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SEISMIC DESIGN AND VERIFICATION OF A NPP STRUCTURE FOR THE STORAGE OF RADIOACTIVE WASTE COMPONENTS IN SWITZERLAND

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ABSTRACT

Seismic design and qualification of safety and/or radiological relevant structures for Swiss NPPs are subjected to rigorous procedures. Structures have to meet high safety standards, be robustly designed and therefore cover a wide range of parameter uncertainties both on seismic action and capacity side.
In the framework of a wide Facility Power Retrofit at the Swiss Leibstadt NPP the Project ZENT (acronym of radioactive waste storage building) aims to erect two new structures on the site. Due to operational and radiological reasons, these new structures have to be built very close respectively above to the already mentioned existing structures. One structure has to be founded on piles above the existing NPP main cooling pipelines. Approval of this new seismic foundation system by Swiss Federal Nuclear Safety Inspectorate (ENSI) required extended seismic design. This paper attempts to describe the analyses performed by the Owner for seismic qualification and verification of structural integrity at planning stage.

INTRODUCTION

Leibstadt Swiss Nuclear Power Plant

The Swiss NPP Leibstadt (abbreviated KKL) is located in the municipality of Leibstadt (canton Aargau) on the Rhine River close to the Aare Delta and in proximity of the German border. After twelve years of construction, Leibstadt NPP was commissioned on May 24, 1984 and is the youngest nuclear power station built in Switzerland. With a boiling water reactor having 1,245 MW of electrical power, Leibstadt NPP is also the most powerful of the five existing nuclear power stations.

Radioactive Waste Storage Building Project (acronym ZENT)

In the framework of a wide renewal and retrofitting scheme at Leibstadt NPP it was decided to erect two new structures adjacent to the east façade of the turbine building with the primarily scope of storing radioactive waste components (Project ZENT). The new structures have to be built adjacent to many existing one having safety and/or radiological relevance: turbine building, condensate storage tank building, supply channels, auxiliary building complex, radioactive waste building. Moreover, the ZENT north building has to be erected above the existing NPP main cooling pipeline. Nonetheless the irrelevance of the main cooling pipelines for extreme accident management, this core cooling system is of exceptional importance for the electrical power production. Therefore, negative effects due to the new constructions have to be avoided. Based on these considerations, the structural and seismic design for the ZENT north building was carried out by means of extensive analyses. Particular attention was paid to the design and verification of the soil-pile-foundation plate system because due to the fundamental importance of lateral load carrying capacity under earthquake excitations.

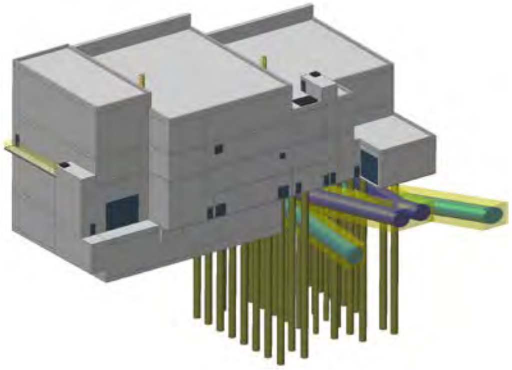


Figure 1. SW perspective view of the projected ZENT south and north buildings above already existing main cooling water pipelines (left) and along east façade of turbine building (right)

PROBABILISTIC SEISMIC HAZARD STUDIES

PEGASOS and PEGASOS Refinement Projects

Starting from later 90's of the past century the Swiss Federal Nuclear Safety Inspectorate ENSI (formerly HSK) identified the need to upgrade the seismic hazard assessments for the Swiss NPPs. A probabilistic seismic hazard analysis (PSHA) according to the rules first established by the “Senior Seismic Hazard Committee” (SSHAC) on behalf of the US-NRC, Department of Energy and EPRI, was considered to best represent the current state-of-the-art. In response to regulators’ request, Swiss NPP operators (licensees) performed a new hazard study between the years 2000-2004 that satisfied SSHAC Level 4 criteria and become know as PEGASOS Project. As hazard results, ground motion exceedence probabilities including aleatory variability and epistemic uncertainty for the four Swiss NPP Sites were carried out. Such extensive hazard computation with the entire input based on expert elicitations and systematic, quantitative assessment of uncertainties was firstly adopted for NPPs worldwide. PEGASOS results were discussed in professional circles worldwide. Especially the unusual large scatter in the ground motion results was judged with criticism by experts. As a consequence, ENSI and licencees decided in year 2007-08 to start the PEGASOS Refinement Project (SSHAC Level 4 criteria as well).

