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Dynamic In-Situ Assessment for Seismic Analysis

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Abstract

Seismic assessment of important existing structures that are designed according to prior seismic codes or even without considering any earthquake load is one of the most important issues for minimizing seismic vulnerability. Within this paper the seismic assessment of an existing building for radioactive waste management is presented. By means of dynamic measurements followed by finite element analysis seismic vulnerability assessment was carried out. The application uses a mobile vibration generator for forced vibration testing and offers the possibility of measuring and assessing the structure as well the soil conditions with accurate boundary conditions. By means of forced vibration tests an adequate frequency range and vibration level could be induced in the structure and surrounding soil surface. The advantages of forced excitation are the short test duration, the good controllability and the favourable signal to noise ratio. The benefits of using field testing and an overview about new developments will be given.

Keywords: field testing, seismic analysis, vulnerability assessment.

1 Introduction

Field experiments will significantly enhance the fundamental knowledge of earthquake effects associated with the behaviour of structures and geological layers, thereby reducing loss-of-life and economic losses from future earthquakes. In-situ investigations are in general necessary not only for earthquake assessment, but also for health monitoring and evaluation of the existing condition of the structure. In principle it is possible to measure the following physical parameters [9]:

- Strains
- Displacements, velocities, accelerations (transverse and rotational components, even if measurement of rotational components is less easy)

From measured time histories the dynamic properties (modal parameters) can be obtained. Dynamic field tests or in-situ testing methods are necessary for the evaluation of the dynamic behaviour (modal parameters) and the actual condition of existing structures. In-situ measurements are recommended in order to verify the numerical models and to increase the reliability of the numerical approaches. A first FE-model of the tested structure, which is elaborated on the basis of the design documents, can be fitted to measured results by an optimisation approach. This procedure is called "model updating". Dynamic in-situ measurements are also recommended for the verification of the effects of structural changes and strengthening applied to buildings. By means of experimental modal analysis the identification of the modal parameters (natural frequencies, mode shapes, and damping ratios) of a structure from input and/or output measurements is possible. For this purpose, the mode shapes of a structure must be excited measurably. In the case of input/output measurements also the frequency dependent impedance (force vibration velocity ratio) is obtained. The modal parameters are used to perform ongoing earthquake analysis or assessment. Since the updated model is based on measured results, it represents in a realistic manner the behaviour of the structure during the starting phase of a seismic event. Using this model for an earthquake analysis it is possible to forecast and therefore to point out the weak spots in structural members, if any.

To measure the vibration response of a structure, a sufficient excitation should be available. This excitation is basically possible in two different ways, on the one hand by forced excitation and on the other by ambient excitation. This paper discusses the pro and cons from forced and ambient field testing methods in context of seismic vulnerability assessment.

1.2 Industrial Structures

Industrial buildings usually differ from office buildings or residential buildings because of the following characteristics [9]:

a) Increased safety requirements and seismic design actions in particular where industrial facilities processing toxic, nuclear and explosive materials may cause hazards to the population and environment. Such structures with increased risk for the population are beyond the scope of the current design codes.
b) Properties of industrial structures related to the introduction of seismic actions from equipments and installations and to the type of framing the layout of which should be variable enough to allow fast adaptations to modern process techniques.
c) Need of particular design rules for ultimate limit state e.g. for anchoring of equipments and structures, protection of equipments from consequences of failures of secondary structures and also for avoiding leakages from excessive deformations.

Due to the fact, that the non-structural units and fluids in tanks interact with the behaviour of the structure (e.g. where the stiffness or eccentricity significantly

contributes to the structural response of the construction), the interaction of the several machineries and fluid structure interaction have to be considered. Major difficulty of the analysis of industrial structures arise due to the fact, that changes and modifications in service loads and structural parts have been done since the erection of the structure. Therefore discrepancies between the current and the initially intended use or structure of the building are present. As a result of e.g. additional equipment, the overall masses have been changed and the dynamic response of the structure differs from the original one. For the investigated structure changes have not been documented sufficiently within, so that the original sketches are not according to the current situations furthermore the accessibility of structural parts is restricted because of additional equipment. Past design codes used during design and erection of the construction were needed to figure out the material properties of the structural components. Depending on the quality and amount of existing sketches and drawings in several situations it was impossible to figure out all structural details, like bond length of reinforcements and further hidden details. An approved method for analysing the actual situation was carried out by dynamic measurements.

