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UNDERSTANDING THE SEISMIC BEHAVIOR OF A COMPLEX PILE FOUNDATION BY REALISTIC MODELING THE SOIL BEHAVIOR FROM A REVIEW PERSPECTIVE

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ABSTRACT

A new building, used for storing low level radioactive used machine parts, close to the reactor of the nuclear power plant in Leibstadt (Switzerland) requires the construction of an uncommon foundation type. The structural safety of the main water cooling supply lines, located beneath the northern part of the new building, must be ensured during Safe Shutdown Earthquake (SSE). Therefore an offshore type pile foundation for the concrete structure was proposed by the operator. The Swiss Federal Nuclear Safety Inspectorate (ENSI), which was in charge of reviewing the proposed design and related analysis, carried out an independent feasibility study and a comprehensive analysis of the foundation safety in collaboration with the engineering company Basler & Hofmann. A three dimensional non-linear dynamic time history analysis of the foundation and the building was carried out with the commercially available finite element (FE) program PLAXIS3D. The analysis, which was based on extensive in situ and laboratory test results, allowed better understanding the behavior of the foundation during SSE. Particular attention has been paid to validate the analysis results with well-known analytical approaches, optimizing the FE Model in order to reduce the influence of numerical error due to mesh dependency and model size as well as boundary conditions. The results showed that the maximal deformations of the structure due to SSE were in a tolerable range. The internal forces in the piles were found to be larger than those calculated without taking into account elasto-plastic strain hardening behavior of the soil. Additional feasibility studies have shown that the capacity design method must be used in order to guarantee the safety of the proposed pile foundation.

INTRODUCTION

The operator of the NPP in Leibstadt (Switzerland) is planning to construct a new building for low level radioactive component storage. The new 50 m by 25 m building will be located close to the existing turbine building just above the existing underground main cooling water supply lines. For the operator it was evident that the new building must not interact with other important buildings during an SSE. Additionally, it was required that the main water supply lines must not be affected by dynamic forces transferred from the new building to the ground. While the problem of building interaction can be solved easily with sufficient openings of the gaps, the second requirement has led to an unusual solution for the foundation of the new building.

The new building is divided into two parts with different foundation systems. The southern part has a typical shallow foundation and will not be further discussed in this paper. The northern part however is supported by 30 concrete piles of 1.2 m diameter and 21.5 m length. In the upper zone of about 10 m the piles are placed in protecting steel tubes with a diameter of about 1.5 m, and are therefore separated from the adjacent soil by an annular gap. Thanks to this solution the horizontal forces occurring during an earthquake will be transferred from the building into the ground below the level of the main water supply lines. In Figure 1 the perspective view of the new building is shown.

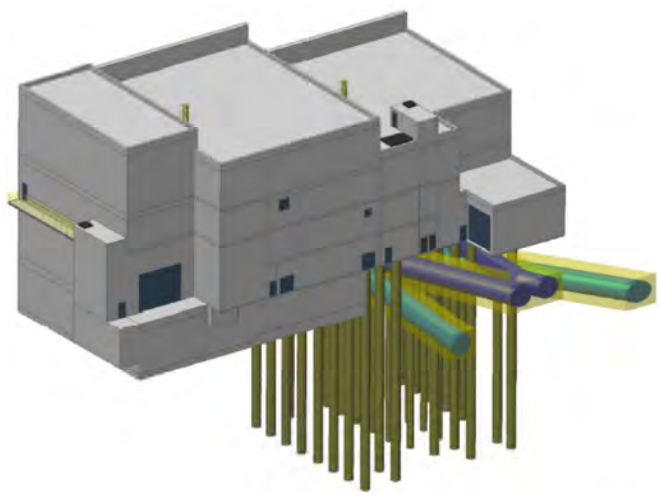

Figure 1. Perspective view of the new building. Left: The southern part with flat foundation. Right: The northern part with pile foundation (Axpo 2013).

The operator used the different 3D FE models of the north part of the building to verify the seismic safety. The displacements of the building and the internal forces in the piles due to SSE were calculated with one model using the response spectrum method and then with another one using the nonlinear time history analysis. In the first model the soil was represented by linear springs distributed along the embedded pile length. In the second model the whole substructure (piles and soil) were represented by nonlinear supports with properties obtained from the preceding nonlinear static pushover analysis of the single pile-soil system.

