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Database-assisted Seismic Analysis of Tall Buildings Subjected to Long Predominant Period Vrancea Earthquakes

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Summary

Facing the new economic environment and public needs, there is an exploratory concern to develop modern tall building structures in Romania. The seismic hazard generated from the Vrancea source is the major concern for the practitioners, especially due to medium-soft soil conditions in the Bucharest-city. Tall buildings therefore might have the highest exposure to significant damage as well as economical and human losses.

The integrated seismic performance analysis system is an unitary format that allows (1) performing the seismic response analysis, energy balance-based analysis, fragility and seismic risk analyses, by using the time series of real/scaled/simulated earthquake ground motions and the induced effects (stresses, efforts, strains, displacements, velocities, accelerations, forces, energies), (2) the design of the structural elements and connections in a straightforward and transparent manner and (3) getting higher performance structures, safer and cost effective. This format becomes much more important in the case of irregular structures, having plan and elevation complex shape, in which the main directions of motion are not obvious.

The paper investigates the applicability of such procedure that makes it possible to estimate the earthquake-induced effects in tall buildings, at a higher level of automation and transparency. Seismic response analyses of a 60 story typical building subjected to long predominant period ground motions are conducted and the main results are emphasized.

KEYWORDS: tall building, database, bi-directional input ground motion, interaction formula, risk

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Mihail Iancovici, Cristian Gavrilescu

1. INTRODUCTION

Achieving modern business and residential infrastructure in Romania is necessary especially in the capital-city Bucharest. The increasing interest of business community and population- according to the new financial environment, to use building structures located in dense urban areas, exposed to multi-hazard natural/artificial sources (repeated strong ground motions, wind, blast, impact, fire etc.), would lead to the completion of tall buildings having high performance, safer and cost effective.

The seismic hazard generated from the Vrancea source in Bucharest is the major concern for practitioners, especially due to medium-soft soil conditions. Tall buildings might have the highest exposure to significant damage as well as economical and human losses.

The modern Romanian construction practice is represented by medium-rise buildings (10~12 stories). The design practice, as well as the construction technology and execution work of high-rise complex structures is well behind other developed, seismic prone countries practice (U.S., Japan, China etc).

The structural analysis methods and procedures have considerably improved due to increasingly computational capabilities.

The objective of the paper is to introduce the database-assisted analysis concept and the advantages of using it in the practical seismic design of large scale structures.

2. TALL BUILDINGS: OVERVIEW AND CURRENT SEISMIC DESIGN PRACTICE

Tall buildings are symbols of cities, the certainty of economic growth, of the force and image of a civilization. In the last centuries, more than in any other historical period, important structures were built mainly due to the modern technology development. What is happening nowadays in the world it is unique and has no precedent.

Dozens of modern cities are erected like over the night in China and India. In the Middle East there is a frenzy of constructions. In Dubai, Qatar, Kuwait, an ocean of cranes is erecting towers, as higher as sophisticated. New airports, museums, stadiums, public spaces and complex transportation and telecommunication networks are continuously developing.

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After years of stand-by in its own self-satisfactory historical conditions, Europe is cosmetizing its appearance with the most interesting concepts and urban developments known up to now.

United States is a model of the continuously changes in urban planning. Thus shortly after Chicago's supremacy in the battle over the sky, New York City takes the major role and produces one of the most innovative and beautiful tall buildings: *Woolworth Building (arch. Cass Gilbert, 1913)*- the tallest building in the world at that time, *Rockefeller Center (arch. Raymond Hood, 1940), Chrysler Building (arch. William Van Alen, 1930)*- a 319 m height famous art deco architectural example, *Empire State Building (arch. Shreve, Lamb and Harmon, 1931)*- the symbol of New York, 102 stories and 381 m height, for 41 years the tallest building in the world.

Today, from one to another side, the world is passing through the same phenomenal economic transformation. Naturally Romania must has not be out of this process, by seriously taking the initiative to develop super-tall buildings, integrated in modern urban developments.

The history of tall buildings in Romania begins in 1800, when the *Coltea Tower* (H=50 m) has been built and was destroyed by the strong earthquake of October 26, 1802 (M_w =8.1). In the modern era, the tallest reinforced concrete *Carlton Building* (14 stories) collapsed during the November 10, 1940 earthquake (M_w=7.6) and about 30 medium-rise buildings were destroyed at the March 4th , 1977 earthquake (M_w=7.4).

