Transactions, **SMiRT-26**
Berlin/Potsdam, Germany, July 10-15, 2022
Division V

INCREASING THE LEVEL OF RESISTANCE TO SEISMICITY AND OTHER EXTREME NATURAL EVENTS NUCLEAR POWER PLANT DUKOVANY, CZECH REPUBLIC

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ABSTRACT

For the process of licensing the extension of the operation of nuclear power plant Dukovany, it was necessary to ensure higher resistance to extreme natural events and the effects of seismicity. These were the effects of wind, snow, and seismicity in extreme values with the return period of 10,000 years. The subject of the presented project of increasing resistance were structures of the nuclear island (NI), where reactors VVER 440 are installed, and turbine island (TI). The article describes the whole process of the event, which took place in 2011 up to 2020 in several phases. Feasibility study, static and dynamic analyses on the spatial model, assessment of the existing load-bearing structures, design of technical measures, project documentation, execution of the proposed measures and final evaluation report for all 4 units of NPP Dukovany.

INTRODUCTION

The nuclear power plant is a set of different civil structures (CS). The subject of the whole licensing process were all the structures for which the increased resistance to seismicity and meteorological events is required. These are the safety-related structures and systems that ensure safe shutdown of the power plant. The buildings and their load-bearing structures have to ensure fulfilment of safety functions with the required reliability for the technical period of service, provided that the defined operating conditions and maintenance requirements are fulfilled. These safety functions with the required reliability have to be ensured not only in terms of normal operating conditions, but also in emergency conditions such as seismicity, and meteorological events. The important buildings are reactor building, a longitudinal and cross-side electrical buildings and turbine hall.

BRIEF DESCRIPTION OF INDIVIDUAL CIVIL STRUCTURES (CS)

Nuclear island consists of CS 800 Reactor Building, which is followed by CS 805 Longitudinal electrical building, connected longitudinally to the Reactor building and CS 806 Cross-side electrical building, which is connected from the sides to the reactor building. Turbine island represented by CS 490 Turbine Hall is connected to longitudinal electrical building. Two reactor buildings form a main power double unit, connected together with electrical buildings and turbine hall. Both parts of the main unit structure are divided by an expansion joint between the individual units. These three buildings form one structural unit, which is spatially heterogeneous. This can be seen in the figure 1.

The main load-bearing structure of the reactor building is robust reinforced concrete consisting of walls, slabs, and columns. The thicknesses of the main walls and slabs are 1,50 m and are strongly reinforced. The reactor pit structure is located in the middle of the building. The roof structure is made of steel frames in the transverse direction, the columns are anchored to the main reinforced concrete walls. Massive reinforced concrete structures of bubble condenser towers are connected to the reactor building. It is about 50 m high reinforced concrete tower with massive walls. The construction of the main steel frames of the reactor room is also connected to these towers. All structures enveloping the primary circuit are designed as massive reinforced concrete structures of thickness 1,5 m and their surfaces are protected by a steel hermetic lining. The hermetic steel lining together with the massive reinforced concrete structure ensures the hermetic separation of the primary circuit from the surrounding environment. Its tightness is tested by pressure tests. The spaces inside this envelope are reffered to as the hermetic zone of the reactor building.

The electrical buildings and turbine hall have load-bearing steel structures formed by frames in the cross-side direction. The individual frames are connected to the main frames of the reactor building. The main frames of the electrical building are anchored to the reinforced concrete walls of the reactor building, which ensures their stiffness in cross-side direction. In the longitudinal direction, stiffness is ensured by a system of roof and wall bracing system.