As expected the piles were the critical parts of the structure. The result for the maximum bending moment and the axial force were found to be plausible. However the calculation of the maximum shear force in the piles was found to be more challenging. It became clear that the assumed value and the distribution of the horizontal spring stiffness have a strong influence on the resulting shear force without affecting the maximum bending moment significantly. To insure that the results provided by the operator are conservative the Swiss Federal Nuclear Safety Inspectorate ENSI, responsible for the reviewing of the project has decided to carry out an independent verification with focus on nonlinear dynamic soil behavior. This additional analysis was performed by the engineering company Basler & Hofmann, Zurich.

LITERATURE REVIEW

A large number of investigations on the dynamic characteristics of pile foundations have been carried out since the first attempts carried out by Novak in the 70’s (see Novak, 1974, Novak and Sheta 1982). The majority of the adopted approaches for studying the soil-pile interaction are based on Winkler spring models (see also Finn, 2005). These models have developed from the first linear springs to the more sophisticated non-linear p-y curves (non-linear Winkler springs). Within this method it is possible to take into account non linearity of the lateral pile soil interaction. The static p-y model is unfortunately still not able to correctly reproduce the dynamic lateral load-displacement response of a pile foundation during e.g. an earthquake. Therefore Makris and Gazetas (1992) developed a more general model based on dynamic frequency dependent non-linear springs and dashpots (Beam on Dynamic Non-linear Winkler Foundation, BDNWF; see also Gazetas et. al, 1993).

Some of the most sophisticated approaches adopting BDNWF can also incorporate cyclic behavior and its consequences for the plastic behavior of the pile (development of plastic hinges, see also Allotey & El Naggar 2008). El Naggar & Bentley (2000) developed a theoretical rigorous approach for obtaining dynamic p-y curves. In general the obtained dynamic p-y curves are later adopted for implementation in linear analysis of transient pile response.

Another way to model dynamic pile-soil interaction is a dynamic analysis with the finite element method in time domain. An early example is given by Wu and Finn (1997). In this study a dedicated finite element program PILE 3D has been adopted. The soil is modeled in this case within volume element and an elasto-plastic (with tensile yielding) constitutive model with viscous damping (Rayleigh Damping).

Generally there are not many examples in the literature if any, of three dimensional dynamic analyses of pile foundations in time domain, in particular adopting complex elasto-plastic strain hardening constitutive models for the soil; this is because the computational effort was too large to be handled in the past. Damping and cyclic behavior of the soil depend in reality on its complex mechanical properties. Therefore a fundamental analysis adopting a realistic constitutive model for the soil could avoid the large effort of determining adequate p-y curves. A model that accounts for these issues and is suitable for dynamic analysis is the "Hardening Soil small" constitutive model (HSs see also Benz, 2007, Brinkgreve et al. 2007) implemented in the finite element code PLAXIS3D.

In the present study an innovative three dimensional finite element analysis of a pile foundation is carried out adopting the HSs for describing the soil and pile soil interaction. No Rayleigh damping parameters have been adopted for the soil to describe its damping behavior during dynamic loading.

SEISMIC ANALYSIS OF THE PILE FOUNDATON

Soil behavior modeling

The mechanical behavior of soil under static loading is rather complex. Adding dynamic loading leads to an even higher degree of complexity. In conventional analysis the soil behavior under dynamic loading is assumed to be non-linear elastic (strain level dependent) and damping characteristics are taken into account within the use of Rayleigh damping. This method allows modeling the energy dissipation in the soil due to dynamic loading. Rayleigh parameters are applied to the mass and the stiffness matrix of the model; therefore they are not a pure material property and produce a cutoff of certain frequency "a priori", which is quite arbitrary (see also Brinkgreve et al. 2007). On the other hand the adoption of this method is the most meaningful, in case of unreliable or nonexistent information on dynamic mechanical properties of the soil. Fortunately, a large amount of laboratory and in situ test investigation was carried out on the site of the Leibstadt nuclear power plant. These results allow the soil behavior under static and dynamic loading to be fully characterized.

