Seismic Upgrading of the Turbine Hall and Electric Building of Unit 5/6 Kozloduy NPP

Varbanov Georgi [1)], Kostov Marin [1)], Kaneva Antoaneta [1)] and Krutzik Norbert [2)]

1) Central Laboratory for Seismic Mechanics and Earthquake Engineering, Bulgarian Academy of Sciences Sofia
2) Framatom ANP, Offenbach a.Main

ABSTRACT

The paper is presenting part of the upgrading activities to increase the seismic safety of critical structures in NPP Kozloduy. The investigation is performed within the Modernization Program for Units 5/6. It concerns the seismic reevaluation of the civil structures of the turbine Hall and the electric Building.

The turbine hall and the electric building are placed next to each other. The gap between them is very small, about 5 cm. They are frame structures. The TB is made of steel and the electric building of reinforced concrete respectively. The seismic capacity of the as-built structures is estimated as insufficient. The critically overloaded parts of the structure and the necessity of the seismic upgrading are defined.

Two general concepts for structural upgrading of the complex "Turbine Hall-Electric Building" are presented. According to the first concept both buildings are upgraded as separate structures. The second concept requires connecting of the both structures and upgrading as complex structure. The advantages and disadvantages of those concepts are discussed.

The upgrading is based on realistic capacity assessment of as built structures and effective use of all available structural elements. Nonstructural elements are upgraded and used as bearing elements, e.g. wall panels are interconnected and used as bearing shear wall. Another technique used is the additional bracing of steel trusses in order to shorten their buckling length.

The upgrading efficiency is demonstrated by comparing the HCLPF determined for as-built and the upgraded structure respectively.

INTRODUCTION

The investigation is performed within the frame of the Modernization Program for Units 5/6.

The Units 5 and 6 of Kozloduy NPP in Bulgaria are of type WWER 1000. Both units are designed and erected in 80's and the beginning of 90's. Generally the safety related structures and equipment are designed for horizontal seismic PGA of 0.1 g. The seismic hazard re-evaluation program that had been started in the end of the 80's defined new review level earthquake (RLE) characterized by broad band free field site specific acceleration response spectra and maximum horizontal PGA of 0.2 g . The vertical acceleration is equal to one half of the horizontal. In addition to the RLE the seismic hazard re-evaluation program postulates a design seismic excitation due to the local seismic sources (LE). These sources are with relatively low focal depth and shaking energy but they can produce vertical seismic accelerations bigger than RLE acceleration in the high frequency range. The maximum horizontal acceleration of LE is 0.16 g and vertical – 0.13 g. The acceleration response spectra for horizontal and vertical components of RLE and LE are given in fig.1 and fig.2. The study described here for is based on the design response spectra that envelop the RLE and LE seismic characteristics

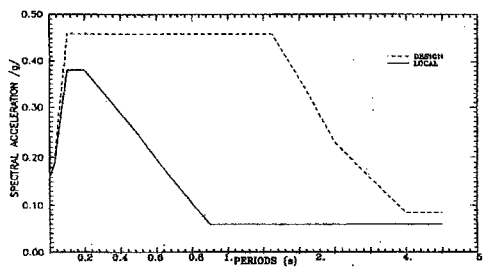

Fig. 1 Horizontal component of RLE and LE

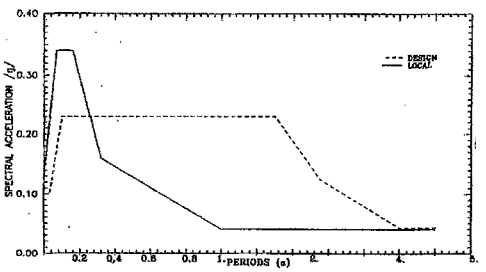

Fig.2 Vertical component of RLE and LE

The turbine hall and electric building are not classified as safety related structures and does not affect directly the plant safety. They are of category 2 and all dynamic analyses are performed using seismic parameters equal to one half of enveloped seismic characteristics.

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

STRUCTURE DESCRIPTION

The Turbine Hall of Unit 5/6 of "Kozloduy" NPP is a steel frame structure comprised of a turbine section and deaerator section. In the turbine section is situated a 1000MW turbine supported by independent reinforced concrete foundation, separated from the bearing structure of the turbine hall. Next to the turbine hall is the electric building. Its is constructed of pre-cast reinforced concrete elements. A vertical cross section of both structures is given in fig. 3.