Soil investigations in the framework of PEGASOS Refinement Project

An extensive field investigation campaign and laboratory tests were carried out in years 2008/2010 at the four Swiss NPP Sites in the framework of the PEGASOS Refinement Project with the aim to reduce uncertainties in dynamic soil parameters.

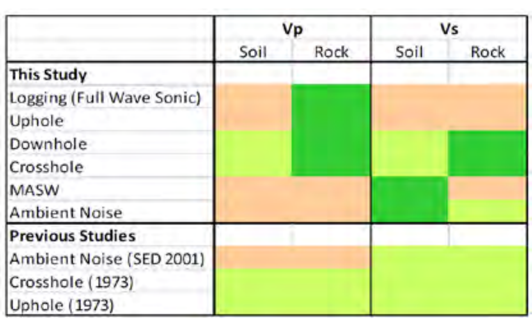

Figure 2. Summary of the methods applied to determine S-wave, P-wave velocities during the 2008-09 field campaign. Light brown (unsuitable), bright green (successful), pale green (unreliable method)

Because of the large variability in the grain size distribution and cohesion characteristics of the soft soil at the NPP Site (see Figure 3), numerous applied methods in the field campaign did not work successfully (see Figure 2). A large band width in the S-wave and P-wave velocity profiles and dynamic soil strain dependent properties from laboratory testing were the consequence. Especially the P-wave velocity profile estimation in soft soil was unsatisfactory.

Figure 3. Photos showing the channel type deposits of gravels and occasional sand lenses. Weathering profile indicates hard (cemented) layers of conglomerates and mainly softer layers of gravel. On the right from the upper section of the outcrop: Alternation of gravels and sandy layers.

Dynamic Soil Properties

Based on results of field campaign and after expert elicitations, a set of three S-wave velocity profiles P1 to P3 and a generic P-wave velocity profile has been defined for subsequent site-response analyses. Profile P1 were judged as preferred S-wave velocity profile by PEGASOS Refinement experts. In Figure 4 is shown, that a wave velocity variability ranging between 10-25% has been considered in the study. For design purposes, the ensemble of the three S-wave velocity profiles P1-P3 and the generic P-wave velocity profile including their variability were considered in the process.

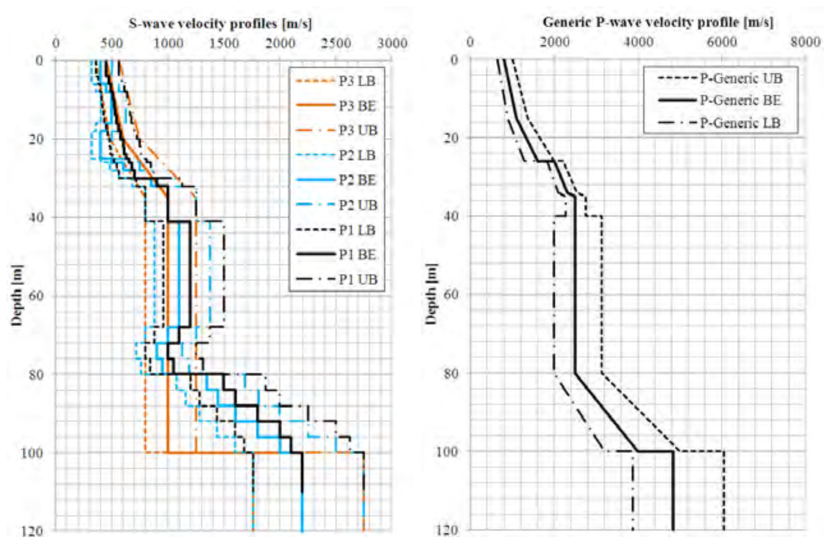

Figure 4. Low strain S-wave (left) and P-wave (right) velocity profiles for lower-bound, best-estimate and upper-bound soil properties considered in the PRP Project at Leibstadt NPP