Therefore, in order to carry out a realistic analysis of the proposed pile foundation, it was decided to adopt a proper constitutive model for the soil. Within the chosen HSs model (Benz 2007) it is possible to reproduce the energy dissipation phenomenon, which occurs in the soil during dynamic loading. Benz suggests that some Rayleigh damping has to be added anyway in order to capture energy dissipation at very small strains if the materials behaves elastically and no plasticity occurs. In the current analysis the energy dissipation in the soil due to dynamic loading is obtained not by adding Rayleigh damping but by adopting both small strain stiffness and strain hardening plasticity.

Calibration and validation of soil constitutive model

The constitutive model has been calibrated with the results of triaxial tests carried out with bender elements. In Figure 2 the calibrated the results of these tests (IGT ETHZ 2009) are compared to the "best fit" curve obtained by calibrating the hardening soil model. The calibration of the constitutive model has been validated comparing the predicted and in situ measured shear wave velocities. A good correlation was found, as shown in Figure 3.

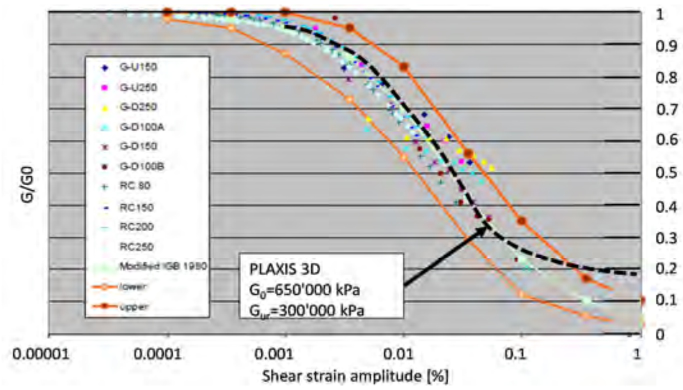

Figure 2. Normalized shear modulus vs. shear strain amplitude: results from triaxial tests (IGT ETHZ 2009) and calibrated curve obtained adopting the hardening soil small constitutive model. G_0 = elastic shear modulus at small strain, G_{ur} = cut off elastic shear modulus.

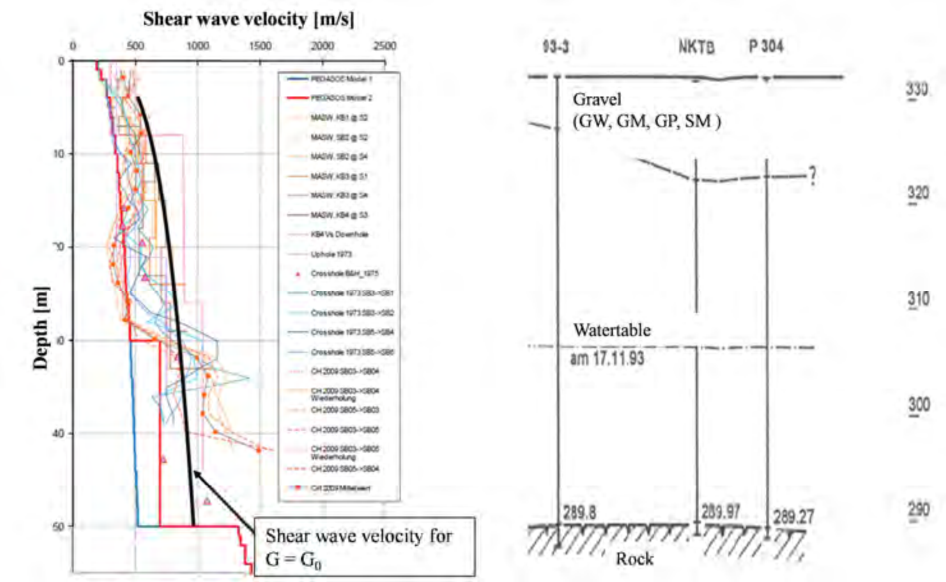

Figure 3. Shear wave velocity in soil vs. depth: measured and predicted (hardening soil small) values (Interoil, 2009) against stratigraphy (GLM, 2010).

