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BLIND PREDICTION OF A FULL-SCALE 3D STEEL FRAME TESTED UNDER DYNAMIC CONDITIONS

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ABSTRACT

A blind analysis contest for a full-scale four-story building was announced in 2007 by the executive committee of the E-Defense steel building project, sponsored by the National Research Institute for Earth Science and Disaster Prevention in Japan. In this contest, each participant should predict the structural response before and after the test was performed using the three-dimensions shaking table located in Miki City, Hyogo Prefecture, Japan.
This paper presents the results obtained from dynamic analyses performed on the building using a fibre element-based program. By comparing the results given by the program with experimental ones, the aim of the work was to demonstrate that reasonably accurate estimation of the nonlinear dynamic response of steel buildings is possible, even by non-necessarily proficient steel structures analysts, as was the case of the authors.

KEYWORDS: steel structure, blind prediction, fibre element, dynamic nonlinear analysis

1. INTRODUCTION

In May 2007 the executive committee of the E-Defense steel building project (NRIESDP, 2007), sponsored by the National Research Institute for Earth Science and Disaster Prevention in Japan, announced the 2007 Blind Analysis Contest for a full-scale four-story steel building, which was tested to collapse in September 2007 on the world's largest three-dimensional shaking table located at Miki City, Hyogo Prefecture, Japan. The test was conducted by applying a scaled version of near-fault motion recorded during the 1995 Kobe earthquake. Each participant had to predict the response before and after the test. Because the actual seismic loads were determined during the course of the testing, the contest was organized in two parts: pre-test analysis based on anticipated earthquake loads, and post-test analysis using the actual loading.

On the base of collected data, a 3D model was constructed to be analyzed in a fibre-based Finite Elements computer program. Different modelling choices (mass discretization, hysteretic rules, damping) were studied in order to obtain the best representation of the real building. A pre-test nonlinear dynamic analysis was performed for an *incipient collapse level*. After the test was carried out, some supplemental data were provided (material test results on concrete and measured based acceleration). The analytical model was thus updated with the new data and post-test analysis was carried for three consecutive seismic levels: *elastic, incipient collapse and collapse level.* Results obtained through computer analyses were compared to experimental ones to assess the nonlinear response prediction capabilities of the adopted modelling strategy.

2. OUTLINE OF THE SPECIMEN

2.1 Main structure

The building is made of four-storey steel moment resisting frames (Figure 1). Along NS-direction there are two frames composed by two bays 5m long, whilst in EW-direction the frames are three, with one bay 6m long. Interstorey height is 3.5m and at the top of the building there is a parapet 0.9m height from the net height of the roof slab, for a total height of the building equal to 15.275m.

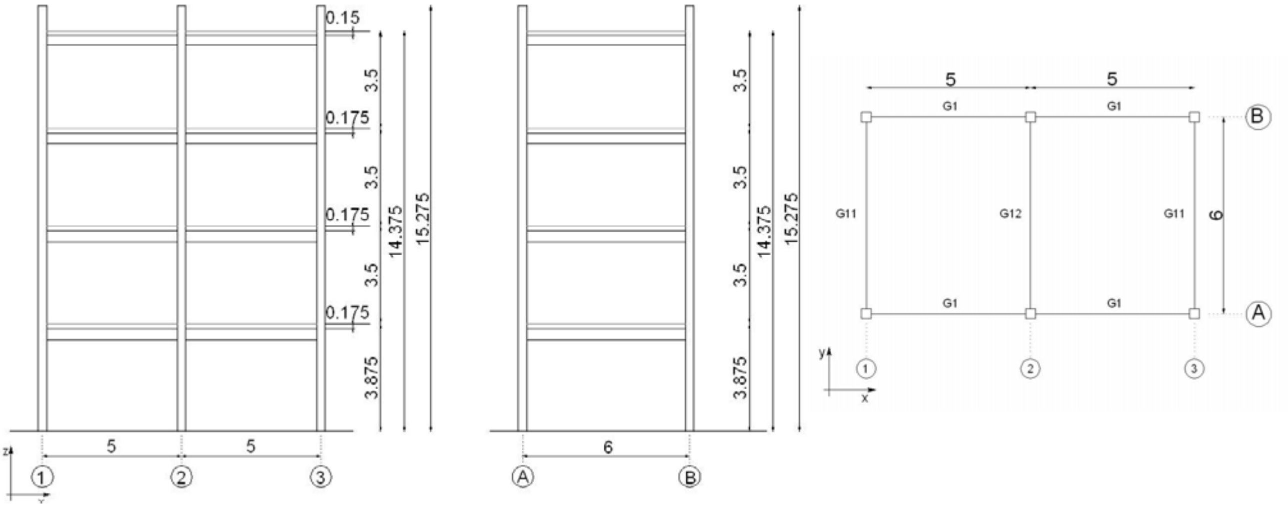

Figure 1 Main structure framing elevations (left) and typical plan (right) (NRIESDP, 2007)