ZENT NORTH BUILDING

Nuclear Building and Earthquake Safety Classification

Structures of Swiss nuclear facilities with safety and/or radiological importance are classified according to the ENSI-Guideline G01 as building class BKI or BKII and earthquake category EK1 or EK2. Classification of structures is governed by either the electrical and/or mechanical systems and components stored in the structure, potential radiological inventory stored or adjacent structures with nuclear classification. Based on the radiological inventory the new ZENT structures fit in the criteria for building class BKII and earthquake class EK 2. This means that the ZENT structures could solely be designed for an Operational-Basis Earthquake (OBE). However, due to the proximity to nuclear structures classified in the highest category BK I/EK 1, the licensee requires to design the new structures for a Safe-Shutdown Earthquake (SSE) with a PGA value of 0.28g at the soil surface. In the case of Leibstadt NPP this means a consideration of two-times of the OBE uniform hazard spectra.

Structural System

The Superstructure consists of a two-storey RC shear wall building with a total height H = 14.80 m above foundation plate at El. +/- 0.00. Planimetric building dimensions of the storage area are L x W = 23.45 m x 21.40 m. An overhanging section with the dimensions L x W = 11.50 m x 5.00 m extends on the east façade and is structurally monolithic connected with the main building (Figure 5). The lateral load resisting system mainly consists in four exterior walls that extend along the four façades of the storage area. Primary bearing capacity is also carried by these elements. A massive foundation plate of t = 1.50-2.00 m thickness ties vertical and lateral loads of the superstructure to the pile system consisted of a total of thirty RC piles of D = 1.20 m diameter. In order to protect the already existing main cooling piping system, piles are separated by a 130 mm gap up to an El. -11.40 m from soft soil (Figure 5). RC piles are coated by a steel shaft D/t = 1220/10 mm along this clear height.

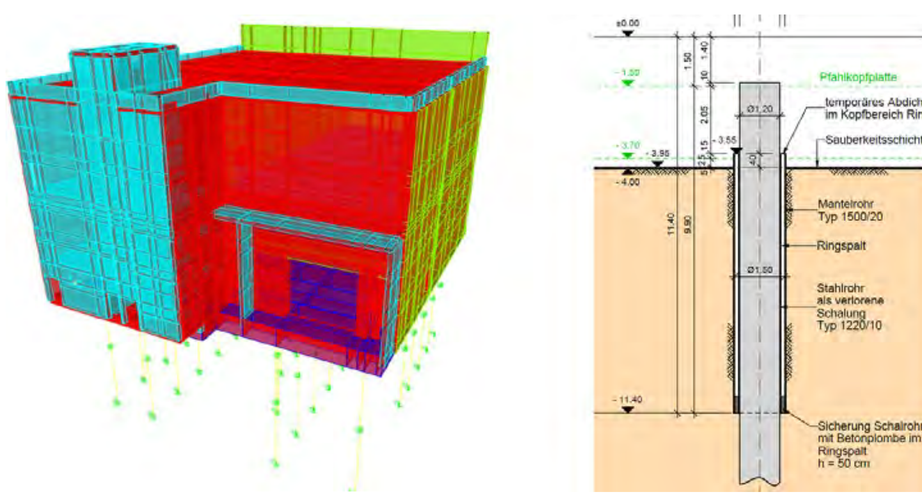

Figure 5. East-Façade perspective view of the 3D FE structural model of ZENT North building (left) and construction detail of a typical steel coated pile from El. -1.50 up to El. -11.40 (right)

Considering the flexibility of the foundation piles with respect to the superstructure it can be stated that soil-foundation-structure interaction under seismic actions is governed by the response of the piles. In recognizing the importance of this structural element extensive analyses were carried out with the scope of accurately represent piles' force-deflection behaviour.