Validation of finite element model (soil) with approach of Gazetas

Dynamic finite element calculations are highly affected by model and mesh size and boundary conditions. Thus the finite element calculation results have to be checked for plausibility by using independent analytical solutions. In this study this goal was achieved by comparing the surface amplified spectrum in the FE model (acceleration applied to the lower boundary of the FE model at – 50 m) to the one obtained within an analytical solution developed by Gazetas (1982). The soil characteristics that can be taken into account within this solution are the same as those implemented in the Hardening Soil small model adopted in the finite element study, in particular shear wave velocity increasing with depth and the damping. The spectrum amplification according to the approach of Gazetas is given by the formula (1).

$$A(a_0) = \frac{2q}{(-0.5+q)(1+\tilde{b})^{-0.5-q}+(0.5+q)(1+\tilde{b})^{-0.5+q}} \tag{1}$$

Where: ω = circular frequency

a_0 = dimensionless frequency factor

$q = \left(\frac{1}{4} - \frac{\omega^2}{[c_0^2 b^2 (1+2i\beta)]}\right)^{0.5}$

$b = 2\frac{c_{H/2}-c_0}{c_0}$ (rate of heterogeneity)

$c_0, c_{H/2}$ = shear wave velocity at the surface and at half depth of the deposit

β = critical damping ratio (constant hysteretic damping)

$\tilde{b} = bH$ (Dimensionless rate of heterogeneity)

This approach is based on two main assumptions: The surface spectrum on the surface only influenced by S waves and hysteretic damping is constant with depth. These simplifications are not totally true for the finite element model, where the hysteretic damping changes by the strain magnitude and other kinds of waves are implicitly calculated.

In order to compare the analytical and the finite element amplified spectrum at the surface a damping ratio β of 10% has to be chosen (equation 1). The 10% damping corresponds to the damping taken into account by the calibrated hardening soil small model in the range of observed strains.

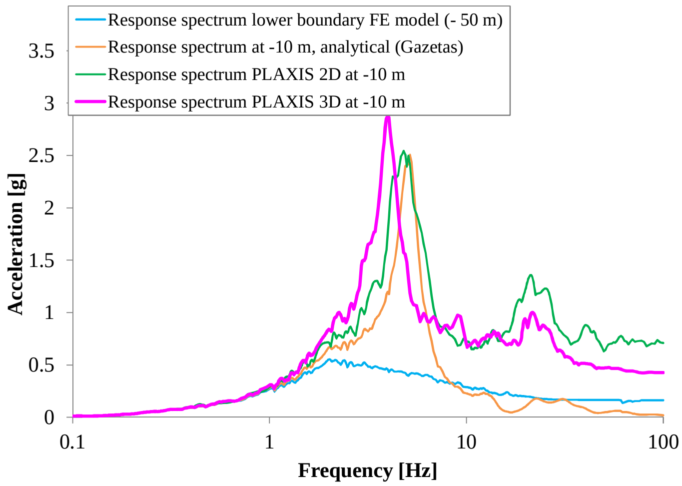

Figure 4. Comparison of analytical and numerical vertical shear wave propagation through the upper soil.

Figure 4 shows the surface response spectrum from the 2D and 3D finite element and analytical solution. The frequency range of high amplification shows a good agreement. The difference of the results from a 2D finite element calculation and the analytical solution is small. That means that the chosen model and element size was adequate for the dynamic analysis. The analytical and the numerical solution differ at frequencies higher than 6 Hz; in particular the finite element calculated amplified spectrum is less damped.

Finite element model

A three dimensional finite element model of the building and the foundation has been implemented in the commercially available software PLAXIS3D, see Figure 5. The piles have been modeled with special beam elements, the "embedded pile model" (see also Engin and Brinkgreve 2009). The validity of these simplified elements, in particular their behavior under lateral loading, has been studied by comparison with volume element piles. The results showed that the two formulations give nearly the same results.

