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Statistical analysis of the variation of floor vibrations in nuclear power plants subject to seismic loads

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HIGHLIGHTS

- Floor flexibility plays a non-negligible role in amplifying horizontal vibrations.
- COV of in-floor horizontal and vertical acceleration are 0.15–0.25 and 0.25–0.55.
- In-floor variation of vibrations is higher in lower floors.
- Floor spectra from limited nodes underestimates vibrations by a factor of 1.5–1.75.

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ABSTRACT

Floor vibration of a reactor building subjected to seismic loads was investigated, with the aim of quantifying the variability of vibrations on each floor. A detailed 3D building model founded on the bedrock was excited simultaneously in three directions by artificial accelerograms compatible with Finnish ground response spectra. Dynamic simulation for 21 s was carried out using explicit time integration. The extracted results of the simulation were acceleration in several floor locations, transformed to pseudo-acceleration (PSA) spectra in the next stage. At first, the monitored locations on the floors were estimated by engineering judgement in order to arrive at a feasible number of floor nodes for post processing of the data. It became apparent that engineering judgment was insufficient to depict the key locations with high floor vibrations, which resulted in un-conservative vibration estimates. For this reason, a more systematic approach was later considered, in which nodes of the floors were selected with a more refined grid of 2 m. With this method, in addition to the highest PSA peaks in all directions, the full vibration distribution in each floor can be determined. A statistical evaluation of the floor responses was also carried out in order to define floor accelerations and PSAs with high confidence of non-exceedance. The conclusion was that in-floor variability can be as high as 50–60% and models with sufficiently dense node grids should be used in order to achieve a realistic estimate of floor vibration under seismic action. The effects of the shape of the input spectra, damping, and flexibility of the floors on floor vibration of nuclear power plant buildings were also investigated.

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1. Introduction

When vibrations affect the buildings of a nuclear power plant (NPP), the focus shifts towards evaluation of the response of buildings. There are two practical points of interest. It must be determined whether the stability of the buildings may be affected, and the propagation of vibrations within the building should be understood in order to qualify the equipment (Fülöp, 2012). The overall aim of this investigation was to demonstrate that no structures, systems and components (SSC) are affected by earthquake-induced shaking. In Finland there are two particularities related to buildings in NPP installations. Firstly, they are founded on hard rock sites, with shear wave velocity $v_s > 3000$ m/s, so that the effect of soil structure interaction (SSI) is eliminated. Secondly, the level of acceleration in terms of peak-ground acceleration (PGA) and the spectral acceleration ordinate corresponding

Abbreviations: ABBSUM, sum of absolute peak values; CDF, cumulative distribution function; COV, coefficient of variation; CQC, complete quadratic combination; EMPF, effective mass participation factor; MDOF, multi-degree of freedom system; PGA, peak-ground acceleration (same as ZPA); PSA, pseudo-acceleration; SDOF, single degree of freedom system; SSC, structure systems and components; SSI, soil structure interaction; SRSS, square-root of sums of squares; ZPA, zero point acceleration.

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to natural frequencies of NPP buildings is very low. Therefore, extensive damage to nuclear buildings is very unlikely. Hence, our studies focused on vibration propagation within buildings and on the uncertainties related to the vibrations affecting the building floors.

In the early days of modelling, when computational resources were limited, numerical studies needed to have a sharper focus on computational efficiency. In this context, dynamic models for nuclear buildings, with the purpose of obtaining in-structure response spectra, were carried out with reduced model sizes by condensation methods or component mode synthesis. For example, Tsai et al. (1974) used the modal superposition method of direct integration in order to analyze the seismic responses of nuclear power plants. In Japan, a lumped mass model with sway and rocking soil springs was used in the analysis and design of nuclear power plants subjected to earthquakes (Nakamura et al., 2008). However, as indicated in the regulatory guide of the US Nuclear Regulatory Commission (1978), mathematical models or simplified models that can be used to generate the earthquake responses of the main structural system of a nuclear power plant may not be sufficient for the detailed and localized analyses of the secondary system and the equipment.

With increasing computational capacity readily available, it later become feasible to carry out the modelling with full 3D models. Studies have shown that results with reduced models tend to overestimate the response (Varpasuo, 1999; Nakamura et al., 2010). However, the floor vibrations may vary significantly within the same floor and this variability needs to be understood when analyzing output from a 3D model. The building typology probably also plays a significant role. Major differences exist between the dynamic properties of a building typology influenced by torsional modes (Baušys et al., 2008) and a compact typology such as that analyzed by Varpasuo (1999). Using a 3D model, Nakamura et al. (2008) investigated the effect of including both horizontal and vertical ground motions in horizontal structural responses of nuclear power plants. It was found that the influence of the vertical ground motion on the horizontal structural response was insignificant.