2 Dynamic in-situ investigations

2.1 Approach to determine dynamic soil parameters

The dynamic soil parameters were used for modelling the soil structure interaction by estimation the initial spring stiffness in the foundation level. The stiffness of the springs was further specified as updating parameter in the modal updating approach. From the measured soil velocities v_s and v_p the dynamic shear modulus can be calculated by equation (1) and the young's modulus by equation (2).

$$G = \rho \cdot v_S^2 \approx \rho \cdot 1{,}08^2 \cdot v_R^2 \tag{1}$$

$$E = v_p^2 \cdot \rho \cdot \frac{(1+\nu)*(1-2\nu)}{1-\nu} \tag{2}$$

By using the modal response spectra method the influence of local ground conditions of the seismic action is accounted with the soil factor *S*. According to EN 1998-1, the soil factor is classified in five different ground types and can be defined by the average shear wave velocity $v_{s,30}$ what should be calculated by equation (3).

$$v_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{v_i}} \tag{3}$$

where h_i is the thickness of layer *i* of *N* layers within the top 30m.

2.1.1 Field testing application

When the ground surface gets excited next to the body waves also surface waves occur. The velocities of compression-, shear- and Rayleigh waves and further elastic- and shear module were evaluated by analysing experimental data. Rayleigh waves are dispersive, which means their phase velocity is a function of wavelength λ. This phenomenon occurs because Rayleigh waves travel only along the surface and their phase velocity is a function of P-wave velocity, S-wave velocity, density and layer thickness. Besides these four properties, phase-velocity is also a function of frequency because high-frequency Rayleigh waves i.e. short wavelengths, only skim the surface and are affected by shallow soil properties, while those waves with greater wavelengths (low frequency) reach deeper into the ground, where they are influenced by soil properties at greater depths [1]. The depth of the Rayleigh wave can be controlled by the frequency of harmonic excitation. About the phase relations of the surface wave at two different points, the corresponding horizontal phase velocity or approximated shear wave velocity of a given frequency can be determined. The dynamic excitation was performed with the reaction mass exciter VICTORIA in terms of harmonic sine sweeps within a specified frequency range. The first inversion technique was based on representative depths: Rayleigh waves of wavelength λ were assumed to travel at a velocity found at depths between $\lambda/2$ and $\lambda/3$ [2], [3]. Given this relationship, a velocity profile range was calculated from an average dispersion curve. The range of shear wave velocity profiles is shown in Figure 1.

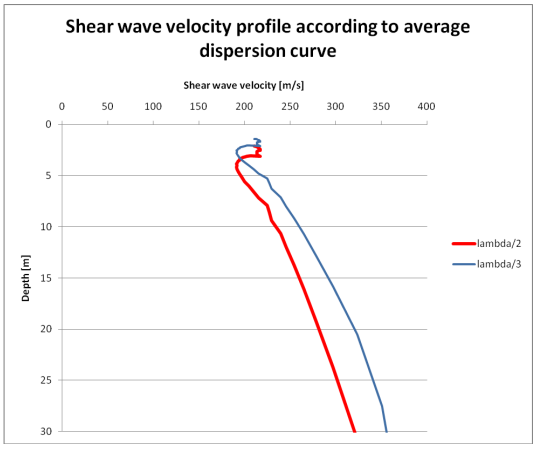


Figure 1: Shear wave velocity profile when using equivalent depth inversion of λ/3 and λ/2 for VICTORIA measurements

For applying the phase velocity testing method sine sweeps in range of 5-60 Hz were used. Phase differences to the reference sensor can be evaluated and Rayleigh wave velocity can be determined for the range of excitation frequencies. The second inversion technique was a series of tests using the software dinver, where an initially simple ground model was gradually increased in complexity to find optimum convergence towards the input dispersion curve [4]. The best model to explain the

measured dispersion curve (minimum misfit of 0.0396) can be seen in Figure 2, showing both P- and S-wave velocity profiles for a ground model consisting of 10 layers over halfspace, assuming increasing velocity with depth. The shear wave velocities correlate well with the ones obtained through equivalent depth inversion.

