SEISMIC SAFETY INVESTIGATIONS OF AN OLD HYDRO-POWER-PLANT

H. E. Ermutlu (I)

SUMMARY

The purpose of the presented seismic investigations is to assess the seismic safety of HPP Mühleberg structures subjected to a ground motion with an intensity level of SSE postulated for NPP Mühleberg site. For this, the two structures of the HPP are idealized by two-dimensional planar finite element models. These models are so detailed that the dynamic behaviour of the structures are well represented and they provide sufficient information for an adequate safety check analysis. The results of the seismic analysis show that, the main water retaining structures of the "over 60 years old" HPP Mühleberg remain stable during an earthquake which is postulated for the neighbouring nuclear power plant.

INTRODUCTION

Hydro-Power-Plant Mühleberg, Switzerland, constructed during 1917-1920, is composed of a Weir Structure and a Machine Building. The water retention parts for both of the plant structures are massive concrete dams. No seismic loading is considered in the original design of the HPP structures.

Recently, envisaged as a part of a future additional emergency cooling system for the NPP Mühleberg, and being situated only approximately 1 km upstream of the NPP site, HPP Mühleberg becomes an important part for the complex of NPP Mühleberg and therefore now it is considered as a "Seismic Category-I" structure. This requires that, the seismic safety of the HPP Mühleberg structures has to be investigated employing dynamic analysis methods, under SSE loading condition postulated for the NPP Mühleberg site.

DESCRIPTION OF PLANT STRUCTURES

The general layout of HPP Mühleberg is given in Fig. 1. The plant is primarily composed of two structures, appr. 70 m. long WEIR at the east side, and appr. 150 m. long MACHINE BUILDING at the west side. Under the roof of the Machine Building, two separate structures are identified, appr. 50 m. long TRANSFORMER BUILDING at the west bank, and appr. 100 m. long TURBINE BUILDING in the middle of the plant. These three main plant structures are separated from each other by means of toothed expansion joints.

WEIR STRUCTURE is a concrete gravity dam with a vertical upstream face. It is also divided into 3 parts with different cross-sections. With a rough comparison, considering the height, weight and foundation width, one can easily distinguish the over-flow section with vertical gates, as the most critical section of the weir structure, from the seismic safety point of view. This section is appr. 21 m. high and appr. 17 m. wide (at level 464.20 m). At level 473.00 m. there is an upper stilling-floor which is supported at down-

(I) Chief Specialist, Emch + Berger Bern AG, 3001 Bern, Switzerland

stream face of the dam by concrete arches. At the downstream of the gravity dam, a stilling-basin, with appr. 17 m. length, is laid down. Although the gravity dam and the basin are constructed as a continuous structure, due to the existance of hollow arches under the basin, it is expected that, the connection zone may easily be cracked whenever the gravity dam is forced to an excessive rocking motion. For this reason, in the analysis for the typical weir section, only the gravity dam part is taken into consideration and the stilling-basin is omitted (Fig. 2).

TURBINE BUILDING houses 8 turbine-generator groups. The distances between the axis of each group is 11.50 m. The width of the structure at the foundation is about 37.60 m. The water retention part is a massive concrete gravity dam, penetrated by the water inlet and outlet channels for each turbine-generator group. At the bottom of the gravity dam and the inlet channels, arched-caverns are constructed, where the uplift pressure is annihilated. With such an arrangement, the uplift forces are reduced extensively.

TRANSFORMER BUILDING houses 2 transformers. The distances between the axis of each transformer is 9.50 m. The width of the structure at the foundation is about 23.40 m. The water retention part is a massive concrete gravity dam. Similar to the turbine building, at the bottom, arched-caverns are constructed to reduce the uplift forces. The structure is anchored into the foundation rock by means of the foundation walls, in order to increase the resistance against sliding (Fig. 3). Comparing the weights, the widths, foundation contact areas and the sizes of the arched-caverns, one can easily conclude that the Transformer Building is less massive than the Turbine Building, and so, it is more critical from the seismic safety point of view.

