COMPDYN 2011
III ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis, V. Plevris (eds.)
Corfu, Greece, 25–28 May 2011

STRUCTURAL DYNAMIC INSTABILITY DUE TO EARTHQUAKE LOADS

Gluck N[1*], Farhat R[1], and Tzadka U.[1]

[1] Sami Shamoon College of Engineering, Dept. of Civil Engineering
Reh. Bialic – Bazel
84100 Beer Sheva, Israel, P.B. 45
mikig@sce.ac.il

Keywords: Vertical oscillations, P-Delta and Secondary Effects, Structural response, Seismic Amplification, Structural Damage,

Abstract. *From a thorough observation of the images taken after major seismic events arises the conclusion that structures collapse vertically due to their lack of stability under vertical load amplification. Overturn of buildings due to earthquake effects happens in case of weak foundations or liquefaction effects (see Niigata, Japan 1964).*

Majority of collapses occurred due to structural instability as P-Delta effects under combined horizontal and vertical loads. Worse, there must be a dynamic effect due to vertical seismic oscillations that leads to amplification of the vertical loads. Vertical effects and P-Delta phenomena are mentioned in each seismic code but the effect is dealt statically for analysis and design purposes.

It is indicated that dynamic analysis should be performed by means of step by step nonlinear analysis.

The present work is intended to initiate a methodology to use the spectral capacity demand analysis to deal with the P-Delta effects compared with the results calculated by means of step by step analysis. The nonlinear effects develop at the bar ends (beams and columns) as plastic hinges with gradual plasticization due to the lateral displacements. Bending moments and shear effects in the structural elements are monitored during the simulations performed by means of the SAP2000 – Nonlinear Software.

1 INTRODUCTION

In his paper, written in 1994, John Lawson, Vice President Kramer & Associates Structural Engineers, Inc. Tustin, California stressed that "…new provisions addressing the effects of vertical seismic acceleration should not be incorporated into the Uniform Building Code without careful consideration." Since then a lot of new research were accomplished most of them based on actual severe seismic events. Nonlinear behavior has been introduced into the methodology of the analysis of structural behavior under seismic events.

Most of the existing codes still use the conventional methodology to check or design the behavior of structures under seismic loads. Horizontal effects solely are taken into account for the seismic loads. Vertical effects are taken into account for specific elements like cantilevers, long presstressed beams or beams that support columns. The secondary effects of earthquakes are taken into account by static methods (due to the weight of each story).

The instability of structures under vertical amplification effects is evident for a lot of collapses that occurred as consequence of seismic movements. Among these effects, the damages of soft floors when the structure lost its first floor are relevant. Figure 1 visualizes some residential buildings at Loma Prieta , California which due to the collapse of their columns of the open floor destroyed the garages underneath the residences.

Figure 1. Loma Prieta, soft story collapse.

The collapse was the consequence of the P-Delta effect combined with the amplification of the vertical oscillation. Similar effects occurred during the 6.8 Richter magnitude event of the Northridge earthquake damaging multistory residential units, parking garages and freeway overpasses. Figure 2 shows the one of the damaged multistory buildings after the 24 October, 1999, 7.6 Richter Magnitude, Taipei earthquake. The structure collapsed due to the plastic hinge developed at the first storey of the building due to the P-Delta effect.

Figure 2 Taipei Structure overturning.

One of the arguments of the author of the upper mentioned paper was the example of the spectacular collapse of the California State University Northridge Parking Structure. A paper in a local newspaper attributed the collapse to the vertical ground acceleration. According to John Lawson the load factors consider the downward vertical acceleration. His reasoning is based upon the dead load of the structure of about 5.75kPa and live load of at least 2.39kPa would lead to consider an ultimate design strength of:
$(1.4 \times 5.75) + (1.7 \times 2.39) = 12.113 kPa$ Since the parking structure was empty at the time of the earthquake, this ultimate strength would have been reached at $(12.113 / 5.75 - 1) = 1.1g$ of effective vertical acceleration. If we compare this effective vertical acceleration with the current UBC's 0.4g effective horizontal peak ground acceleration for Seismic Zone 4 we see that the code's load factors do provide significant reserve strength for upward vertical accelerations compared to horizontal accelerations. The main idea of the paper states "that the additional consideration must be taken on the effects of the vertical acceleration on the P-Delta effects. This situation may create a governing load condition if very large horizontal and vertical accelerations occur simultaneously". To complete this statement is the resonant state which may provide severe damage due to the loss of structural stability in any direction (horizontally or vertically).

