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Seismic Upgrading of the Main Structure of Unit 3/4 Kozloduy NPP

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ABSTRACT

The seismic analysis and the subsequent seismic upgrading design of Unit 3 and 4 of Kozloduy NPP in Bulgaria are described in this paper. The aim of the analysis is to identify the critical scenaria of seismic failures, to assess realistically the seismic capacity of the building structures and to propose seismic upgrading of the structures with highest probability of failure. The analyses and upgrading are performed based on a complex 3D finite element model accounting for the soil-structure interaction and the spatial structure interactions. The results of the analysis of the as-built structures show that the most seismically vulnerable structure is the turbine building, the most probable failure scenario is the damage of the roof structure due to anchor failure, the second important scenario is damage of the RC columns of the turbine hall.

The concept of the seismic strengthening includes three main approaches: local seismic upgrading, structural system upgrading and strengthening of the joints between elements.

The effectiveness of the seismic upgrading is measured by the change of the probability of failure. For the most dangerous scenario, the roof failure, the probability of failure after the introduced upgrading will decrease about 100 times.

INTRODUCTION

The Units 3 and 4 of Kozloduy NPP in Bulgaria are of type VVER 440-230. Both units are designed in the early 70's. Generally the safety related structures do not have seismic design. In 1977 Unit 1, which is of the same type, was subjected to the strong Vrancea earthquake, M=7.2. The shaking was not recorded on site. The maximum acceleration of the strong motion has been assessed to 0.1g. There were no damages observed both, in the structures and the equipment. After that event a program for seismic hazard re-assessment and seismic upgrading has started. In 1992 the last seismic hazard re-assessment program proposed a review level earthquake (RLE) defined by broad band free field site specific response spectrum (fig.1) and maximum acceleration of 0.2g. The studies described here after are based on these RLE characteristics.

Unit 3 and 4 are twin structures. The main building of one unit consists of reactor building, turbine building, longitudinal electric building (electric spreading rooms), transverse electric building (motor control center, electric spreading rooms) and ventilation center. All those buildings are interconnected in one building complex. Unfortunately they have quite different dynamic behavior and the interfaces are very complex [1].

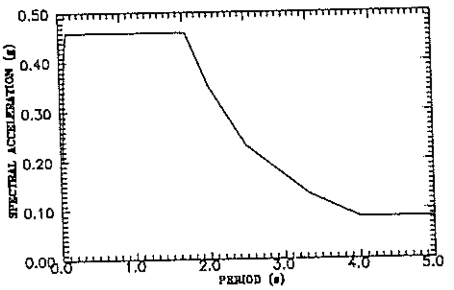

Fig. 1. Free field response spectra for Kozloduy NPP site RLE

All buildings are RC frame structures. Only the reactor compartment is constructed of RC walls. The foundations of the frame structures are single footings. The reactor compartment is founded on a reinforced concrete mat.

Deep alluvium strata characterize the soil profile of the site. The average low strain shear wave velocity is about 450 m/s. The foundation depth is approximately 7m below grade.

DYNAMIC ANALYSIS OF THE AS-BUILT STRUCTURE

The two most important parts of the main building complex are the reactor building and the turbine hall. The longitudinal electric building connects these parts. Attached to the main structure are the ventilation center building and the transverse electrical building. A general layout is presented in fig.2.