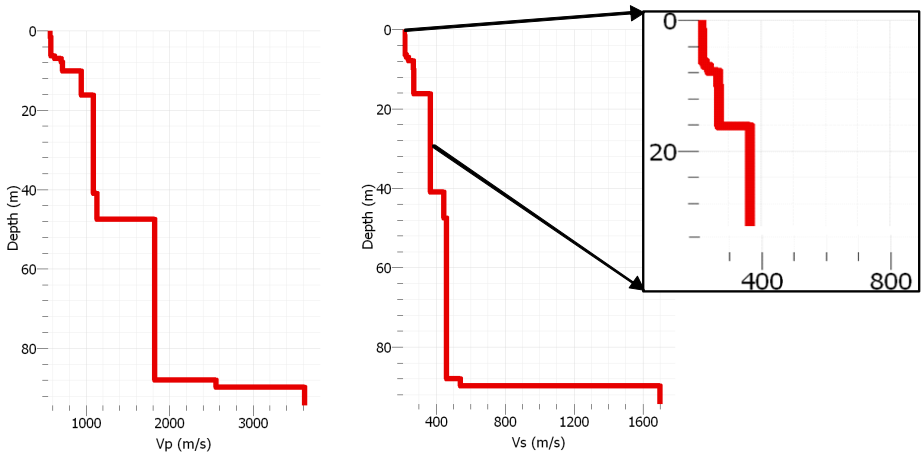


Figure 2: Velocity profiles v_p (left) and v_s (right) from the inversion of the VICTORIA dispersion curve using dinver;
inset shows zoomed area of Vs in the top 30 m

The VICTORIA experiment worked very well and managed to produce velocity profiles at over 30 m depth. The computed average shear wave velocity for the uppermost 30 m of soil was almost the same for the equivalent depth inversions and the dinver calculations, 257m/s for λ/2, 276 m/s for λ/3 and 287m/s, respectively. All three values lie within ground category C according to EC8 with the associated soil factor $S=1.15$ for the response spectra analysis.

2.2 Structural measurements by forced vibration tests

In-situ measurements of significant dynamic properties of structures are recommended in order to verify the numerical models and to increase the reliability of the numerical approaches. During forced vibration tests the modal structural parameters are determined via artificial excitation. The reactions to this known excitation (input) are recorded by sensors (output). From the measurements of structural response and excitation - force transfer functions between each excitation point and all measurement points (FRF = Frequency Response Function) can be calculated. The forced vibration testing techniques have the advantage, that they allow a specific investigation of the modal properties of structures. This includes the selection of the desired frequency range, size and direction of the exciting force amplitude and the possibility to examine non-linearity influences (by correspondingly high dynamic stress). In this type of excitation a measuring period of any length is possible under regular conditions. In particular a greater frequency

resolution can be achieved, which is important for the examination of small changes of natural frequencies due to structural damages. The advantages of forced excitation are the short test duration, the good controllability and the favourable signal to noise ratio.

2.2.1 Field testing application

The experiments for the investigated structure comprised ambient as well forced vibration tests. The planning of the experiment was done in three steps:

a) Preliminary ambient vibration measurement with one triaxial sensor
b) Calculation of expected mode shapes using preliminary Finite-Element model
c) Design of excitation and measurement point scheme.

The preliminary ambient vibration tests were carried out to get a first approximation of structural dynamic behaviour. The results showed very low ambient vibration levels due to the location of the building is a rural area with very few ambient vibration sources. The reasons for choosing forced over ambient vibration excitation were:

- High excitation levels, especially in higher frequencies
- Low amplitude of ambient vibration at this specific site
- High excitation control

Mode shapes calculated with preliminary Finite-Element model allowed the planning of a suitable layout of the measurement point gird and ensured a sufficient resolution of the expected major mode shapes due to in-situ testing. The reaction mass exciter VICTORIA was used as a vibration source. To determine the vibration characteristics highly sensitive accelerometers were arranged at 34 points at four levels. The choice of the points was aligned to determine all representative natural frequencies and mode shapes.

Figure 3 left: position of the exciter mass; right: connection with the building

The excitation was carried out over an axial road chain between the structure and the exciter mass (Figure 3). The excitation angle of about 45° ensures to excite as many natural modes as possible.

