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SEISMIC EVALUATION OF EXISTING NUCLEAR FACILITIES

SEISMIC UPGRADING OF VVER 440-230 STRUCTURES, UNITS 1/2, KOZLODUY NPP

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ABSTRACT:

The purpose of this paper is to present final results from a big amount of computational work in connection with the investigations of the possibilities for upgrading of VVER 440-230 structures, units 1/2, Kozloduy NPP.

1. Introduction

The first NPP's with VVER-type reactor structures have been designed and built without consideration of seismic influences and on the base of simplified calculation. It's the case with NPP Kozloduy. In connection with change of site seismic characteristics and safety demand the necessity rise of checking up and ensuring of technology systems seismic resistance of the existing 440-MW VVER-type units in Bulgaria.

2. Description of the existing structure

The Kozloduy NPP consists of four Units of type VVER-440/230 and two units of type VVER-1000. Units 1 and 2 are of the first type and they are constructed as twin units, i.e. Unit 2 is a mirror image of Unit 1 with a temperature expansion joint in between. The layout of Units 1 and 2 is schematically shown in Figure 1. The expansion joint is in axis 22. The main building is composed essentially of two parts - the reactor building (between rows C and D - Figure 2) and the turbine hall (between rows A and B). Between them is located the longitudinal intermediate part. Next to the reactor building are the smaller buildings of the ventilation centre and the control rooms connected to the reactor building.

The reactor building consists of many massive reinforced concrete walls, shells and slabs irregularly distributed. The roof structure is constructed by steel trusses mounted on reinforced concrete columns. The turbine hall is a regular frame structure - longitudinal RC frames and steel trusses in transverse direction with hinge joints between them and the RC columns. The later are founded on separate foundations. The roof is made of prefabricated RC panels. The turbine hall structure is divided in two equal parts by an expansion joint of 5 cm in axis 12. The longitudinal intermediate building is constructed mainly by precast RC girders and floor panels. This part connects the reactor building and the first part of the turbine hall. The second part is independent.

3. The three dimensional model of the structure

Previous investigations (1,2,3,4) proved the necessity of creating a complex 3D model of the main structure. The entirely different dynamic behaviour of the reactor building and the turbine hall lead to some spatial effects in the seismic response of the structure. The two parts of the turbine hall structure (separated by an expansion joint) have also different behaviour.

Unfavourable torsional effects appear and dominate the seismic response. As a result some structural elements (beams and columns in the turbine hall structure) as well as the longitudinal intermediate building will be overloaded during on earthquakes. That is why a detailed three-dimensional finite element model of the soil-structure system has been used.

The seismic input motion at the foundation level is computed by deconvolution of the design "free field" motion represented by three components of a generated acceleration time history. The local geological conditions are taken into consideration. The design "free field" spectrum and the response spectrum of the N-S component at foundation level are shown in Figure 3.

In the model the soil is represented by springs and dashpots corresponding to its stiffness and damping characteristics.

The spatial structural model consists of 3-D beam elements with 6 degrees of freedom at each node and 3-D rectangular hybrid finite elements with 5 degrees of freedom per node. All columns, longitudinal beams and girders and roof trusses are modelled as beam elements. The roof and floor panels, slabs and shells are modelled with rectangular elements.

4. Investigation of the original structure

Dynamic and static analyses of the original structure are performed using program STARDYNE. The verification of the mathematical model is made using the results from a full-scale test. The first mode of vibration is shown in Figure 4 and the fourth mode - in Figure 5. The main characteristics of the response are:
- the response of the independent part of the turbine hall building (between axes 1 and 12) is primarily in transverse direction. The rotation of that part can be clearly seen.
- the response of the other part (between axes 12 and 24) is predominantly in longitudinal direction.
- the intermediate building is loaded in an unfavourable way because of the different stiffness of the reactor building and the turbine hall.
- the displacements of some control nodal points (at the ends of the turbine hall "tail") are larger than the permissible ones.