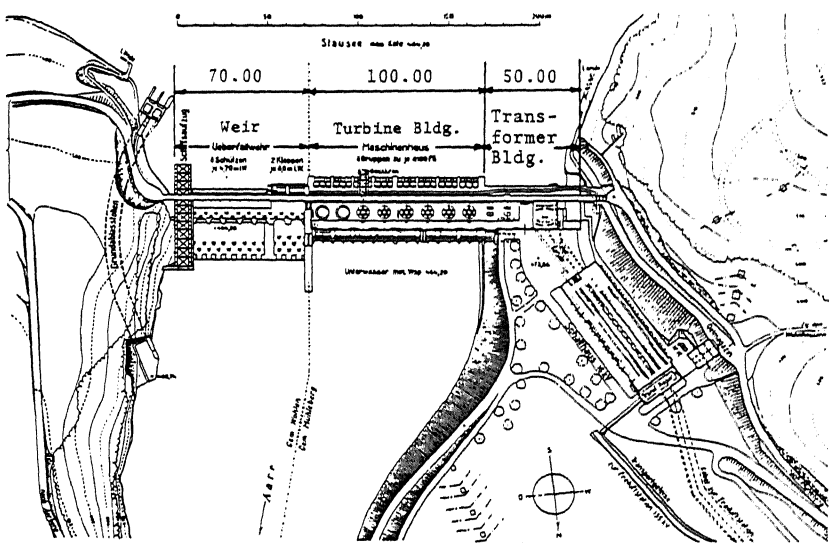

Fig. 1 General layout of HPP Mühleberg

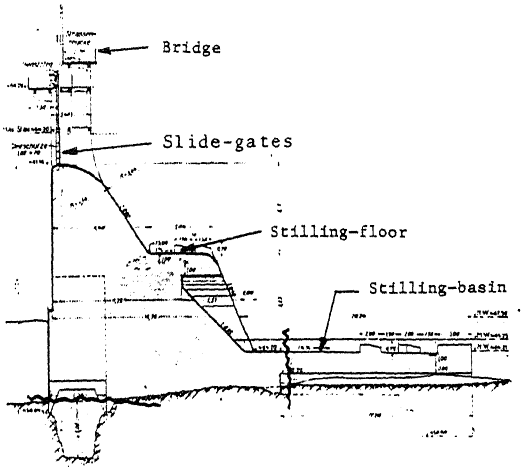

Fig. 2 Weir, overflow section with vertical slide-gates

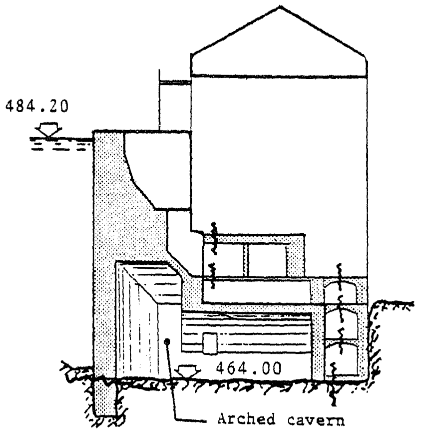

Fig. 3 Typical section of Transformer Building

FOUNDATION SOIL PROPERTIES

The geological and geotechnical properties of the foundation formations at the HPP Mühleberg have been investigated extensively and the results of these investigations have been presented in Ref. 1.

The soil profile under the HPP Mühleberg structures is given in Fig. 4. The properties of the layers above 439.00 m. are established as based on the results of the cross-hole survey. The properties of the layers below 439.00 m. are estimated under the light of the present knowledge and experience gained from other sites having similar geological formations. As the results of the cross-hole survey show a rather high degree of regularity, it is decided to use only one set of dynamic soil properties, and no further parametric investigation is foreseen.

The minimum compressive strength of the rock material determined by laboratory tests is 300 t/m2. This value is utilized for the safety check of the structures due to overturning.