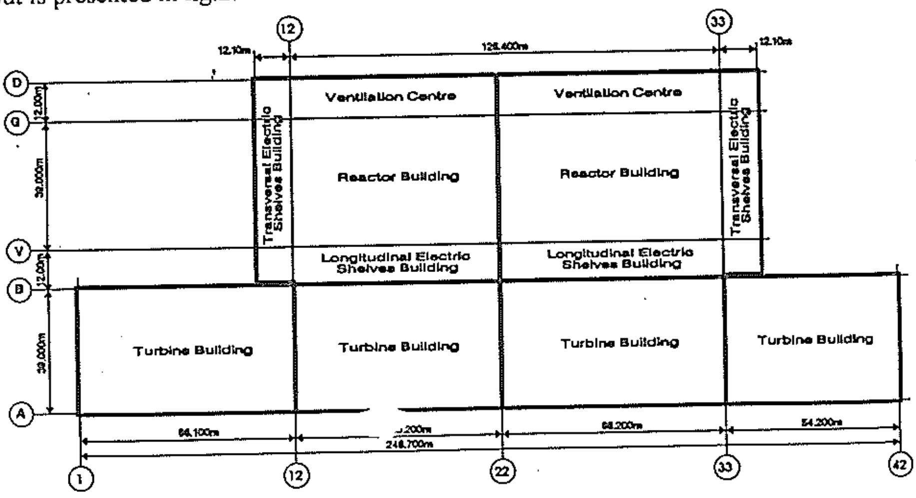

Fig. 2. General Layout of Unit 3/4

The turbine hall is monolithic reinforced concrete frame structure. It has 39m span. The roof consists of steel trusses covered by heavy RC panels.

The reactor building is also monolithic reinforced concrete structure. It consists of walls up to the elevation 10.5m (reactor compartment). Above that elevation the reactor building is frame structure with 39m span. The roof is similar to that of the turbine hall.

The longitudinal electric building connects the reactor building and the turbine hall. It is constructed on the columns of both buildings which are connected by prefabricated beams and panels (the floors of the electric building).

As referent unit in the analysis is used unit 3.

Both static and dynamic (seismic) analyses are performed [1]. The loads are the dead weights of the structure and equipment, the live loads (including also the snow) and the seismic loads.

A detailed 3D finite element model is developed. The model consists of 1345 beam elements and 2298 plate elements. The RC slabs and walls are modeled by plate elements. The frame structures are modeled by beams. The outer wall panels (they are not bearing elements) are modeled only as lumped masses. Attention is paid to represent accurately in the model the actual releases of the prefabricated elements. The designed documentation is checked by walk-downs to reflect the as-built status of the structures.

The primary coolant system is modeled by beam elements and introduced in the structure model. This is done because of the significant weight of the reactor systems and the possible interaction with the building structure. The steam generators are attached to the structure by snubbers. Their dynamic behavior is also of interest.

The soil-structure interaction is modeled by spring-dashpot systems. The seismic excitation on free field is transferred to the foundation level by deconvolution.

The dynamic analysis is performed by the Stardyne 4.0 computer code. In Table 1 the first 10 eigen frequencies are presented.

Table 1. Eigen frequencies of the as-built structure model

Mode	Frequency	Period	Mode	Frequency	Period
1	1.46	0.68	6	2.81	0.35
2	2.37	0.42	7	2.92	0.34
3	2.54	0.39	8	3.17	0.31
4	2.58	0.38	9	3.25	0.307
5	2.69	0.37	10	3.33	0.30

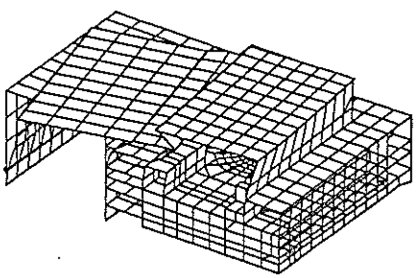

Fig. 3. First mode of vibration

The first mode of vibration is presented in fig.3. The analysis of the mode shapes is showing that there is intensive rotation of all attached buildings (turbine hall, electric building and ventilation center) around the stiff reactor building. There is also intensive rotational motion in upper frame part of the reactor building.

The seismic effects are computed by time history analysis. The three components of the strong motion are applied simultaneously. The modal superposition technique is used. In the analysis 350 modes are computed. The activated modal mass is about 90 percent of the total mass. The material damping used is 0.07. The radiation damping is introduced for each footing. The modal damping is a composite one. For capacity analysis only the maximal computed effects are used.

The seismic effects (the member forces) are combined with the static one. The most unfavorable combination is aimed. In most of the cases these combinations are very conservative, e.g. maximum bending moments from static and dynamic analysis are added while the axial forces from the static analysis are reduced by the normal forces from dynamic excitation.