Several variants of combination of the internal forces due to the static and dynamic loading are performed in order to get the most unfavourable loading condition. The position of the crane is changed in different places of the turbine hall. The bearing capacity of all structural elements is checked. The girders at the upper levels in the longitudinal frames of the turbine hall are assessed that they could not resist the respective forces. The bearing capacity of almost all columns in raw "A" and some of columns in row "B" is found as insufficient. The final conclusion is that the structure should be upgraded.

5. Investigation of the upgraded structure

The basic idea of the structure upgrading consists in an increasing of the stiffness by adding of additional elements - mainly diagonal bracing for the RC frames, steel stretch bars, girders, etc. The distribution of those elements in the existing structure is of great importance because the stiffness concentration should be avoided. The places of the additional elements should be in accordance with the technological requirements too. The stiffness of the existing structure could be increased also by strengthening of the structural elements cross section. The existing rigid structural elements (e.g. the stair-cases) should be used to support the more flexible parts of the structure.

Different variants of strengthening are analyzed (5). For each separate case a capacity checking of the bearing structural elements is performed. The displacements of some nodal points are controlled also.

Several alternatives of upgrading are investigated in order to find the optimum solution. The final variant (Figure 6) incorporates the following additional elements and connections:

1. Steel bracing diagonals 2L 125/125/10 in the spans of the RC frames shown in Figure 6.

2. Steel stretch bars I30 along axis 10 (from row"B" up to the end span of diagonals) at four levels in control room.

3. The girders at level 18.70 m in whole row "A" and in the "tail" only of row "B" are strengthened connecting the two girders in a "box-like" cross section.

4. All columns in row "A" and the columns in the "tail" of row "B" are strengthened in the upper part (over the crane path) by steel plates with dimensions 20/2 cm placed in the corners of each column.

5. There are stiff beam connections between two adjacent columns (forming the expansion joint) at all levels of girders in longitudinal frames.

6. In the intermediate building (between rows "B" and "W") in the spaces between levels 20.80 m and 28.40 m "K-bracing" 2L 125/125/10 are put.

7. There is a steel connection (2L 125/125/10) between turbine hall, row "B" at level 28.40 m and the reactor building, row "W" at level 33.60 m.

8. There are two inclined connections (2U20) between control rooms and the frame in row "B".

9. Additional steel girders are put between the control room and the longitudinal frame of the turbine hall.

The dynamic analysis of the "upgraded model" shows that the added stiffness is sufficient - the displacements of some characteristic nodal points are smaller than the allowable ones. The reduction of the displacements is shown in Table 1. The first mode of vibration is shown in Figure 7. The fundamental period is reduced considerably.

A capacity check of all structural elements is performed according to Bulgarian Code for design of concrete and reinforced concrete structures (Sofia, 1988). The checking is done for eccentric compression or eccentric tension about the two principle axes of the cross section. As a criterion for safety of the elements is assumed the ratio of the bending moment of maximum internal forces to the moment of the external forces. A ductility factor is used element by element to account the nonlinear capacity of the RC members. The factor used for columns is 1.25 and for girders 1.5 respectively.

Several positions of the crane are investigated. First the crane is put in axes 20,21,22. The result is overloading of the columns at these axes. This is the reason to put "K-bracing" in the intermediate building. The second position of the crane is in axes 10,11 and 12. This place is a typical one because there is an expansion joint between the two parts of the building. The third position of the crane is in axes 5,6 and 7 (middle of the "tail" structure).

Looking at the shear force values in all elements there are not considerable problems. Nevertheless additional connection are needed between the top of columns (level 28.40) and steel trusses.

