Requirements and Possible Upgrading Concept for the VVER-440/213 - Mochovce NPP Structures Under Seismic Conditions

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Abstract

The Mochovce-Nuclear Power Plant is one of the VVER-440/213 plants which has been designed against earthquake. Nevertheless, the design earthquake has not been assessed adequately to the seismic hazard at the site. A new seismic design shall include an increased seismic input and assure an acceptable standard of safety.

This contribution is related to some design aspects of civil structures for this nearly finished plant, such as

- existing design and its margins with regard to the employed codes.
- requirements for a new design concept.
- effects to be expected by an increased design earthquake.
- applicable design methods; use of inelastic design spectra, behavior factors and capacity design.
- feasible upgrading measures.

1 Structural Layout

The Main Building consists of the reinforced-concrete structures of the reactor building comprising the confinement with the barbotage tower, and of a steel shop superstructure to which an intermediate six-story building for the electrical equipment as well as a part of the turbine house are linked (see also Fig. 1). A four-story cross wing for the switchgear building is attached to the reactor building aside.

The box structures of the confinement in thicknesses up to 1.5 m are built using prefabricated spacial truss elements consisting of the reinforcing steel, a steel liner including its stiffeners and anchorage, which were mounted, welded together and filled with concrete.

The floor slabs in storeys with equipment are made of reinforced concrete poured on steel sheeting supported by steel girders. Precast elements are widely used wherever possible.

The overall dimensions of the structures for one reactor-block connected without a joint are about 96 x 136 m in plan. The maximum height is about 56 m. A total amount of structural steel built in one block is about 5400 t.

The structural members with a lateral-force-resisting capacity have a spacing of 12 m, intermediate members having only a vertical load capacity are arranged in a half-spacing. The stability to resist lateral forces in the cross-direction is ensured by cantilevered columns of the turbine house, by moment-resisting frame structures and, above all, by the reinforced concrete structure of barbotage tower. Vertical bracings have been designed for lateral forces acting in the longitudinal direction. Horizontal bracings at different roof levels complete the structural layout.

Considering a clear arrangement of bearing and stiffening members, and the elaborated construction in detail, the steel structures appear to be well designed in a classic manner for all conventional load conditions. The used connections correspond to prevailing mostly simple static systems. In all defined hinge-beams the shear is transferred to columns by cleats, and the horizontal forces are carried by rough bolts or screws of a normal strength. Considerable efforts have been made to reduce stresses due to restraint deformations, e.g. pendulum bearings are used for hinge piers.

2 Original Seismic Design

A characteristic feature of the design in Mochovce is the linkage of the all steel structures to the barbotage tower for which a special horizontal bracing has been designed. As can be seen from the Figure, a considerable structural irregularity is to be traced when designing for horizontal forces. This problem has been well recognized in the original earthquake design. A three-dimensional overall-model was used in the dynamic analyses performed in the former Soviet Union. The earthquake loads were determined using a RSMA method for which an inelastic ground spectrum according to a soviet standard for structures in seismic regions was employed.

The basic criteria for design of structures and components were comprised in the former soviet standard VSN 15-78 issued as a preliminary standard in the nuclear-field application. In compliance with international safety requirements, it defines the basic safety goals, the classification of structures and systems, two design earthquakes, general requirements for earthquake calculations of civil structures, systems and components including load combinations and stress allowables, rules for antiseismic measures and further applicable standards.

Unfortunately, the design earthquake values have not been assessed adequately to the seismic hazard at the site. The maximum design earthquake for civil design has been based on a acceleration of 0.05 g assumed as a resulting acceleration vector in the space. Thus the effective ZPA-value used in the calculations varies between 0.025 and 0.033 g. Floor design response spectra as determined for the Ukraine plant in Rovno, which is of the same type and similarly founded on bedrock, were employed for design of components and systems. These spectra are based on an accelerogram of the Vrancea-earthquake with a ZPA of about 0.05 g as measured in Nis.

A new seismic input is still under discussion. However, it is obvious that an input with a ZPA below 0.1 g will not be acceptable with regard to the actual seismic hazard, and the IAEA-recommendations as well. The possible increase of floor accelerations in the reactor building is shown in Figure 2. The "best-estimate" values are based on USNRC-Spectra with ZPA of 0.1 g.

3 Proposed Scope of Activities

As the most items have been already finished, considerable efforts are needed to demonstrate their safety under the increased earthquake conditions and to reduce the upgradings to a necessary minimum.

