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EARTHQUAKE-RESISTANT-DESIGN OF HIGH-RISE-BUILDINGS USING DIFFERENT STRUCTURAL MODELS AND METHODS

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ABSTRACT

The earthquake effects on high-rise-buildings according to ENV 1998 and ÖNORM B4015-1, 2 were investigated. The computer codes ANSYS and DYNA were used. The results obtained by response spectra analysis and time history have been compared. For the time history analysis both linear and non-linear material behaviour has been considered. An appropriate simplified model for the non-linear time history analysis has been found and from a non-linear time history analysis the available ductility was obtained.

KEYWORDS

Ductility assessment; high-rise-building; calibrated model, non-linear behaviour

LINEAR BEHAVIOUR

Two different high-rise-buildings have been modelled. The first one was a 11-storey dwelling-house in Wiener Neustadt.

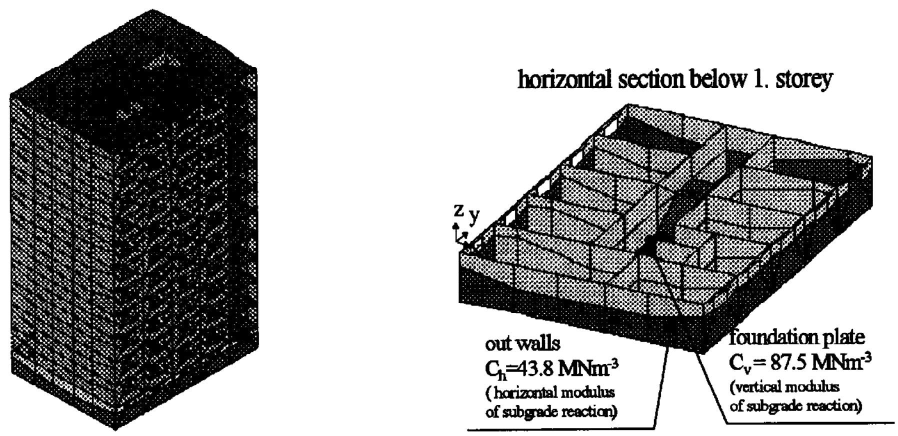


Fig. 1. FEM model of high-rise building in Wiener Neustadt

A detailed 3-D finite elements model was elaborated (Fig. 1). The monolithic concrete structure (15m x 20.7m, height 33.5m) is situated on gravel and sandy clay layers. The physical features of the soil are reported in (Slaštan, 1993) and were modelled by formulas for soil springs well known from literature.

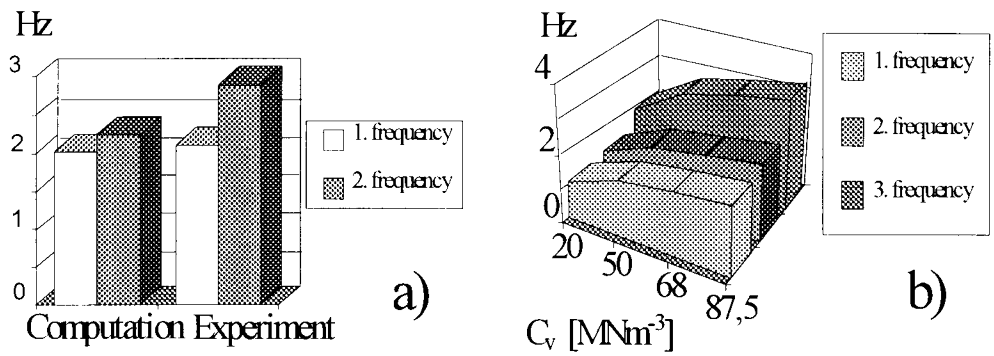


Fig. 2. Eigen frequencies (Wr. Neustadt building) a) Comparison experiment - computation; b) Influence of the modulus of subgrade reaction C_v on the eigen frequencies

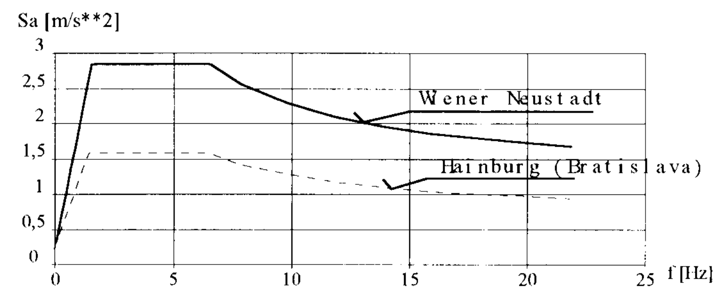


Fig. 3. Response spectrum for Wiener Neustadt and Hainburg (Bratislava) according to ÖNORM B4015-2

As eigenfrequencies have been obtained in-situ, the results are compared with the calculated values in Fig. 2a. It is shown in fig 2b that the modulus of subgrade reaction has a significant influence on the calculated frequencies, hence for this building type (stiff structure) an accurate estimate of soil parameters is very important.