6. Optional upgrading of the structure

The idea is to use the possibility of fixing the parking position of the crane and perform upgrading for that particular load case. In that way a considerable amount of upgrading structures could be saved. The new option is described as it follows:

1. Crane in the turbine hall is located in axes 12 13 and 14. This is one usual parking position of that crane.

2. Static and dynamic calculation of the complete building is performed only for that particular crane position.

As a result of this analyses the following reduction of the already proposed upgrades can be done:

- remove the upgrades of the RC columns above level 18. m in row "A" and "B" everywhere except row"A", axes 13 and 14.
- remove the "K-bracing" in axes 22,21,20 and 19.

A precondition for acceptance of that optional proposal is the elaboration and the strict control for implementation of an operational procedure to use the crane in turbine hall. The procedure should minimize the crane stay in other positions and should assure the parking position as described.

7. Conclusion

The main building of Unit 1/2 is a complicated structure - there is a great variety in height and stiffness of the different structural parts. The analyses of the seismic behaviour have shown weak elements that should be improved. This is achieved by strengthening of the building. There are two main requirements - decreasing the large displacements and increasing the bearing capacity of some structural elements. In the case of Unit 1/2 the structure is upgraded by adding additional stiffness but also by connecting stiff structural elements with more flexible parts in order to redistribute seismic forces. As a whole the structural stiffness is increased and the displacements due to seismic response are decreased. The bearing capacity of the upgraded structure is checked, the dynamic behaviour of the upgraded building satisfies the requirements for seismic safety of critical facilities.

The seismic upgrading of existing NPP is usually very complicated and expensive. The engineering solution is to find the optimum variant between the economy and safety of the upgrading and to satisfy these conflicting requirements.

8. Acknowledgements

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9. References

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TABLE 1

NODAL POINT DISPLACEMENT (m)
ORIGINAL AND UPGRADED MODELS

Nodal point	Along X-direction		Along Y-direction	
	Original model	Upgraded model	Original model	Upgraded model
Row "A" , Level 28.40				
797	.3642	.0374	.1293	.0294
792	.3387	.0362	.1294	.0294
Row "B" , Level 28.40				
803	.3643	.0374	.0661	.0271
796	.3387	.0336	.0662	.0271
Row "A" , Level 18.70				
594	.2719	.0287	.0863	.0304