SINGLE PILE ANALYTICAL STUDIES

Study on Pile-Soil-Interaction

Extensive analytical and experimental studies on vertical load bearing capacity and flexibility can be found in literature. Nonetheless, extremely reduced information were found on the force-deflection behaviour of the piles of large diameter D > 1.00 m when subjected to lateral loads. As a consequence, extensive analyses using Plaxis software were carried out with an aim to accurately describe soil flexibility under different pile actions. Soil flexibility criteria were modelled starting from results of equivalent-linear soil column response analyses carried out using profiles showed in Figure 4. Soil failure were considered by assuming Mohr-Coulomb-Criteria as well. Results of the analyses shows, that piles behave mostly linear when subjected to lateral loads in the range of V = 1000 kN (point load on top). The reduced horizontal soil deflection due to horizontal pile push-pull confirms the excellent soft soil characteristics at Leibstadt NPP Site. Resulting horizontal and vertical soil flexibility for the embedded pile regions are listed in Table 1 and were used in subsequent analyses.

Table 1: Values for horizontal and vertical pile-soil-stiffness according to 3D FE-Model in Plaxis

Soil Properties	G-Soil Profile	Hor. Soil Stiffness	Mantel Friction	Tip Stiffness
[-]	[-]	[kN/m/m’]	[kN/m/m’]	[kN/m]
Lower Bound (LB)	$G_{BE}/1.50$	364’000	197’000	614’302
Best Estimate (BE)	G_{BE}	477’000	199’000	920’704
Upper Bound (LB)	$G_{BE}\cdot 1.50$	616’000	199’000	1’381’401

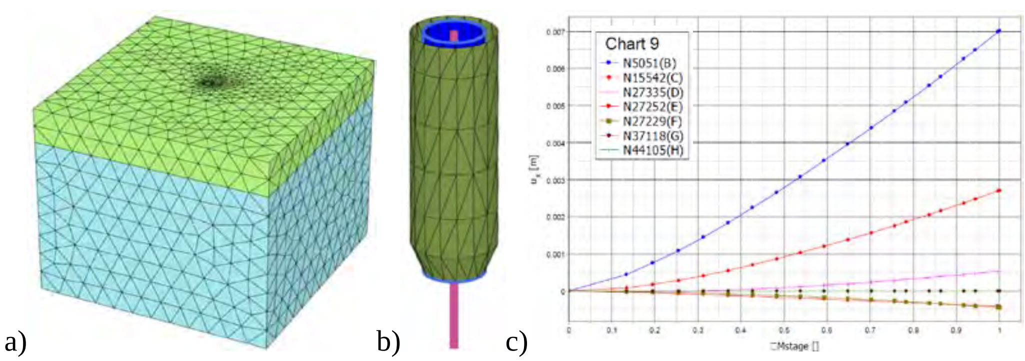

Figure 6. 3D FE-Model of embedded pile in Plaxis: a) Soil Layers and Mesh, b) Embedded RC pile with soil gap of 130 mm and steel tube, c) Incremental horizontal pile deflection at selected nodes (top to bottom, B-F at El. -11.0 up to -15.0 each meter, G-H at El. -18.35 and El. -28.35)

Study on static nonlinear force-deflection-relation of RC pile

Pushover analyses of a single pile (clamped-clamped restraints) having a clear length L = 10.40 m were performed with the finite element program ATENA. The analyses considered different axial (compression) load conditions for P = 0kN to 3000 kN reflecting the axial load variability acting on the thirty piles. FE-Model took into account concrete cracking, crushing, reinforcement yielding and strain-hardening up to failure. Tensile cracking nonlinear material model was based on fracture mechanics. Concrete compressive crushing and steel yielding were based on plasticity theory. Interface between steel coating and concrete was modeled by Mohr-Coulomb friction criteria. A small friction coefficient equal to μ = 0.1 was used. Results of performed static nonlinear analyses in ATENA can be seen on Figure 7. For the considered axial load range an increasing in compression force leads to an increasing of pre-plateau stiffness and maximal plateau lateral force capacity (10% in maximum). P-Δ-Effects in post-peak load were amplified as well, leading to a sign change in plateau force-deflection-slope. However axial load ratio affects ultimate pile displacement capacity but does not modify yield displacement. Consequently, displacement ductility of RC members is affected by axial load ratio as reported in literature.