These important events have drawn the attention on the low accuracy level in reproducing real behavior of medium-tall buildings through standard design provisions. A summary of tall buildings built prior 2008 is given in figure 1 and table 1 (Iancovici, 2007).

Figure 1. Tall buildings in Bucharest, 2008

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Table 1. Summary of tall buildings in Bucharest, 2008

No.	Building	Construction yr.	Height, m	Structure
1	Free Press Building	1957	104	reinforced
2	Tower Center	2007	100	steel
3	*BRD* Tower	2001	87	reinforced
4	The Parliament Palace	1986	86	reinforced
5	Bucharest Financial	1997	83	reinforced
6	The Intercontinental	1970	77	reinforced
7	Millennium Business	2006	72	steel
8	Charles de Gaulle Plaza	2006	70	steel

While currently the tallest building in Romania still is the *Free Press Building* (1975) in Bucharest, the world tallest building is the *Taipei 101* from Taiwan (101 stories, 508 m, including the telecommunication system), about five times the height of the tallest building in Romania. However *Burj of Dubai Building* is expected to be in 2008 the tallest building ever built (~940 m).

Due to the lack of design and construction practice of tall buildings (>180m), there is no enough evidence on how these structures will perform under strong ground motions in the Bucharest soil condition and other exposed cities to seismic hazard.

The seismic design standards do not provide details about the analysis and design of tall buildings. The standard structural design is done in terms of static equivalent seismic loads, obtained from the absolute acceleration response spectra. Little guidance is provided on using time series of ground motions-either recorded or artificially generated, structural 3D modeling etc.; full analyses procedures in the *time*-domain are not yet available in a standardized manner while the time-history analysis tool is mostly used for structural behavior checking purposes.

The *structural seismic performance concept* (SEAOC, 1995) consists of tuning the structural and the earthquake ground motion properties in order to obtain a predictable behavior associated to various pre-defined limit states (no damage, minor damage, serviceability, reparability, safety) with a higher level of accuracy, making use of vulnerability and risk analyses.

In the *Romanian seismic design standard P100-2006* (EC 8 format), a single level of performance is stated. This addresses to the ultimate limit state (i.e. life safety) and the prescribed peak acceleration of ground (*PGA*) is related to a 100 years mean return interval (*MRI*), having 10% probability of exceedance. Any other performance level is not explicitly stated yet.

For tall buildings however, the current analysis procedure may lead to an inappropriate performance, resulting in highly exposed to seismic risk structures. The response spectrum method approach has no the capacity to accurately

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reproduce the high flexibility effect, structural mass and stiffness dissymmetry effect, damping distribution effect, geometric and physical nonlinear behavior, stability loss, the effect of non-structural elements interaction, of soil-structure interaction etc.

Therefore, a higher level of structural analysis and design format is needed, in order to more accurately predict the seismic performance, primarily using the large computational capabilities.

3. DATABASE-ASSISTED ANALYSIS AND DESIGN OF TALL BUILDINGS

Beginning with the *Moment distribution method* (Cross, 1930), continuing with the *Indeterminate coefficients method* (Filipescu, 1935), the *Direct stiffness matrix method* (Turner, 1958) and the effective computational algorithms based on the *finite element technique* (Courant, 1942; Clough, 1960), the structural analysis methods and procedures has considerably improved due to increasingly computational capabilities.

The integrated seismic structural performance analysis uses the concept called *database-assisted analysis and design*. Initially developed for the analysis and design of structures subjected to wind (Whalen et *al.*, 2000; Simiu et *al.*, 2003; Iancovici et *al.*, 2003), the concept can be adopted successfully for the earthquake ground motion case.

This is an unitary structural performance analysis format that allows (1) performing the seismic response analysis, energy balance-based, fragility and seismic risk analyses, using the time series of ground motion and the induced effects (stresses, efforts, strains, displacements, velocities, accelerations, forces, energies), (2) the design of structural elements and connections in a straightforward and transparent manner and (3) getting higher performance structures, safer and cost effective.

