21st International Conference ENGINEERING MECHANICS 2015
Svratka, Czech Republic, May 11 –14, 2015
Paper #196, pp. 18–19

Changes of Dynamic Properties of a Timber Frame Due to Simulated Seismic Load: A Case Study

Jan Bayer[1,a]*, Stanislav Pospíšil[1], Shota Urushadze[1], Bohumil Kasal[2]
[1] ITAM AS CR, v. v. i., Prosecká 809/76, 190 00 Praha 9, Czech Republic
[2] Fraunhofer Wilhelm-Klauditz Institut, Bienroder Weg 54E, 38108 Braunschweig, Germany
[a] bayer@itam.cas.cz

Keywords: modal analysis, frequency response function, structural health monitoring, timber frame, stiffened joints

Abstract: The data from a laboratory seismic resistance test of a three-story timber frame stiffened with steel joints were used to compare sweep and random excitation techniques on the platform of experimental modal analysis. The changes of mechanical properties due to increasing seismic load of the timber structure are demonstrated on corresponding changes of vibration modes and on changes of averaged frequency response functions. The comparison of averaged frequency response functions turned out to be at least as efficient tool for damage indication as the comparison of modal properties in this case. The experiences obtained in this case study are aimed at increasing the efficiency of dynamic tests and methods of Structural Health Monitoring.

Evaluation of experiments

In the scope of the research project Seventh Framework Program of EU [FP7/2007-2013] for access to the dynamic testing facility of the University of Bristol, UK, under grant agreement No.227887 [1], the acquired data from a seismic loading test were used for subsequent analysis. X-modal-III software of the University of Cincinnati was applied for the evaluation [2].

The "Output-only" measurements, as it is the usual case in the practice, using sweep excitation and random excitation of the intact structure were available for the evaluation. Fourteen modes could be extracted from the sweep test and only 7 from the random test. There are differences of identified natural frequencies between the two tests from +0,54 to -2,01 % with the average difference of approximately 1,3% which is explainable through the nonlinear behavior of the timber frame, because the intensity of loading was higher in the case of the random test.

The wooden frame was anchored on the vibration table which simulated the seismic load in four subsequent cycles with the intensity of 0.1g, 0.3g, 0.5g, 1.0g, and 2.0g. During the last test (2.0 g) the structure failed. Experimental modal analysis using random excitation test was carried out after each loading step in order to monitor the progressive damage. The results are summarized in the Table 1.

If we assume the uncertainty of identified modal frequencies about ±1% we can see that clearly apparent changes occurred just after the application of the seismic load with the intensity of 1g (N7). The evaluation of damping for the purposes of damage detection did not seem to be promising. Attention was also dedicated to damage indication from averaged frequency response functions, which seems to be simpler and not less reliable than the modal analysis. However, it is possible to find measured special quantities like frequency response functions of measured strains at particular locations that reflect the progressing damage since the first load step (see Fig.1).