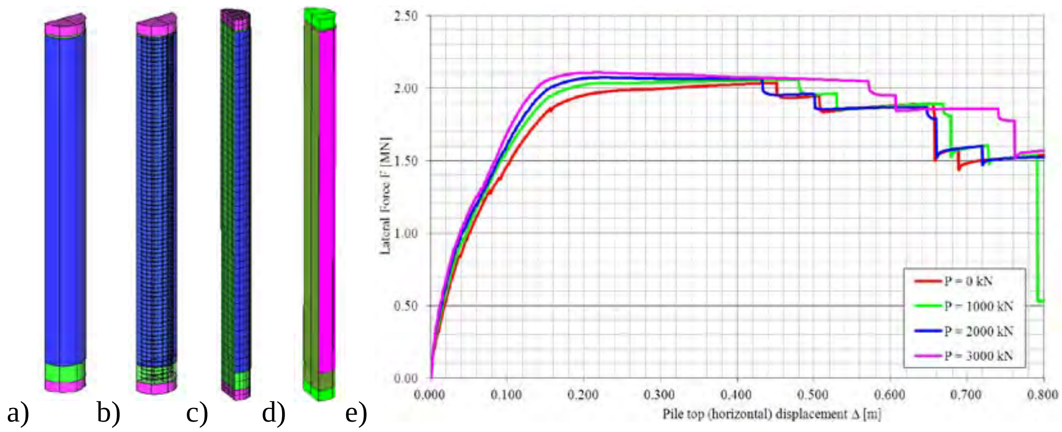

Figure 7. Pile FE-Model in ATENA a) elements, b) concrete, reinforcement, c) FE-Mesh d) steel coat ranging from El. -1.55 to -11.40 in pink and e) results of pushover analyses for different loads P

Member failure was governed by rupture of longitudinal rebars. An expected positive effect on concrete compressive stress-strain-behavior (concrete confinement) due to steel coating was observed in the analyses. Due to the particular pile construction scheme (Figure 5), pile deflection at El. -3.65 (steel mantels' top elevation) with respect pile-top deflection was estimated as well (~80% of top displacement). Considering a 130 mm gap between RC steel coated pile and steel mantel an approximate maximal displacement of 160 mm (top) is allowed, if dynamic pile-mantel-pounding effects have to be avoided. It means, that potential pile lateral displacement capacity as been reported in Figure 7 can not be achieved in reality due to the presence of a steel mantel around each pile of diameter D = 1500 mm.

Comparison and discussion of soil pile flexibility

Based on linear theory for a continuous supported beam on elastic foundation, a global horizontal soil stiffness was calculated. For the purpose, best-estimate soil properties and stiffness parameters according to Table 1 were used. Similarly, uncracked pile stiffness for a clamped-clamped beam of length L = 10.40 m with circular cross section D = 1.20 m was calculated according to Formula 1.

$$k_{pile} = \frac{12EI}{L^3} = \frac{12E}{L^3} \cdot \frac{\pi D^4}{64} \tag{1}$$

$$\frac{1}{k_{pile-soil}} = \frac{1}{k_{pile}} + \frac{1}{k_{soil}} \tag{2}$$

Even if uncracked pile stiffness were assumed, pile flexibility is considerably higher (~7.6-times) when compared to lateral soil flexibility (see Table 2). Consequently, flexibility of clear pile length governs dynamic behavior of overall ZENT structural system for horizontal seismic actions. Thus, soil stiffness characteristics and their variability barely modified dynamic properties of the structural system. Such effect becomes even greater as seismic demand increases. For vertical flexibility, pile to soil stiffness ratio becomes larger (around ~3.5-times). As a consequence, structural behavior will more be affected by pile-soil-interaction effects in case of vertical seismic actions.