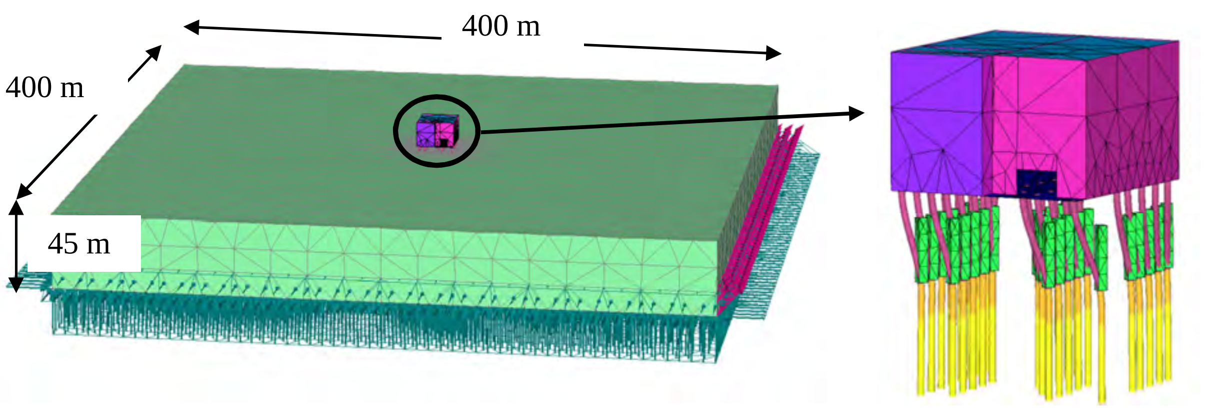

Figure 5. Left: Overview of the finite element model of the building, pile foundation and soil. Right: Extracted model of the building and pile foundation including the steel collars (green) shielding the piles from the soil.

The model is built using 10 nodes tetrahedral elements and consists of 165'962 nodes. The viscous boundary conditions are applied at a distance of approximately 200 m from the building. The averaged element size close to the building is approximately 2 m.

Calibration of earthquake input

Figure 6 shows the resulting earthquake input at foundation level -10 m (pile restraint). Additionally the relevant first eigen-frequencies of the FE model are shown. Due to the free standing pile foundation and the very stiff building structure, the building behaves similar to a horizontal single degree of freedom (SDOF) system. The resulting first horizontal eigenmodes in x- und y-direction have frequencies of about 1.2 Hz. Thus to get an adequate overall behavior of the system and to investigate the non-linear soil behavior at the pile restraint, the earthquake input in PLAXIS3D at the level of -10 m (pile restraint) has to meet the SSE input mainly in the frequency region of 0.5 to 2 Hz. This goal is well reached by simply using a uniformly down-scaled SSE input at the lower boundary of the PLAXIS3D model at the level of -50 m (blue curve in Figure 4). Thus an elaborate calibration of the earthquake input over the whole frequency range using soil column analysis was not necessary for this review study.

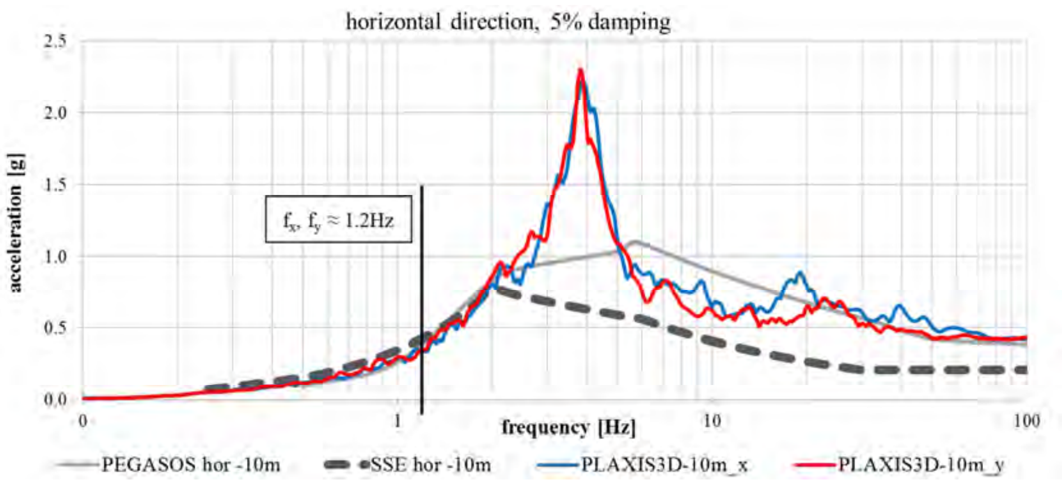

Figure 6. Horizontal earthquake input in PLAXIS3D at foundation level -10 m (pile restraint).