Tab. 1: Identified vibration modes and their changes along with loading history

N4- 0.1 g		N5- 0.3 g		N6- 0.5 g		N7- 1.0 g		Frequency: difference [%]			Damping: difference [%]		
[Hz]	ν [%]	[Hz]	ν [%]	[Hz]	ν [%]	[Hz]	ν [%]	N5	N6	N7	N5	N6	N7
4,29	*16,97*	4,21	*17,18*	4,24	*13,23*	3,49	*12,76*	-1,93	-1,23	-18,71	*1,20*	*-22,04*	*-24,81*
5,39	*2,98*	5,37	*3,30*	5,30	*3,47*	4,74	*6,51*	-0,43	-1,61	-12,01	*10,72*	*16,50*	*118,78*
15,67	*3,87*	15,70	*3,55*	15,67	*2,86*	14,50	*5,32*	0,22	0,01	-7,44	*-8,31*	*-26,30*	*37,22*
16,59	*3,16*	16,50	*3,59*	16,39	*3,32*			-0,58	-1,21		*13,56*	*4,93*	*-64,22*
18,99	*3,23*												
21,36	*1,05*	21,38	*0,91*	21,42	*1,09*	21,15	*1,13*	0,10	0,25	-0,99	*-13,36*	*4,01*	*22,04*
21,82	*1,44*	21,79	*1,46*	21,72	*1,31*			-0,13	-0,44		*0,97*	*-9,02*	
25,29	*1,00*	25,17	*1,05*	25,14	*1,15*	25,25	*1,28*	-0,50	-0,61	-0,15	*4,91*	*15,53*	*69,64*
27,60	*1,74*			27,59	*2,17*				-0,04			*24,56*	
28,86	*2,04*	28,79	*1,84*	28,88	*1,44*	28,50	*1,69*	-0,25	0,07	-1,26	*-9,75*	*-29,40*	
31,37	*1,20*												*-53,88*
33,44	*1,36*	33,54	*1,20*	33,06	*0,56*			0,31	-1,12		*-11,41*	*-58,76*	
34,07	*1,39*			33,86	*2,70*	32,89	*0,55*		-0,64	-3,47		*93,96*	*63,17*
38,07	*2,04*	38,31	*1,74*	38,28	*1,28*			0,63	0,55		*-14,91*	*-37,27*	
39,70	*2,59*	39,72	*2,40*	39,93	*1,88*	39,61	*2,27*	0,07	0,59	-0,22	*-7,11*	*-27,32*	
Average changes:								**-0,23**	**-0,42**	**-5,53**	***-3,05***	***-3,89***	***20,99***

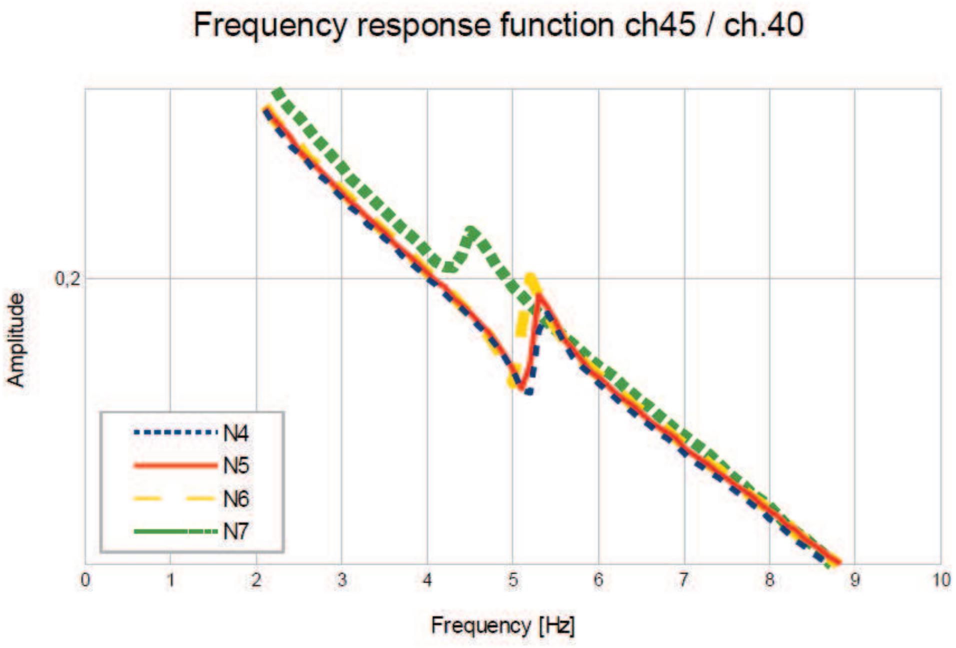

Fig. 1: Frequency response function between the strains of the first column and the accelerations of the third floor in the direction X.

References

[1] Bohumil Kasal, Andreas Heiduschke, Stanislav Pospíšil, Shota Urushadze, Shake table tests of a three-storey spatial timber frame with moment connections, World conference on timber engineering, Riva del Garda, Italy, 2010

[2] R.J. Allemang, The Modal Assurance Criterion – Twenty Years of Use and Abuse, Sound and Vibration, Aug. 2003, 14 – 21.