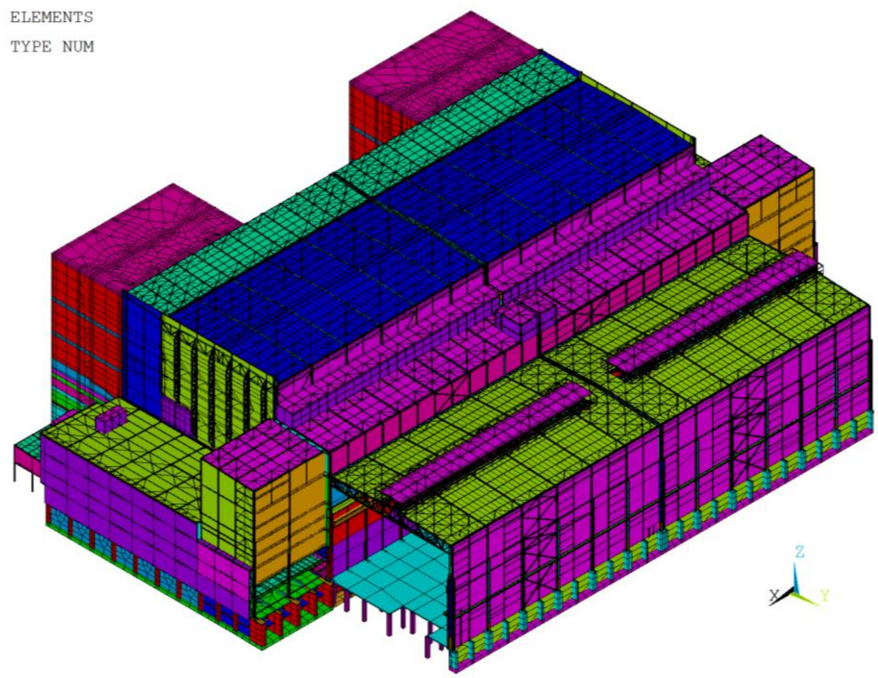

Figure 1. Scheme of the finite element model of buildings forming one structural unit

PRE-PROJECT PHASE

The process of pre-project phase was focused on obtaining all the necessary information and documents for the creation of calculation model of main power unit structures. Regarding the necessary detail of the calculation model and regarding the history of the actual execution of the power plant, it was necessary to verify number of documents and data on load-bearing structures on site. Detailed inspections of load-bearing structures were performed, especially details of joints in steel structures, including their detailed measuring. This process was very demanding, because most steel structures are located at a considerable height above the sensitive technological equipment, especially in the hall of the reactor building and turbine hall, where the height is greater than 30 m above the floor level. Several teams of designers were established under the guidance of an experienced structural engineer, who also had to be authorized to work at heights, including experience with movement using climbing technique. The teams documented all types of decisive joints, including the detailed surveying of individual welds. Detailed drawings, photo documentation and static evaluations were created for each inspected site. These were the basic documents for the subsequent structural assessment of individual elements and especially joints. Many places in this phase of the preparatory work could not be verified due to inaccessibility and these places were documented only in the process of execution when they could have been uncovered.

STATIC A DYNAMIC CALCULATIONS

An important part of the project preparation were static and dynamic calculations of load-bearing structures. The structures of main power unit buildings are strongly structurally and technologically interconnected and form a spatially highly heterogeneous single structural unit. This means that the correct solution of the seismic response can be obtained just during solving the whole connected group of the structures at the same time. A detailed assessment of all load-bearing structures was required, therefore all load-bearing elements had to be modelled without significant simplifications. This was leading to the obligation to create a complex spatial model, where all critical load-bearing structures and critical masses from the installed technological equipment were modelled. The complex model was created in SW ANSYS. The calculation model consisted of 358 233 elements located at 280 586 nodes and led to 939 308 degrees of freedom. The main reinforced concrete structures of reactor building, including the bubble condenser shaft, were modelled by final elements SOLID45 (see the figure 2), ceiling structures in electrical buildings, partition walls, roof and cladding panels with wall-boarded elements SHELL43. The load-bearing elements of steel structures are modelled in detail by bar elements BEAM44 (see the figure 3), mainly with assigning of the cross-sections generated from the areas. In this way, complex segmented cross-sections could be assigned. Auxiliary elements were used for modelling the anchors, local stiffeners, location masses representing equipment. The preparation of the model and calculations were performed in close cooperation with a team of specialists from VUT Brno under the leadership of prof. Štěpánek and Salajka.