Results

The main goal of the study was to verify and to quantify the influence of the nonlinear dynamic soil behavior on the internal pile forces and on the displacement demand of the pile. The history of the displacement between pile and steel collars is plotted in Figure 7 for the corner piles (with the largest displacement demand). It can be observed that the maximal admissible displacement of 13 cm is not reached during an SSE. Figure 7 shows also the maximal internal forces in one of the corner piles. These results show somewhat larger displacements and pile internal forces than those obtained by the operator with the linear calculation. The reasons therefore are: the hysteretic behavior of the soil (cumulated displacement) as well as the small strain stiffness and strain hardening elasto-plastic behavior of the soil (internal shear forces increased due to hardening of the soil under repeated loading at the constraint).

It can be observed that the maximum shear and bending moments occur at the pile head below the 2 m thick foundation slab and at 11 m depth corresponding to the bottom of the steel collars, where the pile gets embedded in the soil. At this level a ~ 0.5 m thick concrete ring is required to fix the pile in position during construction. This ring has been included in the FE model by horizontal shell elements. The sharp edge of the bending moment diagram occurs due to this local stiffness discontinuity and also due to modeling of the ring using one layer of shell elements instead of multi-layered volume elements. Thus it is expected, that the FE model slightly overestimates the maximal internal bending and shear forces. The effective length of the restraint in the soil is about 3.5 m as seen in Figure 7. This value is smaller than the one estimated by the operator using linear soil springs. Thus the non-linear soil behavior and the concrete ring lead to an increase of shear force in the restraint zone. This finding is important in the design of the pile reinforcement.

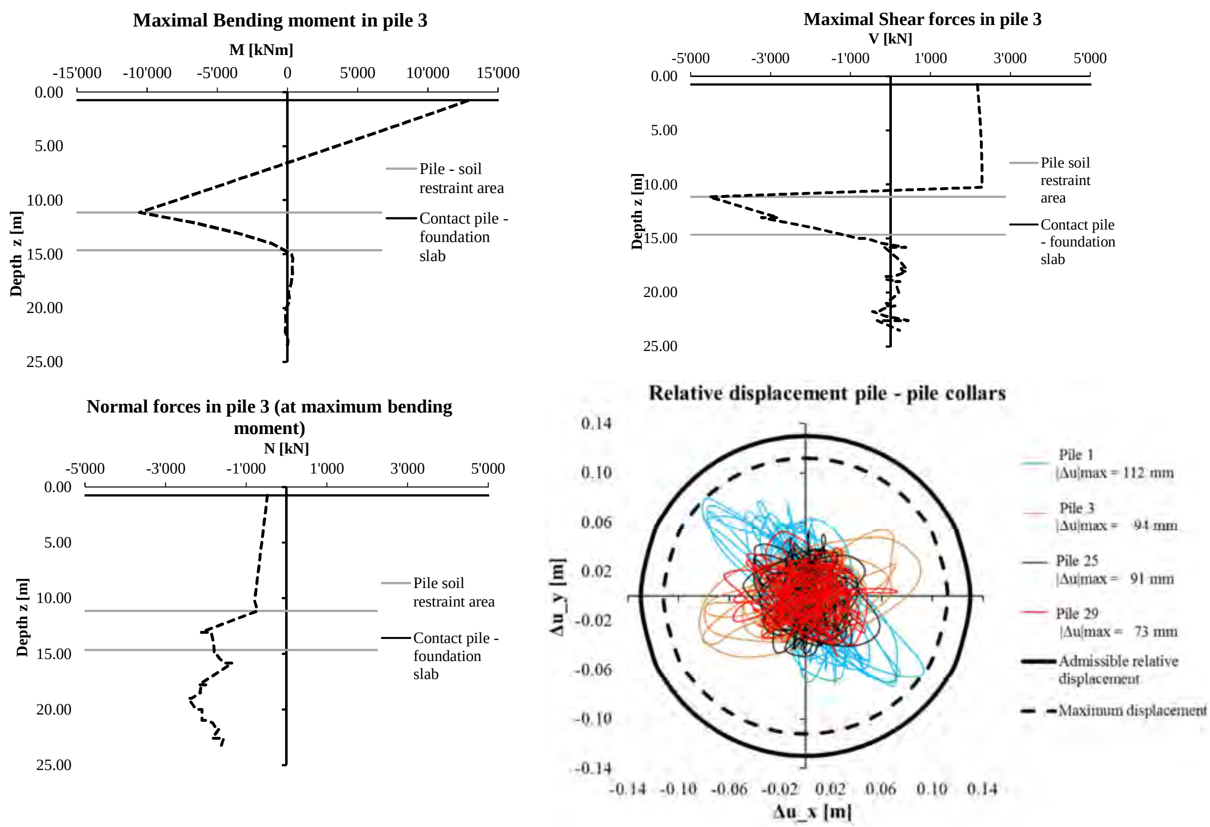

Figure 7. Maximum internal forces in corner pile no. 3 and relative displacement history of pile – pile collars for the 4 corner piles (largest demand).