3 Numerical analysis

3.1 Analysed structure

The analysed structure is situated in Austrian highest seismic region and was designed to prior standards but without adequate consideration of seismic impact. Due to several reconstructions at the building and new assessment of the seismic hazard a detailed analysis was carried out. By consideration of the soil amplification factor a design ground acceleration of $a_g=1.8m/s^2$ results [5].
The building is designed as a water treatment plant with six collecting tanks supported by eight RC columns. Each tank has a capacity of 80m³, with a restriction of maximum filling of five tanks at the same time it leads to additional mass of 400 to (see Figure 4). The tanks are constructed as upside down pyramids with a cubical base. The total height of the building is 7.2m, the centre of the mass is situated in a height of about 4m. The structure is surrounded by two adjacent buildings with a height of 14.2m and the building in the background with a height of 3.3m (see Figure 4). Due to different foundation and mass distribution the buildings are connected only with elastic building joints.

3.2 Finite Element Model

For analysis, a three dimensional model of the building was created in ANSYS (Figure 4). This model includes only a few simplifications and thus provides very accurate results and information on the utilization of individual structural elements. For considering the real mass distribution the lightweight roof construction was modelled in a simplified manner as equivalent mass at the last slab. It was assumed that it has minor impact on dynamic and static properties of the whole building.

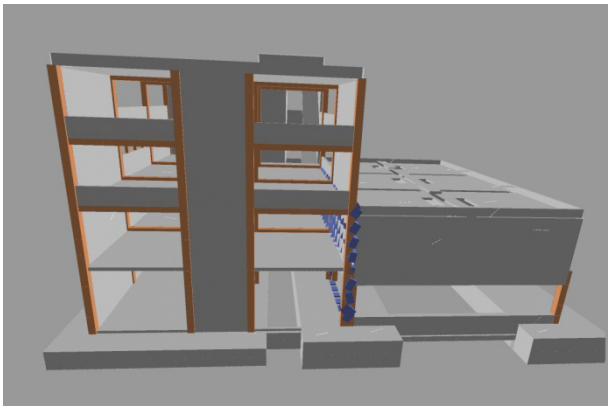


Figure 4: Model of entire building complex

Next to the structural system i.e. bearing walls, ceilings, beams and columns, also fluid elements (Fluid80) were implemented to model the realistic behaviour of water response in the tanks under dynamic excitation. The fluid elements are defined by

eight nodes having three degrees of freedom at each node: translation in the nodal x, y, and z directions [8]. The element input data includes the isotropic material properties for the fluid. Ex, which is interpreted as the "fluid elastic modulus", should be the bulk modulus of the fluid (Ex=2.1E09 Pascal for water). The viscosity property is used to compute a damping matrix for dynamic analyses. A typical viscosity value for water can be assumed with 1E-03 Pascal-sec [7]. The connection between fluid and RC walls was conducted out with respect to the realistic behaviour of degrees of freedom for fluid elements. At the boundary area the fluid elements have been coupled in perpendicular direction with the adjacent concrete surface elements. All other degrees of freedom are free and water can move in a realistic way under modal analysis and transient analysis, see Figure 5).