The characteristics of the principal structural elements are given in Table 1, below. At each floor there are also secondary beams**,** the geometry of which varies at each floor level. Slabs at second, third and forth level consist in composite deck floor, 175mm height. Instead, roof floor is a reinforced concrete slab, with a flat steel deck at its bottom, for an overall thickness of 150mm. (see Figure 2)

All connections (see Figure 3) are made using details and fabrication practice developed following the 1995 Hyogoken-Nanbu earthquake. Column bases are connected to concrete blocks (1.5m height) with steel plates, anchored to them through steel bolts; these concrete blocks create the connection with the shaking table

Table 1 Members characteristics (NRIESDP, 2007)

Beam				**Column**	
Floor	G1	G11	G12	Story	
R	H- 346x174x6x9	H- 346x174x6x9	H- 346x174x6x9	4	RHS- 300x300x9
4	H- 350x175x7x11	H- 350x175x7x11	H- 340x175x9x14	3	RHS- 300x300x9
3	H- 396x199x7x11	H- 400x200x8x13	H- 400x200x8x13	2	RHS- 300x300x9
2	H- 400x200x8x13	H- 400x200x8x13	H- 390x200x10x16	1	RHS- 300x300x9
H- height x width x web thickness x flange thickness, RHS- height x width x thickness					

2.2 Non structural elements

External cladding consists of ALC (autoclaved, aerated concrete) panels, 0.125m thick, fixed on to the support beams at the top and the bottom of their edges. These panels are connected between them using the HDR (Hebel Dry Rocking) method, which does not use mortar but rabbet type panel that are allowed to rock in case of earthquake. Internal partitions are instead made of LGS (light gauge steel) backing board installed on aluminium frames. Glass windows with aluminium sash were installed within external ALC panels openings and reinforced by steel angles. A lightgauge steel was used to hang the ceiling panels at each floor. (see Figure 4)

3. STRUCTURAL MODELLING

All analysis have been carried out using SeismoStruct (SeismoSoft, 2007), a finite element analysis program used for seismic analysis of framed buildings. The software uses fibre beam-column elements, whereby material inelasticity is spread both along the element length and across its section depth. Its accuracy in predicting the cyclic response of steel frames had been shown by Pietra *et al*. (2006).

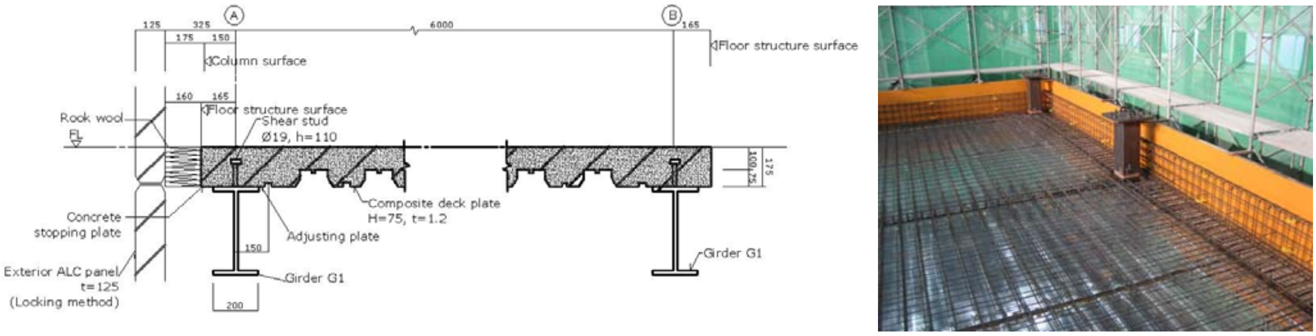

Figure 2 Details of intermediate floor slabs (left) and top floor slab (right) (NRIESDP, 2007)

Figure 3 Details of base and beam-column connections (NRIESDP, 2007)

Figure 4 Test frame before (left) and after (right) the introduction of non-structural panels (NRIESDP, 2007)

3.1 Gravity Loads on beams

The organizing committee supplied the spread floor weigh for each level. Subtracting the main structure self weight from the latter, it was thus possible to determine the additional mass that had to be applied on each beam element (Table 2). The tributary area of the girders was defined based on the geometry of the problem, assuming a distributed mass over the diaphragm. Mass was internally converted into vertical load by the software.

