SEISMIC RESPONSE ANALYSIS AND UPGRADING DESIGN OF PUMP HOUSES OF KOZLODUY NPP UNITS 5&

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CONTENTS

ABSTRACT 2
1. INTRODUCTION 2
2. STRUCTURE DESCRIPTION 2
3. MATEMATICAL MODELS 5
4. SEISMIC EXCITATION AND SOIL MODELING 7
5. SSI PARAMETERS AND SEISMIC RESPONSE ANALYSES 11
6. CAPACITY EVALUATION OF BUILDING STRUCTURES 11
7. STRENGTHENING CONCEPTS 12
7.1. Side Bracing 12
7.2. Dismounting of Upper Structure and Shielding of the Equipment 12
7.3. Replacement of Heavy Roof with Light Roof Structure 13
8. CONCLUSIONS 14
REFERENCES 14

LIST OF FIGURES

Figure 1 General Layout 3
Figure 2 Typical Transverse and Longitudinal Cross-section of SWP structure 4
Figure 3 Models of EB2 and EB1 sections (West side view) 5
Figure 4 Models of AA and SWP sections (East side view) 6
Figure 5 Models of SSWP and AWFPP sections (East side view) 6
Figure 6 Model of FS section (Internal view) 6
Figure 7 Free Field RLE and LLE response spectra 7
Figure 8 Free Field Time Histories 8
Figure 9 Comparison of Low and High Strain Soil Properties 9
Figure 10 Comparison of Free Field Spectra with Loess-Cement Layer 3 Spectra 10
Figure 11 Force Balance at Connections Due to Horizontal Seismic Loading 12
Figure 12 Side Bracing for AWFPP (East and West side) 13
Figure 13 Dismounting of Upper Structure 13
Figure 14 Replacement of Heavy Roof with Light Steel Structure 13

ABSTRACT

The main objective of the presented project was to perform a feasibility study for seismic/structural evaluation of the safety related structures at Kozloduy NPP Units 5&6 for the new site seismicity and determine if they satisfy current international safety standards. The evaluation of the Pump House 3 (PH3) building is addressed in this paper, which was carried out by applying appropriate modeling techniques combined with failure mode and seismic margin analyses. The scope of the work defined was to present the required enhancement of the seismic capacity of the Pump House structures.

1. INTRODUCTION

The project for modernization of Kozloduy NPP VVER-1000 MW Units started in late 1998. It was carried out by European Consortium Kozloduy arranged by Siemens, Framatome, and Atomenergoexport. Siemens in cooperation with EQE Bulgaria was involved in the project for seismic evaluation and conceptual upgrading design of PH3 at the plant. The scope of the work was covered by applying appropriate modeling techniques combined with failure mode and seismic margin analyses. This paper presents the results of the study as well as required enhancement of the seismic capacity of the PH3 structures.

To meet the scope project specific criteria and methodology specifications were developed [1]. Design and acceptance criteria were based on applicable International, American and Bulgarian standards and codes. The general approach followed in the study was consistent with the now-a-day practice for seismic evaluation and strengthening of the existing NPPs. The following activities were performed to accomplish the objectives of the project: PH3 building walkdown, 3-D modeling, SSI seismic response analyses, capacity evaluation, development of strengthening concepts, feasibility study and selection of the upgrading measures, generation of floor response spectra (FRS) for as-built and upgraded structure.

2. STRUCTURE DESCRIPTION

Pump House 3 building (PH3) at Kozloduy NPP servicing VVER-1000 MW units consists of the following sections divided by structural gaps:

- Assembly Area (AA) – axes 1-3, rows A-D;
- Service Water Pumps (SWP) – axes 3’-10, rows A-D;
- Secondary Service Water Pumps (SSWP) – axes 10’-14, rows A-D;
- Additional Water and Fire Protection Pumps (AWFPP) – axes 14’-18, rows A-D;
- Electrical Building 1 (EB1) – axes 2’-10, rows E-K;
- Electrical Building 2 (EB2) – axes 10’-17’, rows E-3;
- Filter Section (FS) – axes 3’-10, rows K-L.

- Crane Way and Intake Structure (CWIS) – east of row A. It consist of 3 different sections, attached without gaps to AA, SWP and SSWP. CWIS is not considered as an independent section.

Pump Hall extends over sections AA, SWP, SSWP, and AWFPP. Only AWFPP section is considered as a category I structure, whereas all other sections are considered as category II structures. All structures of PH3 building except AA section consist of embedded concrete structure and upper RC frame structure.

The embedded structures are founded on different elevations but structure-to-structure through soil interaction was neglected in the study. Embedded structures house service water intake pumps, sieves, secondary pumps, additional and fire protection pumps, service water pipelines, fire protection pipelines and supporting systems pipelines. The SWP and SSWP sections are filled with water in the intake part of the relevant structures. Pipelines from SWP, SSWP, and AFWPP sections pass through embedded structures of EB1 and EB2, whereas in the FS section heavy filters are arranged on service water pipelines. The embedded structures are monolithic, constructed with thick concrete walls and slabs, solid blocks, girders and columns. The foundation slabs of different sections are 1.5m thick. The down parts of intake structure have rounded forms for better water flow. Under the pumps-intake there are huge concrete blocks. Concrete beds are arranged for the pre-cast concrete columns from the upper structures. The embedded structures have adequate reinforcement and stiffness. The AA section is founded on monolithic grid foundation with backfill between the strips. Concrete beds are arranged for the pre-cast concrete columns from the upper part. Pre-cast panels are used to form the floor.

