1. Introduction.

The Kozloduy NPP is located in the north-west part of Bulgaria on the Danube river. The plant consists of four Units of type VVER -440/230 and two Units of type VVER 1000. The first two units are operational since 1974/5. Subsequently the Units 3/4 (1981/2), Unit 5 (1987) and 6 (1991) are completed.

During the operational time of Kozloduy NPP four strong, intermediate depth earthquakes affected the site in 1977, 1986 and twice in 1990. The epicenters of those earthquakes are located in Vrancea region - 300-350 km north-east from Kozloduy. Vrancea seismic zone is characterized by intermediate depth earthquakes - 70 to 130 km. In 1985 and 1986 moderate shallow earthquake sequence /major event M=5.8/ occurred with epicenters 100 km south-east from Kozloduy site. The strongest local earthquake recorded /1987/ was of magnitude 3.6 and epicentral distance of 21 km from the NPP site. All those events are clearly pointing out that the earthquake hazard is an important issue which has to be considered in the plant safety analyses.

Reevaluation and upgrading of Kozloduy NPP has started after 1977 Vrancea earthquake. New SSE level was defined, upgrading of the most structural elements and equipment was performed, seismic instrumentation was installed /1/.

New investigations have been initiated after the 1990 IAEA mission. A comprehensive site conformation project has been started with a subsequent structural and equipment reevaluation and upgrading.

In this paper a short overview of the seismic problems recognized as results of the performed and the on-going analyses on the structures of Unit 1-4, Kozloduy NPP is presented. The seismic problems of systems and equipment have not been subject of this presentation.
2. Seismic Design Basis

The first two units were designed according to the standard building practice for seismic intensity of IV-V (MSK). After the Vrancea earthquake (1977) the SSE was increased to MSK intensity VII as a result of a seismic hazard assessment procedure. The response spectrum of the accelerogram, recorded in Bucharest was modified for the site of Kozloduy and partially used as a review spectrum.

In 1992 the Site conformation project (2) concluded a Review Level Earthquake with $a_{max}=0.2$ and a broad design spectrum shown in fig.1.

![ACCELERATION RESPONSE SPECTRA](image)

Fig.1. Design acceleration response spectrum.

3. Plant Layout.

The layout of the Kozloduy plant is shown in fig.2. The Units 1 and 2, respectively Units 3 and 4 are coupled. Unit 2 is a mirror symmetrically placed to Unit 1, respectively Unit 4 is mirror symmetrically placed to Unit 3. Unit 1 and 2 together are symmetrically placed to Unit 3 and 4. The reactor building of Units 1 and 2 is common for both of them. The same is valid for Units 3 and 4. The turbine hall is shared by all four units and it is placed next to the reactor buildings.

Between reactor building and the turbine hall is located an intermediate building. That building is continuous from Unit 1 to Unit 4.

Units 1 and 2 are served by a common diesel building and a common pump house. There are separate diesel building and pump house for Units 3 and 4.
The service water is supplied by a channel from the Danube river. The channel is connecting the pump stations of all 6 units. A pump house on the river site is pumping the Danube water into the channel.

All of the six units are using a common spent fuel storage building, located near to Units 3/4.

Fig. 2 Kozloduy NPP Layout

Fig. 3. The primary circuit, Kozloduy NPP, VVER 440/213

The primary circuit consists of six loops which containing a horizontal steamgenerator, main circulation pump and a main isolation valve fig. 3. The pressuriser is connected to the primary circuit via two surge lines and a spray line. The accident mitigation system consists of a high pressure injection system and a spray system. There is also a low pressure injection system for units 3 and 4.
4. Main Building.

4.1. Description.

The Main building consists of the reactor building, the turbine hall, the intermediate building, the ventilation center and the electric shelves. Layout of the main building is shown in fig. 4.