The following activities are recommended to be performed in a prephase:
- Assessment on the terms of reference for earthquake design including the final assessment of the seismic hazard, completion of seismic studies, and definition of design earthquake (SL-1, SL-2) and of the pertinent design ground spectra.
- New earthquake design calculations and spectra of the reactor building with coupled steel structures.
- Main features of the upgrading concept.

In a second (design) phase:
- Elaboration of the terms of reference for earthquake design
- Design interference with other upgrading requirements
- Collection of the documentation, walk-downs to perform as built comparisons

In the execution phase:
- Design calculations and upgrading concept in detail:
- Completion of design or redesign of structures, systems and components including anchorage, corresponding walk-downs
- Qualification procedures for non-structural elements
- Upgrading works

A proposed specification comprising the earthquake terms of reference shall summarize the design principles and give the application rules for all class 1-structures, systems and components with the aim to show their adequacy to the state-of-the-art as required by the corresponding design principles of IAEA. In doing so, the original soviet and national standards (SNiP and CSN-Standards) will be employed enhanced by consistent international standards if necessary or appropriate.

Besides, the specification will be used to define the design conditions to all participants and suppliers involved in calculations, design and execution of upgrading works.

Preliminary check of specific items whose documentation has not yet been sorted over or which are supposed to considerably affect times and costs, such as existing safety margin of the precast members of the ventilator fan building, or quantities of brickwork, and of wall-facing panels possibly subjected to different upgrading measures in particular buildings shall be performed in a prephase.

4 Existing Margin and Design Concept

When making proposals for the seismic requalification of the civil structures of the Mochovce-Plant, the following design features should be considered:

i) The structural members shall be designed to demonstrate their seismic resistance at design earthquake within specified limits. The original design of Mochovce is a design in limit states. However, the margin of safety contained in the used standards is lower than in other corresponding standards in Europe or in USA. As can be seen in the example related to design of steel structures, the safety participation factors for loads implemented in the Czechoslovakian standards are lower than those required according to the western standards.

Global Safety Factor for Steel Structures $\gamma_F * \gamma_M * \psi$

	DL	LL (max Q_k)	LL ($Q_{i.k}$)
DIN 18800 (EC 3)	1.35 *1,1 = 1.49	1.5 * 1.1 = 1.65	1.5 * 1.1 *0.9 = 1.49
LFRD-AISC	≥1.4*1.1 = 1.54 ≥1.2*1.1 = 1.32	- ≥1.6 * 1.1 = 1.76	
CSN 730035 731401	1.1 * 1.15 = 1.27	≥1.2 * 1.15 = 1.38	1.2 * 1.15 * 0.9 = 1.24

As a consequence, a steel structure designed for conventional load combinations according to these eastern standards will have less weight and less margin of overburdening than the same structure designed according to the western standards.

ii) The actual percentage of the non-permanent loads acting within a seismic load combination, has not been utilized in the original design. Thus, it will be rather worthy to distinguish between the variable loads to be considered as acting on the building structure during the design earthquake and those which may acting locally on a single structural member. The load plans should be checked and reviewed in this respect.

iii) Lasting (plastic) deformations after design earthquake are permitted. However, further requirements, e.g. on permeability of special structural parts, shall be met in serviceability limit states. There are several restrictions related to plastic design in the CSN-standard for design of steel structures.

iv) Using of behavior (ductility) factors either to obtain in- elastic design spectra or to reduce earthquake stresses resultant from linear-elastic calculations are commonly permitted for non-Class 1 structures or for the seismic verification respectively. The most questionable item is the level of structural ductility.

The presupposition for the use of structural ductility is
- presence of inherently ductile elements in the structure (ductile design in detail),
- the arrangement of these elements so as their dissipative capacity may be actually exploited during dynamic response,
- the proportioning of non-ductile elements to remain undamaged and to allow the dissipative elements to perform as expected.

The typical desired mechanisms are
- the formation of plastic hinges at all beam extremities and at the base of the columns in sway frames
- at the base of the walls and yielding in all coupling beams of the wall structures.

The unwanted mechanisms are
- hinging at the extremities of the columns (with some few exceptions)
- inelastic shear deformations in beams, walls, columns
- inelastic behaviour due to cracking and loss-of-bond in beam-to-column joints
- inelastic behavior of foundation structures and -soil

As can be seen from the Figure 3, several upgradings may be useful to increase the ductility of the steel structural members to be enabled to carry earthquake loads with energy dissipation and load redistribution. However, any redistribution of earthquake loads is bound to increase horizontal earthquake forces affecting the anchorage in the concrete of the reactor building.