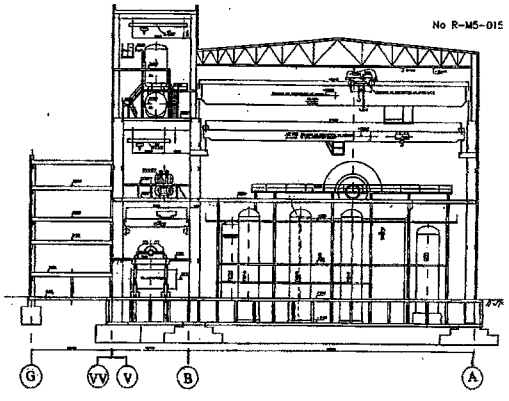

Fig.3 Cross section of the TH and EB

The turbine hall in transverse direction is steel two-bay frame structure. The main columns are along rows "A", "B" and "V". The turbine section is between rows "A" and "B". It spans 45m and is about 38m high at the ridge of the roof. The deaerator section is between rows "B" and "V". Its transverse bay is 12m wide and its height is about 42m. In longitudinal direction there are 12 column axes along each of the three rows. The longitudinal bay spacing is 12m except between axes 1 and 2 where it is 4.5m.

The electric building is next to the turbine hall. It is a reinforced concrete structure between rows "VV" and "G". The steel columns from row "V" and the concrete columns of row "VV" have common foundations. The transversal bay of the electric building is 12 m. The structure height is about 21m. The longitudinal spacing of the columns follows the spacing of the turbine hall. Prefabricated columns and longitudinal girders and cast in situ transversal girders shape longitudinal and transversal moment resisting frames.

The foundations of the columns in column row A and B are separate. The foundations in column row "V" are common for the steel and reinforced concrete columns of both buildings.

The roof structure of the turbine section is of steel trusses between column rows "A" and "B", covered with light panels. The roof structures between rows "B-V" and "VV-G" are of pre-cast pre-stressed concrete elements.

The main load bearing elements of the turbine hall are made of steel. In this part of the structure reinforced concrete elements are only the wall panels and the floor and roof pre-stressed panels in the deaerator section. The structure of the electric building is made only of reinforced concrete except the part between axes 1 and 2 where the structure is mixed.

DYNAMIC ANALYSIS AND CAPACITY CHECK OF AS BUILD STRUCTURE

As referent in the analysis are used the turbine hall and the electric building of Unit 5. A detailed 3-D finite element model is developed using STARMAP Computer Code. The model consists of 1944 nodes, 3548 beam elements, 473 general plate elements. The total number of degrees of freedom is 10769. Beam elements with 6 DOF per node and plate elements with 5 or 2 DOF per node are used.

The RC slabs are modeled by plate elements. The frame structures are modeled by beam elements. The façade wall panels as well as the heavy equipment are represented only as lumped masses. The deaerator vessels are modeled by beam elements with respective weight. Attention is paid to the accurate representation of the actual releases of all bearing elements (especially vertical diagonal connections and prefabricated RC elements). The design documentation is checked by walk-downs to reflect the as built status of the structure.

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

Both static and dynamic (seismic) analyses are performed by the Stardyne 4.0 computer code. The analysis of the natural modes of vibrations of the as built structures shows the following main features of their dynamic response:

- The first mode of vibrations (T=1.79s) is primarily response of the turbine hall in longitudinal direction, with some torsion of the whole structure, mainly due to the heavy deaerators vessels. This mode appears due to the fact that the supporting structures of the deaerators are not designed for lateral loads. The frames of the truss supporting structure of the deaerators are pin supported to the girders.
- The second mode of vibration (T=1.56s) is complex response of the turbine hall and electric building in transversal direction. The interaction between the turbine hall and the electrical building is due to the common foundation in row V and the connections between the steel and reinforced concrete columns in rows V and VV
- The fifth mode (T=1.09s) is representing response primarily of the electric building in longitudinal direction. It corresponds to the first mode of the electric building.