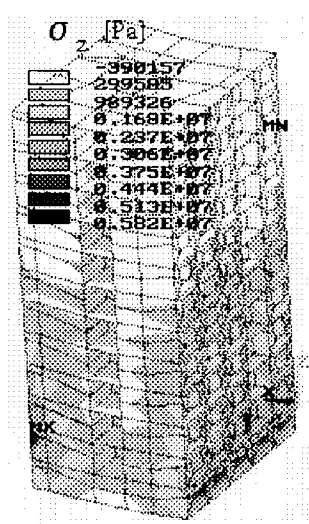


Fig. 4. Normal bending stresses σ_z (response spectrum analysis)

The code spectrum B and 1,13 m/s² maximum acceleration were used for the calculation (Fig. 3). The normal bending stresses are shown in Fig. 4.

There was the feeling, that a structure with less influence from soil would be preferable before going on with the study of the Wiener Neustadt high-rise building. Therefore a 24-storey faculty building of the University of Bratislava has been selected. A detailed 3-D modelling has been carried out (Fig. 5a). The soil was modelled by horizontal and vertical springs (Fig. 5b). Eigenfrequencies and modeshapes of the building have been measured earlier (Juhásová, 1985). Measured and calculated values are in good agreement. In this case the influence of the modulus of subgrade reaction on the eigenfrequencies is not significant (Fig. 5c). For the analysis, response spectrum B and 0,63 m/s² maximum acceleration have been used. The results for excitation in y-direction are are shown in Fig. 6a.