While some of the input ground motion components are usually neglected, the actual format allows the use of large number of full sets of accelerograms.

Ideally, a structural system must have equal performance to earthquake ground motions from all possible directions, resulting a uniformly exposed to risk structure. For a class of structures however, e.g. having non-uniform plan and elevation mass and stiffness distributions, the main directions of motion are not obvious. In order to cover all the structural cases, an incremental directivity of ground motion is considered (Iancovici et *al.*, 2006).

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Mihail Iancovici, Cristian Gavrilescu

In a *3D modal superposition method* any number of vibration modes can be considered. The induced effects are expressed as time-series. There is no need to consider static load combinations.

The design of structural elements and connections is done in a straightforward and transparent manner, directly from the time-histories of efforts; there is no more need of making use of various rules for maxima superposition (e.g. "*30%* " rule, SRSS, CQC, CQC3 etc.). The design is made by applying given interaction formulas (e.g. AISC-LRFD, EC3 etc.).

The use of complete sections databases allows the design with an improved level of automation. The distribution of material can be more efficient and economical.

This analysis format represents the decision support for structural type changing, of incorporating various seismic energy dissipation devices and techniques (active, semi-active, passive and hybrid) and for the assessment of the structural optimization need and effectiveness, as a higher analysis level.

The *integrated performance analysis system* deals directly with different classes of uncertainties in input ground motion and structural behavior. The use of a large number of data allows a full probabilistic assessment of structural performance, an appropriate approach from the point-of-view of implementing the performance concept.

This analysis format requires the use of:

(a) Input ground motion database,

By using a large number of time-series of ground acceleration components, either recorded or artificially generated. The input motion is fully described through amplitude, frequency and phase content, duration, power and energy content, in both *time* and *frequency* domains.

The amount of significant records, from the structural point-of-view, provided by the two seismic observation networks in Romania (INCERC, about 166 accelerometers installed in free field, in boreholes and buildings, about 55 in Bucharest; INFP-18 seismic stations) is relatively small but offers important information for the synthetic accelerograms simulation. These can be obtained by realistically scaling/simulating a large number of sets of accelerograms or series of accelerograms (called "pulse trains", Iancovici et *al.*, 2006) using available techniques (Gasparini et *al.*, 1976; Abrahamson, 2006).

Vertical and rotational component of the ground motion- mostly induced due to the non-synchronism of the spatial motion, can be simulated using similar techniques.

Such valuable information can be efficiently stored and represented using the GIS technology in the form of user-friendly databases.

(b) *Structural systems models database,*

Regarding the overall model, either a simplified one or a full FEM model.

The structural systems may be defined in terms of “classes”, consisting of dual systems: (i) moment resisting frames with braces – dedicated to medium heights range, (ii) central core and perimeter frames – dedicated to high-rise buildings and (iii) hybrid systems. The materials are usually high-strength reinforced concrete (60-120 MPa) and steel (800-1000 MPa). The constitutive laws of the structural elements are either analytical or experimental-based.

The effects of mass and stiffness plan and elevation dissymmetry can be primarily reproduced by the vibration modes correlation. One of the modes correlation descriptors is given by (Der Kiureghian, 1980)

$$\rho_{jk} = \frac{8\sqrt{\xi_j \xi_k}\,(\beta_{jk}\xi_j + \xi_k)\beta_{jk}^{3/2}}{(1-\beta_{jk}^2)^2 + 4\xi_j\xi_k\beta_{jk}(1+\beta_{jk}^2) + 4(\xi_j^2+\xi_k^2)\beta_{jk}^2} \tag{1}$$

where,

ξ_j, ξ_k are the damping ratios corresponding to the vibration modes j and k, β_{jk} is the ratio of the natural circular frequencies of the j^{th} and k^{th} vibration modes.

The results of experimental tests on actual buildings damping properties, revealed the dependency of damping ratio by the natural vibration frequency and amplitude. Assigning a specific value of modal damping ratios can be made using various proposed empirical relationships (Jeary, 1986; Lagomarsino, 1993), code proposals or experimental databases (e.g. *Japanese Damping Database*). The experimental databases however allow a higher level of accuracy in assigning the damping characteristic.

By given knowledge on the site conditions, on the initial structural system and the pre-defined performance levels, the analysis and design will provide realistic solutions for the structural system (fig. 2).

