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VIBRATION-BASED IDENTIFICATION AND ASSESSMENT OF A 3D COMPOSITE FRAME STRUCTURE AT DIFFERENT DAMAGE LEVELS

W. SALVATORE[1], O. S. BURSI[2], M. MOLINARI[3], L. NARDINI[4], D. ZONTA[5]

SUMMARY

In seismic design, partial-strength composite steel-concrete moment resisting (MR) frame structures represent an open research field, both from a theoretical and an experimental standpoint. This paper presents the first results of vibration experiments, carried out with the scope of identifying changes in the dynamic response of a MR frame structure, following to the application of pseudo-dynamic (PsD) loadings. The specimen has been constructed and tested at the European Laboratory for Structural Assessment (ELSA) of the Joint Research Centre (JRC) in Ispra, and consists of a full-scale two-storeys two-bays frame presenting 12.8 m by 7.4 m dimensions in plan and 7.0 m in height. Resistance to lateral seismic loads is provided by three parallel MR frames in the main direction and by a bracing system in the orthogonal direction. Frames are made of composite beams and partially encased composite columns, connected by an innovative partial strength joint, and have been designed according to Eurocode 8 (EC8) rules. The testing program includes a sequence of PsD tests, simulating earthquakes with peak ground acceleration (pga) scaled up to the collapse limit state, followed by a final cyclic test. A dynamic characterization of the structure has been carried out before damaging the structure; it has been repeated after performing the ultimate limit state PsD test; and it has again been repeated after the PsD test at the collapse limit state and the final cyclic test. During each testing phase, the structure has been instrumented in three different configurations: the first one aimed at capturing the overall response of the building, while the other two focused on the characteristics of an interior and an exterior beam-to-column joints. Each characterization phase included shock tests, performed using an instrumented sledge-hammer, and stepped-sine tests, executed by means of an electromagnetic shaker. Data processing allowed the dynamic characterization of the overall structure from a global viewpoint to be performed and the variation of modal frequencies and damping ratio to be estimated.

STRUCTURAL ASSESSMENT BY VIBRATION-BASED TESTING

Vibration-based structural monitoring is widely spread in the last years for the effectiveness both in the approval and verification of structures and in their survey and control. Dynamic proprieties are function of

[1] *Assistant Professor, Dept. of Struct. Eng., Univ. of Pisa, Italy, walter@ing.unipi.it*
[2] *Full Professor, Dept. of Mech. and Struct. Eng., Univ. of Trento, Italy, oreste.bursi@ing.unitn.it*
[3] *Ph. D. Student, Dept. of Mech. and Structural Eng., Univ. of Trento, Italy, marco.molinari@ing.unitn.it*
[4] *Ph. D. Student, Dept. of Struct. Eng., University of Pisa, Italy, l.nardini@ing.unipi.it*
[5] *Assistant Professor, Dept. of Mech. and Struct. Eng., Univ. of Trento, Italy, daniele.zonta@ing.unitn.it*

mechanical characteristics and therefore their variation, like for example damage, reflects itself on the global dynamic behaviour of the structure. Hence, it is possible to evaluate any damage occurred both as a consequence of exceptional actions like earthquakes, explosions or accidental impacts and as derived from the usual service conditions of the structure.

So far, various dynamic experimental methods have been developed in several engineering fields, more diffusely applied to mechanical or aerospace structures [1, 2].

At the beginning, dynamic techniques received a decisive impulse in the field of oil search infrastructures realization and in aeronautic constructions. In the first area they were employed for monitoring the conservation state of off-shore platforms, whose immersed parts turn out inaccessible to visual inspections. In the field of aeronautic constructions they allowed to verify the conditions of structural parts of aircrafts during flight. At last in the field of mechanical constructions, they were applied to the control of rotating elements and suspension devices.

Recently, the interest has remarkably grown even in the ambit of civil constructions, mainly thanks to the success obtained in the applications to single structural elements [3-5], for evaluating material properties or localizing possible imperfections, such as damage or cracks.

For in situ monitoring, one of the most interesting field is that of bridges [2, 6, 7]; more complicated appeared the applications to buildings, often for the greater difficulty in the realization of precise mechanical models of existing buildings.

