THE PUBLISHING HOUSE
OF THE ROMANIAN ACADEMY

PROCEEDINGS OF THE ROMANIAN ACADEMY, Series A,
Volume 7, Number 2/2006, pp. 000-000

MODIFICATION OF THE BUILDING DYNAMIC RESPONSE DUE TO STRUCTURAL DAMAGING DURING STRONG EARTHQUAKEs.

Tudor SIRETEANU*, Ana Maria MITU*, Gheorghe GHITA*, Viorel SERBAN**

*Institute of Solid Mechanics – Romanian Academy 15 Constantin .Mille Street, RO-010141, Bucharest
E-mail: siret@imsar.bu.edu.ro
**Subsidiary of Technology and Engineering for Nuclear Projects, 409 Atomistilor Street, Bucharest Magurele, Romania

The paper is an analysis of buildings behavior during strong earthquakes, considering the modifications of the structural dynamic characteristics due to strong earthquake - generated damages. The study is focused on the difference in a building behavior, function of the mode the structure is situated relative to the seismic ground motion from the spectral point of view: above resonance or below resonance. Experimental and simulation results are presented for simplified mechanical models of building structures with stiffness degradation. As a convenient measure of the effect of duration and severity of the building seismic loads, the total energy dissipated through hysteresis is considered.

Key words: *Earthquake, building damage, degrading systems, hysteresis, Kelvin-Voigt model.*

1. INTRODUCTION

The acceptance of plastic hinges occurrence in a building according to the seismic design standards [1-3] leads to a degradation of the structural restoring force and to an increase of the structural damping. The first effect could be beneficial if the natural vibration periods of the building are longer than those of the main spectral components of the ground motion. In this case, by structural stiffness degradation the building is "pulled" from the resonance regime resulting in a reduction of seismic response. On the other hand, if the main spectral components of the seismic ground motion are longer than the building natural periods, then the structure could be "dragged" to resonance with a significant increase of the seismic response, which can result in important building damages or even in collapse. The increase of the building structural damping capacity due to the occurrence of plastic hinges is beneficial in both cases as more of the kinetic energy injected to the building by the seismic action is consumed as the structure experiences repeated stress reversals. However, this increase of structural damping is not so important such as to dramatically reduce the vibration amplification within the resonance range [4].

Inspection of damaged buildings after major earthquakes reveals in many cases breakages of structural elements between second and third floor. A possible explanation of this damage pattern is discussed in [5], where a vertical cantilever beam with imposed displacement of the embedded base is used as a simple mechanical model of a multiple storey building excited by seismic horizontal ground motion. Studies carried out on both experimental and analytical models showed that different vibration patterns occur when the period of the base imposed displacement is longer (above resonance) or shorter (below resonance vibration regime) than the period of the first vibration mode (vibration regimes above or below resonance, respectively). In both cases, it is assumed that the ground motion has dominant spectral components with periods around the first natural period of the building.

For same absolute displacement amplification factor (the ratio of the maximum response amplitude at the free end to the input displacement magnitude), below resonance both bending moment and shear force along the beam are bigger than above resonance. Moreover, below resonance the shear force has a maximum value at an almost fixed transversal section of the structure, which for reasonable values of the absolute displacement transmissibility, is located around one third of the beam length. The maximum value of the

Recommended by Radu P.VOINEA, member of the Romanian Academy shear force could be about 20% bigger than the value encountered at the clamped end (almost the same in both cases) and about 45-50% higher at the pseudo-node position. Therefore, the building response excited by the ground motion below resonance could be more damaging to the structural elements than in the above resonance case. This assertion is also advocated by the fact that, for same value of the amplification factor, the inter-storey drift is larger above resonance (when the building top level and base motions are almost in opposite phase) than below resonance (when the building top level and base motions are almost in phase).

By accepting the controlled damaging of a conventional building due to material degradation by over-loading (plastic hinges), the building vibration regime could change from the one above resonance to the one bellow resonance, even within the relatively short duration of the seismic action. This dynamic behaviour could be a possible explanation of the building breakages between the 2^{nd} and 3^{rd} floor.

The aim of this paper is to illustrate the change in dynamic behavior of structures associated with system degradation. Simple mechanical models with harmonic and seismic inputs are used for both experimental and analytical approach of this problem.

2. EXPERIMENTAL ANALYSIS

To illustrate the change of dynamic behaviour of a structure, due to system degradation, an experiment was conducted on a simple mechanical structure. A cantilever beam from composite material with a concentrated mass on top was mounted vertically on an electro-dynamic shaker. From practical point of view, this experimental study is relevant when the beam vibrates in the neighborhood of the first resonance vibration mode because in this case the dynamic structural output is strongly amplified by favored transfer of the kinetic energy from the base imposed motion to the structure and damaging shear forces can develop.