The capacity assessment is performed according to the Bulgarian standard for reinforced structures (limit state concept). The nonlinear reinforced concrete behavior is accounted by introducing member ductility. The typical member ductility for columns is 1.25 and for beams 1.5. No ductility is allowed for ties and connections.

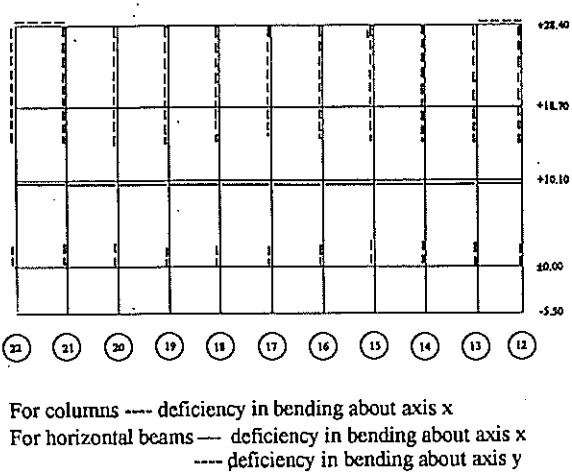

Fig. 4. Row A. Capacity check results

A typical result of the capacity check is presented in fig.4. It shows the failure zones in column row A (outer column row of turbine hall). Generally all the outer column rows are overloaded. The deficiency in the stiffness is presented both in longitudinal and transversal direction. The computed displacements at several control points are bigger than the available gaps of the expansion joints, i.e dynamic impact between adjacent structure is possible. All these results are the reason for the proposed seismic upgrading.

DESCRIPTION OF THE SEISMIC UPGRADING

The seismic upgrading is performed accounting for the requirements of the existing technology scheme of the plant, i.e all the proposed measures should not affect the main technological requirements for arrangement of the piping and equipment. Another requirement is to optimize the upgrades in order to achieve maximum results, i.e. high seismic capacity and to reduce cost of construction and losses due to long outage.

Three different approaches are used for upgrading of the structure. The first approach is based on the local strengthening of elements with low capacity. It means in most of the cases to add more material, i.e. to increase cross section or inertia moment of the element. Usually this could be realized by jacketing of the existing elements or by introducing new elements acting with the existing ones simultaneously and in the same way, e.g. a parallel beam or column. Usually this approach is not very effective.

The second approach is based on the understanding of the global seismic behavior of the structure. By adding new elements it is aimed to improve the unfavorable response modes. Basically by adding new elements a better load path is achieved, the overloaded elements are unloaded and the not fully utilized elements are additionally loaded. This approach could be very effective.

The third approach is especially important for prefabricated structures. Usually they have RC elements of good quality. The loading capacity of those elements can not be used thoroughly because of the bad implementation of the element joints. The local improvement of the joints would immensely contribute for better dynamic behavior of the whole structure.

A general view of the model of the upgraded structure is presented in fig.5.

The most unfavorable mode of seismic response for the main structure is the rotation of all the attached buildings along vertical axis going through the virtual center of stiffness, which is located in the reactor building. Because of this predominant feature of the response the most important upgrading are the new diagonal braces in the column rows placed at the periphery of the structure. Although these upgrading could not change the character of the response they contribute very much for improving the seismic safety of the frame structures. The second major upgrading is the roof connection between turbine hall and reactor building.

The aim of these new elements is also to help resisting the rotation response of the structure, to create continuity in the roof disk, to reduce the horizontal shear force in the columns of reactor building due to response of turbine hall. Improving the joints of the prefabricated elements of the longitudinal electric building increases the effect of these upgrading. Detail of the upgrading is shown in fig.6. The improvement (upgrading) of the joints is proposed also for the transversal electric building and for the roof-to-column connections in the reactor building and turbine hall.The first 10 eigen frequencies of the upgraded structure model are presented in table 2.

