

# Requalification for Beznau (Switzerland) NPP Auxiliary Building

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## Abstract

The Beznau Nuclear Power Plant Auxiliary Building A has been designed in a time period, when less severe earthquake design criteria were in use. Furthermore several changes in the structural system have been introduced since its completion. This led the owner of Beznau NPP to carry out a requalification study of the Auxiliary Building A. A linear dynamic analysis with a detailed 3-D-FE-model has been carried out. The forces have been calculated using the Response Spectra Method. To assess the safety of the structure the failure modes and the ductility of the different structural elements have been studied. The requalification analysis has shown, that it is possible to achieve the required earthquake resistance of an existing building with minor structural strengthening.

## Introduction

The Beznau Nuclear Power Plant Auxiliary Building A is a two storey reinforced concrete structure of 30 m length and 15 m width having (on top of the second floor) a two storey penthouse to accommodate the ventilation system. The structural system is, as shown in Fig. 1, made up of the foundation slab, the floor diaphragms, the columns and the shear walls. The ground floor of the building is considerably less stiff than the first floor. Furthermore heavy installations in the first floor add to the mass of the first floor slab (elevation 332.46). The building has been designed in a time period, when less severe earthquake design criteria were in use. Furthermore several changes in the structural system have been made since its completion. This led the owner of the Beznau NPP to carry out a requalification study of the Auxiliary Building A. The Objective of this paper is to describe the analysis methods adopted and to show what kind of strengthening is needed to fulfil the earthquake safety requirements.

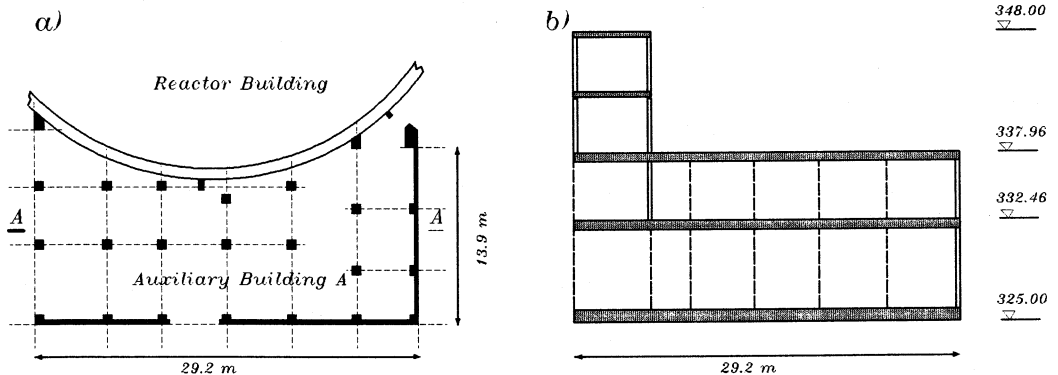


Fig. 1 Auxiliary Building A; a) Plan view of ground floor; b) Section A-A

### Analysis Method

For the requalification of the NPP Auxiliary Building a linear elastic dynamic analysis has been carried out to determine the forces in the individual structural elements of the building. To calculate the resistance of the structural members the nonlinear behaviour has been taken into account. This procedure is appropriate, as the installations in the building do not allow large deformations

### Dynamic Analysis

For the dynamic analysis an elastic 3-D-FE-model has been created, using the FE-Program STARDYNE (1). The model consists of beam and plate elements, having a total of 500 degrees of freedom (see Fig. 2). The member forces under earthquake excitation have been determined with the response spectra method, taking into account the first ten modes. The superposition of the individual modes as well as the three excitation directions has been performed with the RSS method. The influence of the soil has been taken into account by introducing a spring-dashpot-element attached to the centre of the rigid foundation slab. The stiffness and damping characteristics have been computed using the theory of layered half-space (Christiano, 1974).

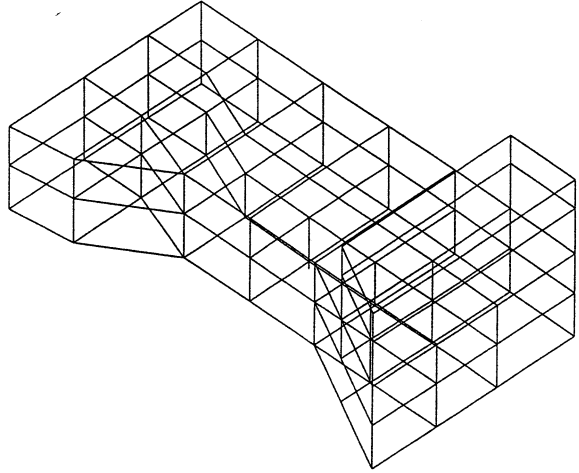


Fig. 2 FE-model for Auxiliary Building A

The behaviour of the structure has been analysed for two levels of earthquake excitation: the operating base earthquake (OBE) with a peak horizontal acceleration of 0.1 g and the Safe Shutdown Earthquake (SSE) with 0.21 g. The US-NRC-Spectra have been used.