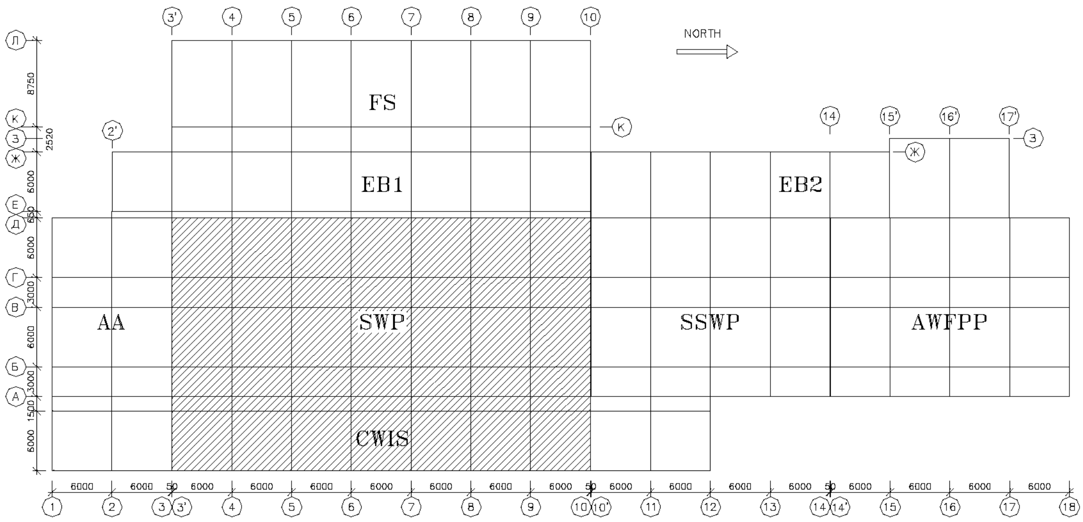

Figure 1 General Layout

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The upper structure consists of pre-cast concrete elements, steel braces and facade panels. Roof beams are arranged in longitudinal direction over the top of the pre-cast columns and 2T pre-stressed roof panels 18m long are mounted on the beams. The structural members involved in transverse seismic response are columns, roof beams (bending, torsion and shear), roof 2T-shaped pre-stressed panels. The structural members involved in longitudinal seismic response are columns, T-shaped crane beams, roof beams, steel braces. The facade panels give some additional longitudinal stiffening also. The roof beam-to-column connections and roof panel-to-beam connections are accomplished with embedded steel plates and welds. The connections and their anchorage details are inadequate for seismic loading. The upper structure as a whole is not adequate for seismic impact in transverse direction. Typical cross-sections of SWP structure are presented on Figure 2.

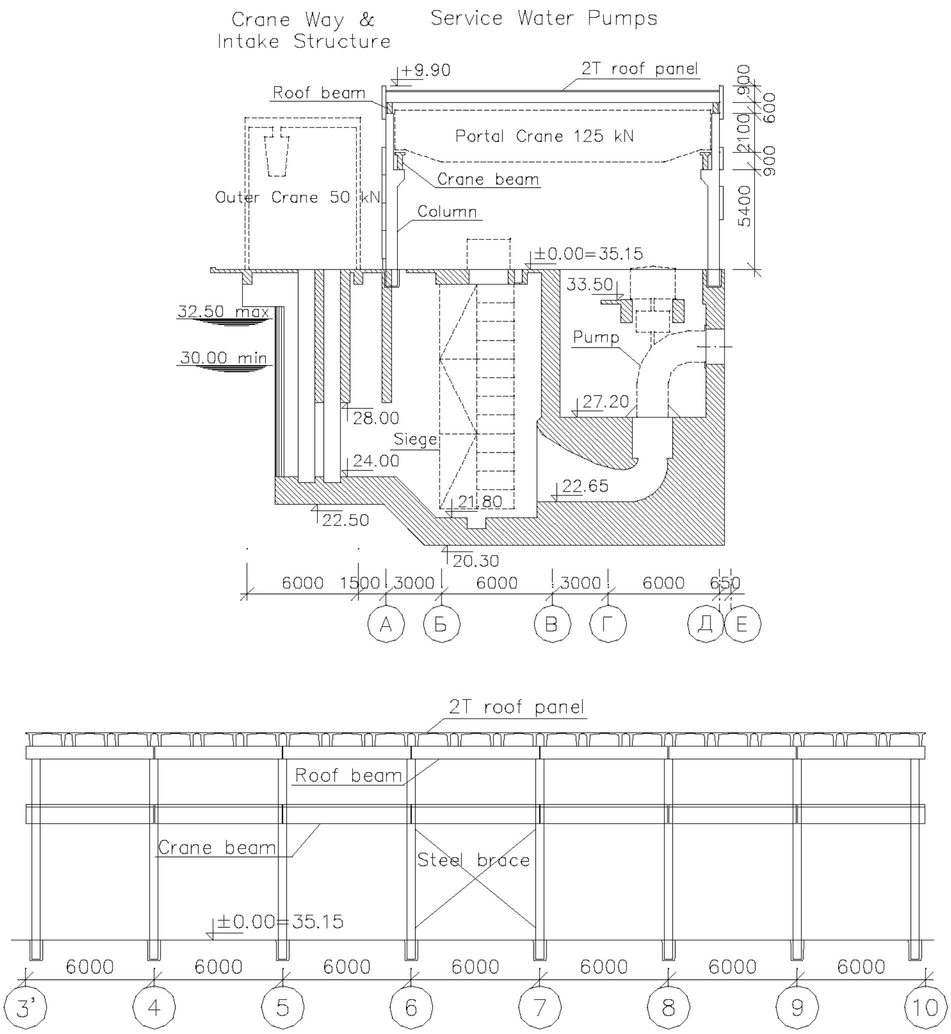


Figure 2 Typical Transverse and Longitudinal Cross-section of SWP structure

3. MATEMATICAL MODELS

3-D models for each of the structures in the PH3 building were created using BEAM, SHELL, and SOLID elements (Figures 3-6). The stiffnesses of the members was defined in accordance with their geometrical characteristics. The masses of the reinforced concrete and steel structural elements are distributed over model elements through their equivalent mass densities. The masses of the equipment are taken from the data presented by the Client, by evaluations during the walkdowns or derived by expert judgement. Masses of heavy equipment (with mass exceeding 1 t) are modeled as lumped masses. Masses of facade panels and miscellaneous brick walls are lumped in corresponding nodes of the columns. The water in pump chambers is lumped at the adjacent nodes taking into account its maximum operational level. It is assumed that in case of vertical motion the water follows rigidly the motion of the foundation and for horizontal motion – the motion of the corresponding walls in the relevant direction. The masses in horizontal directions are increased with 10% to take into account the impulsive water masses outside the intake structure section. Generally the masses of the cranes and crane ways are taken into account without any lifting load mass. The cranes' normal parking positions, identified by walkdowns, are considered. Lumped masses are used for crane modeling.