The seismic effects are computed by time history analysis. The three components of the strong ground motion are

applied simultaneously. The modal superposition technique is used. In the analysis 600 modes are computed and the activated modal mass is about 90 percents. The material damping is 0.07. Radiation damping is introduced in each footing. The modal damping is a composite one. For capacity analysis only the maximal computed effects are used.

The seismic member forces are combined with the static one. The most unfavorable combinations for each type of construction (steel structures, anchor bolts, foundations, truss elements and RC columns or girders) are aimed. The capacity assessment is performed according to the Bulgarian standards for steel and reinforced concrete structures as well as according to ANSI/AISC, N690-1994, ACI 349-85 and ACI 349R-85.

The Bulgarian and USA codes generally are based on different assumptions (as an example Bulgarian code for steel structures is developed on the basis of "limit state" theory and ANSI/AISC, N690-1994 uses allowable stresses). However the capacity check results performed according those codes are very similar. Some differences appear in the assessment of the bearing capacity of steel elements with very high slenderness due to the different limit values of slenderness ratios in the Codes considered. Differences can be expected in the estimation of the bearing capacity of the steel elements assembled from steel shapes with relatively thin flanges, due to the different influence of the depth-to-thickness ratio in the Bulgarian and N690 codes.

General conclusions of the capacity checks are as follows:

Turbine Hall:

- Bearing capacity of all columns is sufficient.
- Bearing capacity of almost all vertical connections is insufficient, and they have to be modified.
- Bearing capacity of the transversal girder under the deaerator vessels is insufficient and all deaerator bearing structures need upgrading.
- The joint "roof truss-column" has to be modified.
- Some horizontal connections have to be upgraded.

Electric Building:

- The stiffness of the structure is insufficient in both (longitudinal and transversal) directions. Seismic capacity of all columns is insufficient.
- The transversal and longitudinal girders have necessary bearing capacity
- The computed transversal displacements of some control points of the turbine hall and electric building located along row V are bigger than the available gap, i.e. dynamic impact between adjacent structures is possible.

BASIC PRINCIPLES AND DESCRIPTION OF SEISMIC UPGRADING

As it was mentioned above turbine hall and electric building are of category 2 and are not classified as safety related structures and do not affect directly the nuclear safety. However increasing of the seismic reliability of these structures will improve the global safety of the plant. The upgrading of the electric building will reduce the possibilities of accidental plant outages due to loss of off-site power. The upgrading of the turbine hall will prevent damage of the equipment located there and will reduce the costs of possible repair and losses due to long outages.

The upgrading performed has to fulfill the following basic requirements:

- To be compatible with the restrictions of the available technology, i.e minimization of the collisions with the existing equipment and piping.
- Rational usage of the as-built elements of the structure;
- Minimum expenditure with maximum efficiency of the upgrading.

The assessment of the dynamic response of the structure of the turbine hall shows that the main bearing columns have enough seismic capacity. Only local upgrading of the vertical and horizontal connections between the columns and transversal girders between rows "B" and "V" is necessary. The upgrading of the existing vertical and horizontal connections can be done mainly by local strengthening, i.e. by decreasing the slenderness both in- and out of the plane. These upgrading will not change the character of the seismic response but will improve the seismic safety of the whole structure.

The analysis of the capacity check of main bearing element of the electric building shows that longitudinal and transversal dynamic stiffness of the structure is insufficient. The higher seismic bearing capacity have to be achieved by global upgrading of the structure - general change of the static scheme in order to improve the forces "path" and their distribution that can be reached by adding new bearing elements and transformation of non structural elements into bearing elements.

General scheme of the upgrading of the complex "Turbine hall – Electric building" is shown in fig.4.