### Dynamic Behaviour

The dynamic behaviour of the Auxiliary Building A is primarily influenced by the asymmetric configuration of the shear walls in the ground floor, as shown in Fig. 1. This leads, as shown in Fig. 3, to a rotational motion of the building in the first mode, with the centre of motion near the intersection of the two perpendicular shear walls. This in turn produces large moment and shear forces in the outer columns (left side of the building in Fig. 1).

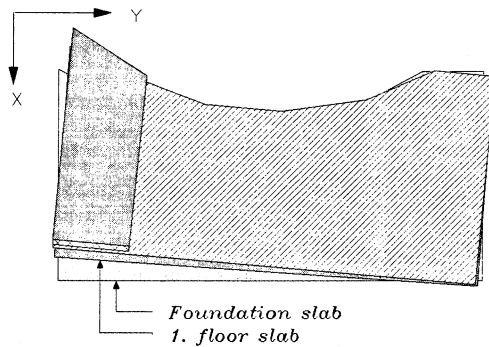


Fig. 3 Displacement of floor and diaphragms in 1. mode.

### Earthquake Resistance

The NOK safety requirements for the NPP-Buildings prescribe that under the SSE a safety factor of 1.2 for ductile failure mode and of 1.4 for brittle failure mode has to be achieved. In the Auxiliary Building A the critical elements are the outer columns in the ground floor, a brick wall in the 1. floor and some of the shear walls.

The *outer columns* in the ground floor have to resist, as previously mentioned, large shear and bending forces due to the rotational motion in the first mode. Fig. 4 shows the shear forces and the shear resistance of the columns in the ground floor. The critical failure mode of the columns is the shear failure, a brittle failure mode, for which a safety factor of 1.4 is required. No advantage can be taken therefore of the ductility of the columns, although they have a displacement ductility of 2 to 3.

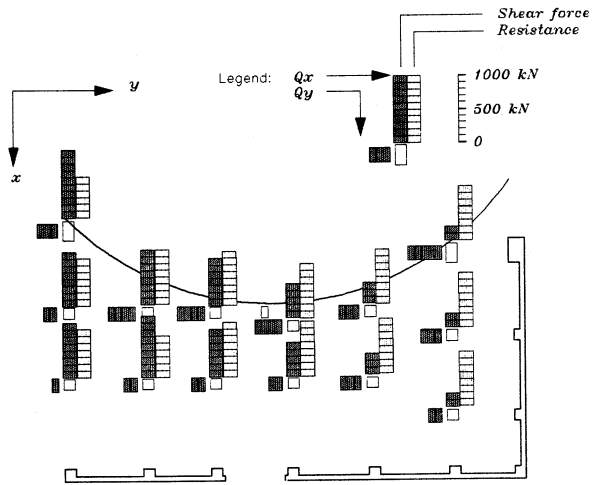


Fig. 4 Shear forces and shear resistance of the columns in the ground floor.

The *brick wall* in the 1. floor is of the infill panel type, with a reinforced concrete frame. Normally these infill walls act as compression diagonals and can transmit large shear forces. However, as the brick wall in the Auxiliary Building A contains a large opening, the shear resistance is considerably reduced. Therefore the analysis has also been carried out for the case where the brick wall has completely failed. It could be shown that the forces are transferred to other structural elements and that sufficient shear resistance is available.

The *shear walls* in the ground floor are of the framed shear wall type, as shown in Fig. 5; the foundation slab and the 1. floor diaphragm form the horizontal members of the frame, the integrated columns the vertical members. Framed shear walls exhibit basically seven distinct failure modes:

1. Flexural reinforcement yielding
2. Shear friction (large crack along the base of panel)
3. Diagonal shear cracking
4. Panel reinforcement yielding
5. Frame reinforcement yielding
6. Panel crushing (compression diagonals)
7. Frame shear failure

Some of these failure modes can provoke collapse individually, i.e. the occurrence of only one failure mode is sufficient for complete failure, some failure modes provoke complete failure only in combination with other failure modes. The above mentioned seven failure modes can be represented in a failure tree as a system of serial and parallel links, as shown in Fig. 6.