Few specific studies were carried out to asses the influence of facade panels stiffness on seismic behavior of upper structures as well as to asses the stiffness of roof structure. A detailed model of the roof structure was created, taking into account all the eccentricities due to the roof panel-to-beam connection and beam-to-column connection. The connections between the panels were also modeled. A simplified equivalent model of the roof was created taking into account the vertical deformations of the panels and the mutual displacements in both horizontal directions. As result the simplified roof model was used in the seismic response studies and the facade panels stiffness was not considered in them.

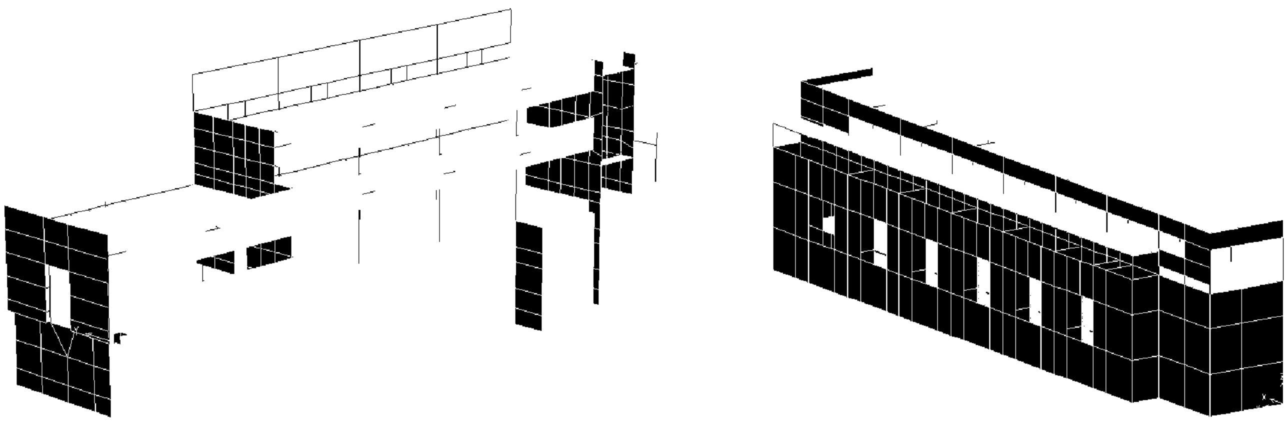


Figure 3 Models of EB2 and EB1 sections (West side view)

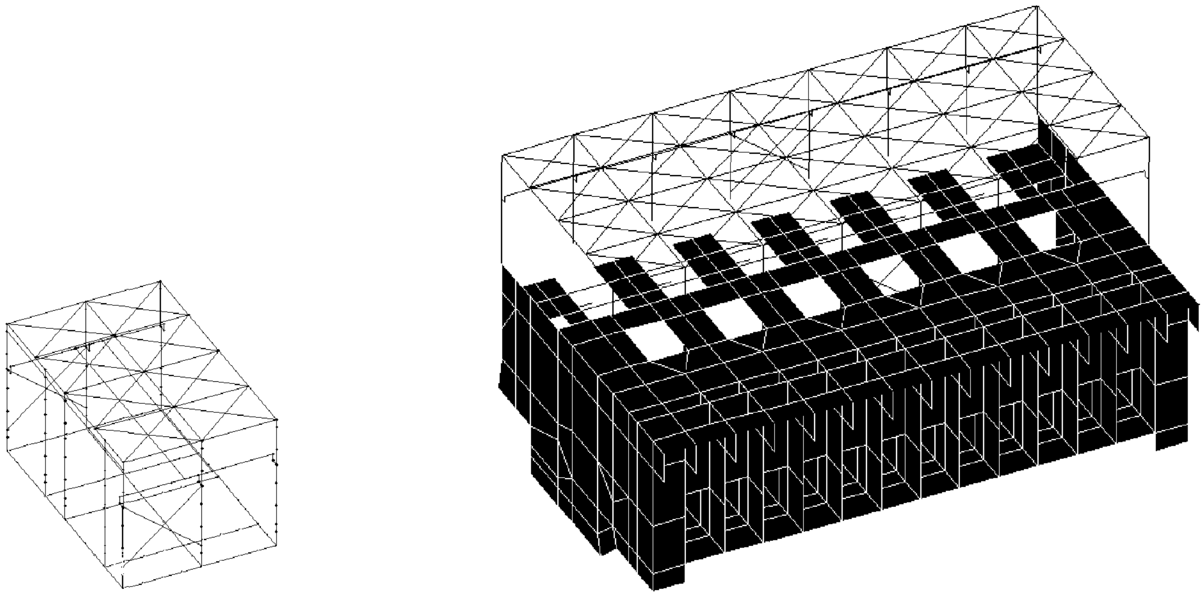


Figure 4 Models of AA and SWP sections (East side view)