Set of accelerograms has been used as input for seismic analyses in accordance with IAEA documents. Following SL-2 seismic level with peak acceleration values is specified for the nuclear power plant Dukovany:

PGA_{RLE} = 0,100 g in horizontal direction
PGA_{RLE} = 0,067 g in vertical direction

Loads from extreme meteorological events were assigned in accordance with methodologies [1] and [2] and document [3] as static load.

The result of the calculation of the response to the self-weight load and to the seismic load at the level of the basement slab were:

- displacements in the direction of the coordinate axes in all nodes of the model,
- internal forces of bar elements,
- stress tensor components and internal forces in 2D elements,
- stress tensor components in 3D finite elements.

The results were calculated in 2501 time steps for 3 load cases of horizontal arrangement of accelerogram components. From these values, in the ANSYS system the files have been created for subsequent assessment of the responses to seismic load in combination with the static response. The time required was considerable, one complete recalculation with evaluation took 30-40 days. The results were used as a basis for a detailed assessment of the elements of load-bearing structures. The effects at the anchorage points were used as an input basis for partial detailed 3D models of individual anchoring elements and connections.

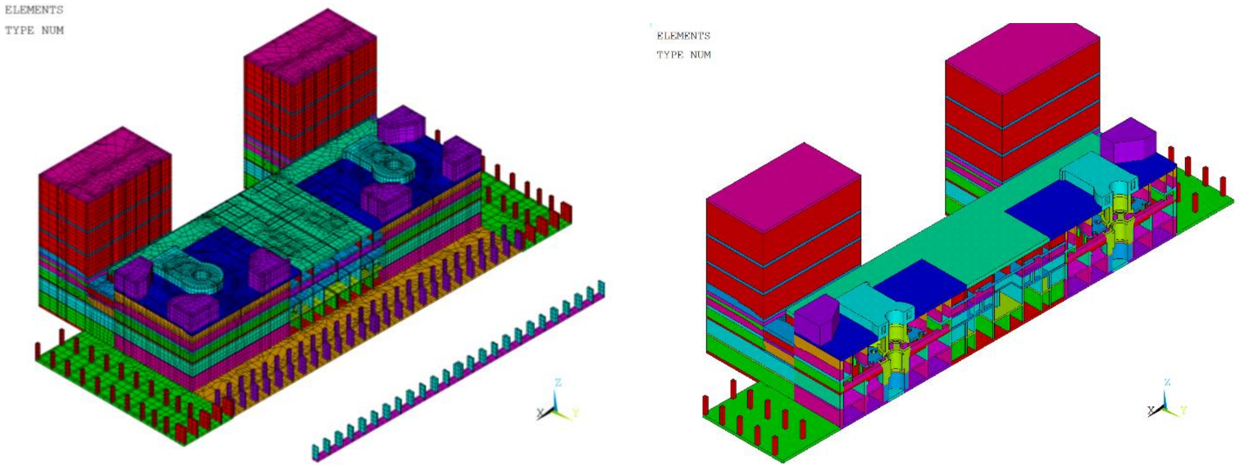

Figure 2. Part of the model of the main concrete structures

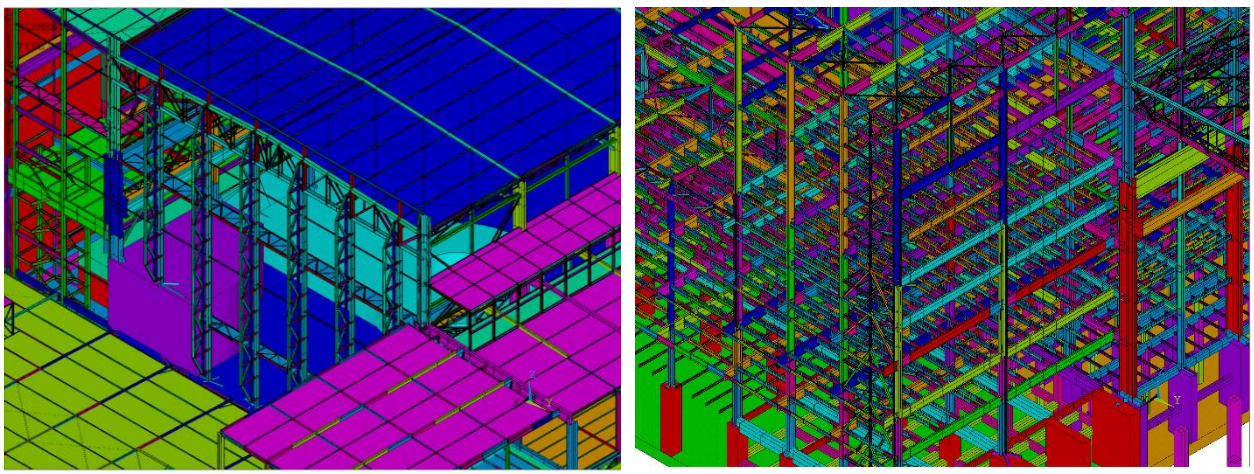

Figure 3. Part of the model of the main steel structures

PROJECT DOCUMENTATION, DESIGN OF TECHNICAL MEASURES

Based on static and dynamic calculations of the real state of load-bearing structures, all elements of load-bearing structures were assessed in detail, especially steel structures, their joints, and anchors. The range involved thousands of elements and joints and hundreds of anchor connections. The technical measures needed to ensure the required seismic resistance and resistance to other natural events were implemented. All phases of project documentation were prepared, from the documentation for the site permit through the documentation for building permit, up to the detail design. At the end of the completed execution, the as-built documentation followed, including the recalculation of the final model, which included all designed and implemented technical measures.