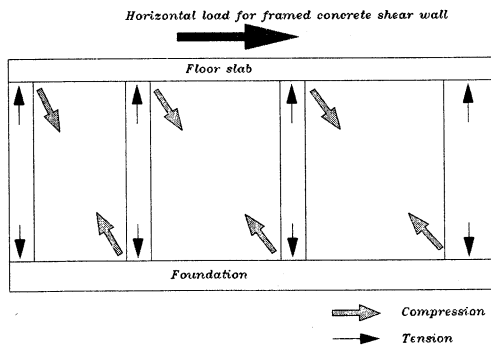


Fig. 5 Framed concrete shear wall

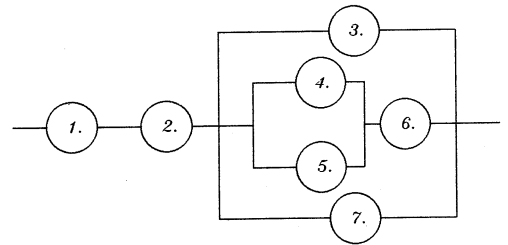


Fig. 6 Failure diagram for framed shear wall

Failure modes 1 and 2 can lead to collapse individually, the other five failure modes only in combination. For the squat shear walls of the Auxiliary Building A the most probable failure modes are mode 3 (link 3 in Fig. 6) and subsequently the failure mode 4, 5 and 7 or 6 and 7. Failure mode 3, i.e. small diagonal cracks occur at comparatively low load levels or even due to shrinkage. For sufficiently reinforced panels they do not pose any problems. In failure mode 4 and 5 a truss system with vertical tension elements and diagonal compression elements is formed. If the reinforcement of the frame and the panels is high enough diagonal shear cracking occurs, i.e. failure mode 6 takes place.

For the analysis of the framed shear walls of the Auxiliary Building A the failure modes 4, 5 and 6 are relevant, i.e. a truss model, where the vertical frame and panel reinforcement represent the tension elements and the concrete diagonals represent the compression elements. For all shear walls the safety factor exceeds the required value of 1.4.

### Structural strengthening

The analysis of the Auxiliary Building A has shown, that structural strengthening of the outer columns in the ground floor is required. The strengthening is best achieved by introducing two new infill walls as shown in Fig. 7. Special attention has to be paid to the connection of the new infill walls to the existing structure. The reinforcement of the infill wall has to be anchored in the existing structure by means of a two compound adhesive, which assures that the yield strength of the reinforcement is reached before the reinforcement is pulled out.

With the new infill walls an approximately symmetric distribution of the shear resisting elements is achieved. The horizontal loads on the columns are reduced by as much as 90%. The first eigenfrequency is raised from 3.5 to 5 Hz.

This analysis has shown, that it is possible to achieve the required resistance of an existing building even with minor structural strengthening.

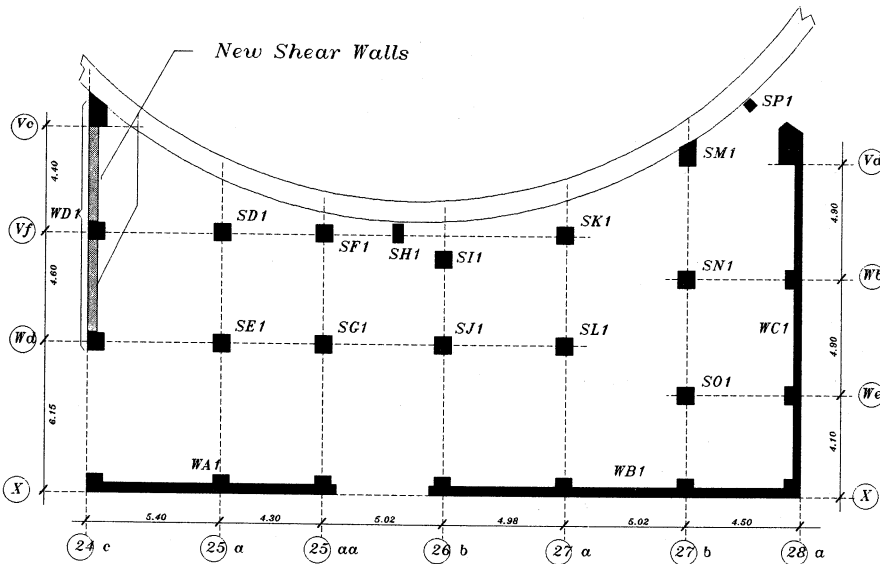


Fig. 7 Structural strengthening with two infill walls

### References

- Christiano, Rizzo, Jarecki (1974). Compliances of layered elastic systems. Proc. Instn. Civ. Engrs. STARDYNE: Structural Analysis Systems. General Microelectronics Corporation; San Diego.