Table 2 Additional mass distributed on girders

Floor number j	Net Floor weight $W_{j\,net}$ (kN)	Element	Tributary Area (m^2)	Load on the element (kN)	Length (m)	Load (kN/m)	Additional Mass (ton/m)
5	437.028	RG1	6.2511	45.532	5	11.326	1.155
		RG11	8.7489	63.725	6	12.841	1.309
		RG12	17.4978	127.450	6	21.242	2.165
4	356.401	4G1	6.2511	37.132	5	7.426	0.757
		4G11	8.7489	51.969	6	8.661	0.883
		4G12	17.4978	103.937	6	17.323	1.766
3	342.675	3G1	6.2511	35.702	5	7.140	0.728
		3G11	8.7489	49.967	6	8.328	0.849
		3G12	17.4978	99.934	6	16.656	1.698
2	340.870	2G1	6.2511	35.514	5	7.103	0.724
		2G11	8.7489	49.704	6	8.284	0.844
		2G12	17.4978	99.408	6	16.568	1.689

3.2 Materials

3.2.1 Concrete

In the computer model, a nonlinear constant confinement concrete model (Mander et al, 1988) was adopted for the concrete of slabs. It is a uniaxial model in which a constant confining pressure is assumed through the entire stress-strain range. Five parameters that characterized the model were calibrated. For the cylinder compressive strength f_{cu} and the strain at peak stress ε_c, data spread by the contest committee were used (Table 3). The tensile strength f_t was considered to be equal to 1/10 of the compressive one. The constant confinement factor k_c, used to scale up the stress-strain relationship throughout all the strain range and defined as the ratio between the confined and the unconfined compressive stress of the concrete, was set equal to 1.0 for unconfined concrete and 1.2 for confined one. Finally, the concrete specific weight was specified to be equal to 24kN/m^3.

Table 3 Characteristic parameters for concrete (NRIESDP, 2007)

Floor	f_{cu} (MPa)	f_t (MPa)	ε_c
2-nd	33.53	3.35	0.00154
3-rd	28.8	2.88	0.0012
4-th	27.54	2.754	0.00127
Roof	35.9	3.59	0.002467

3.2.2 Steel

For steel elements all experimental data regarding constitutive law were released by the committee. Two different kinds of steel were utilized in the specimen: SN400B for beams and BCR295 for column. The probable yield stress for SN400B was 300 MPa while for BCR295 was 380 MPa.

In the FE model, a bilinear curve was adopted to represent the constitutive law for the steel and to offer the best linear regression of the experimental response of each element. The yield point was defined by the given data, whist the hardening parameter *r* was computed so as to render the area under the bilinear stress-strain relationship equal to that of the experimental material response curve. The model parameters are given in Table 4, below.

Table 4 Steel bi-liner constitutive model characteristic parameters

Element	**E (MPa)**	**σ_y (MPa)**	**r**
Column 1	200000	330	0.05
Column 2	200000	332	0.05
2G1-2G11-3G11-3G12	203750	326	0.12
4G12	212830	308.6	0.1
RG1-RG11-RG12	201820	333	0.15
4G1-4G11	223480	301.7	0.13
2G12	174770	279.1	0.2
3G1	199550	311.1	0.18

3.3 Connections

The contest committee did provide a series results on of preliminary cyclic tests of beam and column assemblies, which could, in principle, be employed to define and calibrate relatively accurate beam-column joint models. Lack of time, and some difficulty in interpreting the experimental results, lead however to the pragmatic decision of not introducing any sort of advanced joint modeling. In other words, the decision was to try to maintain the model as simple as possible, neglecting the deformation given by panel zone yielding and column base buckling, and assuming that such absence could perhaps be somehow "compensated" by (i) the deformation of the column and beam elements, extended all the way to the intersection point of their respective section baricentres, in the panel zone, (ii) and by the non modeling of the internal infill panels (which could have stiffened the structure a tiny bit).