In Italy, the recent analyses carried out on historical and monumental interest's constructions, like the Basilica of Santa Maria di Collemaggio [8], the bell tower of SS. Annunziata church of Roccaverano [9], the Arena of Verona [10], demonstrated the potentialities of this dynamic approach like a tool for safety level assessment. In this ambit, such a use can turn out particularly effective in the case in which the objective of the inquiry is the seismic verification and the localization of any damage occurred as a result of previous earthquakes. An example is constituted by the studies performed on buildings of considerable interest, like schools, hospitals, carried out in the ambit of the Project of the Seismic Observatory of Structures (POSS) of the Italian National Seismic Service [9] [11].

In spite of the potentialities demonstrated in practice, the analysis of applications carried out till now clearly demonstrates that it is impossible to define one general methodology; conversely, a particular approach can appear more effective than others depending on the examined construction, so that it is compulsorily referring to the particular structural typology requirements in research techniques.

In detail in the analysis of frame structures, vibration-based monitoring must focus on the evaluation of floor stiffness and on the damage occurred to beams [12] or to beam-to-column joints [13] [14].

In general, it is possible to utilize numerical model updating techniques and to simulate damage by means of variations of model parameters: reduction of the resisting section; reduction of elastic modulus [15] [16]; and modification of the connection degree between structural elements [17].

Only few researches exist in which numerical simulations have been validated via experimental tests [18], nearly never utilizing full-scale specimens.

An example is constituted by the PRESSS project [19, 20], in which the dynamic response obtained from a precast reinforced concrete structure has been analysed before and after PsD tests, utilizing techniques based on the variation of the modal curvature in view of damage detection [21].

Accomplished efforts have remarkably improved effectiveness and reliability of dynamic measures; however there are still many open problems, essentially connected to an efficient definition of the damage index [22]. Therefore adequate monitoring strategies and methods of dynamic analysis are needed.

The present study, part of two research projects funded by the European Community, i.e. ECSC 7210-pr-250 "*Applicability of composite structures to sway frames* " [23] and ECOLEADER Hpr-ct-1999-00059 "*Cyclic and PsD testing of to 3D steel-concrete composite structure* " [24], illustrates preliminary results of the dynamic identification of a steel-concrete composite frame structure with partial-strength beam-to-column joints [25, 26]. The structure, built at ELSA of JRC at Ispra has been subjected to four PsD tests with increasing pga and to a final cyclic test in order to verify the feasibility of its design, which was in agreement with EC8 rules [27].

Global vibration tests and local vibration tests in correspondence to interior and exterior beam-to-column joints have been carried out on the undamaged structure; after a PsD test corresponding to the ultimate limit state of the structure; and after the final cyclic test which follows the collapse limit state. In such a way, it is possible to relate the damage, occurred as a consequence of seismic actions to the vibration response of the structure; and therefore to study an efficient methodology for structural identification and damage analysis of the structure.
Besides to the description of the dynamic analyses carried out, the paper contains some preliminary results relevant to modal experimental analysis and to the recognition of damage occurred to partial strength beam-to-column joints and column base joints.

THE CASE STUDY: A STEEL-CONCRETE COMPOSITE MR FRAME STRUCTURE

Description of the test structure

The MR frame structure shown in fig. 1, derived from the prototype building illustrated in fig. 2, is made up of three identical moment resisting frames arranged at a spacing of 3.0 meters, with two bays of 5.0 and 7.0 meters and two storeys 3.5 meters in height. It is structurally symmetric in the main direction and has been reinforced with X-shaped braces in the transverse direction. A front and lateral view of the whole structure is shown in fig. 3 and 4, respectively.
The structure is made up of steel-concrete composite beams formed by IPE300 steel profiles of structural steel S235JO connected by studs with a full shear connection to a 15 cm thick C25/30 concrete slab, endowed with profiles sheetings Fe E 250 G. The secondary beams are IPE240 simply supported beam, web bolted at edges. The slab has been reinforced with longitudinal and transversal rebars B450C to activate transfer mechanisms between beams and columns in accordance to EC8 [27].
Total weights of the bottom and top storeys of the bare structure subjected to vibration-based tests are 453.7 kN and 415.4 kN, respectively. However, additional loads of 518.4 kN and 496.2 kN at the bottom and top storeys have been applied during PsD and the final cyclic test in order to reproduce initial stresses in the composite slab owing to permanent and variable loads. With regard to the slab’s geometric centre, there was only a negligible mass eccentricity in the direction of the frames, while a slight eccentricity equal to 0.40 m existed in the transversal direction.
Partially encased composite steel-concrete columns HEB260 and HEB280 made of structural steel S235JO have been utilized; C25/30 cast concrete and B450 C reinforcement bars and stirrups have been interrupted in correspondence to the joints.
Partial-strength beam-to-column joints have been designed to provide a joint rotation of 35 mrad associated to a residual strength of at least 80 % of the maximum strength. A sketch of the specimen and a joint detail are shown in fig. 5 and 6, respectively. Similar performances have to be guaranteed by base joints, composed by a thick extended plate welded to the columns and connected to the reinforced concrete foundation blocks by hoocked rebars threaded at the upper end. The gap between the plate and the foundation has been filled with grout. A complete description of the structure and utilized materials can be found in [24].