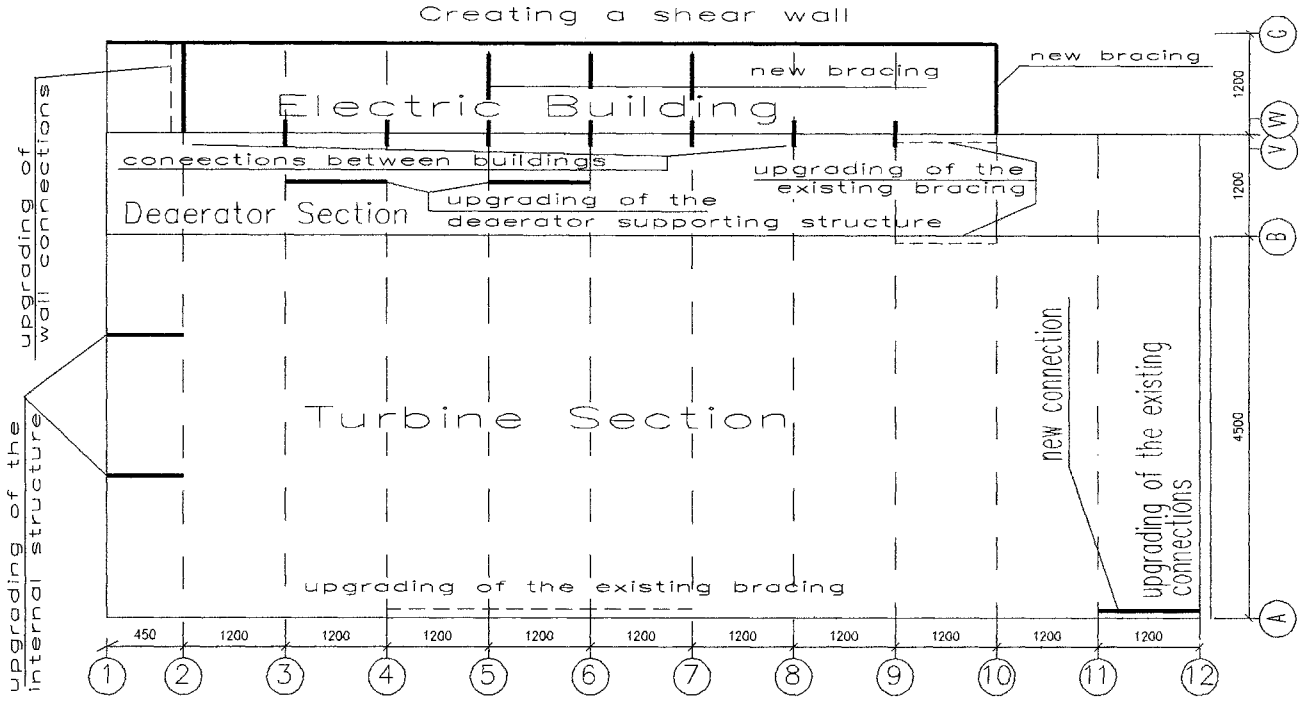

Fig.4. General scheme of the upgrading of the complex "Turbine hall – Electric building

Upgrading of the Electric Building

The capacity check results show the necessity of increasing the transversal stiffness of the electric building. The common approach is to upgrade the structure by implementing proper internal transversal bracing (or shear walls). Another alternative is to create an external structure to support the RC columns. In the particular case it is difficult and not efficient to apply those variants because of restrictions due to existing cabling and equipment. The most limiting condition however is the insufficient gap between the turbine hall and electric building and the possible dynamic impact. Because of those reasons the variant to upgrade the building as separate structure is assessed as unfavourable and is not discussed here further

The other possible realistic approach to increase the transversal stiffness of the electric building is to connect it to the turbine hall and to analyze and upgrade the whole complex "turbine hall-electric building". The detail of the truss connection between steel and RC column is shown in fig.5. Connections are designed at three levels along column's height. Some additional transversal stiffness is introduced in the structure by additional X-bracing between rows VV and G. Due to the technological requirements these bracing are partial (only in three axes and not at all elevations) and can not transfer the seismic forces direct to the foundation. Never the less they improve the global seismic response of the structure.