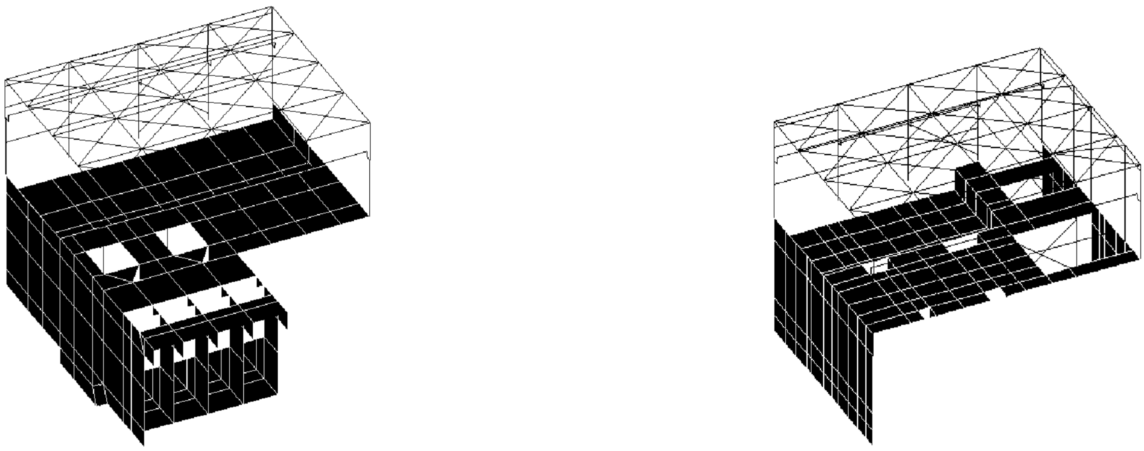


Figure 5 Models of SSWP and AWFPP sections (East side view)

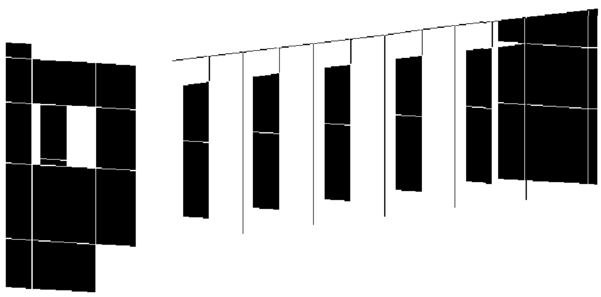


Figure 6 Model of FS section (Internal view)

4. SEISMIC EXCITATION AND SOIL MODELING

The free field ground motion characteristics are defined by free field response spectra, anchored to PGA=0.2g, [1]. For horizontal components the Review Level Earthquake (RLE) spectra envelop the Local Level Earthquake (LLE) spectra. For vertical component the LLE spectrum is higher then RLE spectrum in high frequency range. An envelope spectrum for vertical component of RLE and LLE earthquakes was developed and used in the study [2]. Plots of the ground motion spectra are presented on Figure 7.

The time histories with duration of 61 seconds were generated for the enveloped free field response spectra, Figure 8. The control point of the free field ground motion was assumed on the free surface of the soil deposit. In the Soil-Structure Interaction (SSI) analyses vertically propagating shear waves were assumed as a spatial variation of the free field ground motion.

The development of strain compatible soil properties (i.e. high strain properties) by the site response analysis was performed using available low strain soil properties of the generalized soil profile (Table 1, [3]) and degradation curves obtained for the site. The soil deposit at Kozloduy NPP site is relatively uniform down to a depth of 200m. The layer between –7.00m and –9.00m is loess-cement overlaying the existing layers of clays, sands and sandy clays. Below 265m a compacted clay stratum exists, which can be considered as an elastic half space with shear wave velocity higher then 700m/sec. A backfill layer of loess is considered to overlay the loess-cement layer. A ground water level is at about 7.00m below the ground surface.

Comparisons of the low strain and high strain G-modulus and equivalent viscous damping profile are shown on Figure 9. A variation of the high strain soil properties for the SSI analysis of the structures was defined, considering three cases based on the best estimate properties. The variation coefficients are 0.5, 1.0, 2.0 respectively. The response spectra of the degraded input motion at the loess-cement layer are compared with the response spectra at the free field surface on Figure 10.