Resistance to shear within the foundation and between the structure and its foundation results from the cohesion and internal friction inherent in the materials and

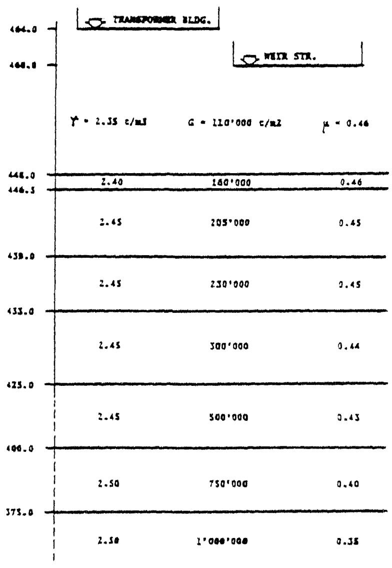

Fig. 4 Dynamic properties of rock layers

at the concrete-rock contact. The minimum values for the cohesion and the internal friction angle of the rock material are determined by laboratory tests as 50 t/m2 and 20°, respectively.

LOADING CONDITIONS

As there exists no specific Swiss Regulation for the seismic resistant design of hydro-power-plant structures, throughout the present analysis, the Design Manuals of USBR, as well as the Regulatory Guides and the Standard Review Plans of USNRC are utilized as guide-lines.

USBR (Ref. 2) defines the loads to be considered for the "Extreme Loading Combination" as follows:

> "Normal design reservoir elevation, with appropriate dead loads, uplift, silt, ice, usual minimum temperatures if applicable, and tailwater, plus the effects of the MAXIMUM CREDIBLE EARTHQUAKE."

As the definition of MCE by the USBR is similar to the definition of SSE by the USNRC, in this present paper instead of MCE the term of SSE is used.

Hydrostatic loads (with normal reservoir elevation at 484.20 m), all relevant dead loads, 100 % of uplift and silt pressure at the upstream face are taken as static loads. The effects of ice and usual minimum temperature are excluded. Hydrostatic load due to tailwater is neglected.

The earthquake at NPP Mühleberg site is specified with the maximum horizontal ground acceleration at rock surface, for SSE condition, as 0.15 g (Ref. 3). USNRC R.G.1.60 Design Response Spectra are applicable at rock surface.

The hydrodynamic pressures acting on the contact surfaces are approximately accounted for by assuming a portion of the water to move with the structure. WESTERGAARD's formulation for the hydrodynamic pressure acting on a rigid structure is employed. The mass of water as computed by this procedure then is appropriately lumped at the grid points of the contact surface.

DYNAMIC EARTHQUAKE ANALYSIS

Modal Response Spectrum method is employed for the dynamic analysis of plant structures. The natural frequencies and the corresponding modal information are determined by using NASTRAN computer code. Response calculations are carried out by using the program MODAL included in the E+B's Dynamic Analysis Package (Ref. 4). All combination procedures applied in the response spectrum analysis are consistent with USNRC R.G.1.92.

For the material damping of the individual structural element, the damping values given in USNRC R.G.1.61 are used, with the exception of the massive concrete elements, for which 5 % material damping is selected. The modal damping ratios are determined by means of a weighted averaging procedure considering the modal strain energy distribution in the structural members having different material damping properties.

STRUCTURAL MODELLING

The two selected critical structural sections are idealized as 2D mathematical models using plane finite elements, bars, springs and rigid elements. In case of existence of caverns, holes, etc., the thickness of the wall is taken directly if the wall repeats in each typical sector. If one hole covers more than one sector, then, the wall thickness is reduced proportionally. In order to maintain a sufficient degree of conservatism, only the main massive water retention parts of the structures are considered in the seismic safety analysis. For this, if a part of the structure seems to be weak then, the stiffness of this part is excluded from the model. The resistance of the omitted parts may be kept as a reserve, and whenever the safety of the main block seems not to be sufficient, then it may be considered as additional resistance. If there is a dynamic interaction between the massive block and the other structural parts, these parts are included in the model, however without increasing the stiffness of the main block.