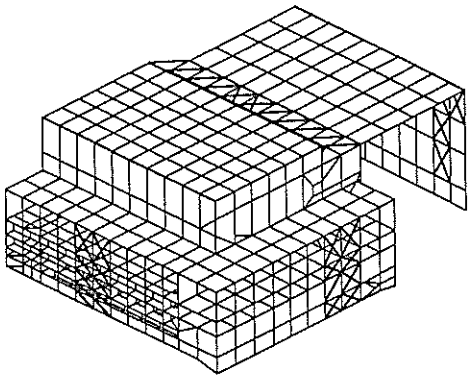

Fig. 5.Upgraded Structure model

The first natural frequency is increased from 1.46Hz to 2.1Hz. This is the only significant shift in the frequencies. Although the seismic upgrading is very limited (it is about 300t steel) the effect on the seismic capacity is significant. After the implementation of the upgrades there is no deficiency in capacity or in structure deflections.

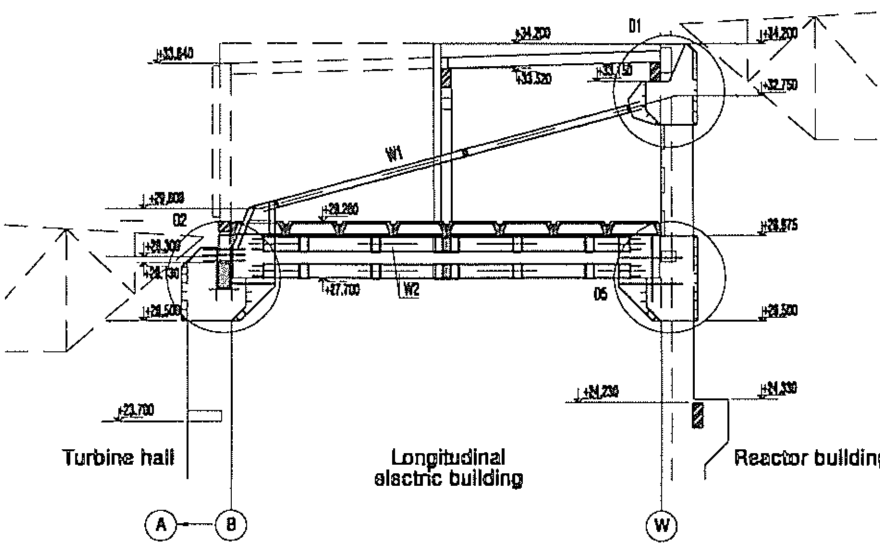

Fig. 6. Detail. Upgrading of roof between turbine hall and reactor building

Table 2. Eigen frequencies of the upgraded structure model

Mode	Frequency	Period	Mode	Frequency	Period
1	2.1	0.47	6	2.91	0.34
2	2.43	0.41	7	3.16	0.31
3	2.56	0.39	8	3.23	0.309
4	2.66	0.37	9	3.33	0.30
5	2.88	0.35	10	3.39	0.29

Table 3. Parameters of the fragility curves for different structures of the upgraded Unit 3/4

REACTOR BUILDING				
	A_F	HCLPF	u^r	u^m
1. Roof failure of the reactor building				
- failure of the truss anchor bolts	0.518	0.29	0.141	0.193
- column failure at level 28.40 – row W	0.630	0.32	0.226	0.193
- column failure at level 23.80 – row G	0.618	0.31	0.226	0.193
2. Column failure at level 18.70 – row W				
- bending failure	0.443	0.23	0.226	0.193
- shear failure	0.422	0.22	0.202	0.187
TURBINE HALL				
3. Roof failure of the turbine hall				
- failure of the truss anchor bolts	0.510	0.29	0.141	0.193
- column failure at level –5.50 – row A	0.518	0.26	0.226	0.193
- column failure at level 18.70 – row A	0.480	0.24	0.226	0.193
TRANSVERSAL ELECTRIC SHELVES				
4. Column failure along axis 10				
- at level -2.80	0.584	0.29	0.226	0.193
- at level 0.00	0.778	0.39	0.226	0.193
5. Failure of the connection between the prefabricated girder and cast in situ wall of the reactor building				
- at level 18.70	0.540	0.26	0.226	0.370
LONGITUDINAL ELECTRIC SHELVES				
6. Failure of the connection between the prefabricated girder and cast in situ wall of the reactor building				
- at level 20.80	0.540	0.23	0.206	0.320
- at level 28.40	0.950	0.31	0.206	0.470
VENTILATION CENTER				
7. Column failure along row D – level 10.10	0.460	0.23	0.226	0.193
10. Failure of the connection between the prefabricated girder and cast in situ wall of the reactor building – level 18.70	0.720	0.26	0.206	0.420