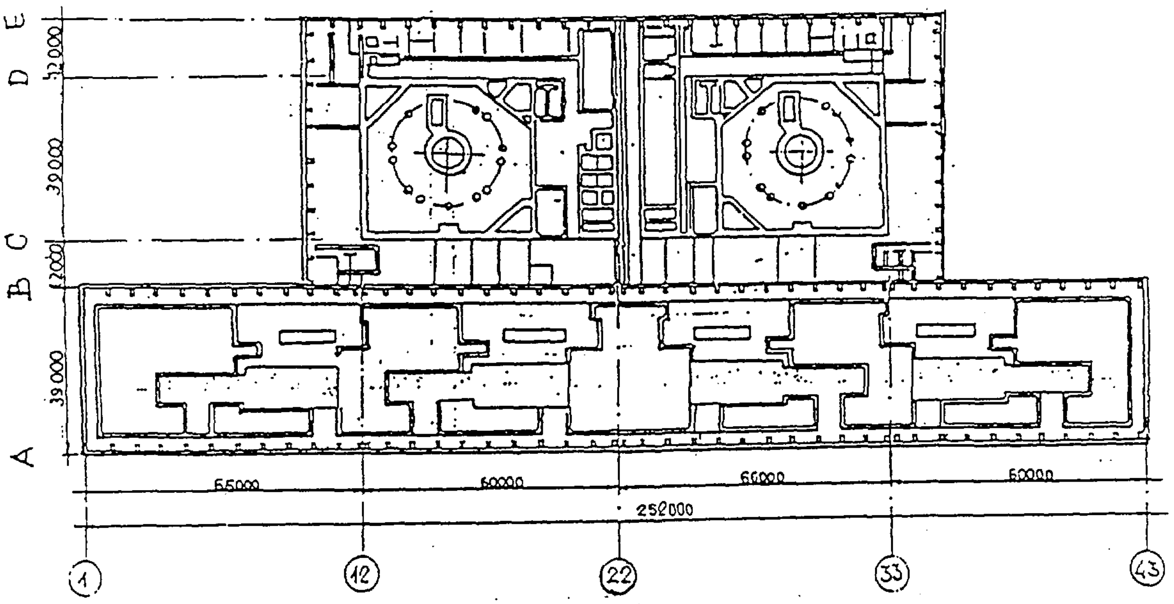

Figure 1. Main building, Layout

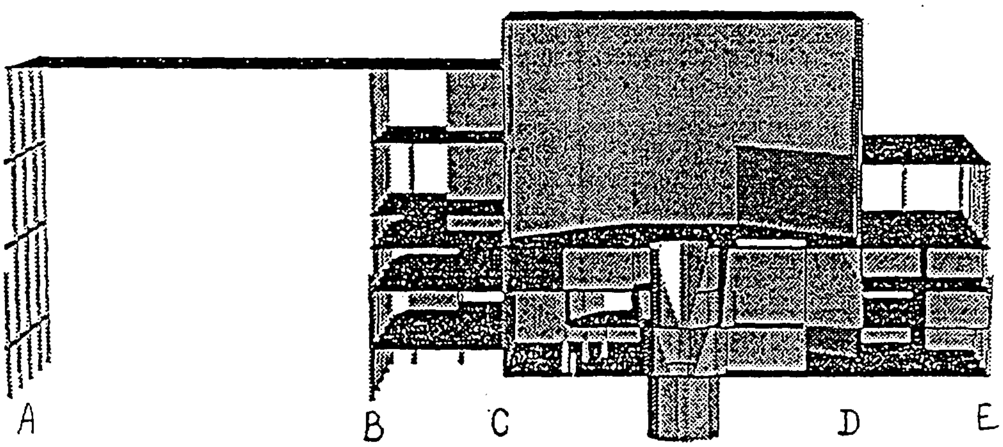

Figure 2. Main building, cross section

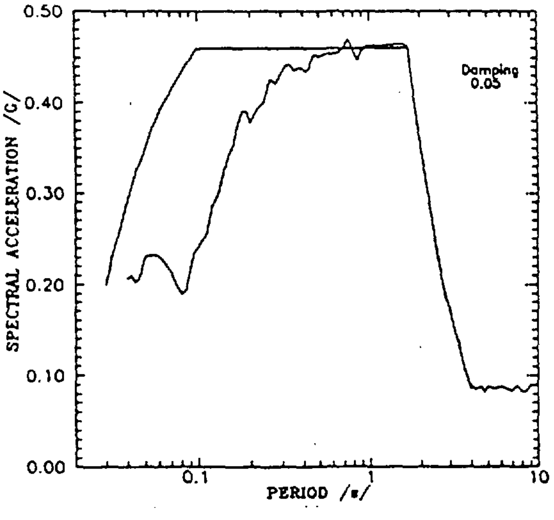

Figure 3. Acceleration response spectrum at foundation level and design spectrum, N-S component