The 2D plane finite element model of the Weir is shown in Fig. 5. This model consists of 27 quadrilateral, 50 triangular, 5 bar elements and 74 grid points. The 2D plane finite element model of the Transformer Building is shown in Fig. 6. This model consists of 36 quadrilateral, 59 triangular, 11 bar elements and 104 grid points.

SOIL MODELLING

The effects of the foundation on the dynamic behaviour of the structures are considered by using the "lumped mass-spring" approach. The soil is first represented by means of a series of soil springs and masses attached to the centroid of the basemat. Then, these concentrated masses and springs are appropriately distributed to the grid points along the bases of the structures. The soil spring constants, soil masses and soil damping ratios for

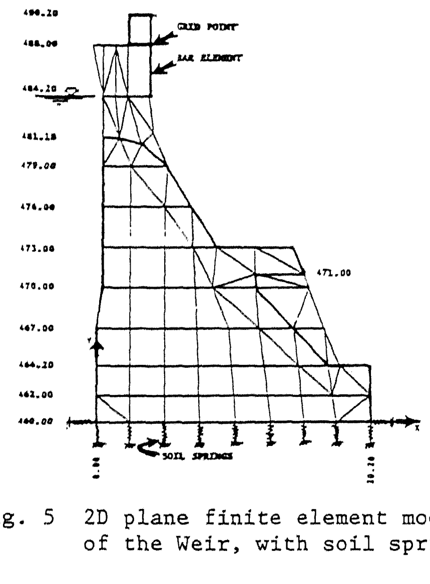

Fig. 5 2D plane finite element model of the Weir, with soil springs

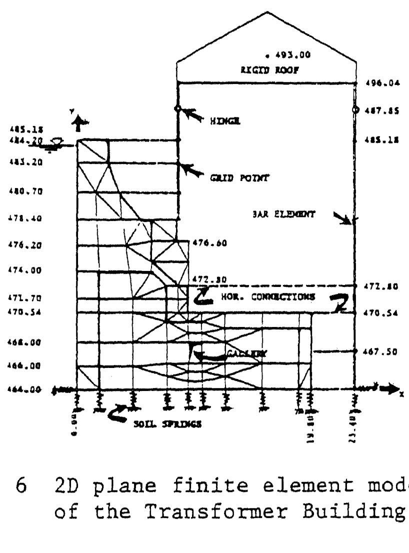

Fig. 6 2D plane finite element model of the Transformer Building horizontal and vertical modes of vibration are calculated by using "compliance function method". Soil parameters related with the rocking mode of vibration is approximately represented by means of the distributed vertical soil springs and masses. Attention has to be drawn that, such a procedure results with higher modal damping ratios for rocking modes. This requires a sensitivity analysis taking the soil damping ratio as the basic parameter.

The layering of the foundation soil is taken into consideration, and Christiano's approach (Ref. 5) is employed for the determination of equivalent shear modulus and Poisson's ratios for the layered soil system. Radiation damping ratios for horizontal and vertical mode of vibration are calculated by using the formulae given by Richart et.al.(Ref. 6).

RESULTS OF ANALYSIS

Up to about 50 Hz, 8 modes for the Weir and 10 modes for the Transformer Building are determined. The fundamental rocking mode shapes for the Weir and the Transformer Building are shown in Fig.s 7 and 8, respectively.

For the determination of the maximum stresses induced during earthquake excitation, the SRSS combination of the principal stresses may not give realistic values, since the axis of the principal stresses usually vary for each mode. However, regardless of the principal axis, the SRSS combination of the absolute maximum modal element stresses gives a conservative estimate for the maximum normal and shear stresses for a plate element. In the cases that, the principal axis closely coincide, or only one mode predominates the stresses, then such a procedure may give quite realistic results. Only for a few elements, in order to avoid over-conservatism, the critical combinations are checked by taking into consideration the directions of the principal axis.