The designed measures can be divided into several basic groups.

Reinforcement of partial elements of segmented bars of steel structures by increasing the cross-section along its entire length by welding sheets or another profile. Can be seen in the figure 4.

Figure 4. Reinforced upper and lower steel truss strip – diaphragm – 1. unit

An interesting solution was chosen for the steel columns in the gable wall of the reactor building, where it was not possible to design the reinforcement from inside the building due to spatial reasons. Therefore, the external steel segmented structure from the outside was used in this case. The external frame structure was connected to the existing columns through the cladding panels. This method required the solution of a number of complex details and execution procedures, especially at the points of penetration through peripheral cladding at the place of connection. This solution sufficiently increased the required bending stiffness of relatively slim steel columns.

Another group of designed measures was the addition of roof and wall bracing in order to increase the spatial stiffness of steel structures. This reinforcement was followed by the reinforcement and modification of individual joints. The main measures consisted of strengthening the welds, enlarging the joint sheets, and welding the additional reinforcements.

Probably the most demanding group of measures was the reinforcing of the anchoring of steel structures, namely the anchoring of the steel structures of the longitudinal and cross-side electrical building to the reinforced concrete walls of the reactor building. The existing anchoring of the main floor beams was performed using the anchor brackets, which were welded to pre-embedded steel anchor plates with welded mandrels. Based on the results of the calculations, the load-bearing capacity of these anchor points had to be significantly increased, especially in the horizontal direction. This anchorage is very important for the stability of the steel frame structure of the connected structures. The stiffness of this anchorage practically determines the overall rigidity and stability of connected steel structures of the entire system of main power unit buildings. Due to the spatial fragmentation of main power unit structure, whose stiffness is highly variable, the effects on the anchor points were substantially different.