ORIGINAL STRUCTURE'S MODEL

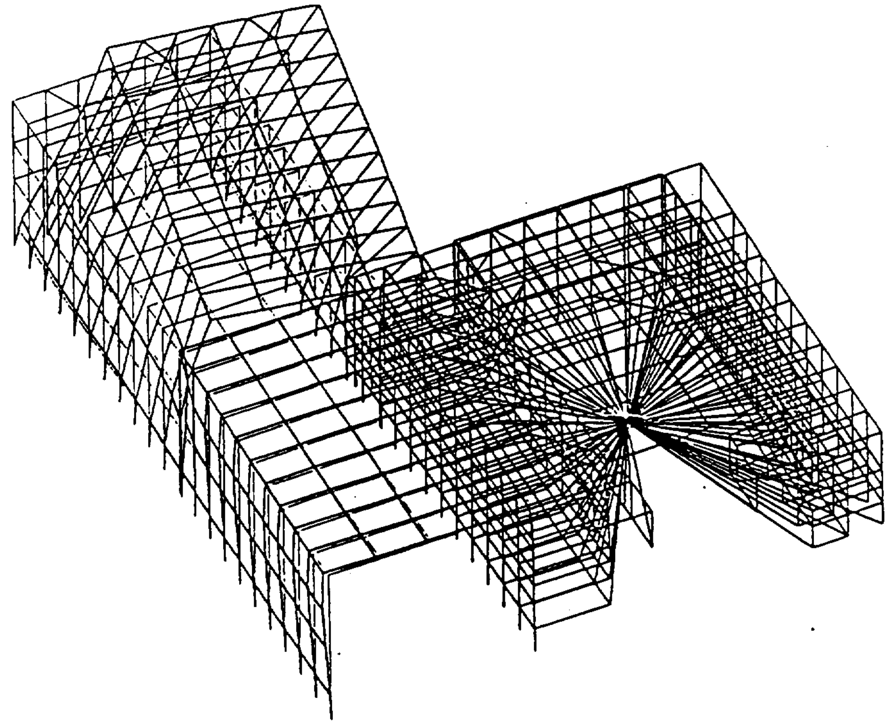

Figure 4. First mode of vibration, T1=1.3915 s

ORIGINAL STRUCTURE'S MODEL

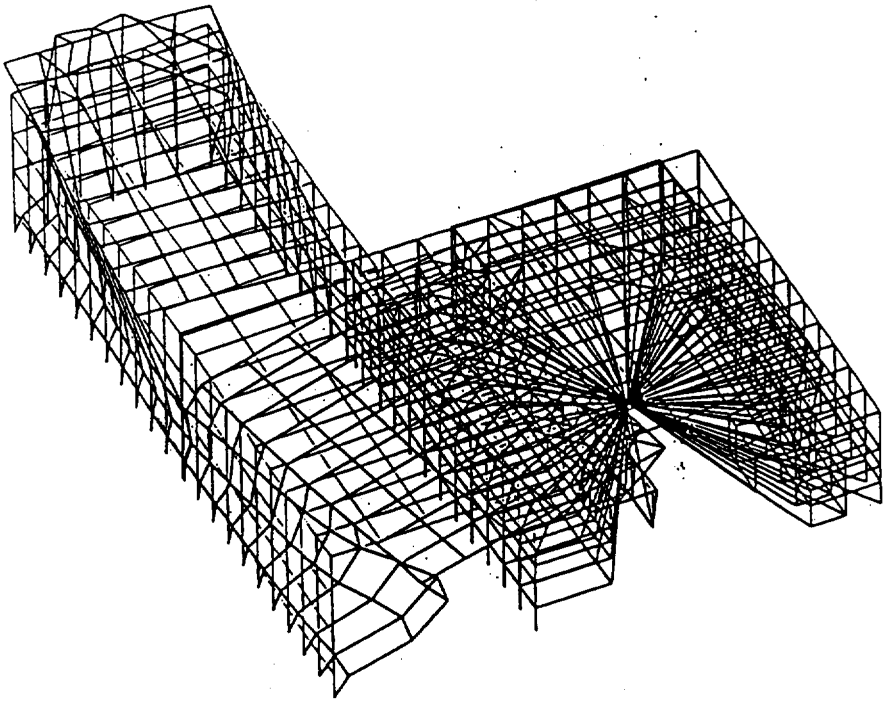

Figure 5. Fourth mode of vibration, T4=0.5377 s