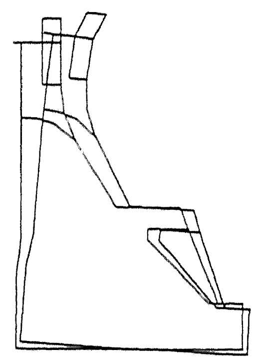

Fig. 7 Rocking mode of Weir Structure

For being examples, the maximum element tensile stresses under extreme load combination (Static + SSE) for the Weir and the Transformer Building are given in Fig.s 9 and 10, respectively.

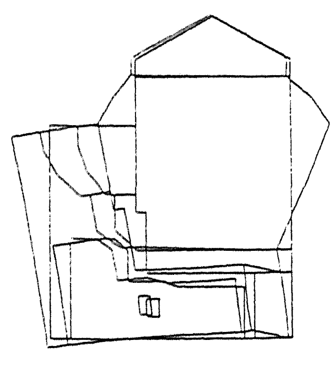

Fig. 8 Rocking mode of Transformer Building

The overall loads transferred to the foundation rock (foundation reactions) in terms of the horizontal and vertical forces, concentrated at the centroid of the foundation contact area and the rocking moment around the centroid are calculated for each static and dynamic load case and are listed in Table 1 for both of the structures. These overall reactions are the basic data used in the overall safety check, which requires an elaborated iterative procedure for the determination of the maximum pressure and shear stresses at the foundation rock. The base pressure distributions assumed for the Extreme Loading Combination are shown in Fig.s 11 and 12 for the Weir Structure and the Transformer Building, respectively.

FACTORS OF SAFETY

The seismic safety of the HPP Mühleberg is assessed by employing the safety criterias, safety factors and calculation procedures defined by the USBR (Ref. 2). These safety factors are based on the minimum ultimate strength of the concrete used in the construction, and the minimum foundation rock strength properties determined during the geotechnical investigations. The ultimate concrete strengths are determined by a probabilistic evaluation of the results of an extensive laboratory investigation carried out on the core samples taken from the actual structures (Ref. 7), which indicated 1120 t/m2 and 100 t/m2, for minimum compressive and tensile strength, respectively. Overall stability for overturning and the overall sliding safety are investigated considering the influence of the cracks which may occur between the concrete and the foundation soil.

It is shown that, all of the safety factors determined by the seismic safety investigations satisfy the requirements of the USBR. Therefore, it is concluded that, the main water retaining structures of the HPP Mühleberg remain stable during an earthquake with a maximum horizontal ground acceleration of 0.15 g.