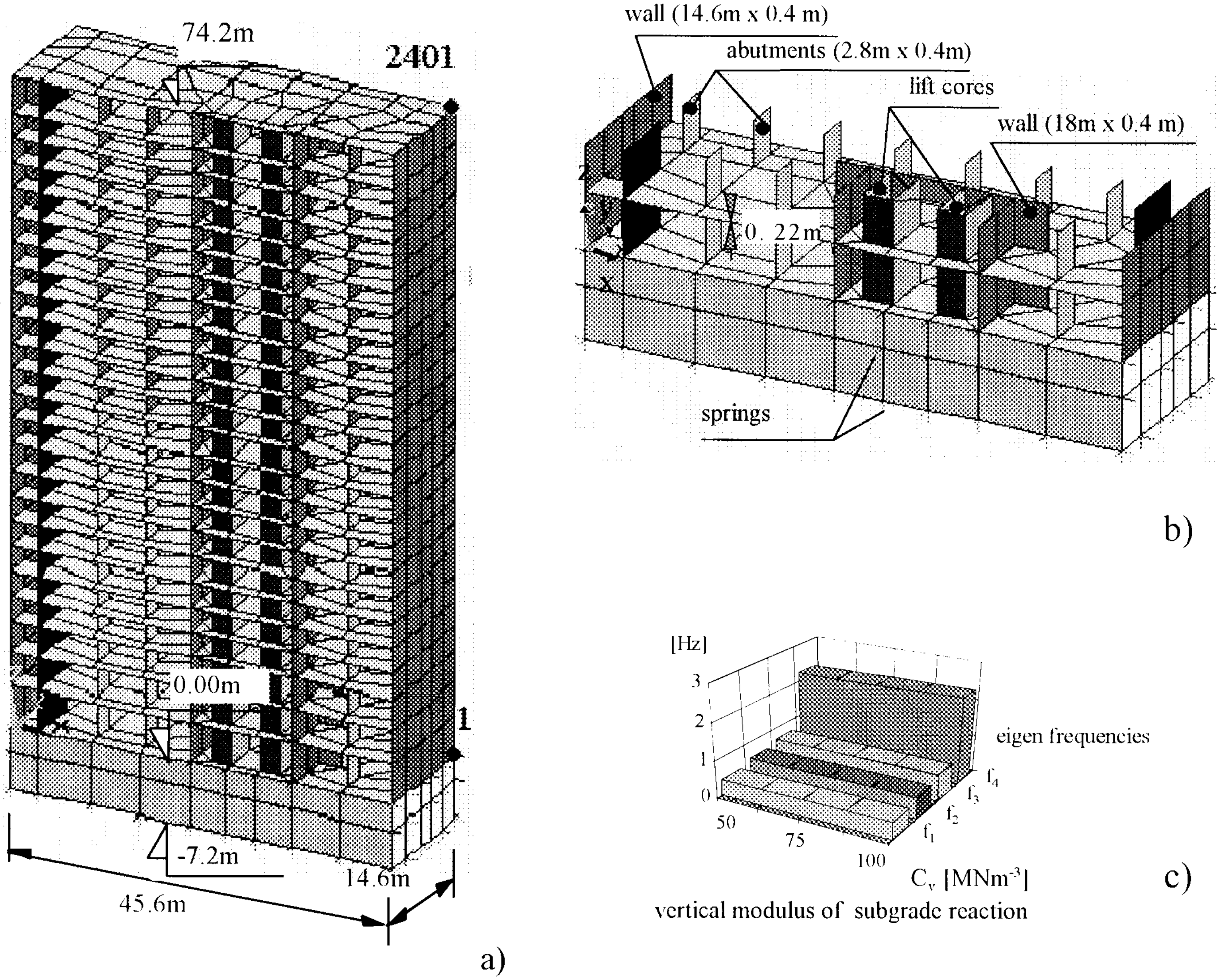


Fig. 5. High-rise building of TU Bratislava, 3-D model a) overall view; b) horizontal section; c) Influence of the modulus of subgrade reaction on the eigen frequencies

The next step was the generation of artificial ground acceleration time histories with 12 s duration. The time histories in Fig. 7a and b are fully compatible with response spectrum B. First, a linear time history analysis was carried out for excitation in y-direction. The 10980 DOF of the original model were reduced applying Guyan's reduction method to 72 MDOF (master DOF). Four nodes at each 4th floor level were defined as MDOFs (each having two horizontal and one vertical degree of freedom). The displacement response of point no. 2401 at the top floor is shown in Fig. 8 for the five acceleration time histories given in Fig. 7. For the time instant of the maximum displacement response to each of the five time histories (i.e. the time instant 1,86 s for acceleration SPEKTR_2, Fig. 8b), the model was transformed back to its original DOF's. The normal bending stresses of the whole structure are shown in Fig. 6b.

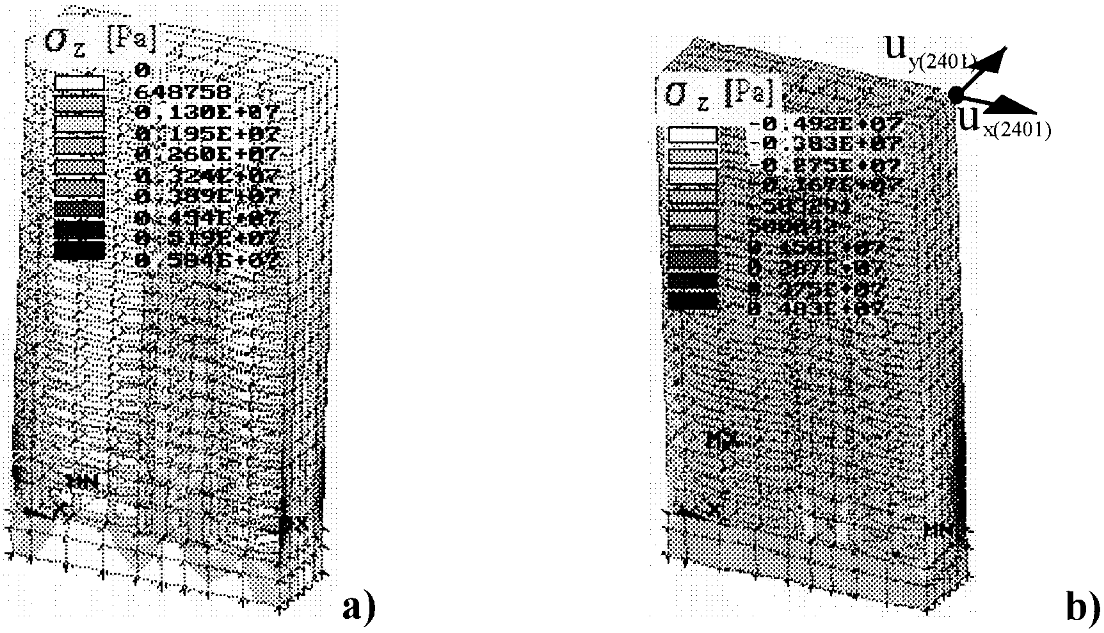


Fig. 6. Normal bending stresses σ_z a) response spectrum analysis b) time-history analysis (high-rise building TU-Bratislava)