UPGRADED STRUCTURE'S MODEL

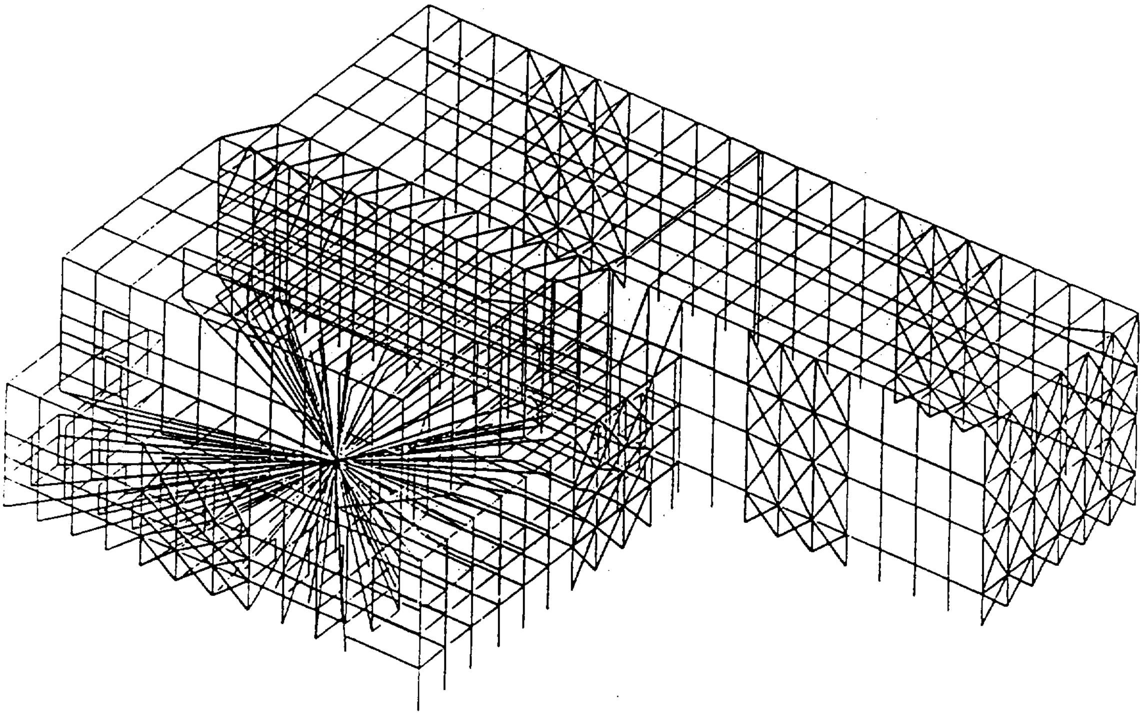

Figure 6. General view

UPGRADED STRUCTURE'S MODEL

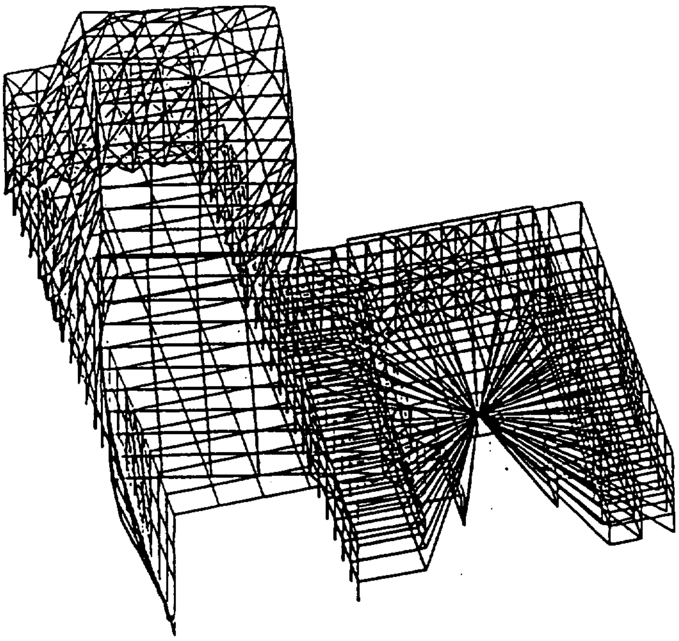

Figure 7. First mode of vibration, T1=0.5815 s