3.4 Damping

Hysteretic damping was implicitly accounted for by the cyclic material relationships described above, employed in the characterization of each individual fiber of the elements cross-sections. In addition to this, a small amount of non-hysteretic equivalent viscous damping (0.5%) was also introduced, for the whole structure, with a view to somehow model sources of energy dissipation that were otherwise not accounted for (e.g. infill panel interaction) and also to improve the numerical stability of the dynamic nonlinear analyses. A tangent stiffness-proportional viscous damping model was employed, as suggested by Priesltey and Grant (2005) and Hall (2006).

3.5 Nodal Constraints and Integration Algorithm

Slabs were modelled as rigid diaphragms with no bending out-of-plane stiffness. At every floor level, therefore, nodes were connected with rigid horizontal links. The HHT numerical integration algorithm (Hilber *et al.*, 1977) was employed, with an alpha coefficient of -0.1.

4. LABORATORY TEST, INPUT MOTIONS, AND REQUESTED ANALYSES

The building was shaken and taken to collapse through the introduction of a scaled version of the near-fault motion recorded in Takatori during the 1995 Kobe earthquake. More 3D shaking table tests were performed consecutively with increasing levels of seismic motion to evaluate the effect of plastic deformation: Takatori scaled to 40% (*elastic level*), Takatori scaled to 60% (*incipient collapse level*) and Takatori in full scale (*collapse level*).

Contest participants were asked to predict the response before and after the test. All analysis results data (e.g. interstory drift, floor displacement, storey shear) had to be submitted for incipient collapse level, whilst for the collapse level participants were instead asked to evaluate the *time* of collapse (defined as the time at which the interstory drift angle, both in x or y-direction, reached 0.13 or -0.13 rad).

4.1 Pre-Test analysis

Pre-test analysis was conducted for an *incipient collapse level,* immediately prior to the building collapse. For this seismic level, the *recorded* ground motion intensity was scaled to 60%. It was required to apply the North-South component parallel to the longest side of the building and East-West component parallel to the shortest one.

4.2 Post-Test analysis

For post-test response prediction, *actual* acceleration records of the shaking table were used. Time history analysis was carried out for all three ground motion intensities consecutively. In the model, ten seconds of delay were left between one motion and the one immediately after (Figure 5). In these lapses of time the building was expected to move in free damped vibration.