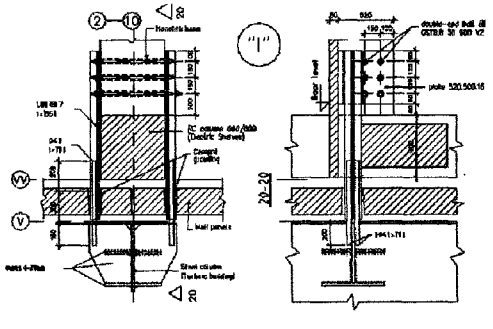

fig.5. Connection between RC and steel column

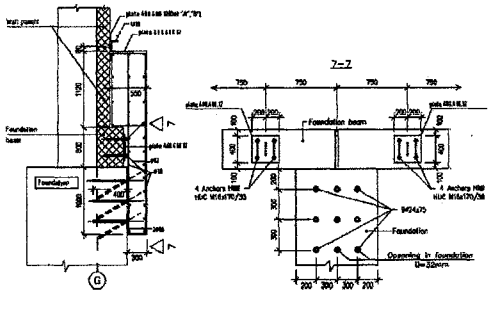

fig.6. Upgrading of the wall foundation

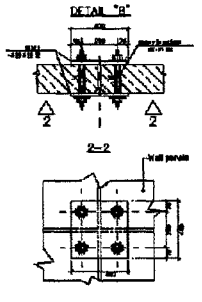

fig.7. Connection between panels

To increase the longitudinal stiffness of the electric building is proposed to interconnect the reinforced concrete façade

panels at row G in a shear wall. For that purpose it is necessary to improve the foundation of the wall panels (fig.6) and create reliable connections between panels as well as to improve the connections of the panels to the columns. Details are shown in fig.7 and fig.8. The reinforcement of the façade panels is insufficient to transfer the seismic shear forces. That is

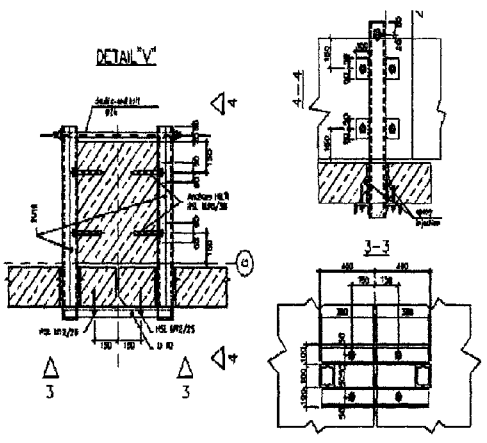

Fig.8. Connection RC column-wall panel

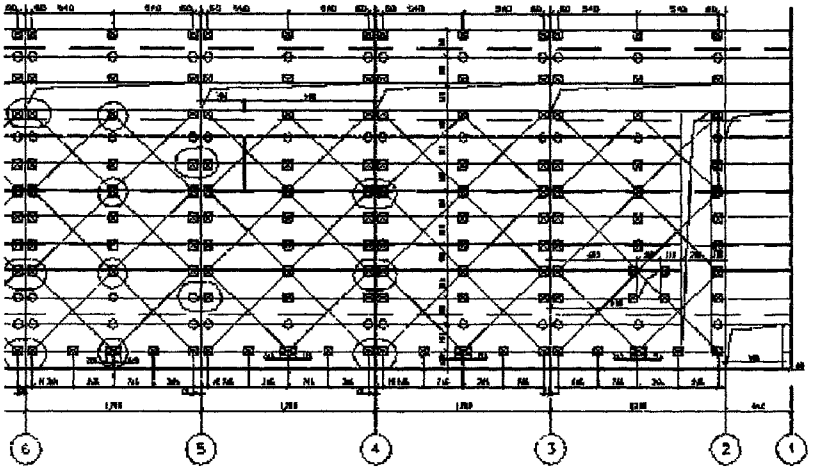

fig.9. Steel bracing of the façade shear wall

way additional steel bracing along the whole façade is provided. The bracing is very slender and is designed to bear only tension forces. A general view of the façade upgrading is given in fig.9.

The wall between electric building and turbine hall is not accessible due to cable traces and equipment installed and upgrading can not be implemented. The center of the rigidity of the system moves to the upgraded wall and the structure will rotate around it. Vertical steel bracing in axis 10 (fig.10) and upgrading of the wall in axis 2 is proposed to resist this tendency.