PsD and Cyclic tests

As shown in table 1, four PsD and one final cyclic test have been carried out in the main direction only.
Some spectrum compatible accelerograms have been built; to perform PsD tests, one of them was sorted out based on the highest level of damage induced in beam-to-column joints and limited value of damage induced in columns bases and column base joints. The duration of the input accelerogram was set equal to 17.5 seconds with an additional free vibration period of 2.5 seconds.
Four PSD tests have been performed according to the pga levels reported in table 1. The first one, characterized by a pga of 0.10 g, aimed at checking the test procedure and at characterizing the elastic structural behaviour; the test structure resulted substantially undamaged.

The second PsD test run at a pga level of 0.25 g led the structure to the serviceability limit state. Thin cracks developed in the mortar under the base plates and cracks developed in the concrete slab. The cracks were found to be more evident at the bottom storey and in exterior beam-to-column joints.

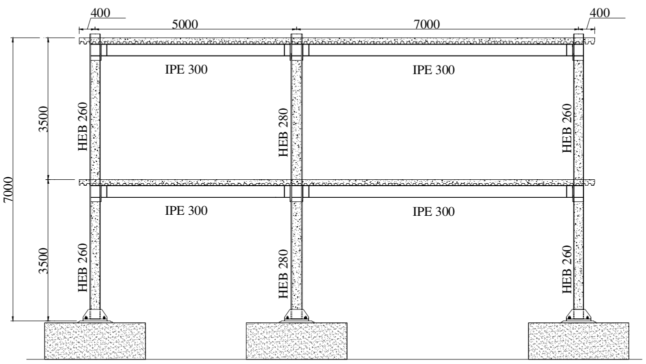

Fig. 1. Side view of the test structure

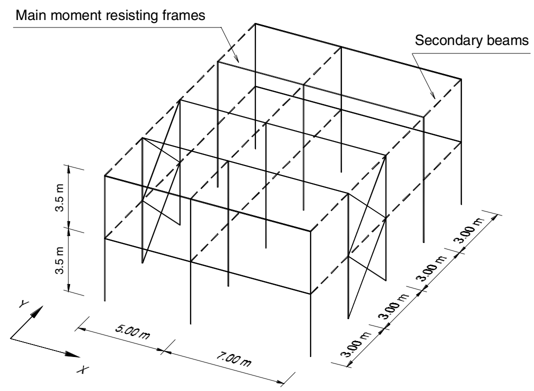

Fig. 2. 3D view of the prototype building

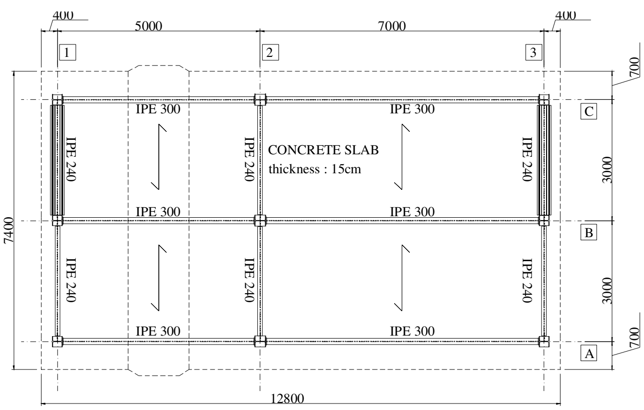

Fig. 3. Plan view of the test structure

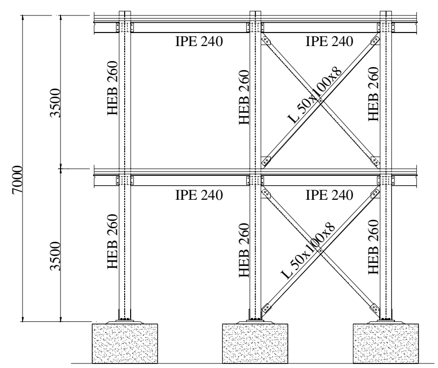

Fig. 4. Front view of the test structure

In the third and fourth PsD tests, a pga of 1.4 g and 1.8 g have been applied in order to approach the ultimate limit state and the collapse limit state, respectively. After the third test, shear yielding of the web panel at the interior joints was observed in partial-strength beam-to-column joints, while flexural deformations of beam end plates were noticed at exterior joints subjected to hogging moment. Only little permanent deformations could be observed in steel elements, while the extent of cracks on the slab increased and spalling of the compressed concrete near columns was observed. Gaps between steel column flanges and concrete slab also developed at all joints. Rotations at base joints under bending moment transferred by columns were observed during testing.