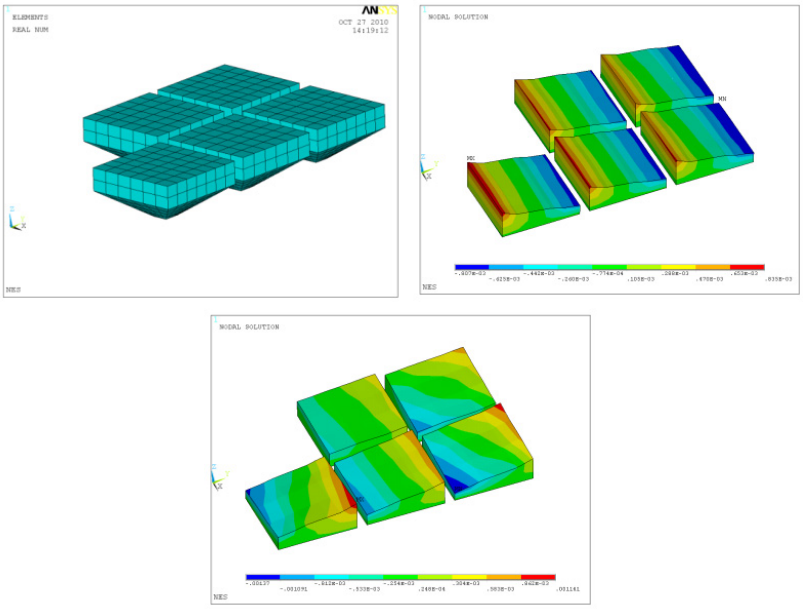


Figure 5: Fluid Elements and movement of water – mode shapes

3.2 Model updating

Natural frequencies and modes shapes of the building were calculated and compared with the obtained parameters from dynamic measurement. The amount of water in the tanks during FE calculation was set to be the same as it actually was during the measurement. The calibration of the FE model was carried out with respect of the first three measured eigenmodes. In this case, the most important updating parameters were: E-modul of RC, soil structure interaction in terms of spring stiffness and E-modul of elastic connection between buildings. Realistic maximum and minimum values of these parameters were assumed, and through series of analysis, whereas they were randomly changing, the most suitable solution was found. In Table 1 the calculated and measured mode shapes of the building are given.

The natural frequencies, as well as the mode shapes of the FE model and measurement were in a good agreement (Table 2).

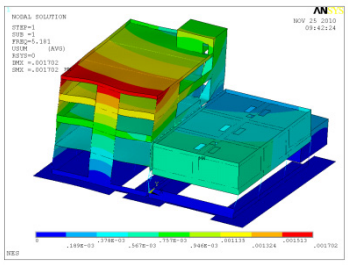
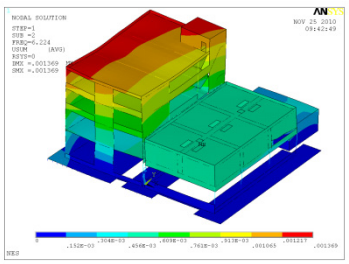
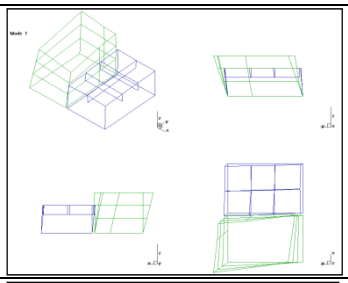
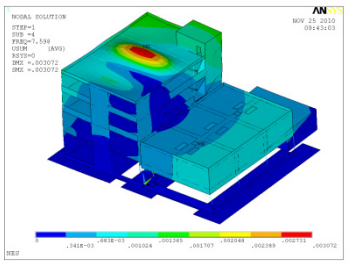
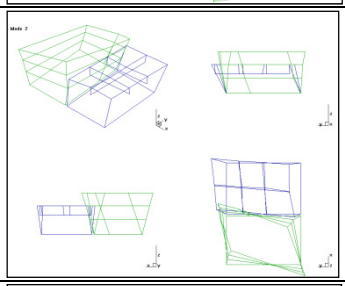
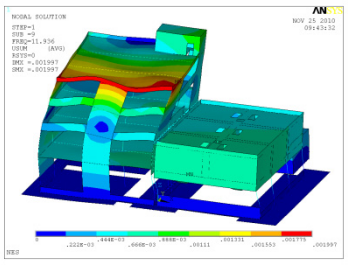
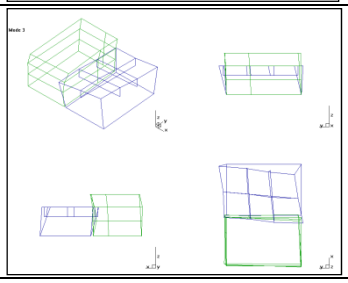
Numerically analysed by Finite Element Software	Experimentally evaluated by In - situ Measurements	Comment
	Not identified by experimental investigation.	Mode 1 - Transversal in x-axis; Numerically analysed Eigenfrequency: f_1=5.2Hz;
		Mode 2 – Transversal in y-axis with influence of torsion; Numerically analysed Eigenfrequency: f_2=6.2Hz; Experimentally evaluated Eigenfrequency: f_2=6.3Hz
		Mode 3 - Torsion Numerically analysed Eigenfrequency: f_3=7.6Hz; Experimentally evaluated Eigenfrequency: f_3=8.7Hz
		Mode 4 – Transversal in x-axis with influence of torsion Numerically analysed Eigenfrequency:f_4=11.9Hz; Experimentally evaluated Eigenfrequency: f_4=11.5Hz

Table 1: Measured and calculated mode shapes

Mode	Natural frequency [Hz]		Deviation
	Measurement	FE model	
1	x	5.2	x
2	6,3	6,2	1,6%
3	8,7	7,6	12,6%
4	11,5	11,9	3.50%
		Average	5,9%