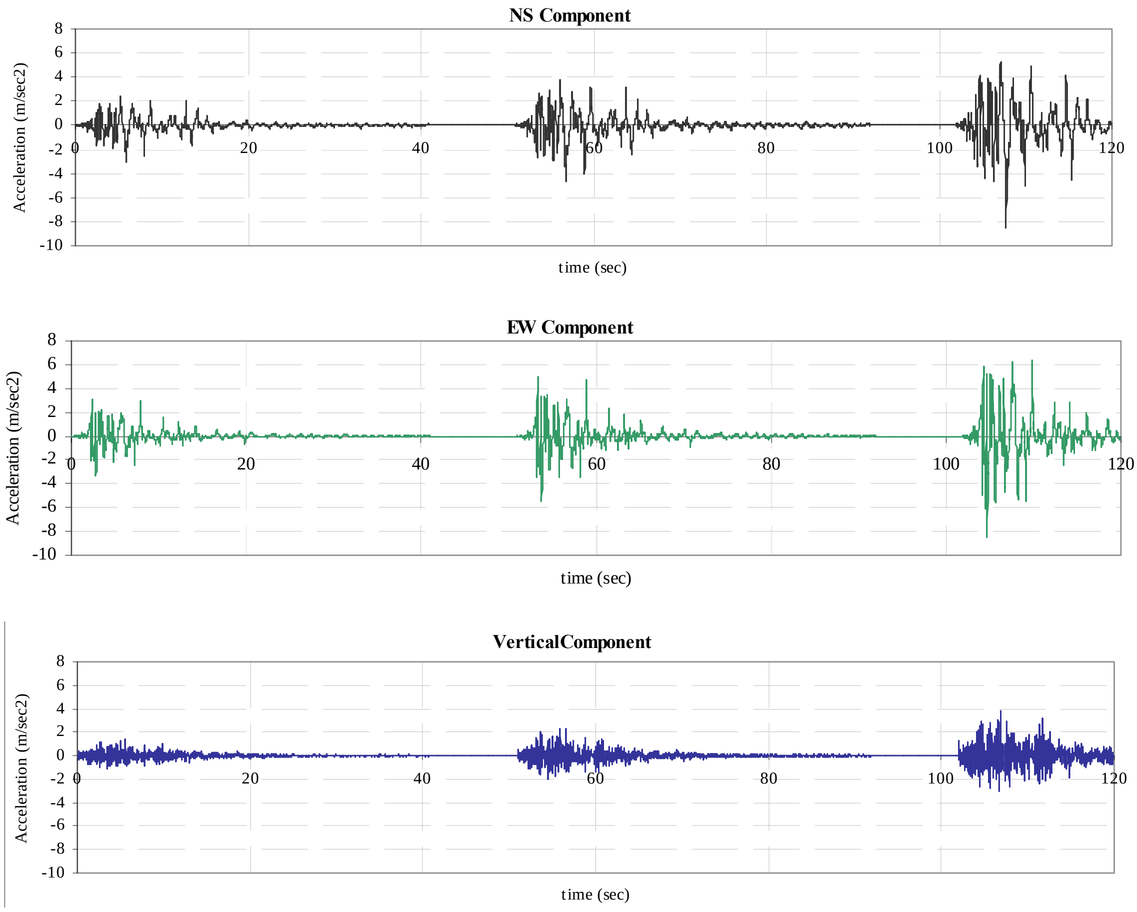

Figure 5 Post-test shaking table acceleration time-histories

5. COMPARISON OF THE RESULTS

As previously mentioned, one of the predictions requested by the contest organizers regarded the expected time of "collapse" (Figure 6), which took place at t=6.14 seconds, during the test, against the 5.98 seconds estimated during the analysis.

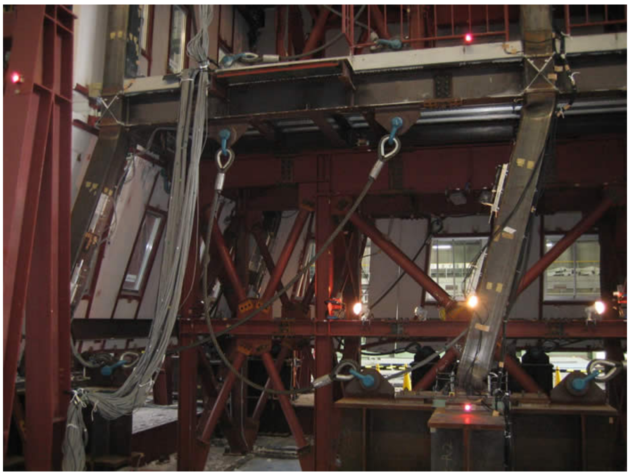

Figure 6 "Collapse" of the structures during the experiment (left) and in the analysis (right)

Another experimental vs. analytical comparison involved the values of displacements and shear forces at each storey, compared in Figures 7 and 8, respectively. It is observed that once the actual input motion was considered in the analyses ("post-test" results), the numerical predictions matched relatively well the experimental results, perhaps with the exception of the ground-storey forces.

Other storey response parameters that were considered in these contest included absolute accelerations, maximum and residual drifts (Pavan, 2008). For these quantities, the numerical predictions were less accurate, somehow rendering evident that the absence of a detailed modeling of the beam-column connections and of non-structural elements (e.g. infill panels) did affect the accuracy of the model, especially for these slightly more local response parameters.