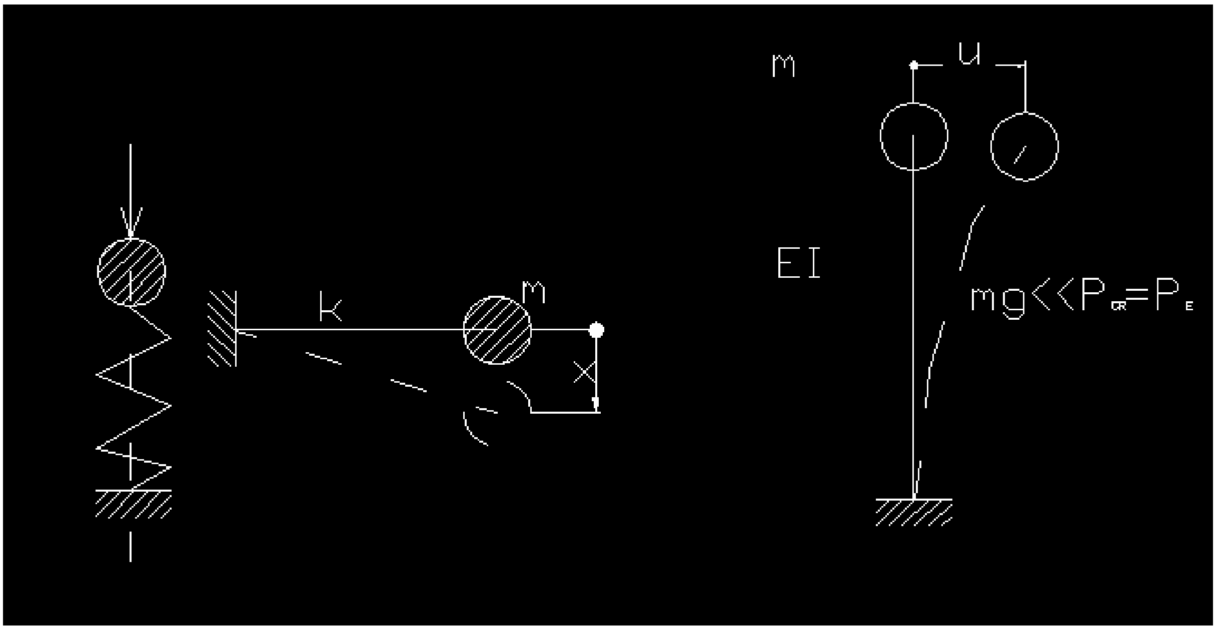

Figure 3 Static and dynamic stability for $mg << P_{cr} = P_E$

2. **STRUCTURAL DYNAMIC INSTABILITY**

The phenomenon of loss of stability occurs as consequence of an essential, sudden modification of the parameters defining the stationary state of the physical system. In case of a mechanical system there may be remarked two stationary states. 1) static equilibrium relative to an inertial system of coordinates, or 2) a movement mainly defined by the generalized coordinates $q, \dot{q}$ according to the minimal action defined by Hamilton.

In the first case the equilibrium loss is a static phenomenon (a sudden increase of the deformations under the critical static loads); in the second case the mechanical system has a perturbation of the main movement trough an unlimited increase of the velocity and acceleration of the system. For both cases the instability phenomenon takes place through the development of great accelerations and a gradual modification of the geometric configuration of the mechanical system and through the modification of the energetic equilibrium by the delivery of kinetical energy (the sudden feature of the loss of stability).

The dynamic systems have been classified as: autonomous systems and non-autonomous ones.