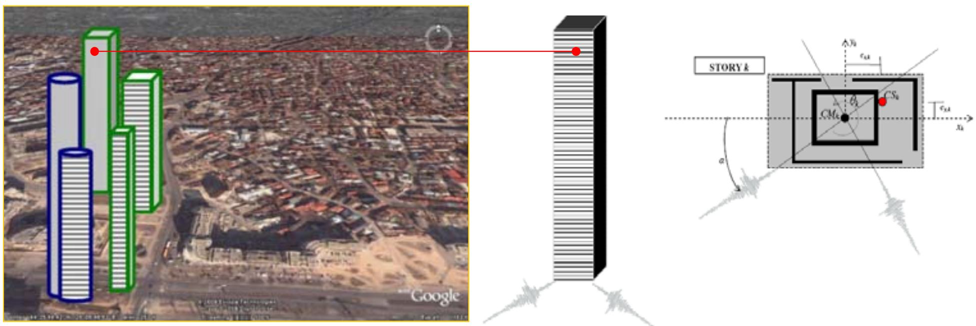

Figure 2. Tall building and typical story plan

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Case study: in order to illustrate the applicability of the concept, a standard tall building is considered (fig. 2).

The building is a 60 stories reinforced concrete structure and has a typical story plan shown in the fig. 2. The plan dimensions are L_x=*30m*, L_y=*18m* and the total height, *H=180m.*

For the sake of clarity, the model is assumed linear-elastic, having equal floor stiffness eccentricity in both directions -10% of radius of gyration, and 3 DOF/floor. A Rayleigh proportional damping model is used, the damping ratio for the first two translational vibration modes was assumed as 5%.

Using firstly a 30^{o} directionality increment orthogonal accelerograms components, recorded on March 4th , 1977 at the INCERC Bucharest site, in all structural elements are obtained the instantaneous induced effects (stresses, efforts, strains, displacements, velocities, accelerations, forces, energies) expressed vectorial in the form

$$E_{e,k,s,\alpha}(t) = f(e,k,s,\alpha,t) \qquad (2)$$

where *e* is the structural element, *k*- the story, *s* is the set of input motion components associated to a certain mean return interval of the earthquake ground

motion, α is the directionality angle of the ground motion set and *t* is time variable.

The structural eigendynamics is summarized in table 2, through the natural periods and the modal correlation coefficients (eq. 1) for the first 12 vibration modes.

Table 2: Vibration modes correlation coefficients, ρ_{ik}

Mode	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**Tn,s**
1	1.000	0.958	0.033	0.012	0.012	0.009	0.009	0.008	0.008	0.008	0.007	0.007	3.991
2	0.958	1.000	0.036	0.013	0.013	0.009	0.009	0.009	0.008	0.008	0.008	0.008	3.908
3	0.033	0.036	1.000	0.066	0.062	0.028	0.027	0.026	0.022	0.022	0.020	0.020	2.260
4	0.012	0.013	0.066	1.000	0.984	0.141	0.134	0.123	0.078	0.076	0.063	0.062	1.344
5	0.012	0.013	0.062	0.984	1.000	0.154	0.146	0.133	0.083	0.081	0.066	0.065	1.316
6	0.009	0.009	0.028	0.141	0.154	1.000	0.993	0.951	0.445	0.421	0.264	0.256	0.807
7	0.009	0.009	0.027	0.134	0.146	0.993	1.000	0.980	0.482	0.455	0.282	0.274	0.790
8	0.008	0.009	0.026	0.123	0.133	0.951	0.980	1.000	0.555	0.523	0.319	0.309	0.761
9	0.008	0.008	0.022	0.078	0.083	0.445	0.482	0.555	1.000	0.996	0.739	0.715	0.577
10	0.008	0.008	0.022	0.076	0.081	0.421	0.455	0.523	0.996	1.000	0.777	0.753	0.565
11	0.007	0.008	0.020	0.063	0.066	0.264	0.282	0.319	0.739	0.777	1.000	0.999	0.457
12	0.007	0.008	0.020	0.062	0.065	0.256	0.274	0.309	0.715	0.753	0.999	1.000	0.449

Considering the induced effects (2) in terms of shear forces, the time-histories in the *x*- and *y*- directions, corresponding to the 30^{o} directivity angle of the input ground motions, in a column at the 10th floor, are given in the fig. 3.