The design of the reinforcement was greatly complicated by the fact that the inspection and detailed measurement of the entire connection was possible only at the beginning of the actual execution because the elements of steel structures are protected in these places by fireproof spraying. Their removal could not be executed in the pre-project and project phase. Regarding the limited load-bearing capacity, to which the existing anchorage could be reinforced, new anchoring elements were designed. The new elements were placed as intermediate, always three new anchors among the existing ones, which were usually 12,0 m in axial distance. The new anchoring elements stabilized the actual reinforced concrete ceiling slab between the main anchors and thus took over a significant part of the overall horizontal effects of the load-bearing structure, especially from the seismicity that was decisive for these structures. The new anchoring elements were designed as additionally installed anchor plates with anchor pins embedded into the drilled holes. Anchor brackets were welded to them. The existing embedded anchoring elements were reinforced with a similar method. The shape of the individual reinforcements and the new anchoring elements had to respect a very limited space and all installed technological equipment and especially the cable trays and other equipment.

Therefore, practically every anchor point required an original technical solution both in terms of shape and in terms of technological procedure. Most of anchor points is located in parts of the reinforced concrete walls, which are part of the hermetic space and from the external part are covered by a full-area steel hermetic liner. For the installation of new anchoring bolts, it was necessary to open the hermetic liner locally and after the installation of the anchoring elements to ensure again their tightness. Special details were designed to allow the creation of welds in a closed inspection space, which could be used to check the given place by pressure tests through pressure nozzles. In this way, all places have been solved where the hermetic liner had to be opened. All these activities had to be executed only during the shutdown, which complicated the whole process.

For a reliable design of reinforcement of anchoring elements, a detailed computational 3D model was compiled for each type of anchoring element, including the nonlinear interaction of the steel plate with the concrete base of the wall. The model also included all elements of the anchor bracket and other newly designed reinforcements, which affected both the tension in the anchor plate and the distribution of effects on the individual anchor bolts. The primary criterion of the design reliability in terms of decisive seismicity was the status of the anchoring element, where the limit will be decided by the plastic failure of the steel, not the brittle failure of the concrete. In parallel with the design of the anchorage reinforcement, which changed the stiffness of the connection itself, it was necessary to reflect these changes in the overall calculation model. By changing the stiffness of the connections, the dynamic characteristics of the construction model has changed. Due to the different stiffness of individual connections and the overall spatial fragmentation of the model as a whole, there were significant changes in the effects on individual anchor points and also changes in the internal forces of the steel structure elements. Therefore, the overall model had to be recalculated many times. It was a long-term iterative process of the influence of the reinforcement design of the structure elements and the anchorage on the dynamic characteristics of the structure as a whole. Regarding the time-difficulty of one recalculation of the complex model, which was about 40 days, it was a long-term process.

THE EXECUTION

The execution of construction works on the strengthening of load-bearing structures was very closely related to the creation of project documentation and especially shop drawings. After the beginning of the preparatory work for the execution, some places were made available, especially in the area of anchoring, where it was possible to verify the actual state and surveying. These were the input data for a detailed static assessment and reinforcement design. This process of close interconnection of design and execution activities accompanied the entire execution of construction work to strengthen the load-bearing structures and was important for the planning of individual activities. For this purpose, "On site" team was created, consisting of structural engineers and designers, which was permanently present on site in the crucial phases of execution. During the activity on anchoring in shutdowns, when had to work continuously in three shifts, the three teams took turns in the same shifts. This ensured that a team of designers was permanently available to deal with all the necessary changes that arose after the uncovering of anchor points protected by fireproof spraying. Modifications of local calculation models of individual details were operatively solved regarding their current status, followed by updating of static assessment and correction of drawing documentation. In this way, the own execution of work was significantly speeded up, which had to be performed only during the shutdown of the unit. These time periods were very short, and therefore was necessary precise preparation and planning of individual activities.

Another important activity performed by this team was the execution of technical measures that ensured that during drilling holes for anchoring bolts into reinforced concrete walls, the load-bearing reinforcement is not damaged and thus the load-bearing structure of the hermetic envelope is not weakened.