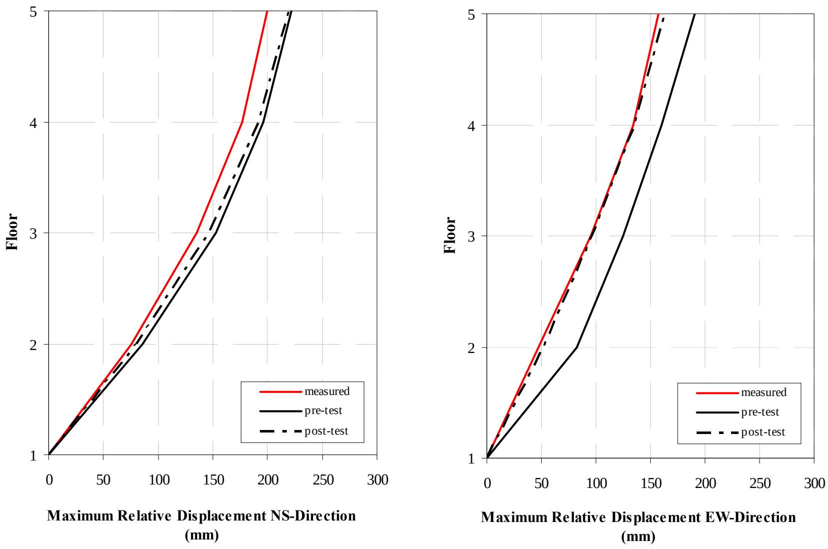

Figure 7 Maximum relative displacements at every level in the two directions

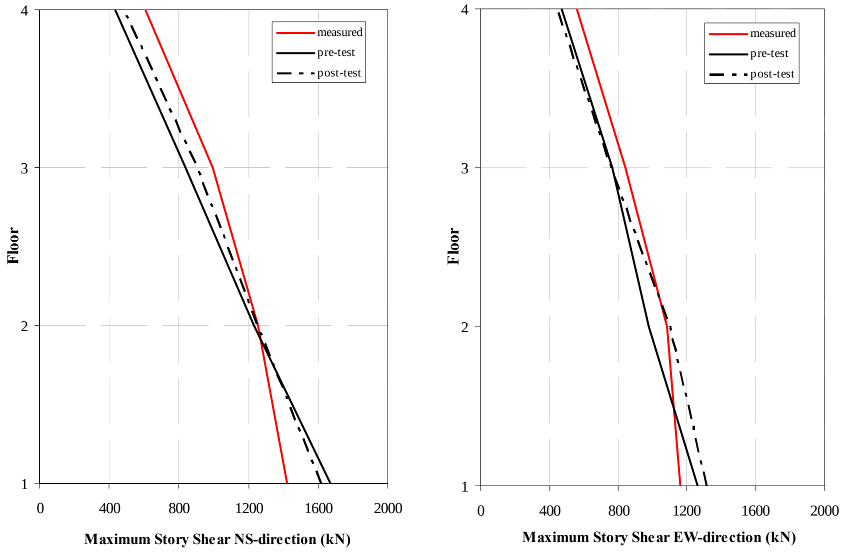

Figure 8 Maximum value of story shear in the two directions

CONCLUSIONS

This work described the blind prediction of the dynamic response of a full-scale steel building tested at the Hyogo Earthquake Engineering Research Center (E-Defense) under the auspices of the National Research Institute for Earth Science and Disaster Prevention. The analyses were carried out essentially by the first author, a graduate student with no previous experience on nonlinear modeling of steel structures, with relatively minor supervision by her two supervisors. The results show that the employment of a fibre modeling strategy did nonetheless cater for the attainment of a relatively accurate prediction of the global nonlinear dynamic response of the building in question, which is certainly reassuring. Clearly, however, the numerical results could be improved further if an explicit modeling of the panel zone deformation and of the presence of the infill panels, both of which not considered in this work, would be included.

REFERENCES

Hall J.F. (2006). Problems encountered from the use (or misuse) of Rayleigh damping. *Earthquake Engineering and Structural Dynamic*: **35**, 525-545.

Hilber H.M., Hughes T.J.R., Taylor R.L. (1977). Improved numerical dissipation for time integration algorithms in structural dynamics. *Earthquake Engineering and Structural Dynamics:* **5(3)**, 283-292.

Mander J.B., Priestley M.J.N., Park R. (1988). Theoretical stress-strain model for confined concrete. *ASCE Journal of Structural Engineering*: **114(8)**, 1804-1826.

National Research Institute for Earth Science and Disaster Prevention (2007). Hyogo Earthquake Engineering Research Center. Blind Analysis Contest. URL: www.bosai.go.jp/hyogo/ehyogo/index.html.

Pavan A. (2008). Blind Prediction of a Full-Scale 3D Steel Frame Tested under Dynamic Conditions. MSc Dissertation, Centre for Post-Graduate Training and Research in Earthquake Engineering and Engineering Seismology (ROSE School), Pavia, Italy.

Pietra D., Pinho R., Antoniou S. (2006). Verification of displacement-based adaptive pushover for seismic assessment of high-rise steel buildings. *Proceedings of the First European Conference on Earthquake Engineering and Seismology*, Geneva, Switzerland, Paper no. 956.

Priestley M.J.N., Grant D.N. (2005) Viscous damping in seismic design and analysis. *Journal of Earthquake Engineering*: **9(SP2)**, 229-255.

SeismoSoft (2007) SeismoStruct - A Computer Program for Static and Dynamic Nonlinear Analysis of Framed Structures. Available from URL: www.seismosoft.com.