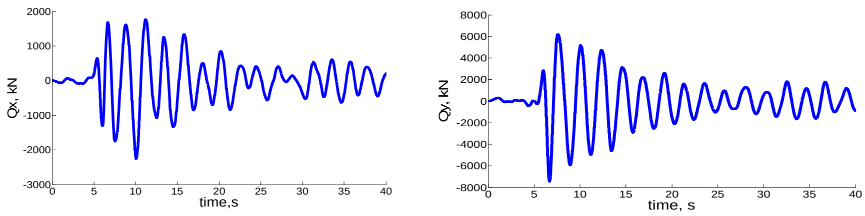

Figure 3. Time-histories of shears in the *x*- and *y*- directions in a typical column

In a similar way, all the time-histories of the induced effects (2) are obtained (efforts, displacements, velocities, accelerations, forces, energies).

The peaks of the base shear forces in the *x*- and *y*- directions in the same column are computed for all 12 directions of the input ground motion. Design values are obtained by practitioners and myriad standard structural analysis software, using common statistical peaks combination rules, e.g. *30% rule*, *SRSS rule* etc.

On the other hand, the time-history of the resultant shear force can be obtained by vectorial composition of time-instant of shears in the *x*- and *y*- directions. This is called the „exact approach”.

The analysis results are presented in the table 3.

Table 3: Design base shear forces in a typical column

Direction,deg.	QmaxX,kN	QmaxY, kN	Q30%	QSRSS	Qvect	Q30%/Qvect	QSRSS/Qvect
0	3463.646	11020.443	6769.779	10156.192	11193.529	0.60	0.91
30	2904.752	7811.405	5248.174	8158.876	8186.598	0.64	1.00
60	3260.544	10032.798	6270.383	10457.000	10152.069	0.62	1.03
90	3154.546	9260.352	5932.651	9782.908	9696.268	0.61	1.01
120	2939.764	8435.801	5470.505	8474.973	8526.183	0.64	0.99
150	3455.964	10523.778	6613.097	10650.323	10827.347	0.61	0.98
180	2732.178	7115.233	4866.748	7466.496	7326.798	0.66	1.02
210	3689.449	11319.372	7085.260	11905.469	11576.779	0.61	1.03
240	2685.088	6671.562	4686.557	7191.623	6897.677	0.68	1.04
270	3631.555	11369.439	7042.387	10240.517	11573.102	0.61	0.88
300	2784.443	7330.900	4983.713	7317.737	7505.929	0.66	0.97
330	3403.330	10670.601	6604.510	11161.437	10822.063	0.61	1.03

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One can be observed that while for some directions the standard combination rules give severe un-conservative results, for others are providing conservative results. The underestimation using *30% rule* goes up to 40% for this case.

The distribution of the peaks design shear forces in a column on the building height, are presented in the table 4, for the 30^{o} directivity angle, due to space limit, shown only for the first 25 stories

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Table 4: De sign shear forces in a typical column, on the building height, 30^{o} directivity