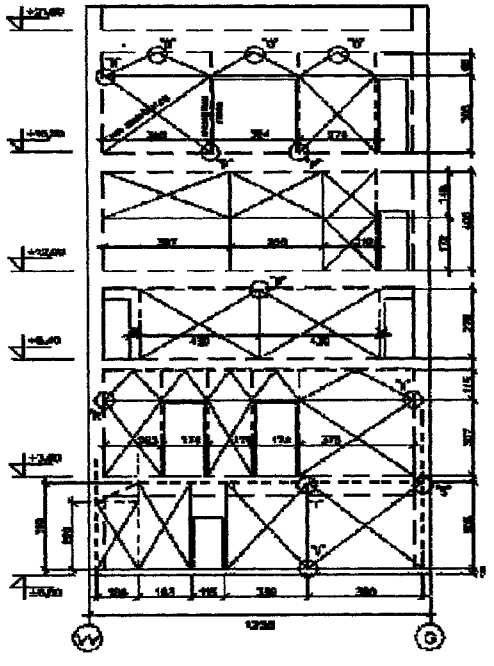

fig.10. Bracing of transversal façade

The advantages of that upgrading are the following:

- The electric building will be connected to the turbine hall and so the possible impact between them will be avoided.
- The main part of the upgrading work will be done from outside (row G) and the technological process will not be disturbed.
- The stiffness in longitudinal direction will be increased significantly by creating the wall in row G.
- The transversal stiffness is increased by connecting steel columns of turbine hall and RC columns of the electric building. The designed X-bracing in transversal direction and improvements in axis 2 increase also the transversal stiffness.

The disadvantage of that upgrading is that there will be dynamic interaction between electric building and turbine hall. In order to control that the connections between both buildings should be designed as ductile members that would soften the dynamic interaction.

Upgrading of the Turbine Hall

The structure of the Turbine hall is regular in plan and height. The proposed upgrading has to keep the regular distribution of the stiffness and mass. In this way the unfavourable overloading of the bearing elements and stress concentration will be avoided.

As it was shown on the general upgrading scheme (fig. 4.) the strengthening of the turbine hall consists of:

Local upgrading of the existing vertical and horizontal connections

The strengthening effect is achieved mainly by decreasing their slenderness in and out of the plane of the connection as well as by adding of the additional material. The prolonging the weld seams and stiffening of the some gusset plates will be necessary too. A typical upgrading scheme for connections at one longitudinal row can be seen in fig. 11.

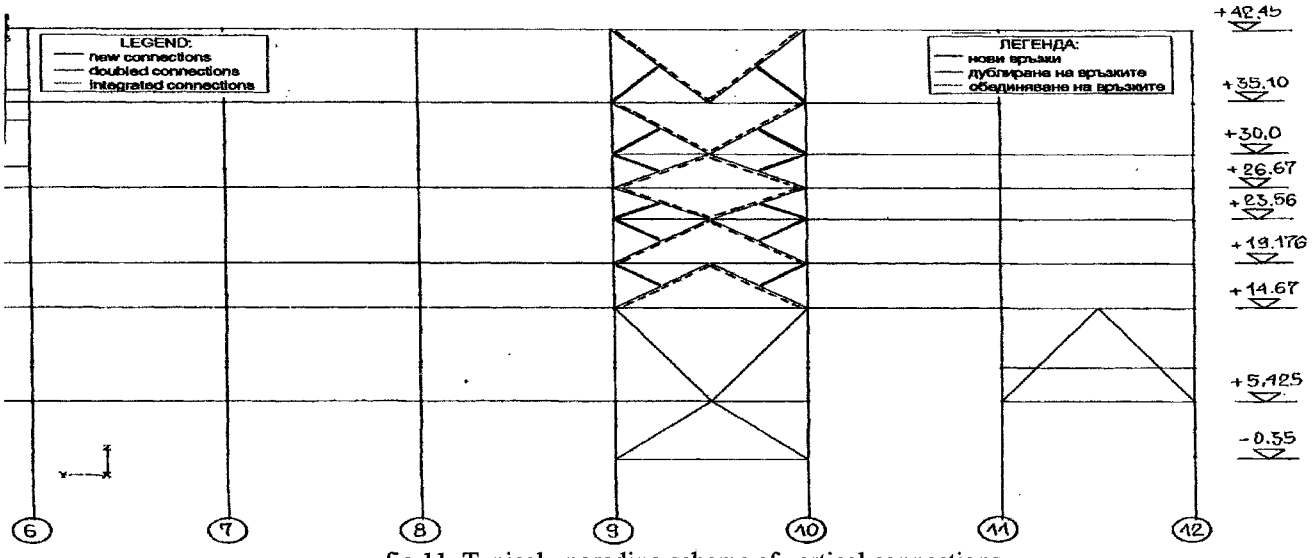

fig.11. Typical upgrading scheme of vertical connections

The main typical cases can be classified as follows:

- **Type1**. The existing vertical connection comprised of two single profiles is integrated into one common bearing element and new diagonals are added to reduce the slenderness ratio in the plane of the connection – fig.12
- **Type2**. The existing single profile connection is doubled with second profile. The two branches are integrated into one common bearing element and new diagonals are added.
- **Type3**. Between the branches of the existing doubled vertical connection a new third steel profile is added to increase the area of the cross section of the tie. The three branches are integrated into one common bearing element and new diagonals are added– fig.13

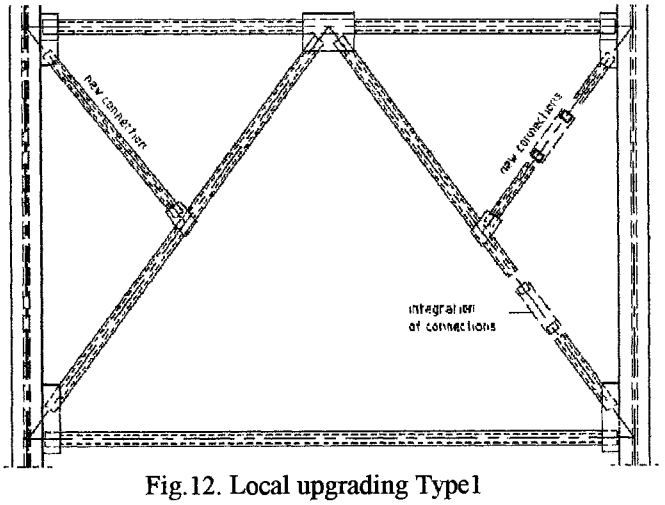

Fig.12. Local upgrading Type1

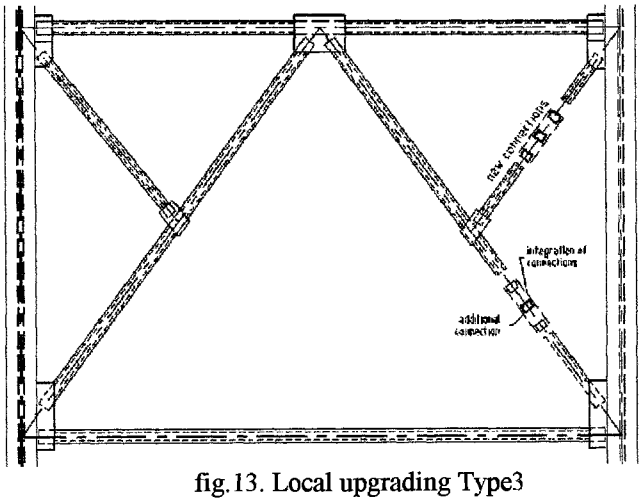

fig.13. Local upgrading Type3

These types of upgrading could not change the character of the seismic response of the structure and respectively the member forces in bracing elements but contribute very much for the improving the seismic safety of the structure.

New diagonal bracing between columns in end axis

This bracing improves the seismic behavior and bearing capacity of the columns in the two end axes near the heavy loaded transversal façade and the big transport openings. The original construction of the opening requires removing of horizontal connections that leads to overloading of the columns in longitudinal direction.

Upgrading of the bracing at transversal facade

After joining the two structures – turbine hall and electric building last two axes of the turbine hall tends to rotate around the point where the stiffness of the joint structure changes. The additional bracing of the transversal façade has to resist this movement.

Upgrading of the support structures of the deaerator vessels

The supporting structure of the deaerators is composed of two rigid frames that are pin-supported to the transversal girders between rows "B" and "V". It has low bearing capacity in longitudinal direction. The seismic forces cause torque and bending moments along the weak axis of the transversal girders. That is an extremely unfavorable loading. The upgrading aims to improve the force path, i.e to transfer directly the inertia forces from the heavy deaerator tanks to the columns of the main bearing structure. As a result the transversal support girders of the deaerators are more favorably loaded. Disadvantage of the upgrading is that the columns in row B and V are loaded at higher levels where their cross section is relatively small.