After the fourth PsD test an increase of maximum rotations values was observed.

A final cyclic test has been performed by imposing displacements of increasing amplitude up to 300 mm at the top storey. The ratio of the reaction force at the bottom and at the top storey was fixed to 0.97. Such a ratio was estimated by considering both modes derived from a modal analysis of the test structure after the fourth PsD test.

Fig. 5. Front-side picture of the test structure

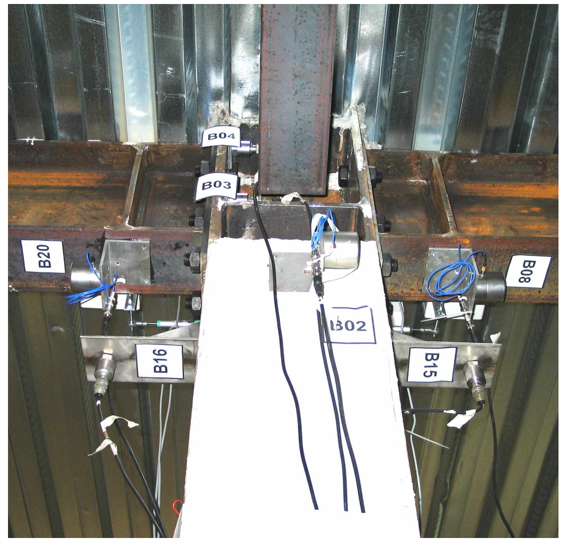

Fig. 6. Instrument configuration of an interior beam-to-column joint

Table 1. Summary of vibration, PsD and cyclic tests

Test	***Vibration test***	***PsD test*** ***pga* [g]**	***Performance objective***
I	Phase 1		*Identification at the undamaged state*
1		0.10	Elastic behaviour
2		0.25	Serviceability Limit State (SLS)
3		1.40	Ultimate Limit State (ULS)
II	Phase 2		*Identification at the Ultimate Limit State*
4		1.80	Collapse Limit State
5		Cyclic	Maximum top displacement equal to 300 mm
III	Phase 3		*Identification at the Collapse Limit State*

VIBRATION ANALYSES

Vibration analysis aims at verifying the possibility of detecting and quantifying damage undergone by the structure following PsD and cyclic tests. To this end, the response of the structure to environmental, forced and impact actions has been measured on the undamaged structure and at two different damage levels, as reported in table 1.

The Stepped Sine Test (SST) entailed the application of sinusoidal type forces, with constant frequency and amplitude, imparted by an electromagnetic shaker ELECTRO-SEIS 400 with 33.79 kg of total mobile mass. The shaker was fixed at the top storey near to column 1A of fig. 3, applying a sinusoidal force along the direction of the accelerometer A3 shown in fig. 7. The accelerometer A0 was connected to the mobile part of the shaker, thus acquiring its effective acceleration and allowing the dynamic force transmitted to the structure to be determined.

An impact force was applied by striking the protruding part of the columns A2 and A3, see fig. 3, above the top storey in the Shock Hammer Test (SHT). An instrumented sledge-hammer PCB 086D50 characterized by a total mass of 10.77 kg was employed; the direction of forces is indicated in fig. 7.