The accelerations of the clamped base and of the beam top were simultaneously recorded on a PC oscilloscope for both forced and free vibrations. The experimental set up is shown in figure1.

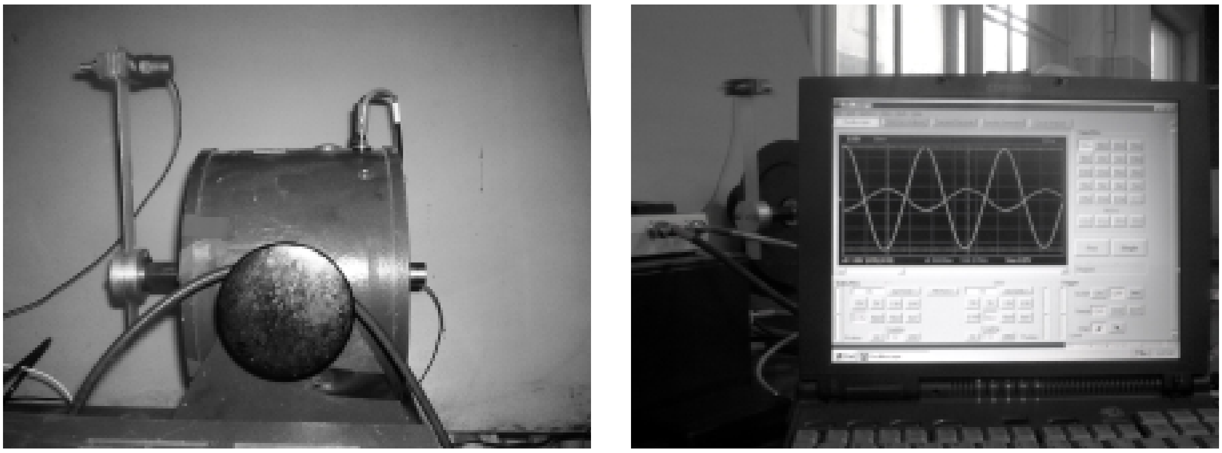


Fig. 1. Experimental set up

The structural degradation was produced by gradually delaminating of the composite material, resulting in a decrease of bending stiffness and a corresponding increase of internal damping. The period of the first bending vibration mode, denoted by T, and the associated modal damping ratio (ς) were obtained from the free vibration records.

In figures 2-4 are presented the experimental results, showing the negative effect of structural degradation on the amplification factor $A_{\mathrm{r.m.s}}$ (defined as the ratio of the r.m.s values of the top and base acceleration). As one can observe, the value of the amplification factor became 2.3 times greater as the bending stiffness degradation was gradually increased. This important increase of $A_{\mathrm{r.m.s}}$ was obtained despite the fact that the damping ratio increased with 60%. During this test, the r.m.s value of the input acceleration and its fundamental period were maintained practically constant ($a_{\mathrm{r.m.s.}} = 0.064g$, $T_{\mathrm{in}} = 0.36\mathrm{s}$) and the period of the beam first vibration mode was in all case less than the base excitation period.

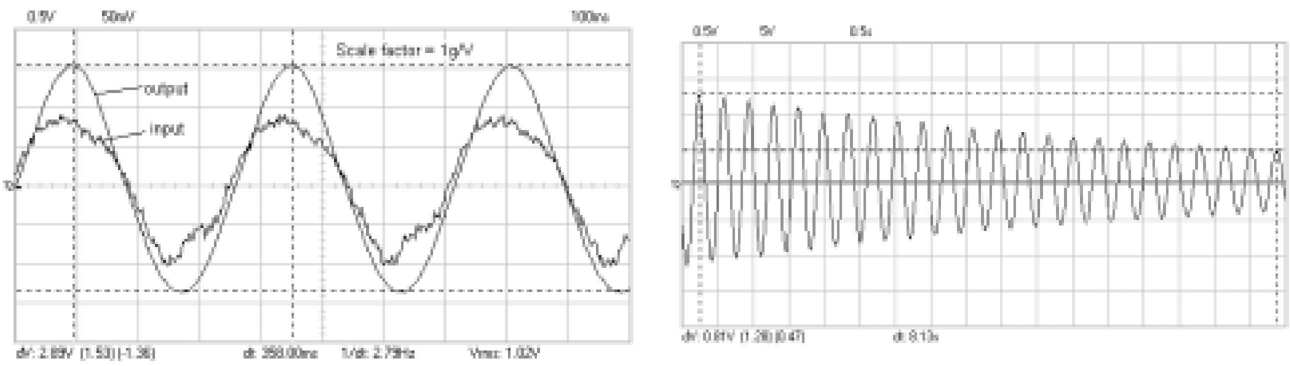


Fig.2. Forced (T_{in} =0.36s, A_{rms}= 16.2) and free vibration of the undamaged beam (T=0.353s, ζ=0.0069).