Viti et al. (1981) discussed the reduction in structural response of floors under seismic excitation using non-linear response spectra in comparison with the linear spectra. Sackman and Kelly (1980) developed an analytical method to estimate the maximum acceleration and displacement of the equipment attached to a structure subjected to a ground motion. The method assumes that a design spectrum for the ground motion is available, and that the structure and equipment have fixed bases. Varpasuo (2013) discussed the analysis of structural response of a reactor building using three different programs for 3D analysis: MSC/PATRAN, MSC/NASTRAN, and Fortum Engineering's in-house program IVO-DIM. It was found that simplified methods obtained by decreasing the size of dynamic models overestimate the response significantly. Specifically, for the structure analyzed by Varpasuo (2013), the reduced models indicated much higher horizontal frequencies (7 Hz, 8 Hz) compared to the 4.5 Hz and 4.9 Hz of the detailed 3D model. Hence, the reduced models were overestimating the horizontal stiffness. On the other hand, the spectral response in the horizontal direction of the reduced models was found to be higher, sometimes by a factor of two. Chen et al. (1997) found that substantial differences in building responses of a nuclear power plant, in some cases 200% or more, resulted from the 3D modelling using the CLASSI and FLUSH Codes. This clearly shows the complexity of modelling and the necessity to thoroughly investigate the variation of floor vibration. Paskalov and Reese (2003) compared floor response spectra using a deterministic and a probabilistic method. The probabilistic method used an analysis of random vibrations and was less computationally demanding than the deterministic method, which used time history analyses. However, the variation of floor vibration due to earthquake excitation was not investigated.

Determining floor spectra based on modal analysis has been proposed by Peters et al. (1977), and is often used in industrial applications. The two drawbacks of the method are its limitation to the elastic part of the response of the structure and the computational cost of extracting a sufficient number of higher modes from a complex 3D model. The problem of floor vibration is not unique to nuclear and industrial applications. In ordinary buildings, attempts to clarify the effects of different factors on the floor spectra were made by Sewell et al. (1986), who concluded that the number of dynamic degrees of freedoms, localization of non-linearities, strength distribution and input ground motion are important parameters. Attention to the design of secondary structural components is considered to be important for traditional buildings (Villaverde, 1997). However, it should be borne in mind that ordinary buildings differ from nuclear applications, because they are more affected by non-linearity in response to earthquakes.

In this work we evaluated the floor vibrations of a reactor building designed for an advanced boiling water reactor (ABWR), with a compact configuration. The building is not prone to torsional effects, its perimeter walls being very stiff. The intention was to keep the results generic for this building configuration. It should be noted that this paper does not include an assessment of the consequences of the structural integrity analysis of the NPP and related damages, which are beyond the scope of this work.

2. Backgrounds

2.1. Estimating floor response with spectra-to-spectra methods

The background of developing floor spectra, using frequency domain calculations, is based on the interaction of a primary multi-degree of freedom (MDOF) system and an attached secondary single-degree of freedom (SDOF) oscillator. It is generally assumed that the secondary oscillator has a negligible effect on the dynamic properties of the primary one, and so the addition of the SDOF oscillator only results in a new vibration mode of the system. But the already established vibration modes of the MDOF system remain unaffected. The primary MDOF oscillator has dynamic properties ω_i, φ_i, Γ_i and ξ_i, the set if $i = 1 \dots N$ circular frequencies, mode shape vectors, modal participation factors and damping ratios, respectively. Once the SDOF is attached the additional terms appear in the eigenvalue solution characterized by the new terms in all vectors describing the dynamics of the structure ω_{SDOF}, φ_{SDOF}, Γ_{SDOF}. The combined MDOF + SDOF system has N + 1 dynamic degrees of freedom, the N + 1th degree representing the additional SDOF oscillator.

The maximum value of acceleration on the N + 1th degree of freedom, corresponding to the mass of the added SDOF oscillator, can be estimated using spectral analysis. If the load on the MDOF is a spectrum "S", each vibration mode $i = 1 \dots N$ adds load according to its corresponding spectral ordinate $S(\omega_i, \xi_i)$. Hence for each mode $i = 1 \dots N$, one can calculate the modal contribution to the maximum acceleration $(a_{i,SDOF})_{max}$. The maximum acceleration on the SDOF degree of freedom can be estimated from the modal components as:

$$a_{SDOF,max} = modal\_comb(a_{i,SDOF,max}) \tag{1}$$

It should be remembered that $a_{SDOF,max}$ is an estimate from peak responses; it is not an exact value. The generally used modal combination rules can be SRSSs, CQC or ABBSUM. The correctness of the $a_{SDOF,max}$ estimate also depends on the number of modes used in the modal combination. Very often it is not possible, or computationally too expensive, to calculate all vibration modes of the

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