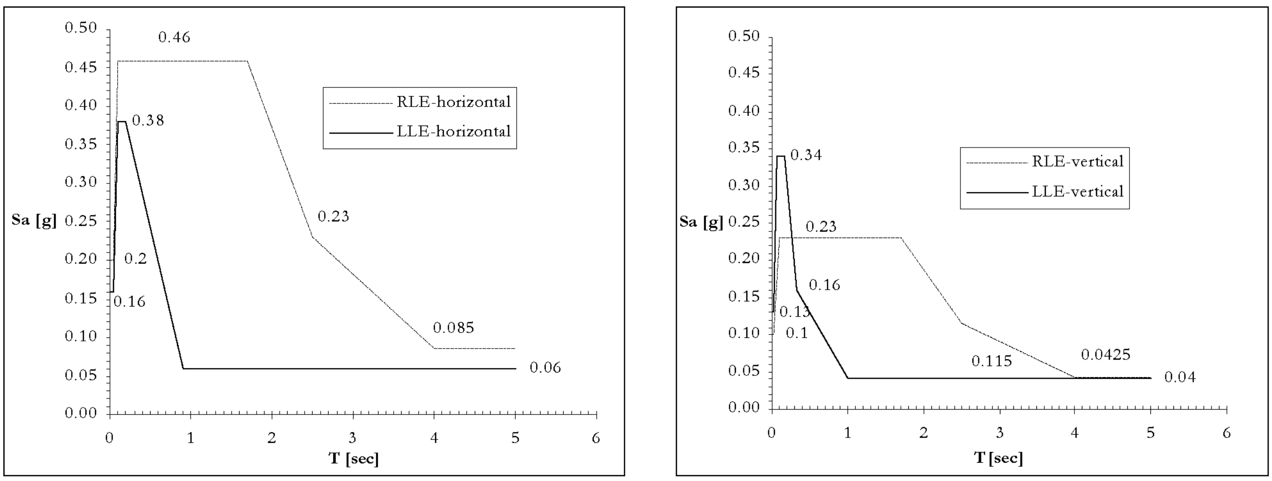


Figure 7 Free Field RLE and LLE response spectra

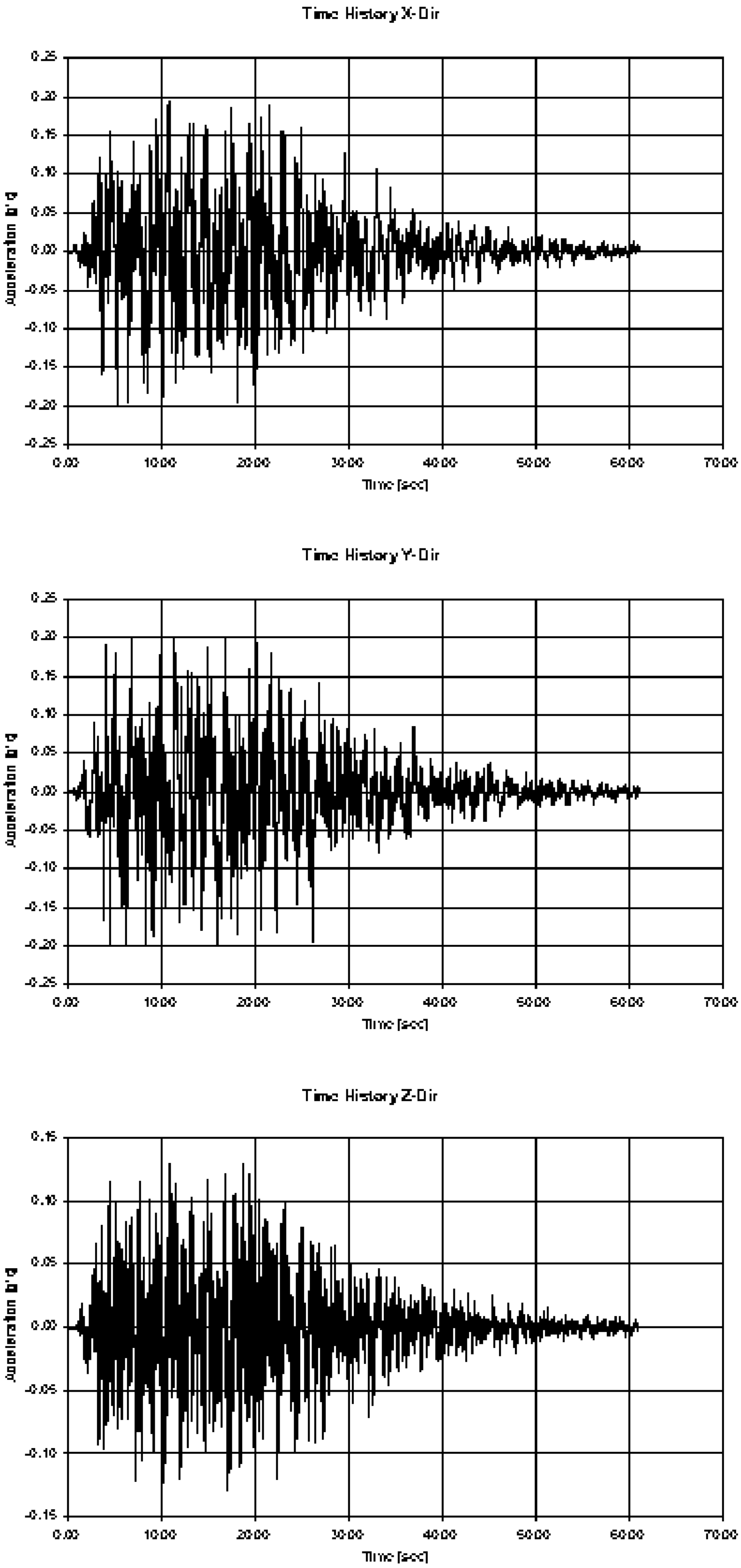
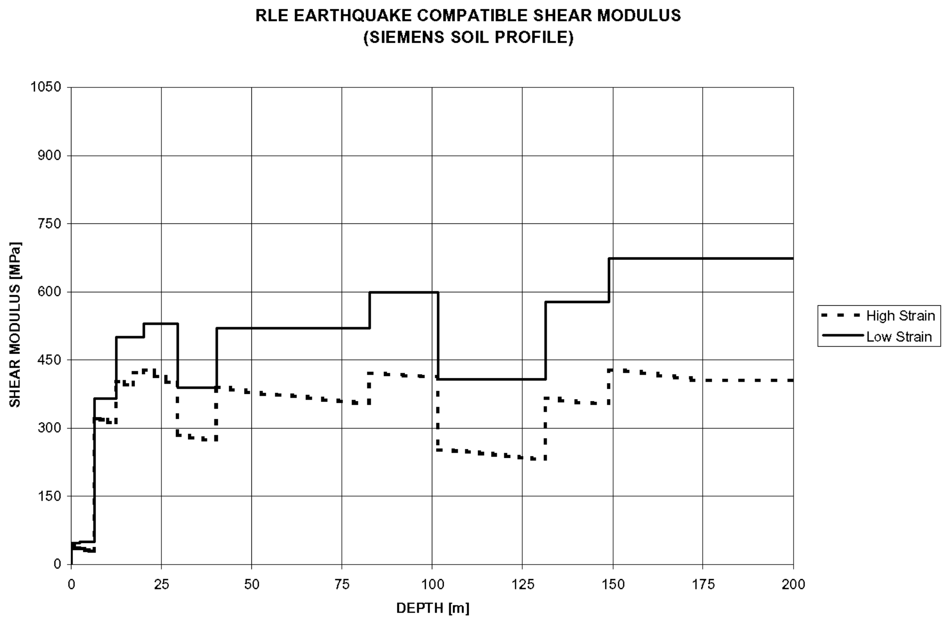


