclude that the frequency content of the ground motion has important implications on the dynamic response and that the interplay between the frequency content of the seismic motion, the vibration characteristics of the structure, and the possible evolution of those characteristics during the shaking has significant influence on the predicted dynamic response. [Popescu, 2002] and [Madabhushi and Schofield, 1993] also state that special attention should be paid to low frequency seismic inputs since the frequencies of the system may decrease during dynamic excitation, due to degradation of the effective shear module of the soil as a result of pore pressure build-up and large shear strains. In this case the frequency of the structure-soil system may come close to the seismic input frequency and thus, resonance conditions result in amplifications of the structural response. [Yoshiaki, 1977] and [Jun-Tsai Hwang, 1994] examine analytically and experimentally the liquefaction potential below structures. Their conclusion is that soils beneath footings would harder liquefy than free field soils.
Selected case histories considering the behaviour of piled foundations are reviewed in [Berrill and Yasuda, 2002] and [Berill et al., 2001]. The main conclusion drawn from these and other studies is that the main threat to piled foundations is from passive lateral soil spreading forces by the non-liquefied crust and not by the drag forces of the liquefied soil itself. There is also evidence that, in the majority of cases, displacement within the liquefied soil is continuous with depth.
Lateral-spreading fields is clearly a very complex phenomenon and raises many controversial issues concerning development of stresses and displacements within the liquefied layer. Should the liquefied soil be modelled as a greatly softened solid or as a viscous fluid? Is the problem better treated as one of applied force or one of imposed deflection? In either case what is the correct value of the design force or deflection? Are the piles laterally supported against buckling inside the liquefied soil? Because of these unresolved questions it is very difficult to establish simple designs rules for piles in lateral spreading fields.
Preliminary analysis of the pit and its piled foundations for various assumed soil conditions suggests that its fundamental period is sufficiently low so that amplifications of its response due to soil softening during liquefaction will not be significant. In addition since only the upper soil layers are expected to liquefy during a seismic event, the piles will not be subjected to passive soil pressures from the non-liquefied soil crust.

5.2 Investigated design scenarios

The adopted empirical methodology for the design of the pit and the piles is mainly suggested by Prof. Dr. G. Gazetas [Gazetas and Gerolymos, 2005]. For design purposes, an idealized soil profile consisting of three layers has been adopted: a liquefiable layer from 0 to 8m; an extremely soft and sensitive layer from 8 to 22.35m; and a rather strong layer from 22.35 to 43.5m. The pit walls and the piles are examined to sustain the maximum design forces of the following three extreme scenarios:

5.2.1 Scenario I

In the first scenario, the pit is considered as a structure fully buried in the soil. To account for the anticipated liquefaction, the soil resistance was ignored from 0 to 22.35m. Only the soil layers below 22.35m were assumed to provide horizontal support for the piles with an average horizontal bedding module of 30MN/m^3. Vertical support of the piles was accounted for by single springs with a stiffness of 304.5MN/m placed at the pile feet.
In this scenario, the structural mass was not subjected to an earthquake action. In accordance with EAK 2000, the walls of the pit were loaded with the soil pressure at rest, the trapezoidal seismic design pressure for rigid structures embedded in soil and the static and dynamic water pressure from inside and outside the pit.

5.2.2 Scenario II

In the second scenario, the pit is handled as a structure above ground, this assumption being more rational and in accordance with EAK 2000. To account for the anticipated liquefaction, the soil resistance was in principle again ignored from 0 to 22.35m. In order to avoid an unnecessarily expensive foundation design, the bottom slab of the pit and the piles were considered as slightly embedded in the layer from 8 to 22.35m by applying a horizontal bedding module of $1MN/m^3$ resp. $2MN/m^3$. Below 23.35m, the same assumptions as for Scenario I were taken. The fundamental period of the piled foundation was calculated to be 0.74s.
In this scenario, earthquake action was applied to the structural mass, but with a reduced design acceleration following the recommendations of Prof. Dr. G. Gazetas. He proposed to use a design acceleration of 0.15g which is half of the effective design acceleration of 0.30g, arguing that due to liquefaction the structure will 'receive' its motion through the piles from which it is 'hanging' in this case. Additionally, the hydrodynamic pressure of the liquefied soil (with an assumed density of $\gamma=2t/m^3$) on the pit walls was taken into account. The hydrodynamic pressure of the liquefied soil on the piles was ignored.