Upgrading of the support of the roof truss structure

The existing roof trusses supports are not able to withstand the horizontal forces from the roof (poor detailing). The support scheme is changed in order to allow proper force transfer from the truss structure to the columns.

Upgrading of the internal structure in the turbine hall

The internal steel structure is situated between axes 1 and 2 (near the reactor building) and supports the main steam lines and mean feed water lines. To improve the seismic safety additional X bracing is designed. New connections are placed in longitudinal direction of the turbine hall (transversal direction of the internal structure). Additionally the welding seams of the floor steel cover should be improved in order to create floor diaphragms connecting axes 1 and 2.

After the implementation of the upgrades a new dynamic analysis and capacity check is performed. There is no deficiency in capacity or in structure deflections.

COMPARISON OF FAILURE PROBABILITIES

Based on the analyses performed both for the as built and for upgraded structure the failure probabilities are computed for several important scenarios. For each scenario a representative fragility curve is developed. The fragility curves are developed according to the methodology described in [1]. The material strength variation, the element ductility, the ultimate capacity, the load combination and the conservatism involved in the analysis are considered. The parameters of the fragility curves are represented in table 1.

Table 1. Comparison between the parameters of the fragility curves for different scenarios before and after upgrading

	as built				upgraded			
Scenario	A_F	HCLPF	u^r	u^m	A_F	HCLPF	u^r	u^m
Roof failure of the turbine hall	0.179	0.102	0.132	0.193	0.356	0.210	0.132	0.193
Failure of the deaerator supporting structure	0.170	0.101	0.132	0.178	0.472	0.284	0.132	0.178
Failure of the vertical connections – row A	0.125	0.075	0.132	0.198	0.404	0.230	0.132	0.226
Failure of the vertical connections – row B	0.147	0.087	0.132	0.198	0.390	0.216	0.132	0.226
Failure of the vertical connections – row V	0.154	0.089	0.132	0.198	0.396	0.219	0.132	0.226
Column failure of the electric building	0.123	0.073	0.132	0.183	0.340	0.204	0.132	0.178

The total probability of failure is computed by integrating the fragility curve and the seismic hazard curve for the site. The comparison between computed probabilities of failure of as built and upgraded structure is shown in table 2.

Table 2. Comparison between 85% total probability of failure for different scenarios before and after upgrading

Scenario	as built	upgraded
Roof failure of the turbine hall	1.242E-03	2.356E-05
Failure of the deaerator supporting structure	1.426E-03	4.448E-06
Failure of the vertical connections – row A	7.872E-03	1.518E-05
Failure of the vertical connections – row B	3.548E-03	1.886E-05
Failure of the vertical connections – row V	2.836E-03	1.717E-05
Column failure of the electric building	3.563E-03	1.061E-05

The analysis of the as built structure shows that all scenarios for failure of the turbine hall and electric building have

very high probability of failure, i.e. 10^{-3}. The implemented upgrading decreases significantly the total probabilities of failure – for electric building the reduction of the probability of failure is about 300 times and for the most overloaded elements of the turbine hall (vertical connections of the column row A) the reduction is about 500 times. Strengthening of the join roof truss-column decreases the risk of the roof failure about 50 times. Upgrading of the deaerator bearing structure enhance the seismic safety about 300 times.

The total risk for all investigated critical elements of the upgraded structure is in the range of 2.10^{-5} that is a reasonable value for structures of category 2.

CONCLUSIONS

The presented analysis shows non-traditional upgrading measures applied in order to account the existing status of the buildings. The measures are aiming to change the structural system in an effective way for better seismic behavior. In particular those measures are:

- Connecting of two existing separate structures in order to take advantage of their joint lateral resistance. The connecting elements have to be very ductile in order to control the possible dynamic interaction.
- Transformation of non-structural elements (prefabricated wall panels) into shear wall. The transformation is achieved by interconnecting the panels and by improving the ties between panels and columns. Additionally the wall foundation is improved as well as ductile wall bracing is introduced.

The HCLPF analysis shows the effectiveness of t he proposed upgrading measures.

REFERENCES

1. Kennedy, R. P., Cornell, C.A., Cambell, R.D., Kaplan, S. and Perla, H. F., "Probabilistic seismic safety study of an existing nuclear power plant,", Nuclear Engineering and Design, Vol. 59, 1980, pp. 315-338.