Figure 8 Free Field Time Histories



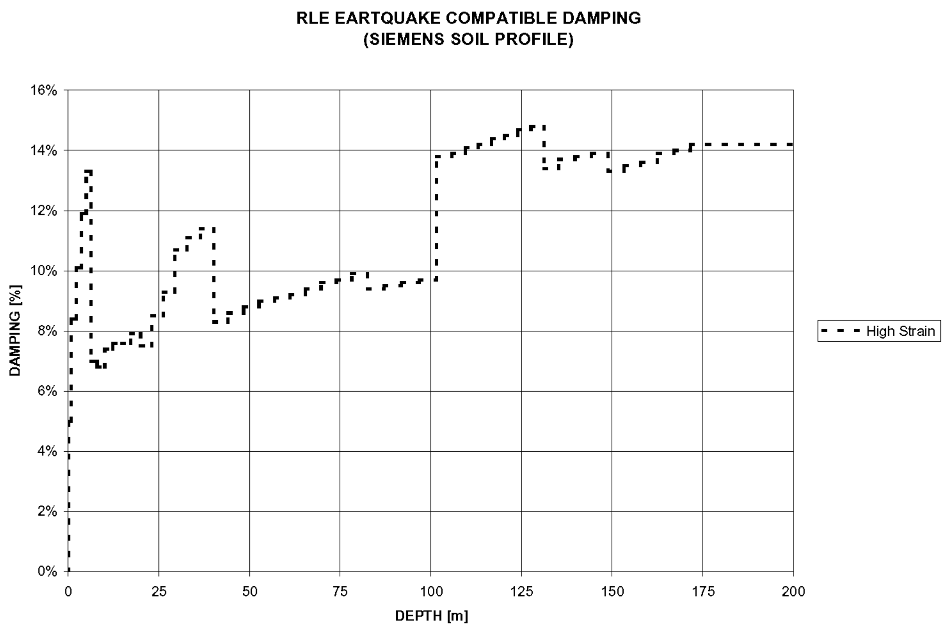


Figure 9 Comparison of Low and High Strain Soil Properties

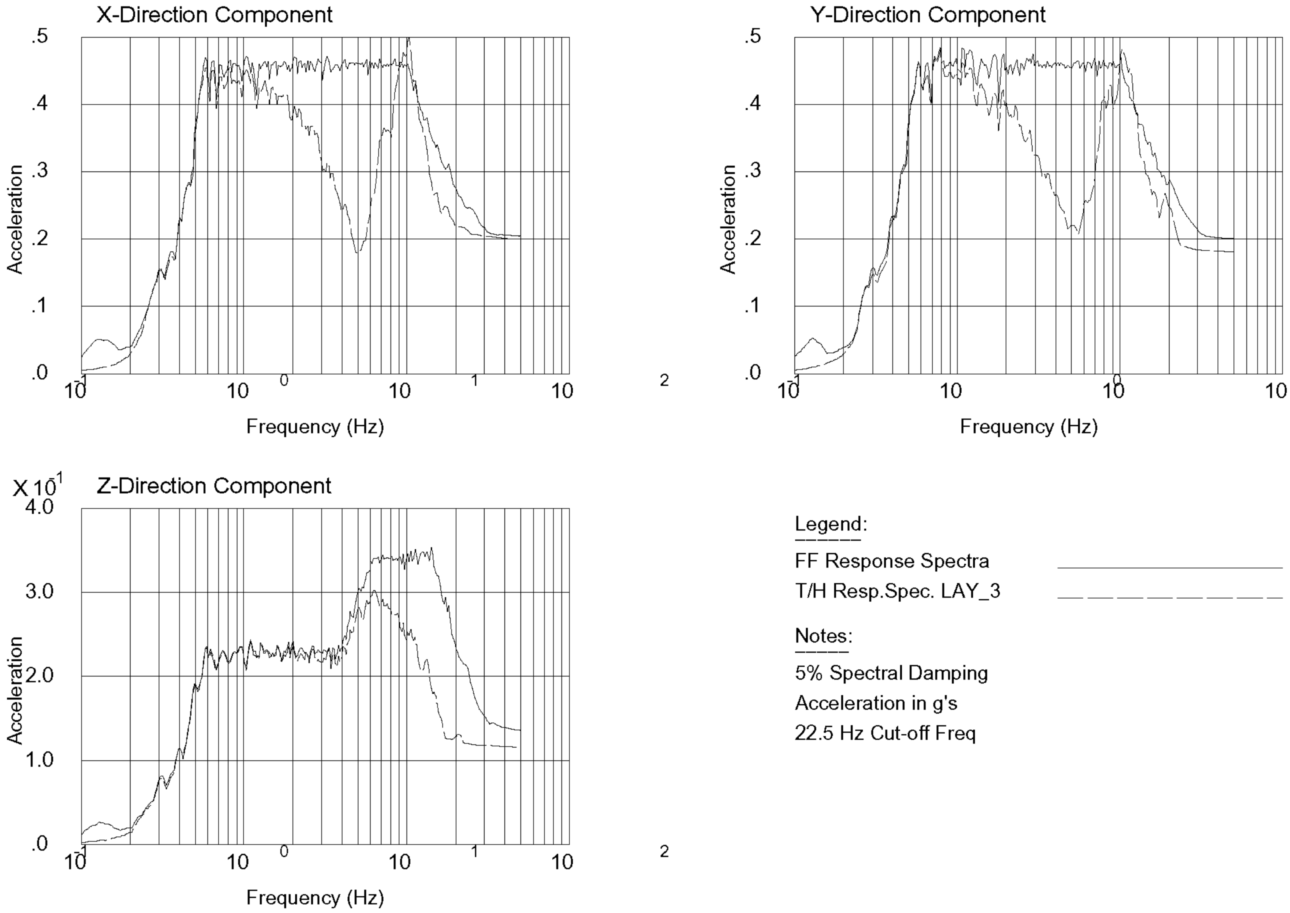


Figure 10 Comparison of Free Field Spectra with Loess-Cement Layer 3 Spectra

Table 1 Kozloduy NPP Low Strain Soil Properties of the Generalised Soil Profile

No.	From-To [m]	Depth [m]	Density [t/m³]	Poisson's Ratio	Sh. Velocity [m/s]	P. Velocity [m/s]	G-modulus [kPa]	Soil Type
1	0.0-3.0	3.0	1.60	0.42	170	470	46240	loess
2	3.0-7.0	4.0	1.60	0.44	175	540	49000	sandy loess
3	7.0-9.0	2.0	1.80	0.41	450	1180	364500	loess cement
4	9.0-13.5	4.5	1.80	0.41	450	1180	364500	clayey loess
5	13.5-18.5	5.0	2.00	0.45	500	1600	500000	gravelly sand
6	18.5-21.5	3.0	2.00	0.45	500	1600	500000	compact clay
7	21.5-31.0	9.5	2.12	0.45	500	1600	530000	sand-fine clayey
8	31.0-42.0	11.0	2.10	0.47	430	1700	388290	sandy clay
9	42.0-84.6	42.6	1.92	0.45	520	1700	519168	sand-fine clayey
10	84.6-104.0	19.4	1.98	0.44	550	1700	598950	sand-fine clayey
11	104.0-133.0	29.0	2.01	0.46	450	1600	407025	sandy clay
12	133.0-151.0	18.0	1.98	0.44	540	1600	577368	marly clay
13	151.0-175.0	24.0	2.00	0.44	580	1750	672800	marly clay
14	175.0-204.0	29.0	1.96	0.43	530	1470	550564	marly clay
15	204.0-224.0	20.0	1.98	0.37	630	1470	785862	marly clay
16	224.0-245.0	21.0	2.00	0.40	680	1700	924800	clayey marl
17	245.0-265.0	20.0	1.96	0.40	705	1760	974169	clayey marl
		265.0	2.00	0.40	>705	>1760		

5. SSI PARAMETERS AND SEISMIC RESPONSE ANALYSES

The SSI parameters refer to the foundation input motion and foundation impedance functions (impedances) that characterize the kinematic and inertial behavior of the soil-foundation system. The impedance and scattering functions are calculated only for the best-estimated soil properties, consistent with the free field spectra at Kozloduy NPP site. The impedance and scattering functions for other soil cases have been obtained from the functions developed for the best-estimated soil properties by applying appropriate scaling factors.

Program CLASSI [4] was used to calculate the impedance functions for each of the PH3 structures except for AA section, which is founded on strip foundations at surface level. A surface founded flat foundation and vertically propagating incident seismic SH-waves were assumed in the calculations. The foundations' geometry was modeled by two-plane symmetric models. 36 frequencies, which cover the whole frequency range of interest (0.1-40.0 Hz), were used in the calculations. The best estimate soil properties for each sublayer from denconvolution analysis were used. It was assumed that the foundation is founded on the loess-cement layer of the generalized soil profile. The soil layers were modeled up to 265.00 m below grade in these calculations, neglecting the backfill layer. The elastic half space was assumed below soil profile model.

Frequency independent equivalent soil springs and dampers for the best estimate soil were developed based on the impedance functions. SSI seismic response analyses in time domain were performed for simplification of the calculations and for direct generation of the design structural response forces, moments, and displacements.