Table 2: Comparison of computed and measured natural frequencies

4 Seismic assessment

First, a sensitivity study was carried out with the aim to identify the most unfavourable combination of filling levels of the tanks. The positioning of the fulfilled tanks was varied with respect to maximum loading for each construction element and considering global torsion effects due to unsymmetrical set ups. The identification of the worst load case situations were obtained by linear spectrum analysis. Next to spectrum analysis, also transient time- history analysis was used. It was necessary to improve the behaviour of fluid elements and to allow material nonlinearities. For the nonlinear dynamic analysis a set of different artificial time-histories [6], which were generated fitting to the EC8 response, were used. As mentioned before the building complex consists of three buildings, linked with a dilatation joint. Whereas the stiffness of this elastic material for small deformations was approximately identified by modal updating, it is difficult to assess its behaviour during the earthquake impact. Due to high deformations caused by seismic impact, connections to adjacent parts of the building may get lost during earthquake. The calculation was performed without connection to the adjacent buildings (Figure 6). Together with the high mass concentrated at the height of about 4 meters the building is extreme vulnerable to horizontal forces.

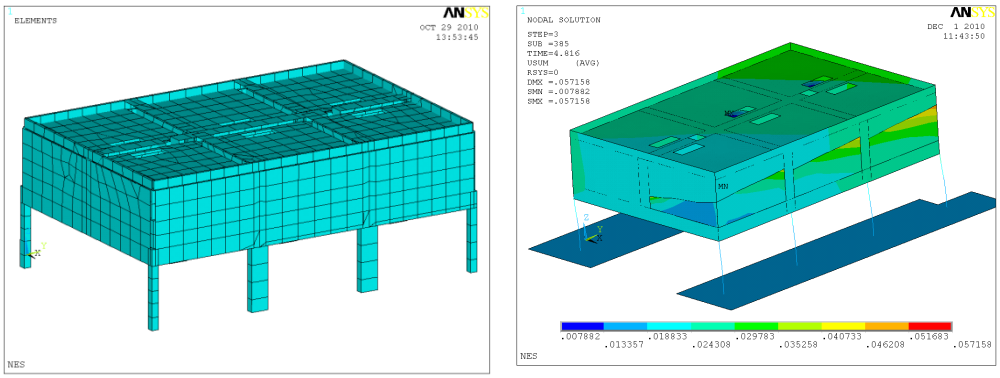


Figure 6: Water tank building

The results from all transient analysis were averaged and the safety factors according the requirements of Eurocode 2 were calculated. Comparing the response spectrum method to analysis results of transient time history method showed that the results match fairly well with a slight overestimation by using the response spectra method. The deformation of the structure during time history analysis is plotted in Figure 7. It is obviously that the most critical components of the structure in case of seismic impact are the eight columns in the ground floor. The structural design analyses were carried out according EC2 regarding possible shear and biaxial bending failure. An important issue in post processing of transient analysis is the synchronism of the forces. Not only maximums are significant, but also the worst combination of all forces in time. For biaxial bending in every time step following simplified criterion (EC2) was used:

$$\left(\frac{M_{Edz}}{M_{Rdz}}\right)^{a} + \left(\frac{M_{Edy}}{M_{Rdy}}\right)^{a} \leq 1 \tag{4}$$

where:
$M_{Edz/y}$ is the design moment around the respective axis
$M_{Rdz/y}$ is the moment resistance in the respective direction
a is the exponent, which depends on the axial force

The example of time- depending bending moments (red and pink) and axial force (blue), as well as results from equation (4) are shown in Figure 7.

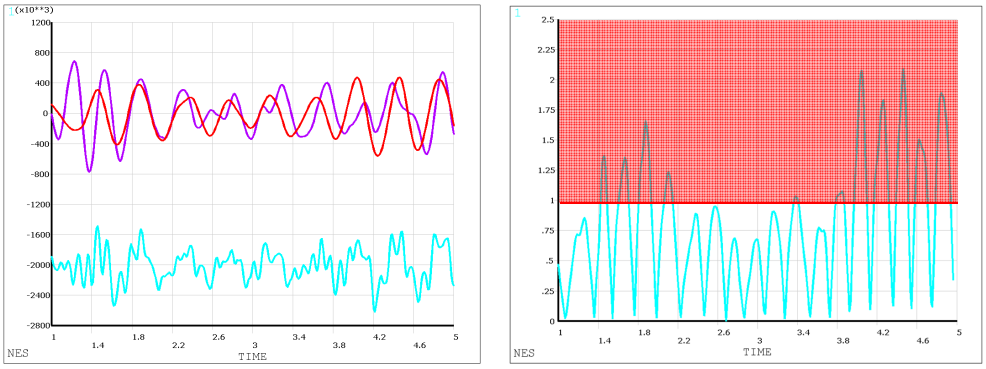


Figure 7: Left: bending moments and axial force; right: results of equation (4)