Table 2: Comparison of soil and pile flexibility from single pile analytical studies. Soil flexibility based on best-estimate characteristics. RC pile flexibility based on pushover curve for axial load of P = 3000 kN

Pile stiffness assumption	hor. pile stiffness	hor. soil stiffness	pile to soil stiffness ratio
[-]	[kN/m]	[kN/m]	[-]
uncracked (initial) stiffness	38’005 (Formula 1)	288’184	1/7.60
secant stiffness to V = 1000 kN	26’250	288’184	1/11.0
secant stiffness to Δ = 160 mm	13’667	288’184	1/21.1

LINEAR AND NONLINEAR ANALYSES ON HOLE MODEL

Three Dimensional Finite Elements Model and Dynamic Properties

A realistic 3D FE-Model in SAP2000 was carried out for design verification. Structural elements such as foundation plate, interior and exteriors walls, decks and roofs were modelled by means of shell elements. RC piles were modelled by means of multi-linear plastic link/support elements. Monotonic force-deflection behaviour was obtained from ATENA analyses (Figure 7) while Takeda hysteresis rule was assumed for cyclic behaviour. Soil stiffness was modelled by means of global (translational) linear springs at El. -11.90 (clamped-end of RC pile with soil). Total assembled structural mass for dynamic analyses corresponds to m = 9820 t. Mass portions consist of self-weight, surcharge loads (i.e. façades, roof construction) components’ weight and portioned live loads.

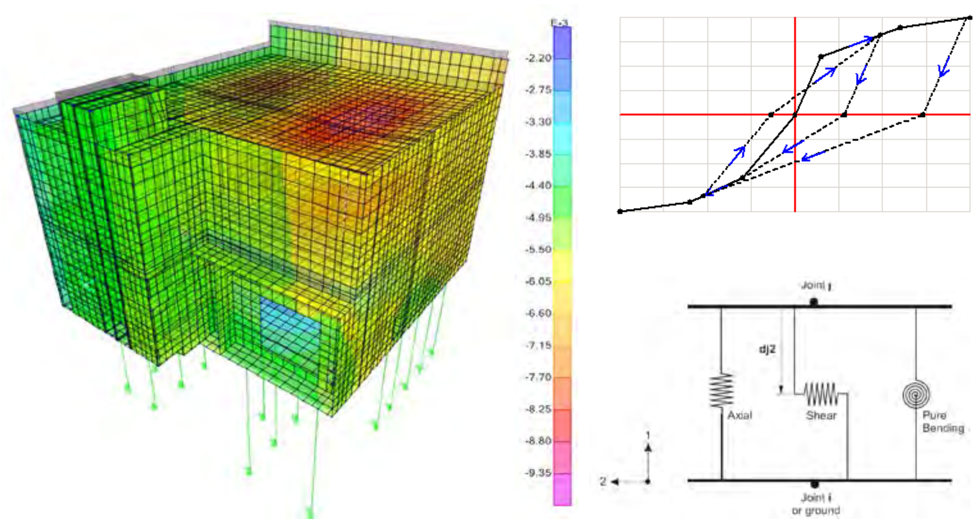

Figure 8. Vertical displacement in [m] of 3D FE model for static load combination (left), link/support element characteristic (right, bottom) and cyclic link behaviour for Takeda-Rule (right, top)

Table 3: Modal analysis results from 3D FE-Model in SAP2000 (uncracked pile stiffness assumption). Cartesian X- and Y-direction in the horizontal plane (NS- resp. EW-dir.) and Z-direction vertical

Mode	Frequency	Damping Ratio	Modal Mass M_X	Modal Mass M_Y	Modal Mass M_Z
[-]	[Hz]	[-]	[t]	[t]	[t]
1	1.30	0.07	3347	4916	-
2	1.32	0.07	6376	2770	-
3	1.43	0.07	9	1989	-
4	5.64	0.07	1	143	50
5	6.34	0.07	60	-	3120
6	7.01	0.07	24	1	5563

Linear Modal Time-History Analysis

Linear modal time-history analysis (LMTHA) was carried out in order to verify results of nonlinear dynamic analyses for SSE earthquake ground motion with a PGA value of 0.21g at El. -11.90. Maximum relative roof displacements in both horizontal directions are smaller than 80 mm and reduces to roughly 90% at foundation plate level (Figure 9). Seismic forces from LMTHA are mostly consistent with design values, obtained response spectral analyses and SRSS modal combination rule assumptions in an equivalent FE-Program.