Three seismic response analyses for each of the as-built structures were performed considering any of the three soil cases. 7% structural damping was used in the analyses. Eigen frequencies, mass participation ratios and composite modal damping values were obtained for each of the soil cases. The composite modal damping values were cut-off in accordance with recommendations of TOR [1]. In the seismic response analyses the deconvolved time histories for loess-cement layer were used. SAP2000 program [5] was used in the seismic response analyses.

6. CAPACITY EVALUATION OF BUILDING STRUCTURES

The governing load combination for D/C (Demand-to-Capacity) ratio assessment is taken in accordance with ACI 349-90 [6]. It is - Design force = Gravity + 0.25xSnow + Earthquake/$F\mu$ ($F = G + 0.25xS + E/F\mu$). $F\mu$ is inelastic energy absorption factor. In seismic response analyses 100% of dead weight masses and 25% of snow masses are used. The corresponding values for $F\mu$ from [1] and [2] for the relevant forces and moments are used. Capacity checks of the structural elements were performed on comparative basis according to different design codes. The D/C ratios were obtained using MathCAD templates. For columns and girders the bending moments in the both directions were taken into account.

It was found that columns in AWFPP, SSWP, SWP, and AA sections are overloaded. The shear capacity of the columns is not enough also. There is not a ductile detailing of the horizontal reinforcement of the columns.

The anchorage details of embedded parts in the pre-cast roof beams were found vulnerable for seismic loading (shear forces and torsional moments) with D/C>1.5 (see Figure 11). Anchorage of embedded parts in the girders is not appropriate for horizontal seismic loading due to the lack of ductile detailing.

The pre-stressed roof panels are very heavy. Their ribs have low capacity in torsion and shear. Anchorage of embedded parts (seats) in the ribs is not appropriate for horizontal loading. The facade panels highly influence the overall behavior of the upper structure. In the reported analyses the facade panels are modeled as masses only. The quality of panel-to-column connections is bad. Strengthening of the connections is proposed to prevent panels' out-of-plane failure or replacement of the panels with light ones as alternative.

Displacements at the top of the roof and at the crane elevation of the as-built structure were obtained. The maximum values show that an impact with neighboring structures is possible, as the structural gaps at the roofs of the sections are only 0.050 m wide.

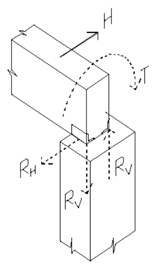


Figure 11 Force Balance at Connections Due to Horizontal Seismic Loading

7. STRENGTHENING CONCEPTS

Few upgrading alternatives were examined for strengthening of the Pump Hall. It was found that FS section does not need upgrading as well as EB2. For EB1 minor upgrading measures were prescribed related with construction of vertical braces in longitudinal direction. The investigated alternatives for Pump Hall are as follows.

7.1. Side Bracing

Side bracing can be achieved by mounting skewed rods at each column axis on East or West side of the Pump Hall. Side bracing elements take loads from the heavy roof and transfer them to strip foundations. Additional longitudinal bracing of skewed rods was considered. This type of strengthening would require dismounting of cladding panels as well as facade panels and mounting of light panels instead.