The first category comprises the systems with equation of movement according the following equation:

$$M\ddot{x} + F(P, x, \ddot{x}) = 0 \quad \text{Eq. 1}$$

Where
M is the mass matrix of the system,
$x, \dot{x}, \ddot{x}$ are the vectors for the displacements, velocities and the accelerations of the system,
P is the vector of the external action on the system.
Function F may be linear or non-linear according to x and $\dot{x}$ for the mechanical system.
Actions which are based upon a potential (gravitation e.g.) are conservative and imply autonomous behavior.
In case of a linear elastic structure with viscous amortization the free oscillation has the following equation of movement:

$$M\ddot{x} + C\dot{x} + Kx = 0 \quad \text{Eq. 2}$$

Where
C is the damping matrix.
K is the rigidity matrix of the structure.
For positive coefficients in the damping matrix the free vibrations decay with time and the dissipated energy of the system may be expressed by a quadratic form:

$$E_d = \frac{1}{2}\dot{x}^T C\dot{x} \quad \text{Eq. 3}$$

The instability of the shape $(x \to \infty)$ and the instability of the oscillations $(\omega \to 0)$ is attained only when the rigidity matrix K becomes singular. If we complete Eq. 2 by means of the axial force participation such as:

$-N\left(x, \frac{\partial x}{\partial z}\right)$ we define the rigidity diminution due to the axial force and thus the loss of stability may be sensed through the Equation 4.

$$\det\left|K - \lambda N\right| = 0 \quad \text{Eq. 4}$$

The increasing series of the λ values which are the roots of the equation 4 are the Eigen values for the external load.

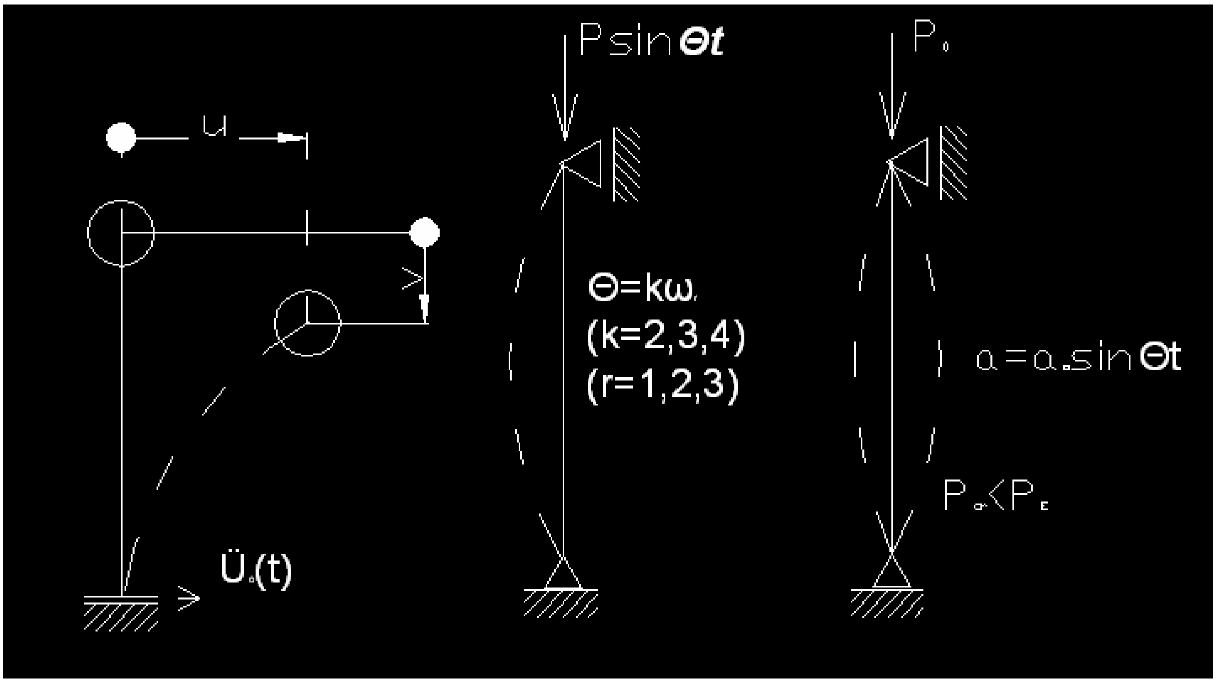

Figure 4 Column under axial load combined with oscillatory perturbation θ

An example of a non-linear system is the pendulum with an elastic wire which suffers a change of its parameters during the oscillation process. This kind of pendulum is a non-autonomous system visualized by means of the Hill type equation:

$$\ddot{\theta} + \omega(t)\theta = 0 \quad \text{Eq. 5}$$

For a non-autonomous linear system the following Equation 6 is the dynamic equilibrium :

$$M\ddot{x} + C(t)\dot{x} + K(t)x = F(t) \qquad \text{Eq. 6}$$

and gives the systems movement.