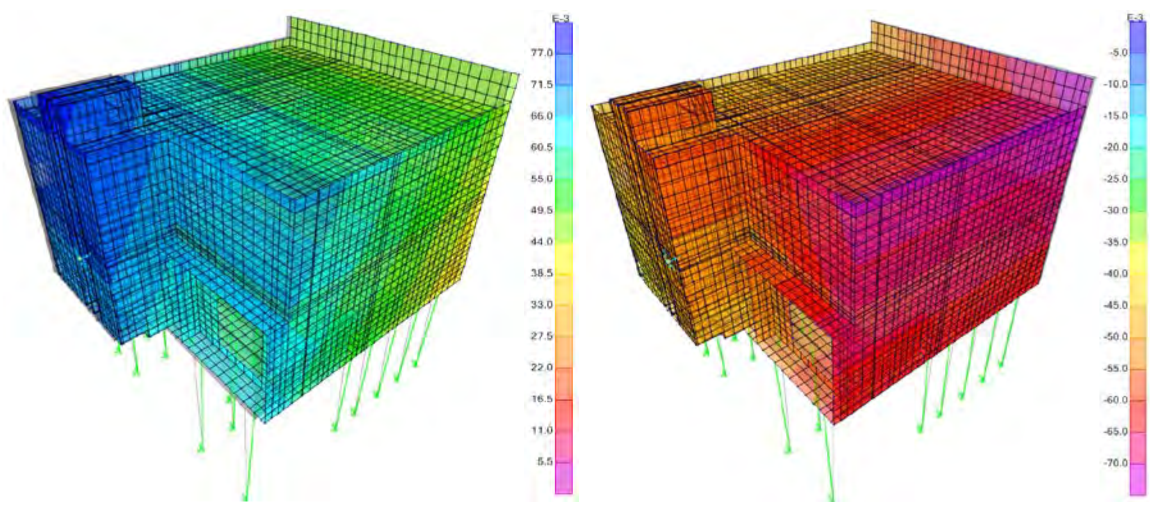

Figure 9. Deformed shape for absolute maximum roof displacement in [m] in X-direction (left) and Y-direction (right) from linear modal time history analysis

Nonlinear Time-History Analysis

Considering the fact that superstructures' stiffness is significantly higher than soil-pile-systems' stiffness, rigid body constraints were applied to all superstructure DOFs. Strongly reduced analysis computation time, small increase in vibration frequencies and therefore reduction of maximal displacements for LMTHA with dynamic properties of Table 2 were the results. Nonlinear time-history analysis verification was performed on the rigid-body constrained model. Hilber-Hughes-Taylor time-step integration algorithm (α = 0.0) and Rayleigh damping were used. A small elastic damping ratio of ξ_{el} = 0.04 was set at frequencies f = 1.00 Hz resp. f = 8.00 Hz in order to avoid overestimation of total modal damping in NLTHA. Results confirm the conservative assumptions made in seismic design and structural robustness of the ZENT north building. Due to particular conservatism, maximum base shear from NLTHA in X- and Y-dir. are roughly 40% lower when compared with design values of response spectra analyses. However, in the vertical Z-dir. almost identical values are obtained. Maximal roof top displacement in the X- and Y-dir. from NLTHA are around 70 mm (≈1.2·59 mm) and therefore at least 30% smaller when compared with design values. As a consequence design values of pile-steel mantel horizontal gap (130 mm) and ZENT building's horizontal gap to adjacent structures (150 mm) are adequate. Especially hysteretic modal damping values of ξ_{hyst} = 0.065-0.073 at peak structural response leaded to total modal damping ratios of $\xi_{tot} = \xi_{el} + \xi_{hyst}$ = 0.098-0.108 for the X- and Y-dir. of loading (see Formula 5.1, were A_h = loop area, F_m = peak load, Δ_m = peak displacement). Design modal damping equal ξ_{tot} = 0.07for SSE earthquake excitations was conservatively assumed to be for all modes.