5.2.3 Scenario III

In the third scenario, the pit is handled again as a structure above ground. To account for the anticipated liquefaction, the soil resistance was ignored from 0 to 8m. It was assumed that the layer from 8 to 22.35m will provide horizontal support to the piles. The idea was that a potentially stiffer structure would have a lower fundamental period and attract higher earthquake loading. The piles and the surrounding soil below 8m were substituted by single springs. The horizontal and rotational stiffness of these springs accounting for the horizontal bedding was determined with an elastic continuum approach. Vertical support of the piles was accounted for by single springs with a stiffness of 304.5MN/m analogous to Scenario I and II. The fundamental period of the piled foundation was calculated to be 0.47s.
In this scenario, the same load cases as for Scenario II were applied.

5.3 Structural design issues

The reinforced concrete piles and the ring beam were designed for the envelope of the results of the above given three scenarios. The load bearing capacity of the Ø 1200mm bored piles was calculated in acc. with DIN 4014 and Eurocode 7, Part 1. Pile group effects were neglected without any loss of accuracy in the results, as the average pile spacing is > 3D (D = pile diameter) and pile-to-pile interaction is extremely small for piles embedded in a very soft soil and bearing on a stiff stratum. The piles were designed for end-bearing and friction below 22.35m with an active pile length of 34.3-22.35=11.95m. As no or only reduced lateral support from surrounding soil was assumed in the three scenarios, additionally a buckling check of the piles was carried out. The piles were reinforced with 16 pcs Ø25 from pile top at +0.2m down to -21.9m and 28 pcs Ø25 from -21.9m down to the pile toe at -34.3m.
The ring beam was considered to be rigidly connected with the piles and the pit walls, such that piles and the pit structure act as a frame. The ring beam was heavily reinforced with each 21 pcs Ø32 bottom and top running in longitudinal direction of the pit, and stirrups Ø20 every 15cm. The arrangement of reinforcement was carried out in accordance with EKOS 2000.

6 CONSTRUCTION OF THE PIT

The construction sequence of the pit was the following:

- Initially a working platform was created backfilling the area up to around +1.00m above mean sea level.
- Sheet piles with an approximate length of 14m were installed on the outer perimeter of the pit creating a cofferdam wall.
- Dewatering wells were installed in the perimeter and in the pit in order to lower the water level. They were operated continuously during the pit construction, starting approximately two weeks before begin of excavation.
- The cofferdam was excavated in stages with installation of reinforcing beams and struts at three levels of the cofferdam wall.
- A gravel layer at the bottom of the pit was placed to act as blinding and to balance the groundwater uplift pressure.
- A 40cm thick reinforced concrete slab was concreted to act as a working platform at the bottom of the pit and to prop the base of excavation (see Figure 4).

- The structure was constructed in pre-defined stages, first the 80cm thick bottom slab and than the 50cm thick vertical walls.
- During construction of the vertical walls the decision was made to found the pit on piles in order to eliminate any problems that may arise from liquefaction induced displacements.
- Piles were bored from the top working platform down to the depth of -34.3m.
- The ring beam was constructed to connect the pit with the piles.
- The area was finally levelled and the structure commissioned.

Figure 4: Pit construction

7 CONCLUSIONS

A functioning cooling water intake pit is a pre-condition for the operation of the power plant after an earthquake. Hence, the appropriate consideration of all geotechnical and seismic aspects during the structural design was of utmost importance. Since issues related to liquefaction and lateral-spreading fields are still controversial and not clearly regulated in the codes, a pragmatic approach was applied by investigating different worst-case scenarios. With the combined effort of all involved parties, a both safe and economic design of the pit and its piled foundation could be achieved.

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