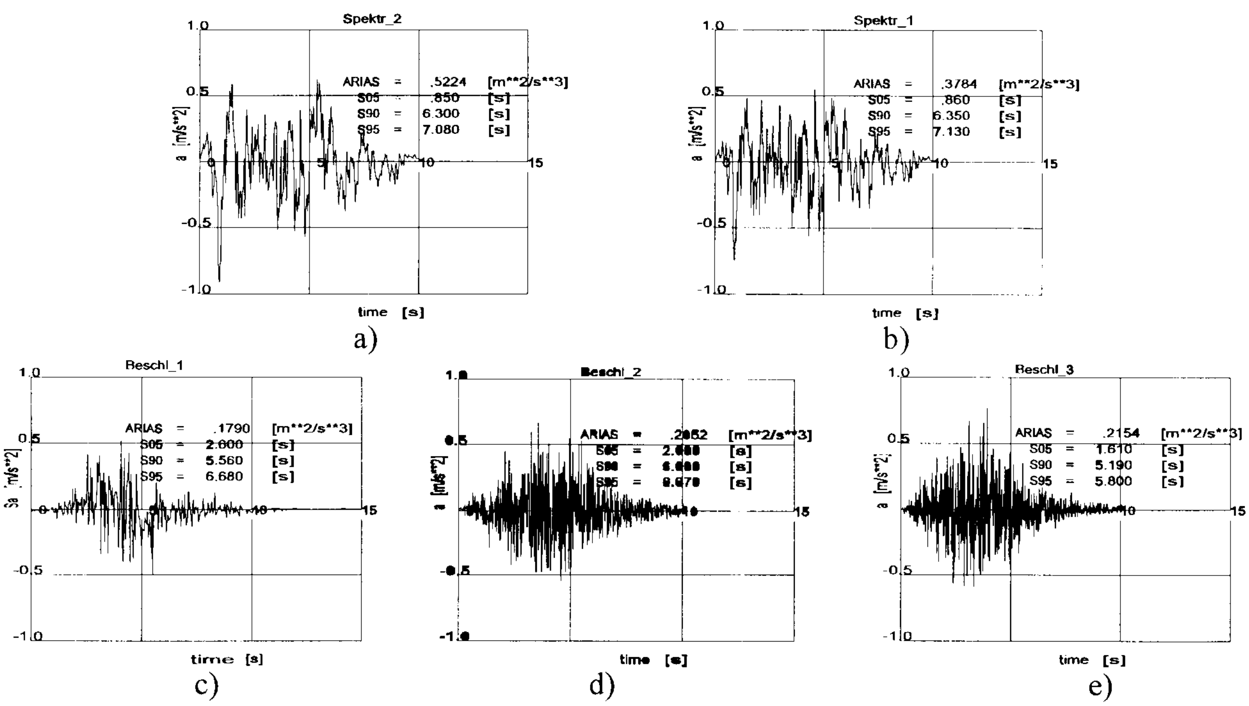
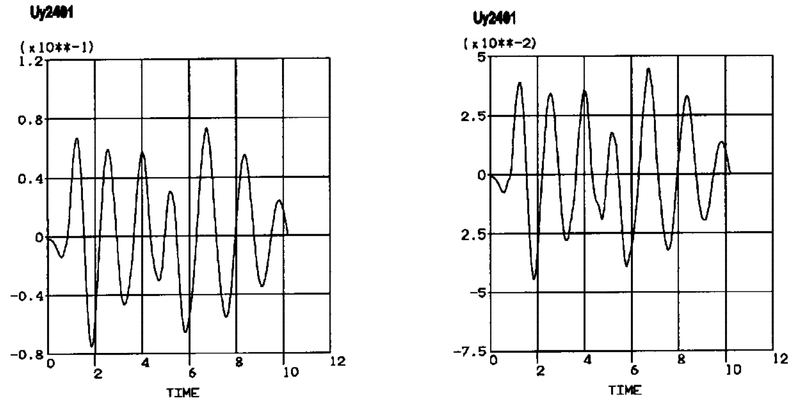