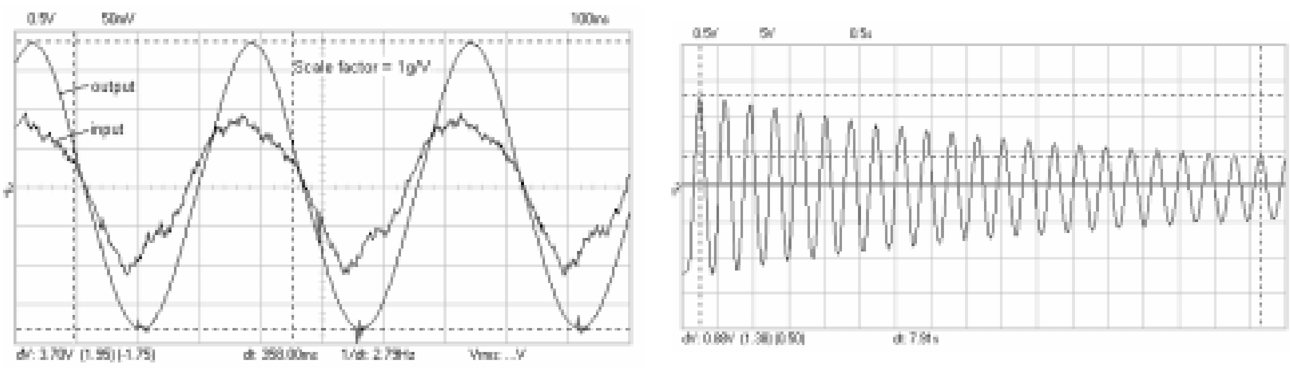


Fig.3. Forced (T_{in} =0.36s, A_{rms}=20.1) and free vibration of the slightly damaged beam (T= 0.359s, ζ=0.0073)

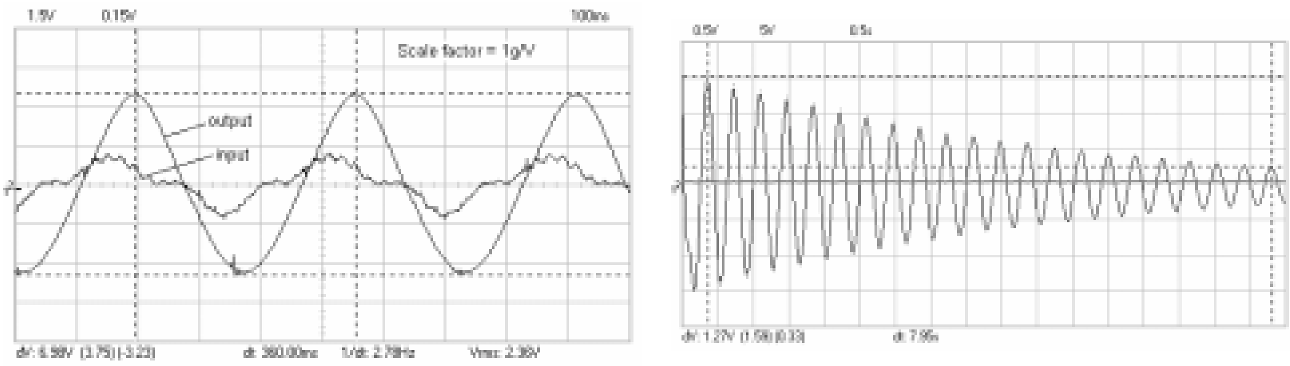
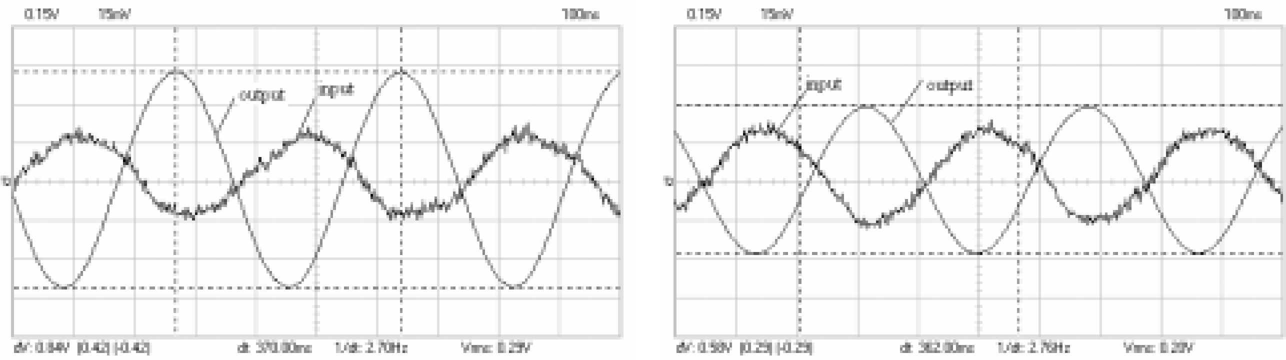


Fig.4. Forced (T_{in} =0.36s, A_{rms}= 36.7) and free vibration of the moderately damaged beam (T=0.379s, ζ=0.012)

The decrease of amplification factor, $A_{\mathrm{r.m.s}}$, when the vibration regime is moving away from resonance is illustrated in figure 5.