7.2. Dismounting of Upper Structure and Shielding of the Equipment

The entire upper structure was considered for dismounting. A portal crane was proposed to be mounted instead. The portal crane path would pass through columns' axes, which exist at present. Placing of removable metal shields over equipment items (pumps, sieves) will protect them from environmental impacts.

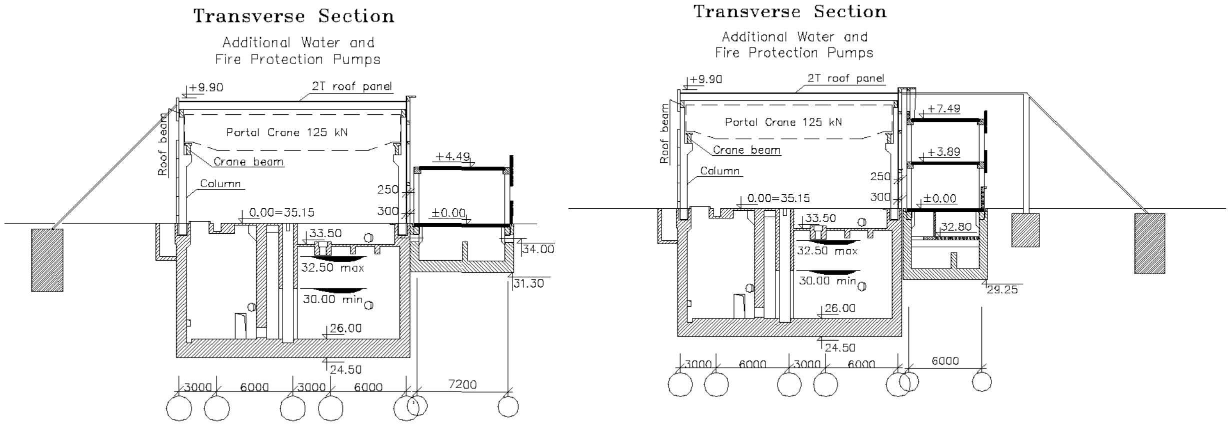


Figure 12 Side Bracing for AWFPP (East and West side)

7.3. Replacement of Heavy Roof with Light Roof Structure

Replacement of the heavy roof with steel trusses and light roof panels as well as replacement of the facade panels with light ones was proposed. This measure involves comprehensive construction works, but is the best one from structural point of view.

Replacement of heavy roof has following advantages:

- Short load path through connections with significant capacity.
- Decreasing of seismic internal forces
- Arrangement of lateral resisting frame, which can transfer seismic forces to the foundation.
- The cost of construction works is comparable or less than that of the other alternatives.
- This measure is applicable for all sections of PH3 Building between axes 1 to 18.

The minor disadvantage is that construction works should be performed during the outage of the unit and preferably during summer. Heavy equipment (crane) would be required during construction works.

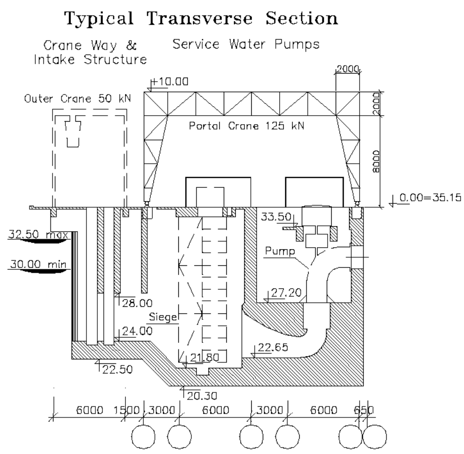


Figure 13 Dismounting of Upper Structure

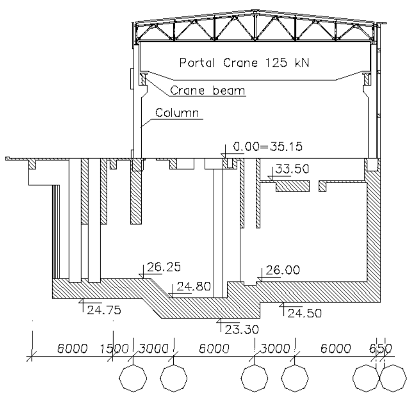


Figure 14 Replacement of Heavy Roof with Light Steel Structure

8. CONCLUSIONS

Based on the results of the evaluations of the forces and stresses a few strengthening concepts were developed and the most appropriate one was selected. The amount of hardware (steel and concrete) and engineering work to retrofit the building structures has been estimated on the basis of the conceptual upgrading and strengthening measures. As result of the study design floor response spectra (FRS) for as-built and upgraded structures were also generated. FRS are needed for the seismic qualification of equipment, piping and components and are also essential in case of replacing equipment, ventilation systems, electric and I&C equipment, cable trays, etc.

Seismic Upgrading of structures of Pump House 3 building will be performed during the next step of the project for Modernization of Units 5 & 6 at Kozloduy NPP. When strengthening measures are implemented the aim of the project for seismic qualification of PH3 for new seismic hazard of the site will be reached.

REFERENCES

1. Terms of References fir Item 21241, BEP of Modernization Program for Units 5&6 of Kozloduy NPP, February 1999
2. Artificial Time Histories of RLE and LLE, Siemens report #0022, 1999
3. Earthquake Adopted Soil Profile for Free-Field at Site Kozloduy, Siemens report #0023, 1999
4. Wong, H. L. and Luco J. E., Soil-Structure Interaction: A Linear Continuum Mechanics Approach (CLASSI), CE 79-03, University of Southern California, Los Angeles, CA, 1980.
5. SAP2000 Nonlinear Version 7.12, Three Dimensional Static and Dynamic Finite Element Analysis and Design of Structures, CSI, Berkeley, California
6. ACI 349-90, Code Requirements for Nuclear Safety Related Concrete Structures, 1990
7. DOE-STD-1020-94, Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities, USA Department of Energy, April 1994