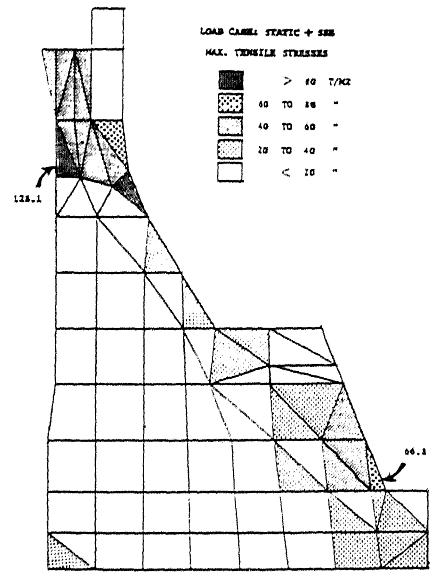

Fig. 9 Weir Structure, combined max. tensile stresses

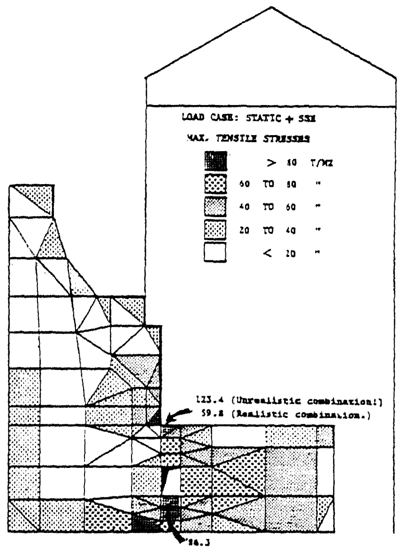

Fig. 10 Transformer Building, combined max tensile stresses

STRUCTURE (OR PART)	REACTION COMP.	STATIC LOADS			STATIC TOTAL	SSE
		DEAD	HYD. STA. (+ SILT)	UPLIFT		
WEIR STRUCTURE	HOR. (t)	0	1578	0	1578	± 772
	VERT. (t)	-3094	0	1271	-1823	± 377
	MOM. (tm)	8144	-12380	-4279	-8515	±10366
TRANSFORMER BLDG. "MAIN PART"	HOR. (t)	-50	1479	-7	1421	± 879
	VERT. (t)	-3740	0	192	-3548	± 488
	MOM. (tm)	5899	-13034	-1535	-8670	±15802
TRANSFORMER BLDG. "DOWNSTREAM SIDE FOOT"	HOR. (t)	50	460	7	517	± 363
	VERT. (t)	-536	0	0	-536	± 59

Table 1 Overall loads transferred to the foundation

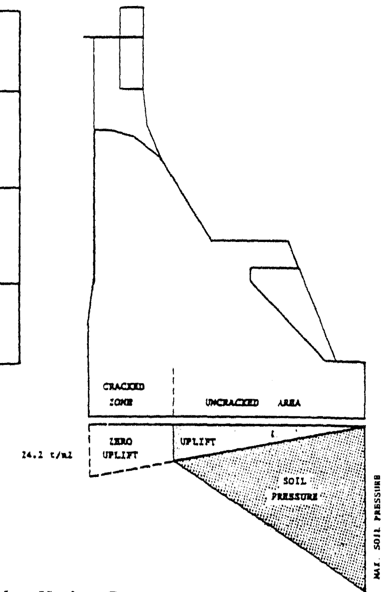

Fig. 11 Weir Structure, foundation base pressures during earthquake

ACKNOWLEDGEMENT

This investigation was carried out under the contract to the owner of both NPP and HPP Mühleberg, the Bernische Kraftwerke AG, Bern, whose support is gratefully acknowledged.

REFERENCES

(1) "BKW, Wasserkraftwerk Mühleberg, Bestimmung von Felskennziffern", Bericht Gt.1989, Geotechnisches Institut, Bern, 1980.
(2) "Design of Gravity Dams", U.S. Bureau of Reclamation, Denver, Colorado, 1976.
(3) "Erdbebenrisiko am Standort KKM", ASK, Basler & Hofmann, Zürich, AN 777-11, 2.11.1978.
(4) "Arbeitsgebiet Dynamik", Emch + Berger Bern AG, Bern, 1980.
(5) Christiano, et.al., "Compliances of Layered Elastic Systems", Proc. of Inst. of Civil Engineers, London, Dec. 1974.
(6) Richart, et.al., "Vibrations of Soils and Foundations", Prentice-Hall Inc., 1970.
(7) Stucky, J.-P., "Interprétation des résultats", Rapport no, 5'735, Lausanne, Juin 1981.

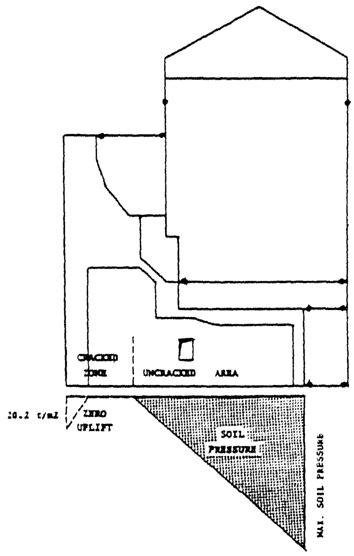

Fig. 12 Transformer Building, foundation base pressures during earthquake