COMPARISON OF FAILURE PROBABILITIES

Based on the analyses performed both on the as-built structure and the upgraded structure the failure probabilities are computed for several important scenarios. For each scenario a representative fragility curve is developed. The probability of failure is computed by convoluting the fragility curve and the seismic hazard curve for the site. Brief summary of the fragility curve characteristics for some of the scenarios are presented in table 3. The corresponding probabilities of failure are presented in table 4.

Table 4. Comparison between 85% total probability of failure of the structures of Unit 3 / 4 before and after the upgrading

Scenario	REACTOR BUILDING		
	Roof failure of the reactor building	**As built**	**Upgraded**
1.	- failure of the truss anchor bolts	3.482E-04	3.259E-06
2.	- column failure at level 28.40 – row W		1.929E -06
3.	- column failure at level 23.80 – row G		2.106E-06
	Column failure at level 18.70 – row W		
4.	- bending failure		1.229E-05
5.	- shear failure		1.197E-05
	TURBINE HALL		
	Roof failure of the turbine hall		
6.	- column failure at level –5.50 – row A		4.953E-06
7.	- column failure at level 18.70 – row A	3.894E-04	9.973E-06
8.	- failure of the truss anchor bolts		7.516E-06
	TRANSVERSAL ELECTRIC SHELVES		
	Column failure along axis 10		
9.	- at level -2.80	1.909E-06	2.733E-06
10.	- at level 0.00		7.203E-07
	Failure of the connection between the prefabricated girder and cast in situ wall of the reactor building		
11.	- at level 18.70	7.170E-05	4.710E-06
	LONGITUDINAL ELECTRIC SHELVES		
	Failure of the connection between the prefabricated girder and cast in situ wall of the reactor building		
12.	- at level 20.80	8.804E-05	1.233E-05
13.	- at level 28.40	1.449E-04	2.313E-06
	VENTILATION CENTER		
	Column failure along axis D		
14.	- at level 10.10		9.595E-06
	Failure of the connection between the prefabricated girder and cast in situ wall of the reactor building		
15.	- at level 18.70		7.047E-06

The fragility curves are developed according to the methodology described in [2]. The material strength variation, the element ductility, the ultimate capacity, the load combination and the conservatism of the analysis performed are considered.

The analysis of the failure probabilities of the as-built structure shows that one probable failure will be damage of prefabricated element joint. The worst consequences are to be expected after failure of the roof structure of turbine hall or the reactor building.

After the proposed upgrading these probabilities are changed (decreased) significantly and the failure probability for the most of the scenarios is about 10^{-6}. It is interesting to mention that because of the applied upgrading approach some of the failure probabilities after the upgrading are changed worse, i.e. there are structural elements which are additionally loaded. Such effect should be expected after the global redistribution of the seismic forces. However the failure probabilities are within acceptable limits.

CONCLUSIONS

The presented analyses are showing that for effective seismic upgrading detailed seismic investigations are needed in order to understand the significant response modes of the structures. In the presented case this is the rotation of the attached flexible structures to the stiff reactor building. Based on this an upgrading approach is applied to increase the seismic resistance for the predominant motion. The second significant approach applied is the strengthening of the prefabricated element joints. Although it is very simple it allows use of the available element capacity.

The use of elements of the probabilistic safety analysis for the aim of the seismic upgrading is very useful. It helps to prioritize the observed deficiencies and to select a proper strategy for improvement. Important benefit is also the possibility to assess the effectiveness of the proposed upgrading.

References

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