The two different bending vibration patterns, which take place above and below resonance for almost same value of the amplification factor ($A_{\mathrm{r.m.s}} \approx 16$), are shown in figures 2 and 5. It is easily seen that above resonance ($T_{\mathrm{in}} = 0.36\mathrm{s}$, $T = 0.35\mathrm{s}$) the base and top motions are almost in phase whereas below resonance ($T_{\mathrm{in}} = 0.36\mathrm{s}$, $T = 0.39\mathrm{s}$) these motions are almost 180^0 out of phase. The gradual change of phase shift with beam degradation can be observed in figures 2-5.

It should be mentioned that the dramatic increase of the motion amplification factor for a rather modest increase of the beam eigenperiod (approximately 8%) was obtained because the test had been conducted in the neighborhood of the resonance range where the variation of the amplification factor is very rapid.



a b

Fig.5. Forced vibration of the moderately damaged beam below resonance:
a) T=3.8s, T_{in} =0.37s, A_{rms}=26.6; b) T = 0.38s, T_{in} =0.36s, A_{rms}=15.7.

3. ANALYTICAL MODEL

Although an multiple-storey building is envisaged, the model adopted (see figure 6) employs only one mass, the aim being to approximate the vibration of the building in the range of its lowest mode. Only lateral motion is considered, the building being treated as a shear structure.

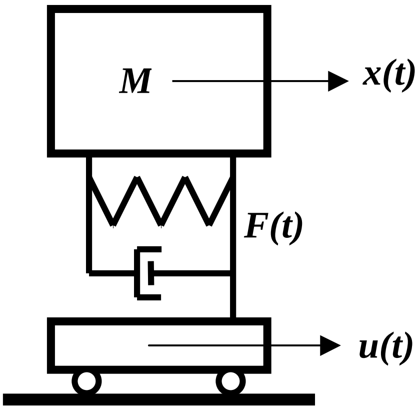


Fig.6. Schematic of mechanical system

The sprung mass, M , is connected to the system base by an element of Kelvin-Voigt type, generating a hysteretic force, $F(t)$, given by

$$F(t) = ky(t) + c\dot{y}(t) \tag{1}$$

where $y(t) = x(t) - u(t)$ is the relative displacement between the top level and the building base. Degradation of the restoring force gradually increases as the structure experiences repeated stress reversals. The degradation due to the occurrence of plastic hinges, leads to a certain increase of energy dissipation by the internal damping mechanism. The viscous damping force term in (1) is view as an equivalent damping of the internal energy dissipation. The parameters in any hysteretic model must become time dependent, if these degradation effects are to be accounted for.

The degradation mechanism can be modeled by allowing the parameters k and c to vary as a function of the response duration and severity [4]. As convenient measure of the combined effect of duration and severity is the total energy dissipated through hysteresis over the time interval $[0,t]$. Taking into account that in the considered hysteretic model the energy dissipation is of viscous type, a simplified model of degradation can be expressed by the following linear functional relationships for the instantaneous values of the system relative damping coefficient, $\varsigma(t)$, and natural undamped pulsation, $\omega(t)$,

$$\varsigma(t) = \varsigma_0 (1 + \alpha \int_0^t \dot{y}^2 \mathrm{d}t) \text{ , } \omega(t) = \omega_0 (1 - \beta \int_0^t \dot{y}^2 \mathrm{d}t) \tag{2}$$

where ς_0 and ω_0 are the initial values of the undamaged structure and α , β are non-negative parameters.