Story	QmaxX,kN	QmaxY, kN	Q30%	QSRSS	Qvect	Q30%/Qvect	QSRSS/Qvect
1	2904.75	7811.41	5248.17	8158.88	8186.60	0.64	1.00
2	2844.20	7765.46	5173.84	7921.92	8002.48	0.65	0.99
3	2780.74	7761.44	5109.17	7695.02	7991.07	0.64	0.96
4	2714.36	7748.34	5038.87	7482.00	7971.25	0.63	0.94
5	2645.02	7725.10	4962.55	7284.30	7941.74	0.62	0.92
6	2572.87	7691.02	4880.17	7098.97	7901.69	0.62	0.90
7	2497.99	7645.61	4791.67	6923.05	7850.50	0.61	0.88
8	2420.61	7588.60	4697.19	6754.68	7787.75	0.60	0.87
9	2340.91	7519.99	4596.90	6592.17	7713.48	0.60	0.85
10	2259.19	7439.85	4491.14	6434.43	7627.64	0.59	0.84
11	2175.64	7348.46	4380.18	6280.59	7530.34	0.58	0.83
12	2090.55	7246.05	4264.36	6129.71	7422.07	0.57	0.83
13	2004.16	7133.06	4144.08	5980.78	7303.15	0.57	0.82
14	1916.71	7009.98	4019.71	5838.28	7173.99	0.56	0.81
15	1828.48	6877.36	3891.68	5697.00	7035.17	0.55	0.81
16	1739.69	6735.81	3760.44	5555.53	6887.37	0.55	0.81
17	1650.61	6585.87	3626.37	5413.95	6731.01	0.54	0.80
18	1561.49	6428.24	3489.97	5272.59	6566.97	0.53	0.80
19	1472.58	6263.57	3351.66	5131.93	6395.70	0.52	0.80
20	1384.18	6092.56	3211.95	4992.58	6218.14	0.52	0.80
21	1296.50	5915.81	3071.25	4855.23	6034.85	0.51	0.80
22	1209.85	5734.20	2930.11	4720.60	5846.54	0.50	0.81
23	1140.32	5548.26	2804.79	4589.34	5653.88	0.50	0.81
24	1083.23	5358.71	2690.84	4462.00	5457.48	0.49	0.82
25	1024.32	5166.13	2574.16	4339.04	5258.25	0.49	0.83

In this case too, the *SRSS rule* gives better estimation of the maximum design shear forces, but the „exact approach" has to be used for analysis and design.

Usually the seismic performance limit states are basically controlled through demand peaks of drift ratios for all the input directions considered, in the *x*- and *y*-directions (fig. 4).

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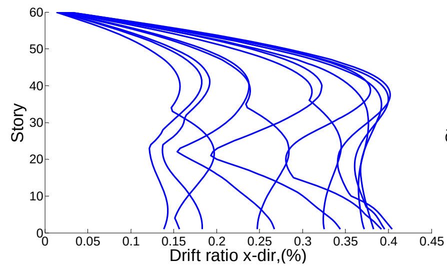

Figure 4. Peaks of drift ratios in the x- and y- directions, 12 input directions

There are a very limited number of records that can serve for the analysis format. Therefore a large number of artificial strong ground motion accelerograms were generated, with the aim of realistically reproduce similar amplitude, frequency content and duration as the real ones, recorded in Bucharest on soft soil conditions.

Artificial ground motions were simulated using the procedure described by Gasparini *et al.*(1976), who's response spectra match the 5% damped elastic acceleration response spectra, given in the Romanian seismic design code P100-2006 (fig.6).

A set of 10 synthetic bi-directional accelerograms having peak ground accelerations of 0.24g, associated to 100 years *MRI*, low predominant frequencies and small frequency bandwidth (Cartwright & Longuet-Higgins) were obtained and used in analyses. Typical orthogonal artificial accelerograms are presented in the fig.5.

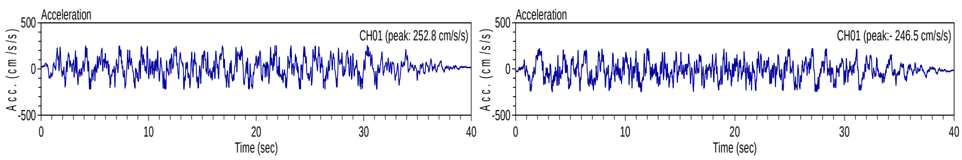

Figure 5. Typical bi-directional artificial input ground motion accelerograms

For the considered input set, the code-based and the computed normalized acceleration spectra and the corresponding normalized power spectral density functions (*PSD*) as well, are shown in the fig. 6.