The reactor building foundation mat is approximately 10.0m under the free surface on artificially improved soil /loess-cement/. Under the loess-cement there are alternations of sand, gravel and clay up to a depth of 200m, the mean shear wave velocity is about 500 m/s. Up to the elevation 10.m the reactor building is constructed by massive concrete walls. In that part of the structure is located the reactor and the primary loop elements. The design basis accident /DBA/is a 100.mm inside diameter line break with unidirectional flow through a 32.0 mm orifice insert. The accident localization compartment functions as a confinement for one bar overpressure resulting from DBA. The reactor building between the mat foundation and the operational floor at elevation 10. m. is very rigid and could be assessed as a structure with high seismic capacity.

From 10.m up to elevation 34.m the reactor building (RB) consists of RC frames with a span 39.m. In longitudinal direction the frames are in every 6.m. There are longitudinal girders and insulation RC walls which are stiffening the structure. In the corners of the reactor area between levels 10.m and 18.m there are additional shear walls forming triangular rigid cores /fig. 5/. The roof consists of steel trusses braced in two directions covered by RC prefabricated plates.

The turbine hall (TB) is placed next to the reactor building. The connection between TB and RB is realized by the intermediate building. The turbine building structure is formed in transversal directions of 39.m one span RC frames founded on separate footings at elevation -5.m. The transversal frames are placed every 6.m in longitudinal direction. There are longitudinal girders at four different elevations of the outer
column row and at 6 levels of the inner columns. The roof is similar to that of the reactor building.

The intermediate building placed between reactor and turbine hall is supported by the columns of those buildings (row B and C). Between them precast girders are placed. The floors are formed primarily by precast panels also.

The ventilation center is attached to the reactor building. The bearing structure consists mainly of poured-in-place RC shear walls.

![Fig.5. Main building cross section](image)

The control room building is also attached to reactor building. It uses partially the bearing columns of the reactor structure. The remaining structure is made of precast elements.

There are no expansion joints between reactor building and the adjacent structures. The turbine hall is divided by joints every 12 axes /72 m/. The expansion joints are very narrow - 5 cm in average.

4.2. Seismic Behaviour

![Fig.6. 3D Model of Unit 1](image)

The seismic behaviour of the main building is analysed by means of a 3D model /3,4/ shown in fig.6. The responses of the turbine building and the reactor building are essentially different. There is also big difference in the behaviour of the turbine building between axes 1 to 12 compared to that of TB between axes 12 to 24.
The response of the TB-tail /axes 1-12/ is primarily in transversal direction. The response of the TB-main /axes 12-24/ is primarily in longitudinal direction - a rotation of the outer column row about the center of rigidity located in the reactor building. Because of the different stiffness of the reactor structure and the turbine hall the intermediate building is loaded quite unfavourable.

The weak points which should be improved are:

- girders in the turbine hall at elevation 18 and 28m;
- columns in the turbine hall - column row A /outer columns/;
- seismic joints /expansion joints/ are very narrow /impact of adjacent structure is possible/;
- displacements of the turbine building /axes 1-12/ are large, secondary effects and lose of stability are probable;
- prefabricated element connections in the intermediate building and in the electric shelves /control room/ have to be checked;
- heavy roof - steel trusses of the roof should be anchored additionally to the RC columns.

5. Diesel Generator Building

The DG building is a complex space non-regular structure in layout and height. The main bearing structure is a frame of two parts with spans of 18.0m and 6.0m between columns in rows A, B and C /fig.7 and fig.8/.

The roof is made of precast prestressed rib panels of 18.0m length and reinforced concrete panels of 6.0m length. Two horizontal elevations of steel and concrete beams and monolithic slabs are designed to serve the six diesel generators. Foundations are separate cast-in-place over loess-cement cushion. In longitudinal direction precast girders are the only connection between the columns. The cast-in-place columns in row B2 are designed without justification of the joints between them.
and the precast columns in row B1. All brick walls are made of clay hollowed bricks without reinforcement. The general conclusion is they do not have sufficient seismic resistance.

After 1977 Vrancea earthquake cracks were observed in the longitudinal beams and in the masonry walls. Additional shear walls were recommended but not implemented in 1978.

Within the WANO program 1992 an analysis and detailed design for upgrading of DG1 building was performed, fig.9 /5/. The main findings of that analysis were the following:

- the stresses in the main bearing elements do not satisfy the requirements for safety; main bearing elements have to be upgraded;
- the heavy roof should be removed;
- additional bracing is needed both in longitudinal and transversal directions;
- columns in row B1 and B2 should be connected and a common jacketing is required;
- masonry walls have to be braced both sided in order to prevent interaction with safety related equipment.