The equation of motion of the system with degradation may be written as

$$\ddot{y} + 2\varsigma(t)\omega(t)\dot{y} + \omega^2(t)\, y = -\ddot{u} \tag{3}$$

For an imposed harmonic motion

$$y(t) = y_c \sin\left(\frac{2\pi}{T_c}t\right) \tag{4}$$

the time histories of parameters $\varsigma(t)$ and $\omega(t)$, as obtained from equations (2) and (4) are given by

$$\begin{gathered}\varsigma(t) = \varsigma_0\left[1 + \alpha\varphi(t)\right],\ \omega(t) = \omega_0\left[1 - \beta\varphi(t)\right] \\ \varphi(t) = \frac{2\pi^2 y_c^2}{T_c^2}\left[t + \frac{T_c}{4\pi}\sin\left(\frac{4\pi}{T_c}t\right)\right]\end{gathered} \tag{5}$$

At the completion of N^{th} loading cycles, i.e. for $T_N = NT_c$, the following values are obtained

$$\varsigma_N = \varsigma_0\left[1 + \alpha\frac{2\pi^2 N y_c^2}{T_c}\right],\ \omega_N = \omega_0\left[1 - \beta\frac{2\pi^2 N y_c^2}{T_c}\right] \tag{6}$$

Equations (6) can be used to assess the values of coefficients α and β, associated with the system degradation. Let us consider a cyclic loading with the amplitude $y_c = 0.2\text{m}$ and the period $T_c = 1\text{s}$. Assuming that after completion of 25 testing cycles (25seconds) the system damping capacity increases 3 times while its stiffness decreases four times, from equation (6) results $\alpha = 0.1\,\text{s/m}^2$ and $\beta = 0.025\,\text{s/m}^2$. From an intuitive point of view, this cyclic loading is fairly realistic if one thinks of building first vibration mode period and of the relative displacement between the building base and top level, which can occur during strong earthquakes. Large relative movements within a building (drift) are liable to fracture strengthening structural elements and may result in serious building damages or even in collapse. For this reason it is desirable to keep drift below 0.5% storey height [6]. Therefore, a total drift of 0.2m could be acceptable for a building height greater than 40m. On the other hand, the limitation of drift by introducing too large damping forces increases the lateral accelerations, which is particularly undesirable in the case of buildings, where equipment and services (gas, water) could be damaged. Indeed some deaths in the 1999 Izmit (Turkey) earthquake were reportedly due to televisions falling on to people sleeping below. Accordingly accelerations should be kept below 3 m/s^2 [6].

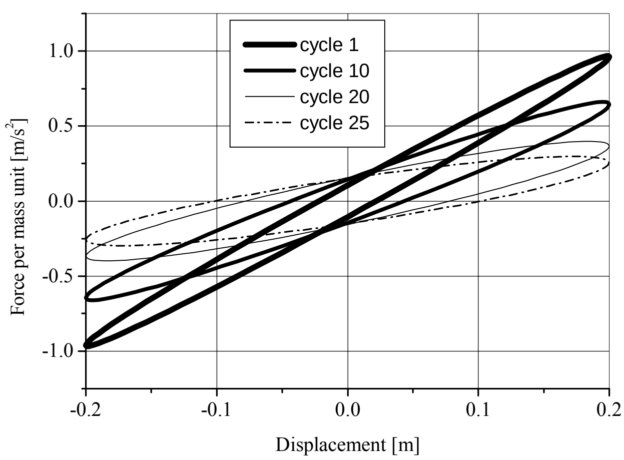


Fig.7. Time evolution of hysteresis loops with the increase of loading cycles number

Figure 7 shows the evolution of the degrading hysteresis loops with the increase of loading cycle number,

Combining the motion equation (3) with the degradation model (2), one obtains an integral-differential equation to portray the dynamic behaviour of a degrading oscillating system perturbed by the base acceleration, $\ddot{u}(t)$.

4. NUMERICAL RESULTS

In this section are shown the results illustrating the modification of dynamic behaviour of buildings with different initial values of the first vibration mode period and degrading hysteretic characteristics, subjected to strong seismic actions. The time history of the ground motion acceleration was chosen from the Internet available seismic records, having the following identification data:

- Origin Time: 1996/10/19 23:44;
- Latitude: 31.8; Longitude: 132.0l; Depth: 39 Km; Magnitude: 6.6;
- Station Code HRS018; Station Latitude: 34.3380; Station Longitude: 132.9094; Station Height: 2m;
- Record Time 1996/10/19 23:46:13; Sampling Frequency: 100Hz; Duration Time: 59s; Direction: E-W;
- Maximum Acceleration: 11gal.

In order to obtain a strong earthquake input, this record was scaled for a maximum acceleration value of 0.5g. In figures 8 and 9 are shown the time history (shown over a time interval of 50s) and response spectra of the seismic ground motion. The response spectra were calculated for damping ratio $\varsigma = 0.05$, the same value that was assigned to the relative damping coefficient, ς_0, of the undamaged system.

It should be pointed out that the considered seismic acceleration input is represented by a narrow bandwidth signal with a dominant spectral component of period $T_{\text{in}} = 0.9\text{s}$.

The integral-differential equation of motion, obtained as a combination of equations (2) and (3), was solved by using the Simulink toolbox of Matlab 6.5.