FEASIBILITY STUDY

ENSI is responsible for granting the approval for the project described. It was obvious from the beginning that the feasibility of the unusual structural system must be carefully examined. For this reason different plausibility checks were performed additionally to the careful review of the calculations done by the operator.

The massive concrete superstructure is rigid compared to the flexible foundation built on 30 partially freestanding piles. The whole structure can thus be considered in a first approach as a SDOF system. For the first plausibility check a force-based approach was used. For the piles the effective stiffness was considered to be about 60% of the initial value in order to take into account the influence of cracking. With a total mass of about 10’000 t and an assumed effective free pile length of 11 m the first natural frequency of the system was found to be about 1.2 Hz (in both directions). This value was in good agreement with results obtained by the operator. The corresponding spectral value of the acceleration in the case of SSE was 0.45 g and the total horizontal force was about 43 MN (1.4 MN for a single pile).

A linear bending moment distribution with equal values on the top of the pile and at the bottom of the effective pile length was assumed. In this case the maximum bending moment of about 7.9 MNm results for a SSE in one direction; about 8.5 MNm if the 100/40 rule is used to take into account contribution of simultaneous earthquakes in orthogonal directions. Based on a static consideration the additional factor for the influence of the torsion was found to be about 1.35. This assumption leads to the maximum effective shear force of about 2.1 MN and elastic moment demand of about 11.5 MNm.

However using a "reasonable" reinforcement ratio of about 3.3% (32 Ø40) a design moment capacity of the pile of maximum 7 MNm per pile could be reached depending on the axial force. Further studies had shown that an enlargement of the pile diameter or increasing number of piles in order to reduce the bending moment would not be effective. The increase of the stiffness and the natural frequency would automatically lead to higher spectral value of acceleration and therefore to reduction of the benefit of this solution. On the other hand, the big load bearing capacity is not really needed in this case. Considering that the piles have a very large displacement capacity, ENSI has decided in this particular case to allow the operator to use concepts of the capacity design method (see section pile design).

The most important requirement for the new building was that the displacement of the building should be limited in the case of SSE. Therefore, a deformation-based analysis was performed. For an elastic SDOF system with a natural frequency of 1.2 Hz spectral displacement of about 8 cm results as shown in Figure 8. It was assumed, that this value also applies to the inelastic SDOF system, which represents the effective behavior of the new building in more realistic way (principal of equal displacements).

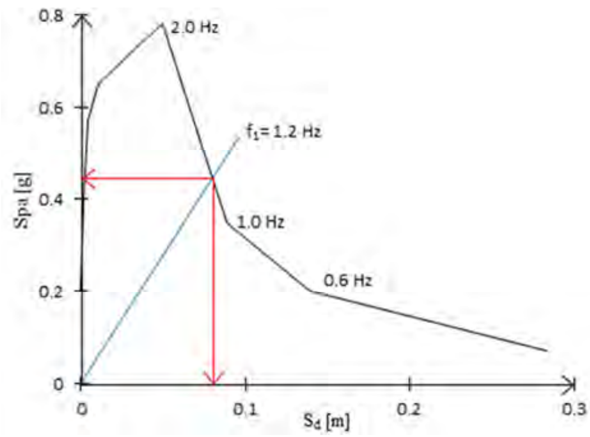

Figure 8. Acceleration-displacement response spectrum (ADRS) for SSE with 5% damping. The displacement demand of about 8 cm results for a SDOF system with the frequency of 1.2 Hz.

It should be noted that the critical point for the pile displacement is about 2 m below foundation slab (top of the steel collar) with about 80% of the maximum value (see Figure 5). For the design of building joints, however, the displacement of the pile head (foundation slab) brings a main contribution to the total displacement of the superstructure. The displacement demand of the SDOF system was determined for the SSE acting in one direction. The superposition of the contributions from two directions was considered using 100/40 rule. Vertical direction of the earthquake was neglected in this consideration. The influence of the torsional movement of the superstructure was taken into account using again a factor 1.35.