$$\xi_{hyst} = \frac{A_h}{2\pi \cdot F_m \cdot \Delta_m} \tag{3}$$

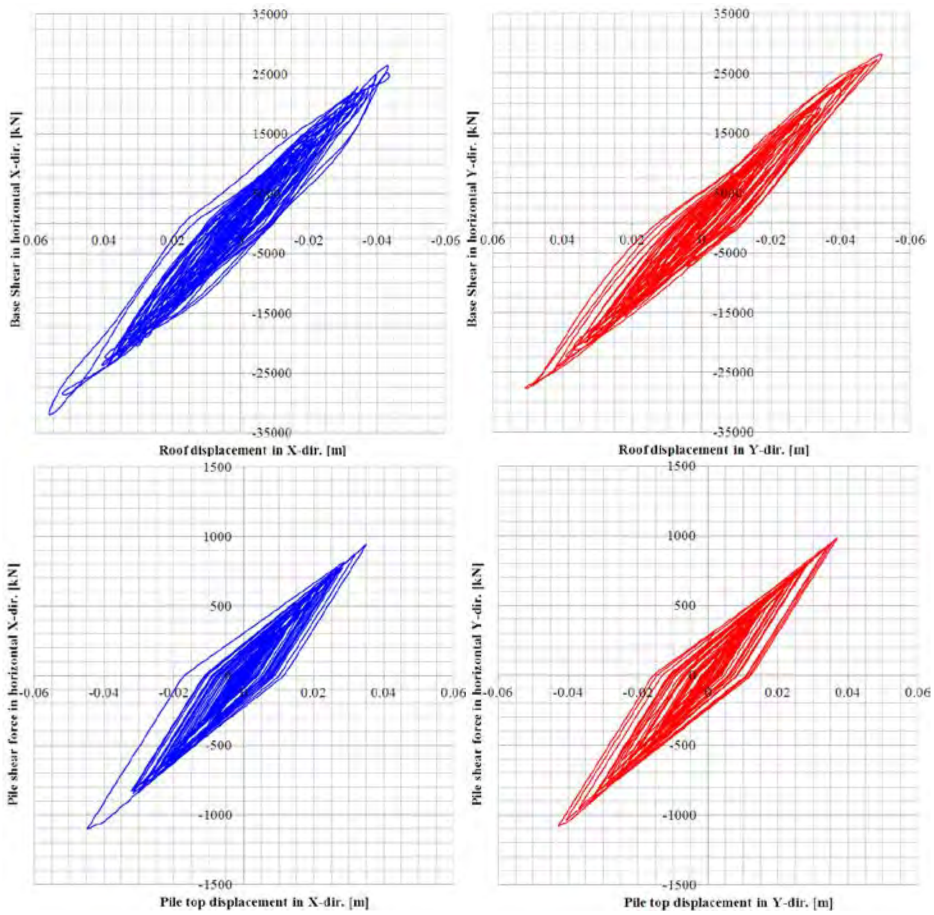

Figure 10. base shear hysteresis loop vs. roof displacement for the X- and Y-dir. from NLTHA (top). shear force hysteresis loops vs. top displacement for the X- and Y-dir. of loading for the controlling corner pile from NLTHA (bottom)

CONCLUSIONS

Structural design for a new structure with radiological importance on a existing Swiss NPP Site has been presented. Results of soil investigations and experts judgment of SSHAC Level 4 PSHA Studies PEGASOS and PEGASOS Refinement Project has fully been considered in the design process. Due to the particular foundation scheme, extensive analytical investigations on pile-soil-interaction and nonlinear pile force-deflection behaviour were carried out at the design stage. Furthermore, results of single pile analytical studies were implemented into a 3D structural FE-Model and dynamic nonlinear time-history analyses were performed for design verification purposes. Findings of this investigation were compared with design assumptions. Pile reinforcement detailing was finally found to be compliant to the capacity design criteria (ensuring a ductile member behaviour). It could be shown that seismic structural design was well performed and conservative with respect to verification. Consequently, a large seismic safety margin for earthquake demands exceeding safe-shutdown-level can be ensured.

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