Detailed working procedures have been developed and tested for this purpose. Individual activities provided by the "on site" team consisted in verifying the actual position of individual reinforcement bars of the load-bearing reinforcement and the resulting correction of the position of individual anchoring bolts. In places where there was no steel liner, the method of scanning the positions of the load-bearing reinforcement was used. The bigger problem was in places with steel liner, where scanning could not be performed. Smaller testing drills were used here for detection the actual position of the individual reinforcement steel bars. After finding the first steel bar, the structural engineer, using the original reinforcement drawings, specified the expected position of the other steel bars in the relevant direction. According to this information, the positions of individual drills were corrected. This process required a very precise approach by the drill operators and their close cooperation with the designers on site. Strictly following of this procedure was ensured that the load-bearing reinforcing steel bars were not disrupted and thus the load bearing reinforced concrete structures were not unacceptably weakened.

An interesting construction detail in the cases of anchoring was used in places where was necessary to install anchoring elements in the wall equipped with a hermetic steel liner. Can be seen in the figure 5.

Figure 5. Detail of Reinforcement procedure of the main anchor and beam connection

It was necessary to ensure full hermetic tightness by the tests with the prescribed overpressure. For these purposes, atypical double sealing metal sheets have been implemented in terms of the installation of the anchoring plates around the openings to form a closing intermediate layer which could be controlled by pressure. The inner boundaries were formed by sealing metal sheets welded to the anchoring bolts and the surrounding steel liner. The external boundary was formed by the main anchor plate welded by load-bearing welds to the anchoring bolts and the sealing metal sheets welded around the perimeter of the anchor plate and to the steel liner. The inspected area created in this way was tested for the prescribed pressure via the pressure nozzle, resp. overpressure. In this way, the required hermetic quality of the steel liner was verified retrospectively. In addition, this method allows to repeat the tightness control of individual points at any time in the future.

The execution of designed measures to reinforce the roof structures, especially the reactor building and turbine hall, required the use of work platforms at a height of more than 20 m above the floor. These were lattice spatial structures with a distance of more than 40 m and the height of beam is about 4,0 m. For allowing these structures at their whole height and platform were designed the spatial structures of system scaffolding that have been installed on crane bridges, which are installed in both buildings. The structures of the platforms were designed to allow them to be moved under another beam after partial dismantling. After moving the base part under the beam, the high parts were installed again. The construction of the platforms had to allow the transfer of heavy parts of steel structures, their preparation and welding, and at the same time to ensure the possibility of dust-tight separation from the surrounding area. Such closing areas were air-conditioned due to the high temperatures in this space and at the same time fire protection was solved. The activities on the roofs of the reactor building and turbine hall had to be performed only during out of shutdown, in order to be possible to use the main bridge cranes. The structures of the platforms were designed in the way to be possible to dismantle at prescribed time in case of operational requirements and the crane could be prepared for standard use. It was a very complex overall unique structure. This can be seen in the figure 6. Even at this phase of execution, the "on site" team of designers was significantly applied, when it carried out detailed inspections and measuring of individual joints of lattice structures, that were available in this way. This was followed by their static assessment and operational design of the relevant reinforcements. These activities were performed in close coordination with other activities to reinforce steel roof structures.

Figure 6. Construction of a work platform on a crane bridge

Reinforcement from the external part of the building had to be designed to reinforce the gable columns of the reactor building. This solution was chosen for spatial reasons and the requirement to minimize dust technologies (grinding and welding) inside the building. It involved the assembly of external frame elements with a length of more than 30 m in a vertical position on gable columns. The assembly took place above the roof of the cross-side electrical building. As a result, the assembly was complicated by the large distance and height, it was necessary to use mobile cranes with maximum tail radius. However, their location was complicated by a number of underground channels, where bridging in the positions of the crane support had to be solved. The assembly required very sensitive handling of the long load during its installation for prepared connecting brackets anchored to the existing columns.

Figure 7. Reinforcement of the columns from the external part of the reactor building

CONCLUSION

After completion of the implementation, the as-built documentation was prepared. This was the basis for the preparation of an independent final evaluation report which proved compliance with the requirements for the resulting resistance of structures to extreme meteorological events and seismicity. This fulfilled the conditions specified by the State Office for Nuclear Safety of the Czech Republic for extended operation of the nuclear power plant in Dukovany.

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