The DG2 building is similar to DG2 except the better implementation and the better connections between the prefabricated elements as well as the existing steel bracing in transversal direction.
6. Pump House

The pump house 1 was also investigated within the first 2 phases of WANO program /6/. The structure consists of two parts: underground massive part where the intakes of the pumps and working whiles are located and an upper structure, which is formed by one span prefabricated RC frames at 6.0m intervals. The underground structure is very rigid and assessed to have high seismic capacity.

Fig.10. The pump house 1, upgraded structure – cross-section.

Fig.11. The pump house 1, upgraded structure.

The upper structure is upgraded /fig.10 and fig.11/ because of the following:
- heavy roof, formed by prefabricated beams and panels;
- pore implementation of the brick walls and forming of short columns;
- not sufficient bracing in longitudinal direction;
- poor implementation of the column anchorage in the massive foundation;
- poor connection between prefabricated elements.

Comprehensive liquefaction analyses /7/ were performed also for the region of the in-take basin of the pump house. The pump
house itself is founded on gravel. All loose sands have been removed during the construction of the foundation. There are loose sands in some area beneath the channel and the channel embankments. In fig.12 is shown result from liquefaction analyses - isolines of the pore pressure ratio are drown / \( r = \frac{P_u}{S_{eff}} \) where \( P_u \) is the pore pressure and \( S_{eff} \) is the effective stress. Sands will liquefy when \( r = 1 \). Different seismic conditions /equivalent number of cycles have been tried - the conclusion has been that sands below the bottom of the channel and in some regions of the embankment could liquefy. That liquefaction has been assessed to be not of primary importance for the plant overall safety because it does not affect the safety related systems and does not cause significant loss of cooling water. However the liquefaction analyses will continue.

\[ \text{Variation of the } r = \frac{P_u}{S_{eff}} \text{ earthquake } N_{eq} = 17.2 \text{ } T = 30 \text{ s} \]

Fig.12. Isolines of the pore pressure ratio, Intake PH1.

7. Conclusions.

The Units 1/2 of Kozloduy NPP have been designed originally to resist a IV - V degree MSK earthquake. They have been subsequently upgraded for a VII degree MSK earthquake. Now that structure basically do not meet the safety requirements to resist the new review earthquake with a maximum acceleration of 0.2g and very broad spectrum. The performed analyses are clearly pointing out that an upgrading for the new earthquake level is possible. The problems common for all of the structures of Kozloduy NPP could be summarized as follows:

- Usually the prefabricated elements are of good quality but the connections are badly implemented and in some cases not adequately designed. Those problems have been well recognized in the Bulgarian construction practice. There is also experience for damages due to similar shortcomings during past earthquakes in Bulgaria. Most of the connections should be checked and upgraded.
- There are roofs made of prefabricated RC element which are very heavy. Such roofs cause big inertia forces which usually could not be transferred to the foundations.
- There are buildings where the longitudinal bearing elements are insufficient.
- Connection of structures with different stiffness and different height is usually made without justification of the respective joints.

- The seismic joints constructed are inefficient because of the small gap between the elements and because of not proper implementation.

- Due to the expected intensive long period excitation most of the buildings will experience very large displacements. In most of the structures not the seismic forces but the large displacements will be critical.

- The masonry walls generally do not have sufficient capacity. There are very rear bearing masonry walls. The king of hollowed brick used are not adequate for seismic design. Most of the masonry requires upgrading.

8. Acknowledgment.

This paper has been prepared with the valuable support of the Nuclear Division of Energoproec, Sofia. The authors are expressing their gratitude to Mrs. Vessela Michailova, Mr. Artin Mamian and Mr. Sergey Danailov for the high professional technical assistance.

9. References.

1. Papazian M., Petrov D., Measures for Upgrading the Seismic Safety of Kozloduy NPP, in "10 Years Operation of Kozloduy NPP", Technika, 1984. (in Bulgarian)