Fig. 7. Groud accelerations

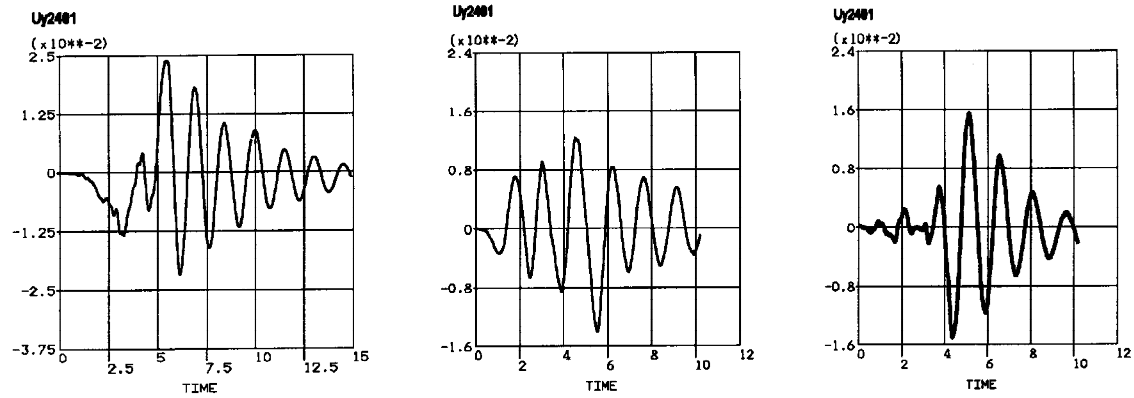
The results of response spectrum and time history analysis are compared in Table 1. The maximum difference was found to be 19 % for stresses and 14 % for displacements.

Table 1. Comparision response spectrum analysis - time-history analysis

value	response spectrum analysis	time-history analysis	distinction
$u_{x,\max}$ [mm]	0.954 (node 2401)	0.825 (node 2401)	15%
$u_{y,\max}$ [mm]	**85.3 (node 2401)**	**75.0 (node 2401)**	**14%**
$\sigma_{z,\max}$ [MPa]	**5.84 (node 1)**	**4.92 (node 1)**	**19%**
$\sigma_{eqv,\max}$ [MPa]	5.88 (node 1)	4.93 (node 1)	19%



a) $u_{y2401,max}(1.86s)$= -75 mm b) $u_{y2401,max}(6.7s)$ =44 mm



c) $u_{y2401,max}(5.61s)$ =24 mm d) $u_{y2401,max}(5.53s)$ = -14 mm e) $u_{y2401,max}(5.15s)$=16 mm

Fig. 8 Displacements of the node 2401 in y-direction

NON-LINEAR BEHAVIOUR

In principle, using high performance computers and appropriate codes also structures with 10000's of DOF could be analyzed in a non-linear way, but it is debatable, if such an approach really gives more realistic results and helps to design safer structures. Some structural details have to be modelled more exactly than in the linear case. It is necessary to have more integration points over the cross section and to consider exactly the steel reinforcement. Because of the cyclic action it is necessary to include such effects as crushing and cracking of concrete, yield of steel reinforcement etc. It was an important goal of this investigation to elaborate an appropriate simplified model for the second building which allows the consideration of the most important phenomenon.

It is emphasized, that the main earthquake resisting elements in direction of the smaller building dimension are external shear walls on both sides (Fig. 5). The wall was modelled as a beam. First the simplified beam model was optimised in order to agree with the 3-D model in the linear domain. Several attempts were necessary to define appropriate and plausible values for the soil springs, the effective masses and equivalent beam stiffness. The final values are given in Fig. 9. With these values the difference of calculated frequencies between 3 D and beam model was only 4 % for the first mode and 6 % for the second mode.

Yield moment, yield curvature and maximum curvature were calculated from the cross sectional data of the wall. At the beam ends non linear rotational springs matching the Takeda rules (Takeda,1970) were applied. In accordance with literature the length of the plastic hinge was assumed to be the half of the cross section height. Hence, the ductility demand (cross sectional ductility) follows from the calculated angle of plastic hinge rotation.