Due to simultaneous dependency of shear and axial force, the calculation was carried out for every time step, and the worst case was taken for verification of the ultimate limite state in case of seismic impact. The structural design of the analysed columns doesn't fulfil the requirements according the standards. Furthermore the internal requirements regarding maximum deformation will exceed what will lead to damage of technical facilities and cause a potential risk for the environment.

5 Structural strengthening

Possible retrofitting designs were limited by requirement of the operator to minimize construction works inside the building due to interference with internal operation. The proposed strengthening solutions should take into account minimizing modifications for existing technical assets and facilities. Aware of these requirements also unorthodox seismic strengthening solutions, like asymmetric strengthening and coupling of building through viscous damping devices were analysed. Due to asymmetric stiffening the dominant torsion mode increase the loading of the corner columns significantly. In fact of this, minor modifications of internal facilities were accepted by the operator and proper retrofitting solutions were investigated. In Figure 8 the selected strengthening solution is presented which suits the technical and economic needs. New walls to be built are marked in red. The

torsion, caused by asymmetry, is reduced by the use of two outside walls, but could not be avoided.

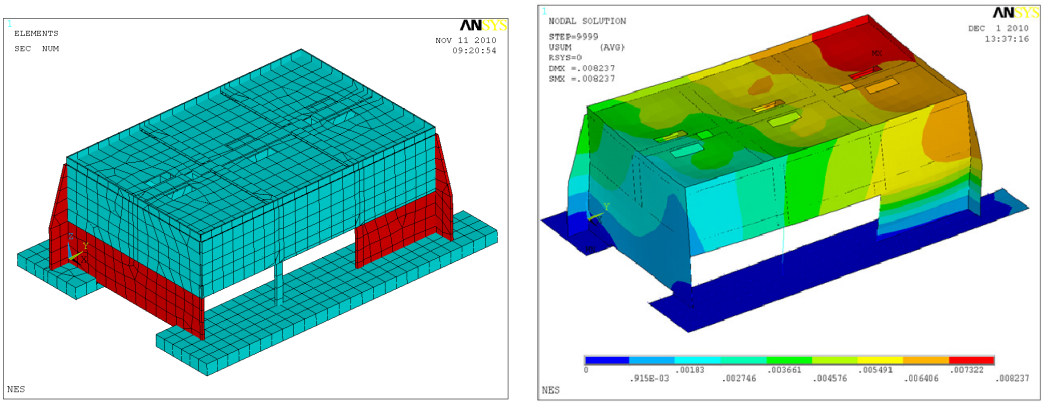


Figure 8: Selected retrofitting variant

6 Conclusion

By means of experimental modal analysis and finite element calculation a very realistic computer model for the entire building complex was created. Due to artificial excitation three mode shapes could be identified. The model adjustment was made on these three modes, whereby in addition to the Young's modulus of the materials, the boundary condition at the base and the elastic bearing joints to adjacent building parts were used as model updating parameters. The comparison of calculated mode shapes with the measured mode shapes showed a very good agreement. With the improved FE model, earthquake analysis under consideration the local conditions and historical seismic events were carried out. Moreover following issues can be highlighted:

i. Considering fluid structure interaction (FSI) allows an adequate simulation of the water behaviour under dynamic excitation and increases the reliability.
ii. By using FSI the local dynamic pressure arising from liquid sloshing along the tank walls will be considered in a proper way [4].
iii. The analysis results without FSI lead to an overestimation of the deformation and stresses of about 20% for the global system.
iv. Post- processing of the transient analysis provides information about simultaneity of bending, shear and axial forces.
v. From the finite element analysis many different strengthening possibilities could be investigated and optimised.
vi. Using a vibration generator for the Rayleigh wave method is favourable, as the transmission of lower frequencies can be achieved, permitting investigations at greater depths.

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