The initial eigenperiod of the undamaged system, $T_0 = 2\pi/\omega_0$, was given values within the time interval 0.5s-1s, where the most important values of the response spectra are found. Note that the lowest natural vibration periods of tall buildings are usually situated within this time interval.

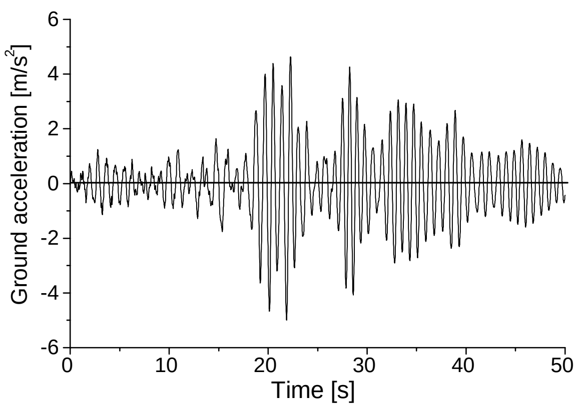


Fig.8. Time history of ground motion acceleration

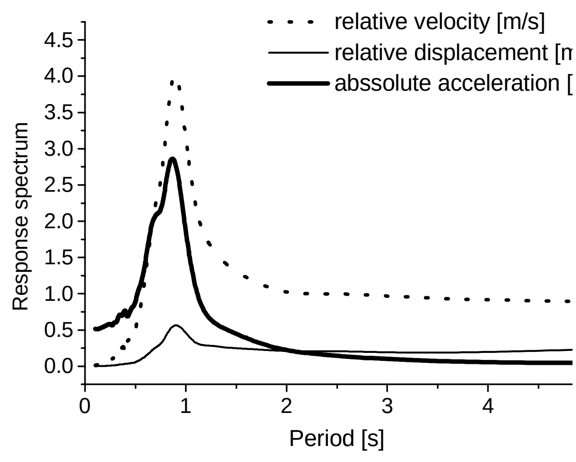


Fig.9. Response spectra of ground motion acceleration

The modifications of system dynamic behaviour due to structural damage are illustrated in figures 10 and 11 for two case studies. The first case corresponds to the situation when the natural period of the undamaged building, $T_{01} = 0.83\text{s}$, is only slightly shorter than the period of the dominant spectral component of the seismic motion, $T_{\text{in}} = 0.9\text{s}$. In this case, the development of controlled structural degradation by allowing the occurrence of plastic hinges is expected to "pull" the system away from the vibration resonance regime, with a beneficial effect on the building seismic response level. In the second case study, the natural period of the undamaged system, was chosen much shorter than the period of the dominant spectral component of ground motion ($T_{02} = 0.63\text{s} < T_{\text{in}} = 0.9\text{s}$). In this case, by structural stiffness degradation the building is "dragged" to the resonance range with and an important amplification of the shear stresses that can lead to severe structural damage or even to building collapse.

To outline the seismic response attenuation or amplification effects due to the system degradation, the output time histories of the system with and without degradation, as well as the evolution of the instantaneous values of relative damping, $\varsigma(t)$, and natural period of vibration, $T(t) = 2\pi/\omega(t)$, are plotted in figures 10 and 11. The scales of the plotted time histories were chosen such as to allow their proper representation on the same plot area.

It is worth mentioning that the modification of the building dynamic behaviour during strong earthquakes is mostly influenced by the structural stiffness degradation and only in a less extent by the associated increase of the internal damping capacity. The time histories plotted in figures 11, showing that an important amplification of the building seismic response occurred despite of the 140% increase of the system relative damping, advocate this assertion.

Note that the building modeled in the first case study is taller (more flexible) than the building modeled in the second case study since $T_{01} > T_{02}$. Therefore, same or even smaller relative displacement values can produce higher shear forces in the second building than in the first one. Moreover, the relatively high number of shear stress cycles with important amplitudes, which occur at the end the seismic action in the less flexible building (see figure11), could produce more damaging effects than the only three or four significant cycles (even with higher amplitudes) produced by the first intensification of the seismic action in the more flexible building (see figure 10). Comparison of the time intervals within the relative displacement amplitude is larger than 0.15m ($\Delta_1 t = 23\text{s-}19\text{s=}4\text{s}$ in the first case and $\Delta_2 t = 41\text{s-}28\text{s=}13\text{s}$ in the second case) is also relevant for evaluation of the two different effects of structural degradation on building dynamic behaviour.