3 SECOND ORDER ANALYSIS OF THE STRUCTURAL RESPONSE

The P-Delta structural response leads to the collapse of the structure by increasing the moments and shear acting at the ends of the bars both columns and beams under seismic effects. All the codes for the seismic analysis deal in a way or in other with the structural behavior due to augmenting the static loads.
The Israeli Design Provisions for earthquake resistance of structures SI 413 takes into account the P –Delta effect by computing a coefficient that shows the ability to stand the drift the structure is going to get as result of the horizontal seismic load:

$$\theta_i = \frac{W\Delta_{el,i}K}{V_i h_i} \quad \text{Eq. 7}$$

Where:
$\Delta_{el,i}$ - the drift at the story “i” according to the elastic analysis.
V_i - the horizontal shear force at story i.
θ_i - the coefficient for the deformation at story i.
W – the total load over the floor i
K – the decrease coefficient of the loads acting on the structure.
h_i - the height for the story i.
According to SI 413 if the θ_i coefficient is less than 0.1 the structure is stable and no further analysis must be accomplished. If $0.10 < \theta_i \le 0.20$ a second order analysis must be done.
As concerns the vertical seismic oscillations certain elements must be loaded by vertical loads.
The structural stability is a function of the axial force which develops in the columns due to the amplification in vertical direction. According to Nakamura (2000) the vertical component of the tremor retains the characteristics of the horizontal tremor at the bedrock.
Based on this assumption the Eurocode 8 defines the vertical spectrum in correlation with the horizontal spectrum (Fig. 5)

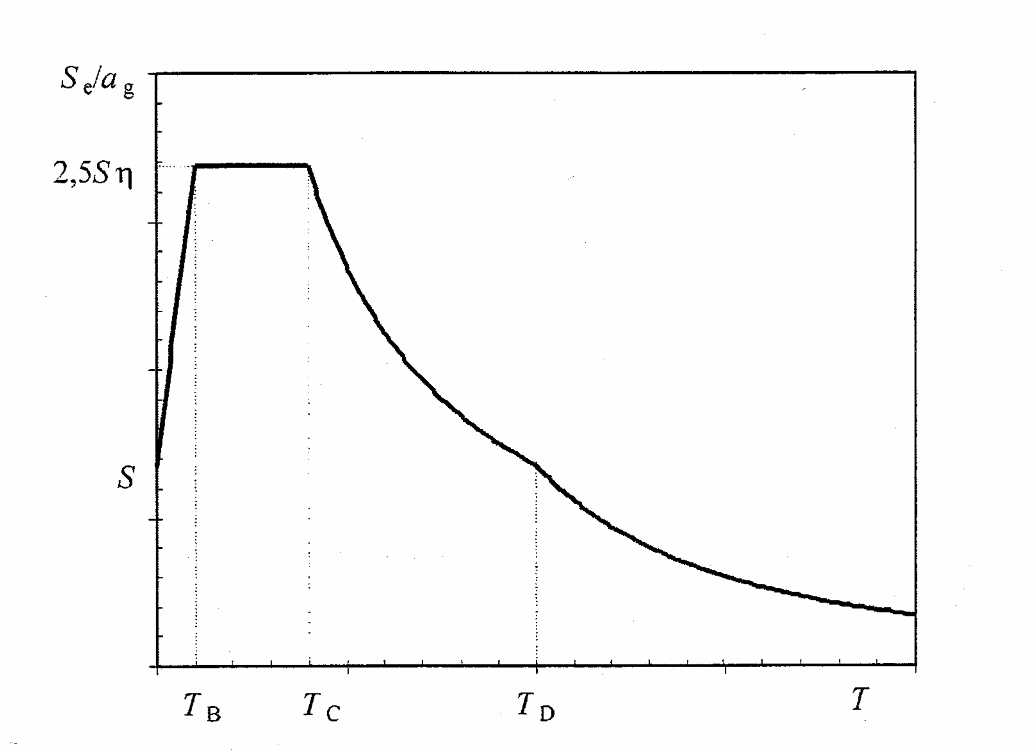

Figure 5 Shape of the elastic response spectrum

The vertical component of the seismic action shall be represented by an elatic response spectrum, $S_{ve}(T)$ derived by using the following expressions:

$$0 \le T \le T_B : S_{ve}(T) = a_{vg}\left[1 + \frac{T}{T_B}(\eta \bullet 3.0 - 1)\right] (3.8)$$

$$T_B \le T \le T_C : S_{ve}(T) = a_{vg}\eta 3.0 \ (3.9)$$

$$T_C \le T \le T_D : S_{ve}(T) = a_{vg}\eta 3.0\left[\frac{T_C}{T}\right] (3.10)$$

$$T_D \le T \le 4s : S_{ve}(T) = a_{vg}\eta 3.0\left[\frac{T_C T_D}{T^2}\right] (3.11)$$