Finally the maximum displacement of 12 cm at the pile head due to SSE was found. The corresponding displacement on the level of upper edge of steel collar is 9 cm. These values are in a good agreement with the calculations of the operator. The described detailed nonlinear analysis carried out by ENSI using PLAXIS3D leads to larger but still admissible displacements on the level of upper edge of steel collar of about 11 cm (see Figure 7).

The plausibility check has shown that the solution chosen for the new building meets the requirements both of the operator and of ENSI. Another benefit of the plausibility check was verification that, due to the displacement controlled structural system, standard NPP design rules did not apply in this case. For this reason, the operator was allowed to take advantage of the big displacement capacity of the pile foundation by conducting a non-linear analysis and applying the rules of capacity design.

PILE DESIGN

To guarantee a safe ductile behavior of this special foundation under earthquake loads the piles and the foundation slab (pile cap) have to be designed using the capacity design method (Paulay, Bachmann, Moser 1995). The plastic hinge region was chosen to be at the top of the pile (index 1), where the maximum bending moment occurs (see Figure 7), and not in the foundation slab or at the restraint of the pile in the soil (index 2). Thus it has to be guaranteed that no premature shear failure of the piles and bending or shear failure of the foundation slab can occur. Based on the above described results of the feasibility study ENSI did an additional plausibility check of the capacity design especially of the shear design of the piles.

There are two main values of the elastic shear forces V_1 and V_2 over the length of the pile due to SSE: V_{1d} = 2·11.5 MNm/11m = 2.1 MN over the free pile length and V_{2d} = 11.5 MNm/3.5m = 3.3 MN according to the restraint in the soil. The effective length of the restraint is about 3.5 m as seen in Figure 7. In the capacity design these shear forces are not as shear design load, but rather the shear forces resulting from the overstrength moment of the piles. The "reasonable" reinforcement ratio of about 3.3% (30 Ø40) at the top of the pile gives a maximum design moment capacity M_{1Rd} = 7 MNm and a maximum overstrength moment capacity of M_{1Rd}^+ = 12 MNm, due to the overstrength of the reinforcement and of the confined concrete. The comparison with the elastic moment demand M_{1d} = 11.5 MNm shows that some piles will slightly yield under the SSE depending on the acting normal force. Thus the design shear forces according to the capacity design method results as: V_{1d}^+ = 2·12MN/11m = 2.2 MN and V_{2d}^+ = 13MN/3.5m = 3.4 MN. These design shear forces lead to a very strong and narrow, 2 to 3 layered spiral shear reinforcement of the piles with reinforcement diameter of 16 mm and spacing of 75 mm.

CONCLUSIONS

A new building for storage of low level radioactive components in NPP Leibstadt (Switzerland) is based on 30 partially free-standing piles. This uncommon foundation type is necessary in order to prevent the existing main cooling water supply lines located under the new building from additional loads in the case of SSE.

The design of the pile foundation carried out by the operator is based on a linear-elastic spring Winkler model for the soil. The piles were modeled as beams. In order to verify the results obtained by the operator, the reviewer decided to carry out a more complex analysis, based on the elasto-plastic strain hardening theory. The performed time history analysis has showed somewhat larger displacements and pile internal forces than those obtained by the operator with the linear calculation. The reasons therefore are: the hysteretic behavior of the soil as well as the small strain stiffness and strain hardening elasto-plastic behavior of the soil.

The independent feasibility study has been carried out by Swiss Nuclear Safety Inspectorate ENSI in the initial phase of the project. The results of the sophisticated studies performed by the operator could be verified well using simplified a SDOF model and displacement based methods. The feasibility study helped understanding dynamic behavior of the structure and to define a procedure for verification of the structural safety of the building considering the big displacement capacity of the pile foundation. It was shown that an elastic design method, commonly used for nuclear structures could not be recommended in this particular case. The application of the capacity-design method was found instead to be more appropriate. Therewith a controlled behavior of the foundation under earthquake loads can be achieved.

The plastic hinge region was chosen to be at the top of the pile. Thus, the other parts of the pile and the foundation slab have to be designed taking into account the overstrength bending moment of the pile. Especially a premature shear failure has to be prevented by using strong shear reinforcement.

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