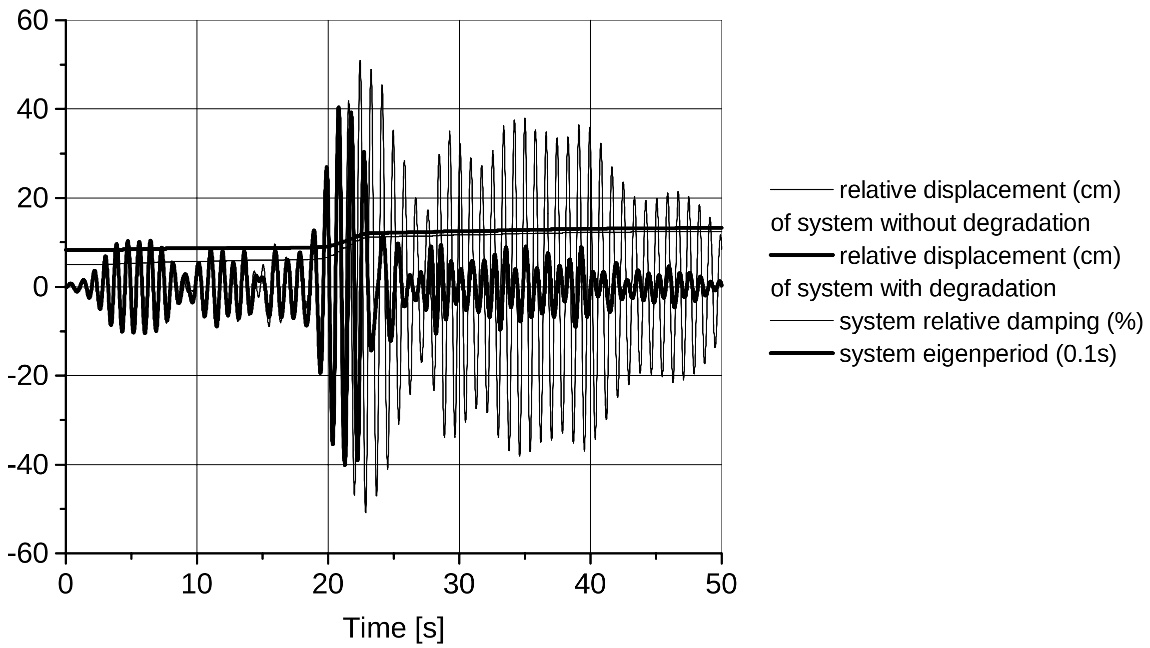


Fig. 10. Attenuation of system seismic output due to structural degradation (first case study : $T_{01} \approx T_{\text{in}}$)

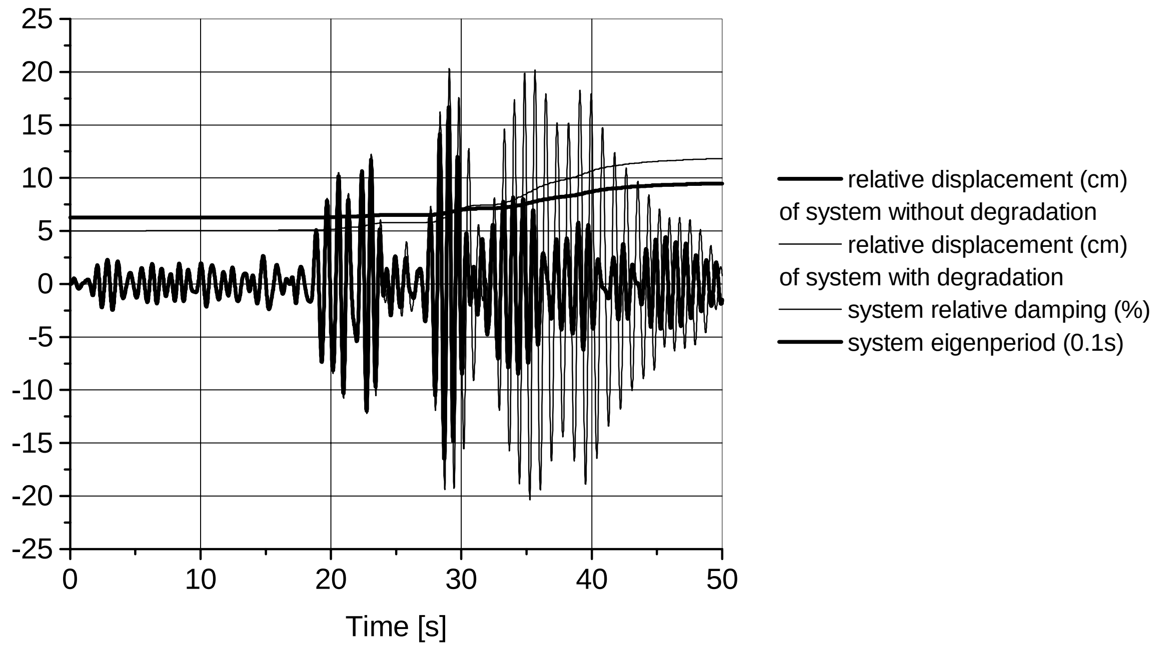


Fig. 11. Amplification of system seismic output due to structural degradation (second case study $T_{02} < T_{in}$)