Thus, the non-damage-criteria of ductility and stability are to be compared with serviceability criteria at high risk potential for the special Class 1 structures; e.g. a global ductility of the hermetic room structures may affect its gas-permeability.

v) Some results of contemporary studies related to the structural design of the main reactor building indicate that only minor improvements on structures might be really necessary. However, a sufficient seismic stability under the increased earthquake loading has to be demonstrated for each class 1-structure.

The extent of recalculations will depend on existing margins in the previous design. So, it may vary between a complete new design calculation for the combination with earthquake loads in an unfavourable case, such as that of steel structures coupled to the reactor building, and a simplified qualification check in a favourable case which contains a wide safety margin. Thus, it is important to check the expected efforts in some specific cases.

vi) A little less attention has been paid to the seismic stability of non-structural elements, such as to brickwork or to light-weight concrete facing panels. Both items widely used all over the plant must be checked at all locations with a safety-related impact. The panels should be subjected to dynamic testing at realistic support conditions and must be upgraded to prevent them from falling down. Unreinforced brickwork near safety-related components must be reinforced or removed.

vii) All anchoring elements should be checked against design earthquake and improved following the principles of seismic detailing. Reinforcing measures can be expected in several horizontal and vertical bracings of steel structures.

Additional bracing might be necessary to improve the seismic capacity of structures made of precast concrete members.

5 References

Miscellaneous design reports of Mochovce
Internal Siemens-Report KWU R322/92/0546