The signals produced by the piezoelectric accelerometers PCB 393 C, characterized by a sensitivity of $9.8*10^{-4}$ m/s^2, and by PCB 393 B12, with a sensitivity of $8*10^{-5}$ m/s^2, have been treated by a PCB 584 amplifier and acquired utilizing a National Instruments PCI-6031E multiplexing device of acquisition. A sampling frequency of 800 Hz was chosen for data recording.

Three different accelerometer configurations have been considered:

i) a global configuration A aimed at describing the overall dynamic behaviour of the structure conceived as a system with two translational DoFs and one rotational DoF per storey. The assumption of rigid floor deck and inextensible columns was implicitly assumed. A total of 6 accelerometers labelled A0 - A6 were utilized, three per floor, as shown in fig. 7.

ii) two local configurations B and C investigated the behaviour both of interior and exterior first floor joints at the interior frame. A maximum of 23 accelerometers was utilized, labelled B1 - B23 for the interior joint as illustrated in fig. 8; while 22 accelerometers, labelled C1 - C22 were installed on the exterior joint, as depicted in fig. 9. A detailed representation of the instrument configurations is reported in fig. 12a) and 12b) for interior and exterior joint, respectively.

The location of accelerometers derives from a mechanical model, presented in fig. 10b and 10d for interior and exterior joints, respectively. The model consists of a mechanical idealization which involves 4 rigid bars connected together by pins with translational and rotational springs. The translational spring simulates the stiffness of the column web panel under shearing; while two rotational springs represent the flexural stiffness of beam-to-column connections. The stiffness of the connection between columns and foundations has also been modelled via rotational springs.

Fig. 11 shows kinematic quantities needed to characterise the joint behaviour. In detail, we can indicate φ_{sp} as panel rotation; $\varphi_{c,t}$ and $\varphi_{c,b}$ as end rotations of the top and bottom column, respectively; $\varphi_{b,l}$ and $\varphi_{b,r}$ as end rotations of the left and right beam, respectively. Then, one gets:

$$\gamma_{sp} = \varphi_{sp} - (\varphi_{c,t} + \varphi_{c,b})/2$$

$$\varphi_{conn,r} = \varphi_{sp} - \varphi_{b,r}$$

for the panel distortion and the right connection rotation, respectively.

Accelerometers have been arranged both on interior and exterior joints as illustrated in fig. 12a) and 12b), respectively. Kinematic quantities $\gamma_{sp},$ $\varphi_{conn,l}$ and $\varphi_{conn,r}$ can be evaluated by means of simple relationships and modal analysis.

Frequency response functions (FRF) and time histories relevant to dynamic tests have been included and commented upon in [28].

STRUCTURAL DYNAMIC IDENTIFICATION

Dynamic experimental analyses allows dynamic properties of the structure like natural frequencies, damping and modal shapes to be estimated in three phases: the undamaged one; the medium damaged one; and the strongly damaged one. Two different techniques for the extraction of modal parameters have been applied in sequence. The first one, called Inverse Receptance Method was applied to the experimental FRFs, allowing a first estimate of natural frequency values and of damping ratios to be estimated. Later, the aforementioned values were utilized as primers for the non-linear regression procedure, called Non-Linear Least Square, which allows modal shapes and more accurate frequency and damping values to be tracked [29].

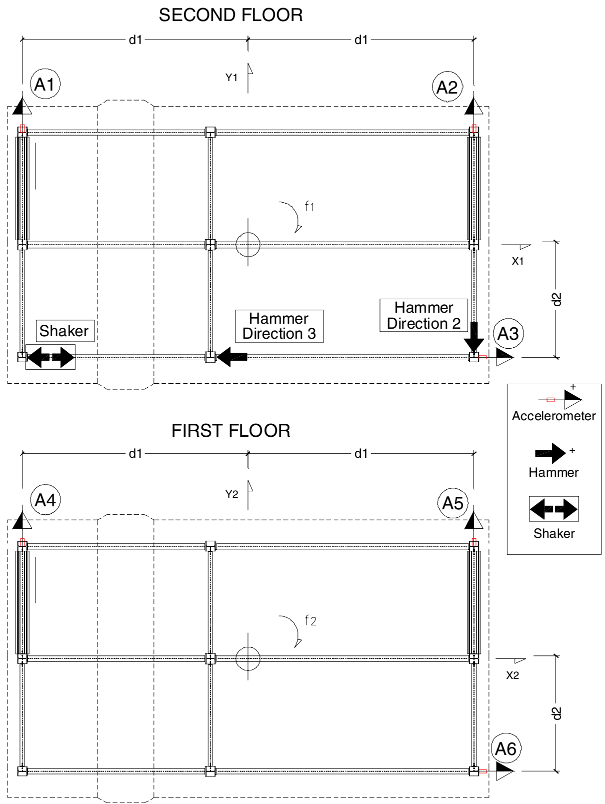

Fig. 7. Location of shaker, hammer impact points and accelerometers at the a) top storey and b) bottom storey