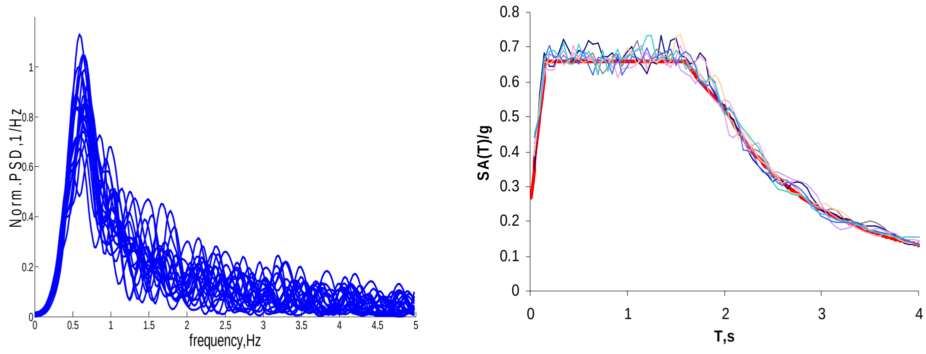

Figure 6. P100-2006 code spectra compatible accelerograms and corresponding normalized PSD functions, 10 sets of artificial accelerograms, PGA = 0.24g, MRI = 100 years

Using same technique, synthetic accelerograms corresponding to various hazard levels can be obtained. For instance for Bucharest, P100-2006 code provides a *PGA* value of 0.36g for ground motions having 475 years *MRI*.

For the generated sets of input ground motion corresponding to 30^{o} directionality angle, the drift ratios in the *x*- and *y*- directions are represented in the fig. 7.

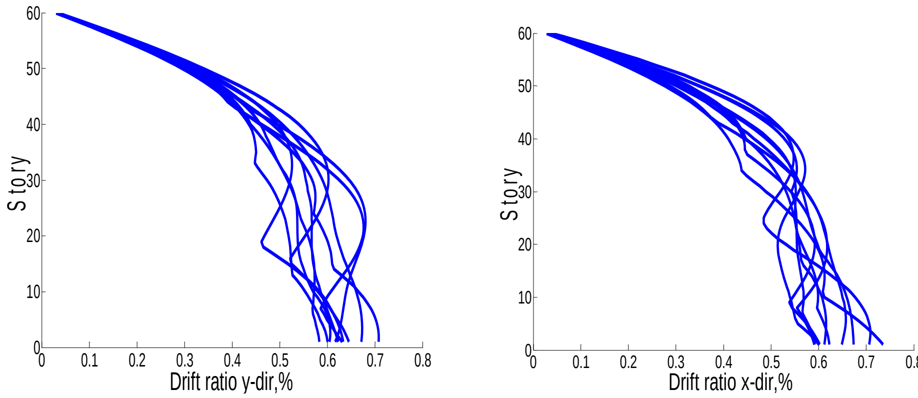

Figure 7. Drift ratios in *x*- and *y*-directions, 10 sets of bi-directional artificial accelerograms, PGA = 0.24g, MRI = 100 years

Large databases of input ground motions allow statistical analyses for high reliability design and associated risk analyses.

3.1. Structural design facts

The time-histories of the induced-efforts (axial forces, shear forces and bending moments) are used in design equations. For instance, for a steel column subjected to large axial load in *Load and Resistance Factor Design*, the following interaction formula has to be fulfilled

$$\frac{N_{e,k,s,\alpha}(t)}{\phi N_{ne}}+\frac{8}{9}\left(\frac{Mx_{e,k,s,\alpha}(t)}{\phi_b M_{nex}}+\frac{My_{e,k,s,\alpha}(t)}{\phi_b M_{ney}}\right)\leq 1 \qquad (3)$$

In the eq. 3, N_{ne}, M_{nex}, and M_{ney} are the nominal axial and flexural strengths of member e, ϕ and ϕ_b are the axial and flexural resistance factors (AISC, 2001), and the quantities in the numerators are the time-instant axial load and bending moments due to ground motion from direction α.

Similar formula is provided in the Eurocode 3 (section 6).
The structural design is done in a straightforward manner, directly form the time-series of the induced effects, by using the full capabilities of the dynamic analysis procedure and computational tool.

4. CONCLUSIONS

The *integrated performance analysis system* consists of a considerable amount of modeling and data processing effort. This is highly supported by large computational capabilities. The approach has clear advantages and represents a higher level, modern and effective tool for practitioners.

The analysis format removes the needs of the current analysis model simplification (e.g. neglecting some degrees of freedom, sub-structuring etc.), the use of the maxima superposition rules.

It was shown that this approach provides a higher accuracy in estimating the structural demand, resulting safer and cost effective structures.

The *integrated performance analysis system* needs further developments by adding full design module as well as integrated seismic vulnerability and risk evaluation modules.

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