The values to be ascribed to T_B, T_C, T_D *and* a_{vg} for each type (shape) of vertical spectrum may be found in its National Annex.

Table 1 Recommended values of parameters describing the vertical elastic response spectra

Spectrum	a_{vg}/a_g	$T_B(s)$	$T_C(s)$	$T_D(s)$
Type 1	0.90	0.05	0.15	1.0
Type 2	0.45	0.05	0.15	1.0

Where:
$S_e(T)$ is the elastic response spectrum;T is the vibration period of a linear single-degree-of-freedom system; a_g is the design ground acceleration on type A ground $(a_g = \gamma_I a_{gR})T_B$ is the lower limit of the period of the constant spectral acceleration branch;
T_C is the upper limit of the period of the constant spectral acceleration branch;
T_D is the value defining the beginning of the constant displacement response range of the spectrum;
S is the soil factor;
η is the damping correction factor with reference value of $\eta = 1$ for 5% viscous damping.

According ASCE/SEI 41-06 $P-\Delta$ effects shall be included in linear and nonlinear analysis procedures. For nonlinear procedures, static $P-\Delta$ effects shall be incorporated in the analysis by including in the mathematical model the nonlinear force-deformation relationship of all components subjected to axial forces.
Dynamic $P-\Delta$ effects are caused by a negative post-yield stiffness that increases story drift and the target displacement. The degree by which dynamic $P-\Delta$ effects increase displacements depends on the following:

1. The ratio of the negative post-yield stiffness to the effective elastic stiffness;
2. The fundamental period of the building;
3. The strength ratio, R;
4. The hysteretic load-deformation relations for each story;
5. The frequency characteristic of the ground motion;
6. The duration of the strong ground motion.

Because of the number of parameters involved, it is difficult to capture dynamic $P-\Delta$ effects in linear and nonlinear static analysis procedures. For the nonlinear static procedure, dynamic instability is measured by the strength ratio R. For the nonlinear dynamic procedure $P-\Delta$ effects are captured explicitly in the analysis.
The strength ratio R shall be calculated in accordance with Eq. 3-15:

$$R = \frac{S_a}{V_y / W} C_m \quad (Eq.3-15)$$

Where
S_a is the response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration;
V_y is the yield strength calculated using results of the nonlinear static procedure for the idealized nonlinear force-displacement curve developed for the building;
W is the seismic weight;
C_m is the effective mass factor. Alternatively, C_m is taken as the effective modal mass participation factor calculated for the fundamental mode using an Eigenvalue analysis.

C_m shall be taken as 1.0 if the fundamental period, T, is greater than 1.0 sec.

4 USE OF SAP2000 SOFTWARE

Building affected by seismic load both horizontally and vertically can be analyzed by means of SAP2000 software statically and dynamically. The static nonlinear methodology leads to the performance point which is placed at the intersection of the demand and capacity curves. If the capacity is determined by taking into account the $P-\Delta$ effect the capacity level is adjusted accordingly. The performance point indicates the influence of the vertical oscillations.
SAP2000 can be used for direct integration of the structural movement leading thus to the ability of the structure to withstand the seismic load till certain limit signaling the limit state of the structure.