5. CONCLUSIONS

The results reported in this paper are based on simple models and possible scenarios portraying the dynamic behaviour of buildings with degrading hysteretic characteristics, during strong earthquakes. Although the model parameters and the ground motion were assigned fairly realistic values or characteristics, the simulation results couldn't be associated with some specific real case. Nevertheless, these results advocate several important qualitative conclusions regarding the effects of structural degradation on building dynamic response to strong earthquakes:

- The behavior of a building subjected to seismically forced motion of the foundation is very much dependent on whether the building is shorter, approximately equal or longer as compared with the periods of the dominant spectral components of the ground motion.
- The controlled damage by allowing the occurrence of plastic hinges as the structure experiences repeated stress reversals during a strong earthquake reduces the overall building stiffness and consequently increases the building natural periods of vibration. Consequently, a building may enter or exit the resonance range of the seismically induced vibration, depending on the relative position between its initial natural vibration periods and the periods of the dominant spectral components of the seismic motion. It should be mentioned that stiffness degradation can occur only for sufficiently large building deflections, which are unlikely to develop unless the building motion is excited within or very close to the resonance range. Otherwise, the building mechanical filtering properties do not allow significant amplification of the base motion or even can lead to its attenuation. Therefore, in this case the building protects itself against the seismic action. Both the experimental and numerical simulation results presented in this paper are addressed to building seismic behaviour for resonant or quasi-resonant vibration regime.
- If the initial values of the building natural period of vibration are situated to the right side of the potential resonance range of the ground motion response spectrum, then by overall stiffness degradation the building is rapidly "pulled away" from resonance and becomes self-isolating against

the seismic motion. Contrariwise, if the initial values of the building natural period are placed to the left side of this period range, then by overall stiffness degradation the building is "dragged to" the resonance range resulting in gradual development of high shear stress levels with severe consequences on building structural integrity.

- The increase of the building self-damping capacity by internal energy dissipation due to development of plastic hinges is not so important as to significantly reduce the building seismic response. Moreover, the acceptance of plastic hinges is neither a safe nor an economic solution. The increase of the building damping capacity should be achieved by special devices that are capable to control and limit the structural deflections rather than by accepting the local breaking of the structural elements (beams and the base columns).
- The acceptance of plastic hinges occurrence in buildings, according to the seismic design standards to enhance the building strength during severe earthquakes, is beneficial only in the case in which the undamaged building natural vibration period is longer than the period of the dominant spectral component of the ground motion.
- By stiffness degradation the building has always the tendency to vibrate above resonance range, since a pure resonance could be possible only for extremely short time intervals. As shown in [1], for this vibration regime, the maximum values of shear forces (stresses) occur in columns at a certain height from foundation, generating in many cases column breakages at about one third of their height from the foundation level. This damaging effect is manly due to the fact that in the above discussed vibration pattern the building baser and top level absolute displacements are in opposite phase as shown by the experimental results presented in this paper.
- Further work should consider more realistic models of building structural degradation as well as synthetic seismic ground motions, compatible with given design response spectra.

ACKNOWLEDGEMENT

The authors would like to express his gratitude to the Romanian Academy for supporting this work through the Grant no.93/2005. Also the opportunity created within the framework of the research programs CERES and CEEX for developing advanced research work in the field of seismology and earthquake engineering is highly appreciated.

REFERENCES

1. P 100-92 – Standard for the seismic design of civil, agricultural and industrial buildings.
2. P 100-1/2004 – Seismic Design Standard.
3. EUROCODE 8, Design Previsions for Earthquake Resistance of Structures. ASCE-7-93—SUA;
4. ROBERTS, J.B., SPANOS, P.D., Random Vibration and Statistical Linearization, Dover Publication, Inc., Mineola, New York, 2003.
5. SERBAN,V., ANDRONE, M., SIRETEANU, T., *Structural degradation effects on the e building seismic response and their assessment in the Seismic Design Standard* P *100-1/2004*, Symposium "Contribution to the Development of Seismology and Earthquake Engineering in Romania", Bucharest March 03, 2006.
6. SIRETEANU, T., A possible explanation of the building breakage between 2nd and 3rd floor when subjected to strong earthquakes, Proceedings of the Romanian Academy, Series A, Volume 6, Number 3/2005, pp. 241-247.
7. STAMMERS, C.W., SIRETEANU, T., *Control of building seismic response by means of three semi-active friction dampers*, Journal of Sound and Vibration, **237**, 5, 2000, p.745-759.

Received May 8, 2006