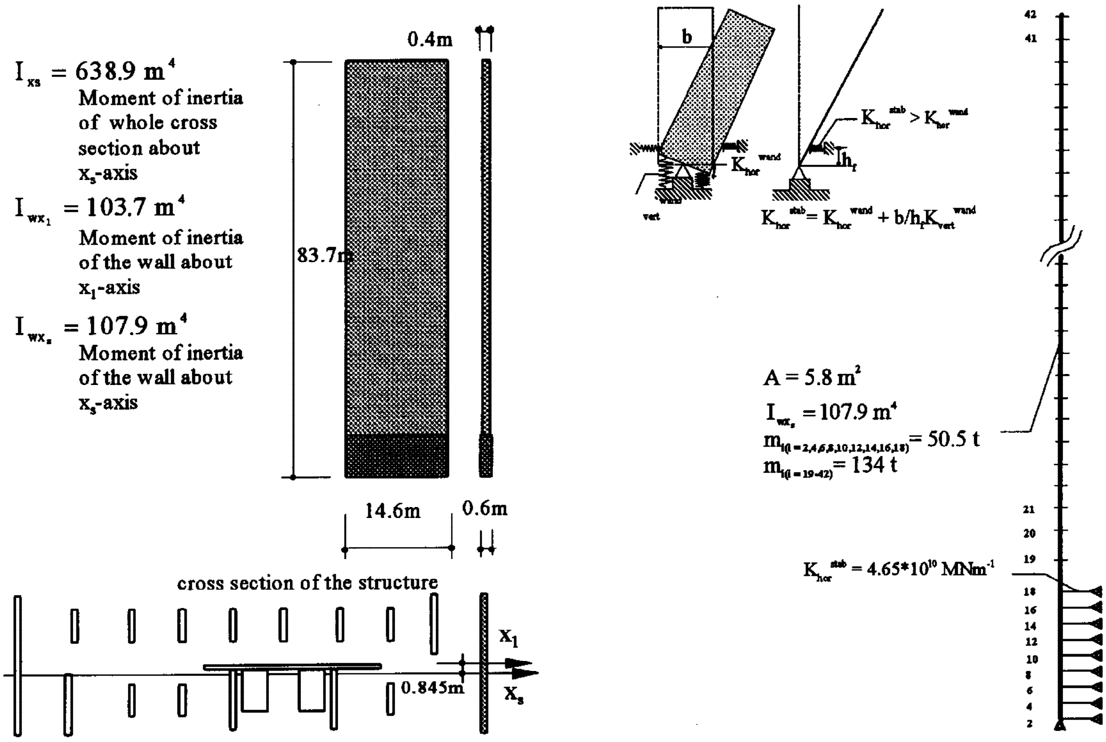


Fig. 9. Simplified beam model

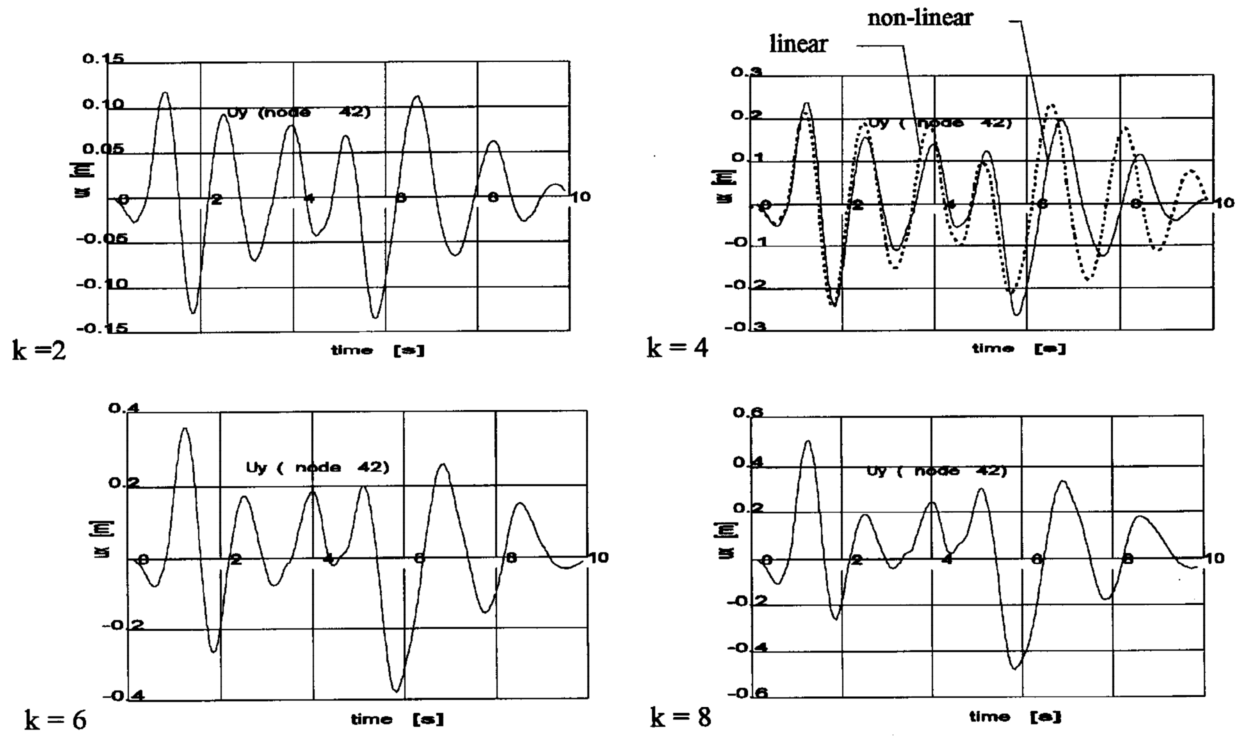


Fig. 10 Displacement of node 42 (non-linear time-history analysis) for ground acceleration SPEKTR_2

The code NILDYN (Meskouris, Krätzig, Elenas, Heiny and Mayer, 1988) was used for the non-linear analysis applying the strip method and the exact cross sectional properties (geometrical data, area and position of reinforcement, steel strength, concrete strength).

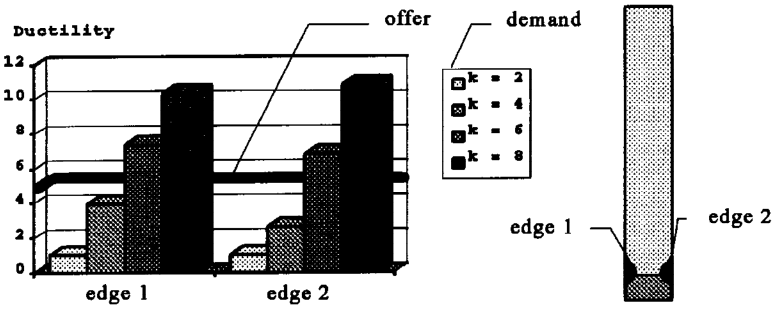


Fig. 11. Ductility balance for ground acceleration SPEKTR_2

The acceleration time histories SPEKTR_2 and SPEKTR_1 were used for the nonlinear analysis. The ground accelerations were calibrated to different earthquake levels using the factor k = 2, 4, 6 and 8 in order to investigate different levels of non-linear effects. The nonlinear displacement response in node 42 to ground acceleration SPEKTR_2 is given in Fig. 10 and to ground acceleration SPEKTR_1 in Fig. 12. Some significant differences between linear and non-linear behaviour for the higher action level can be seen in Fig. 10 and 12 (for k = 4 and 6). The structure evidently looses stiffness, resulting in a longer fundamental period in the last phase of vibration.