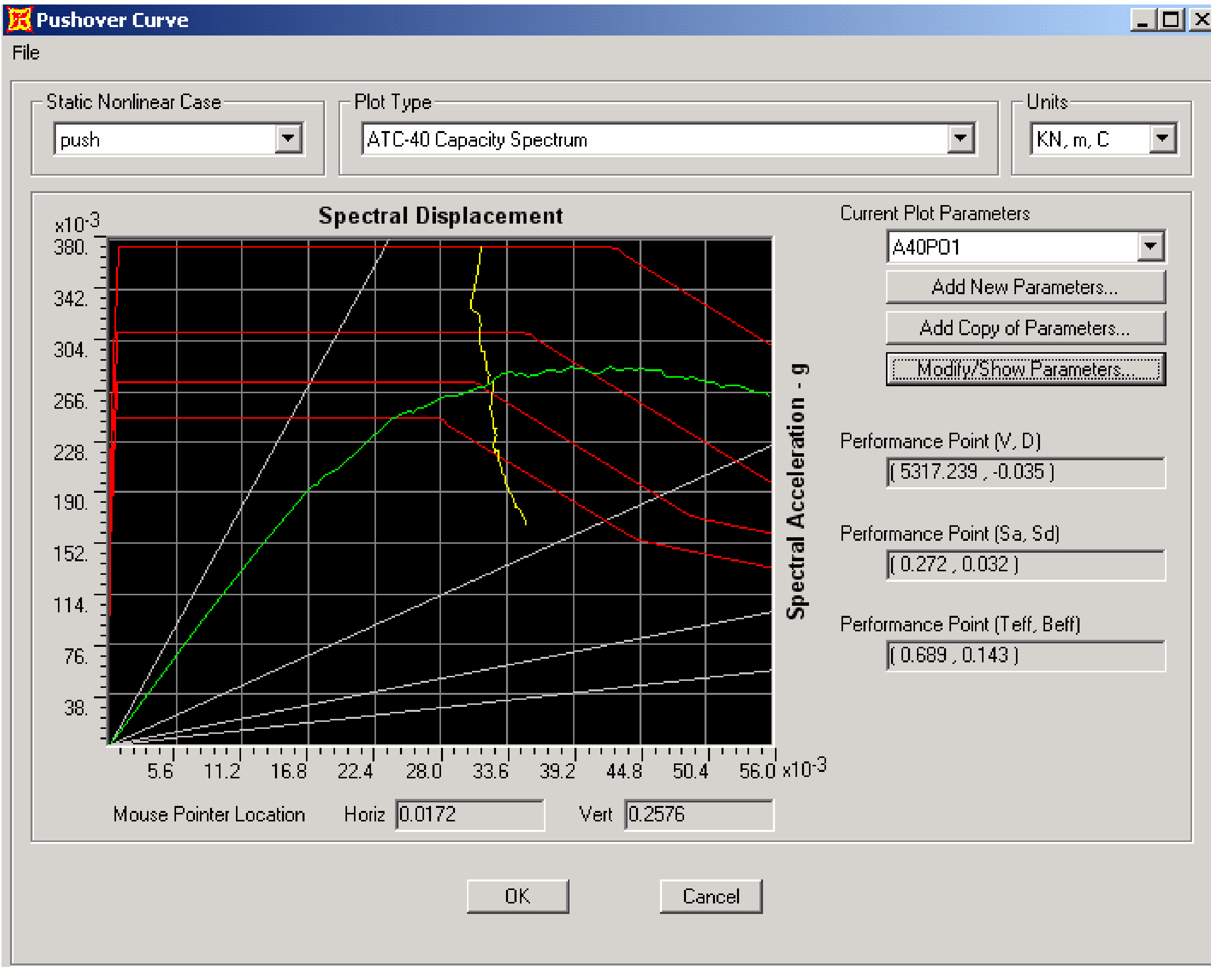

Figure 6 Static behavior under seismic load – performance point determination

5 CONCLUSIONS

Structural instability plays an important role in structural collapse. Since earthquakes are dynamic phenomena dynamic instability occurs during seismic excitations.

A thorough observation of the oscillations which occur during seismic excitations lead to the remark that both horizontal and vertical oscillations take place together.
$P-\Delta$ effects affect the stability of structures.
More, these effects are amplified during the oscillations leading to what is considered the dynamic instability due to the environmental factors and the structural abilities.
This work will be followed by further research and software for

6 REFERENCES

1 R. Marinov, Dynamic Stability Problems in Civil Engineering, Editura Technica –

2 SI 413 Design Provisions for Earthquake Resistance of Structures, 1998

3 Eurocode 8: Design of Structures for Earthquake Resistance 23.04.2004

4 SAP2000 Integrated Solution for Structural Analysis & Design, ver. 14 April 2009.

5 John Lawson, Reflections on the Effects of Vertical Seismic Acceleration, PCI Journal.