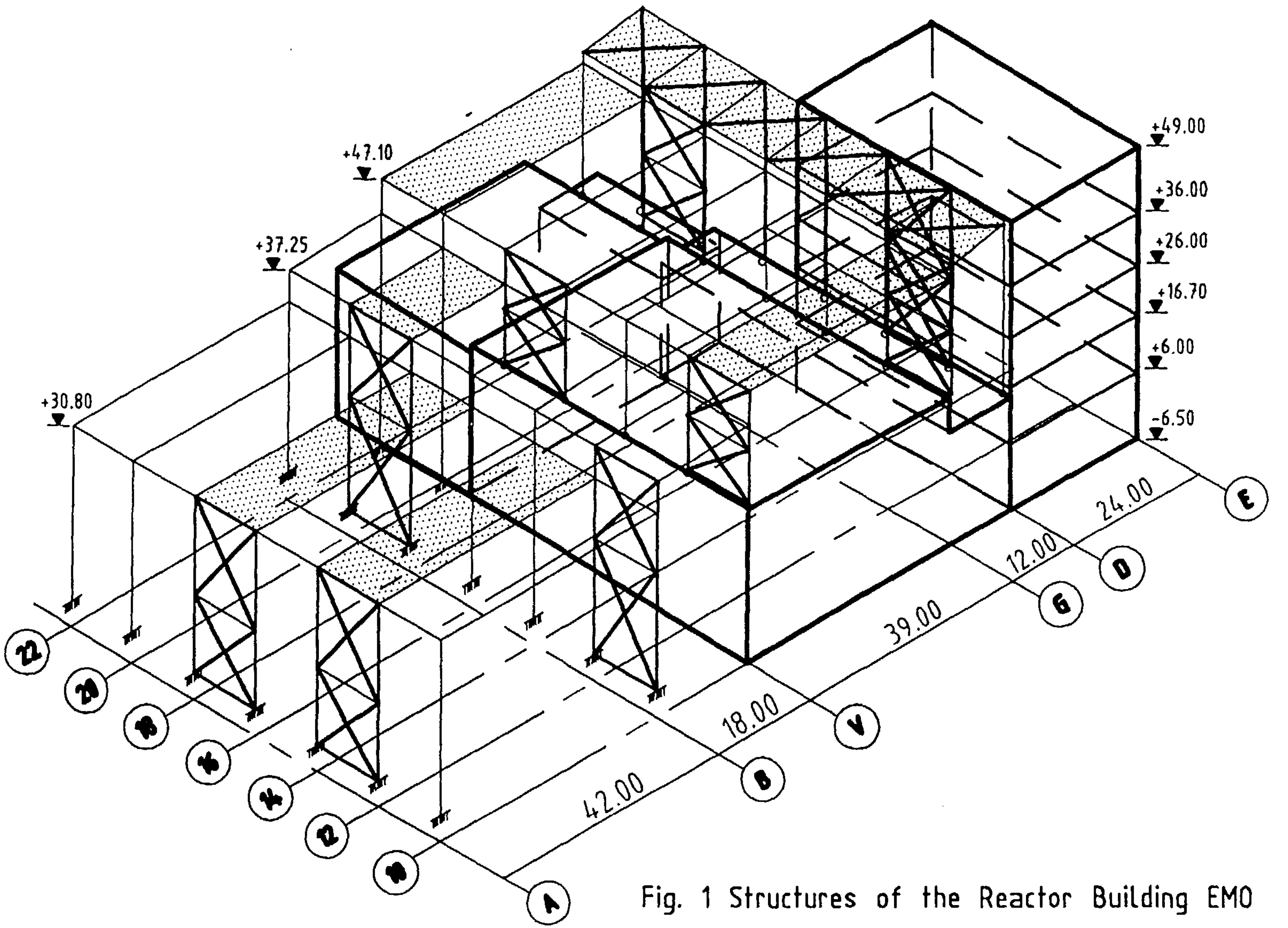


Fig. 1 Structures of the Reactor Building EMO

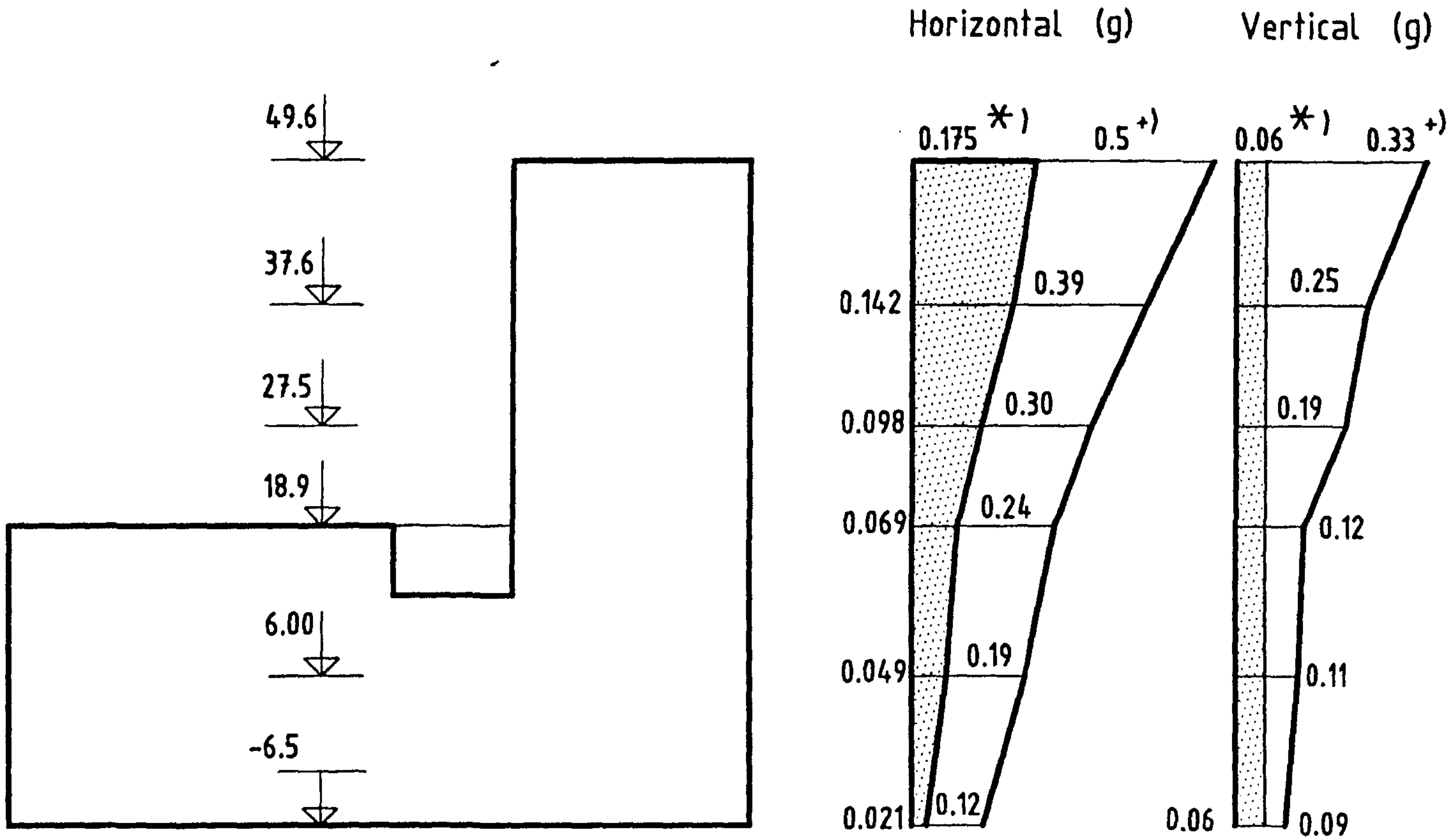
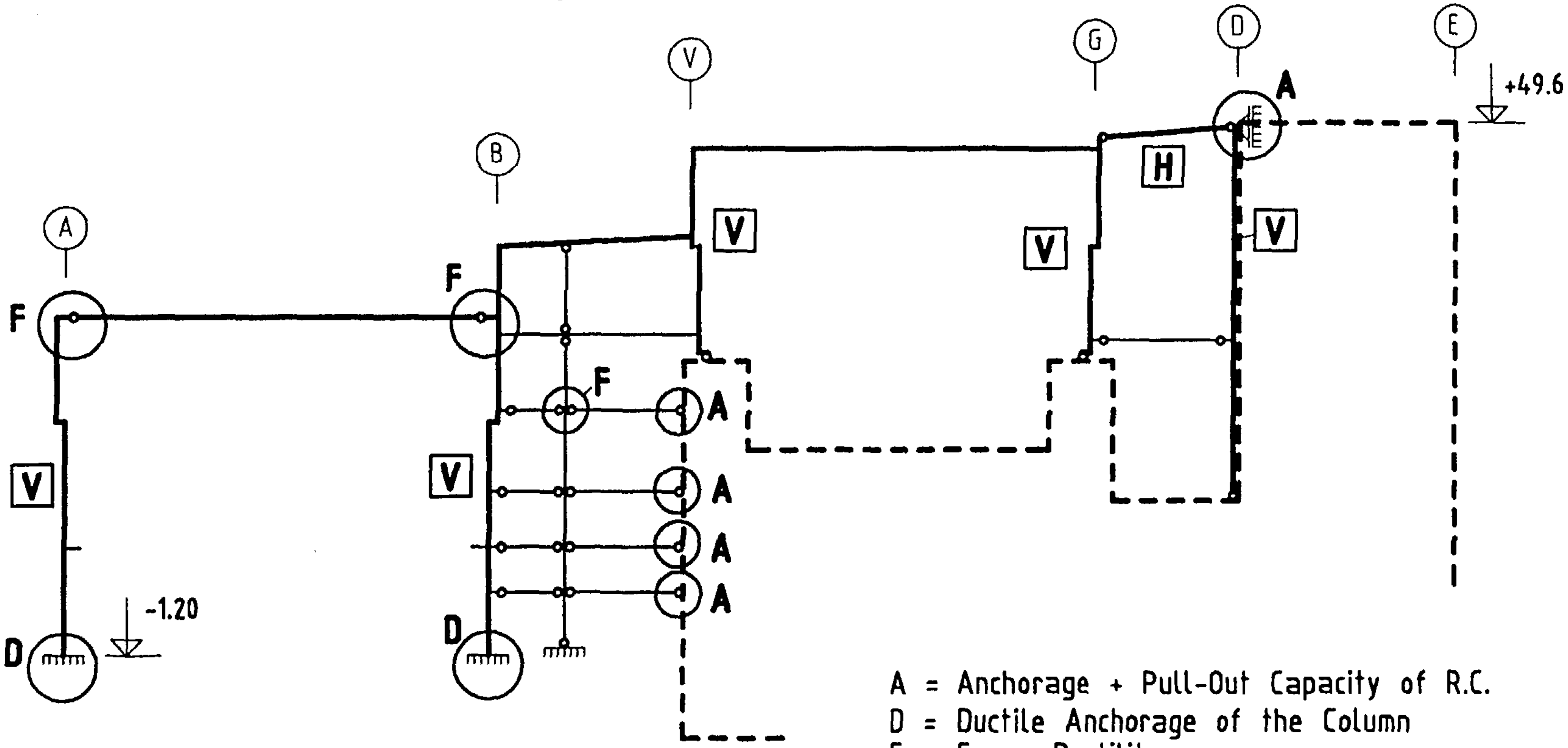


Fig. 2 Zero-Period-Accelerations at Selected Floors of RB
*) Original Design +) "Best Estimate"



A = Anchorage + Pull-Out Capacity of R.C.
D = Ductile Anchorage of the Column
F = Frame Ductility
H = H-Bracing
V = V-Bracing

Fig. 3 Upgrading of Steel Structures