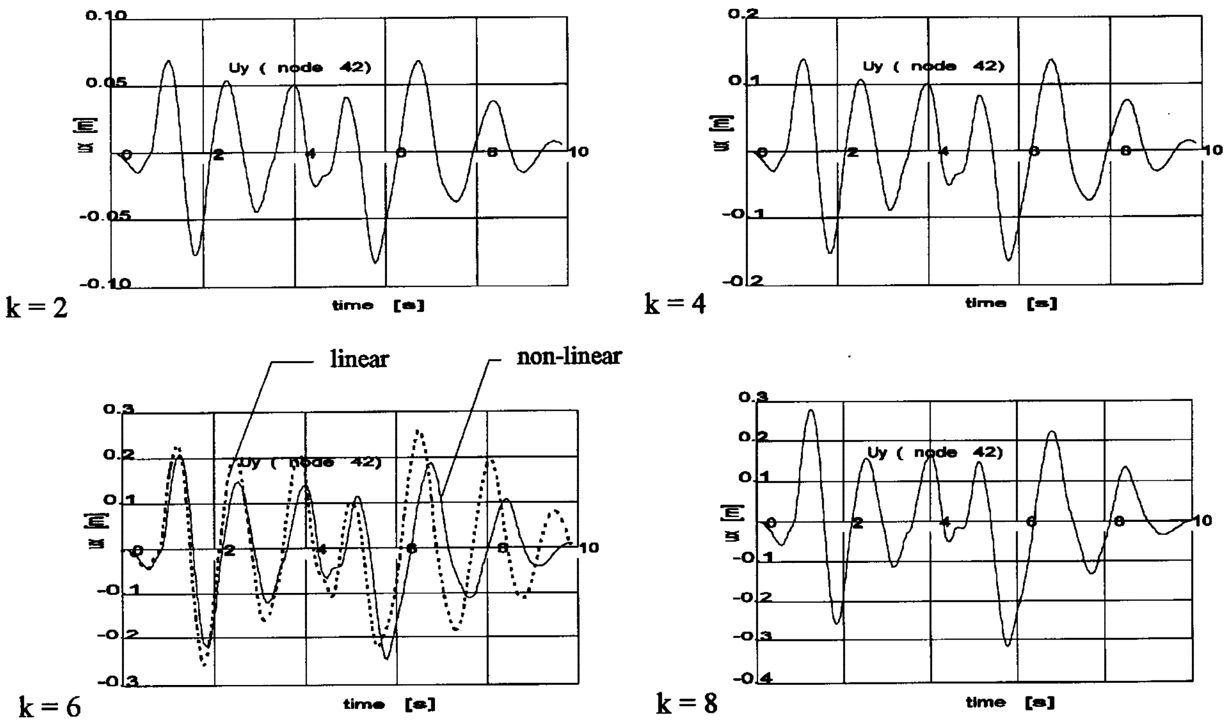


Fig. 12 Displacement of node 42 (non-linear time-history analysis) for ground acceleration SPEKTR_1

It is emphasized that the maximum non-linear displacements are not essentially higher than the linear displacements due to the hysteretic damping in the non-linear region. Similar results have been published by (Hanskötter, Krätzig, Meskouris, 1995). The goal of the non-linear analysis was a ductility balance. The ductility demand is compared with the ductility offer. From Fig. 11 it is evident, that the results for k=6 and k = 8 in Fig. 10 are not realistic, since the ductility demand is higher than the ductility offer. The same fact can be observed in Fig. 13 for k = 8.

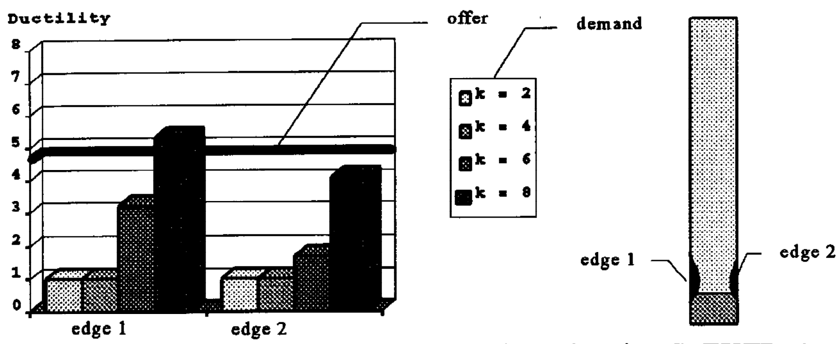


Fig. 13. Ductility balance for ground acceleration SPEKTR_1

CONCLUSION

A good agreement between experimentally obtained and calculated eigenfrequencies has been obtained. Therefore, the FEM models are considered to describe the vibrational behaviour of the buildings for an earthquake resistant design in a sufficient way. The modal analysis clearly demonstrated that in the case of stiff buildings soil parameters have a significant influence on the vibrational behaviour and have to be estimated as exact as possible. On the other hand in the case of soft and slender structures the estimate of soil parameters has much less influence on the results. The values obtained by response spectrum analysis are conservative compared with time history analysis, the maximum stresses being 19 % higher in the first case. The non-linear results demonstrate some significant difference between linear and non-linear behaviour for higher action levels. The structure evidently looses stiffness, resulting in a longer fundamental period in the last phase of vibration. The maximum non-linear displacements are not essentially higher than the linear displacements due to the hysteretic damping in the non-linear region. It was demonstrated that a simplified beam model can be elaborated, giving results which are in good agreement with the results obtained from a detailed 3-D model. The simplified model was mainly used to check the ductility demand against the ductility offer. Hence, ductility balances could be executed for different load levels.

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