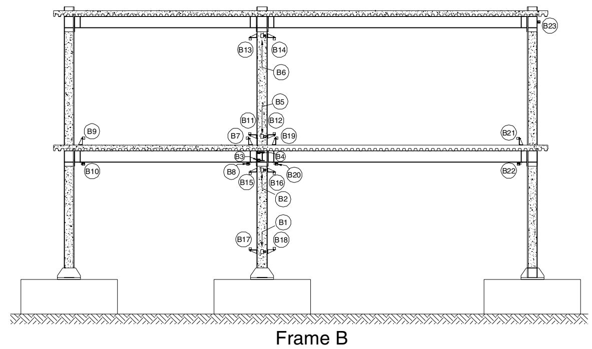

Fig. 8. Location of accelerometers in the configuration B for the interior joint

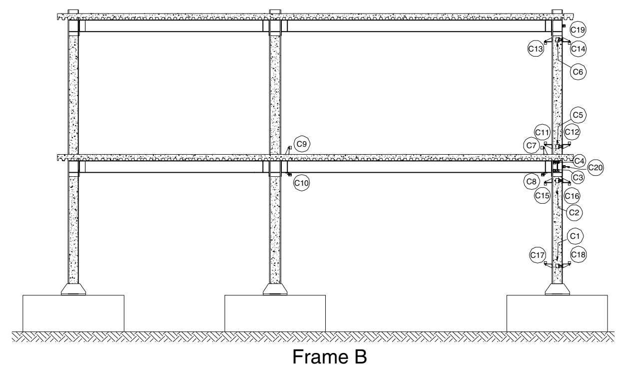

Fig. 9. Location of accelerometers in the configuration B for the exterior joint

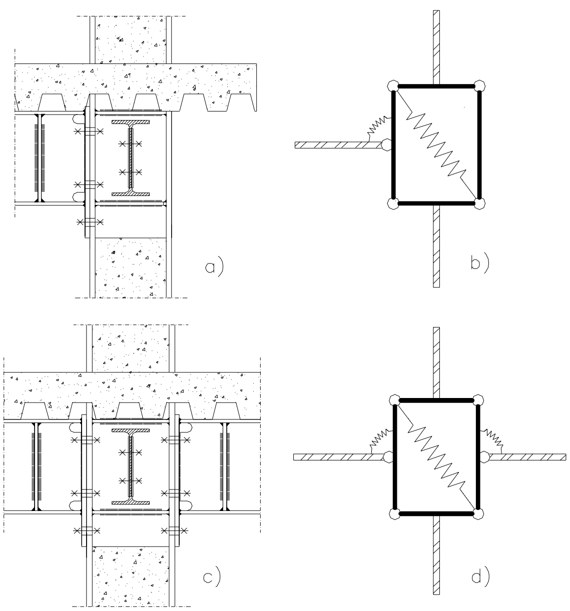

Fig. 10. Joint models (b, d) for the exterior (a) and interior (c) joint

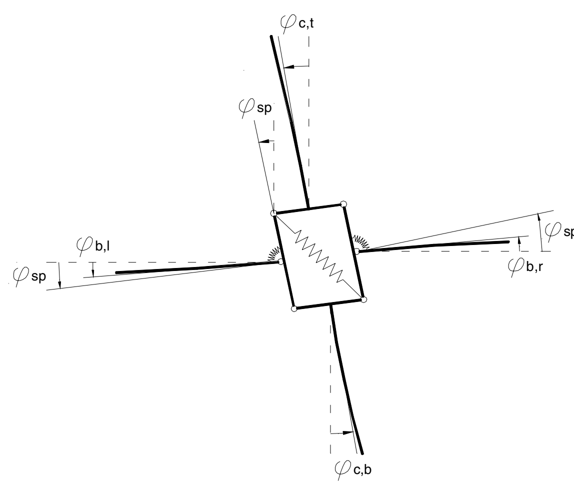

Fig. 11. Kinematic quantities relevant to an interior joint

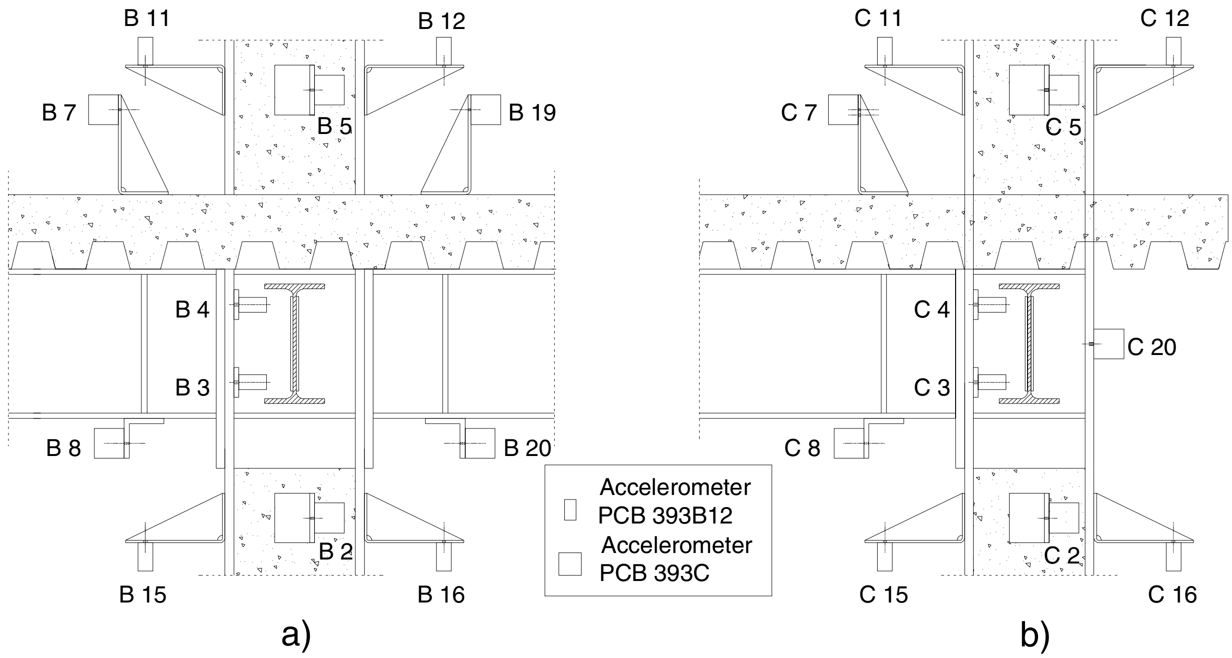

Fig. 12. Accelerometer lay-out: a) configuration B for the interior joint; b) configuration C for exterior joint

In order to verify usual assumptions regarding damping ratios employed in frame structures, a non-proportional viscous model was adopted in analyses as it is more general compared to the Rayleigh's model. Extracted complex values of modal shapes relevant to the A configuration result to be approximately arranged along a straight line inclined to - 45° shown in fig. 13. This property assure that damping can be considered of proportional type in phase I [22, 30]. Therefore, it was possible to extract real values of modal shapes like, for instance, the first, second, third and fourth mode of the structure in the undamaged condition represented in fig. 14. These mode shapes correspond respectively to: i) the first flexural mode in the main frame direction; ii) the first flexural mode in the orthogonal direction; iii) the first torsional mode; iv) the second flexural mode in the main frame direction. Moreover, the fifth and sixth mode correspond the second flexural mode in the orthogonal direction and to the second torsional mode, respectively.

The natural frequencies of the above-mentioned six modes relevant to the three phases I, II and III are reported in table 2. One may observe clearly a general reduction of natural frequencies owing to damage of partial strength beam-to-column joints and column base joints.

Table 2. Frequencies of the first six modes relevant to phases I, II and III.

		Test		
		I	II	III
		f [Hz]	f [Hz]	f [Hz]
Mode	1	3.41	2.45	2.07
	2	5.13	4.49	4.24
	3	7.02	6.29	5.77
	4	11.00	9.06	8.18
	5	16.50	13.87	13.71
	6	22.60	19.90	19.10

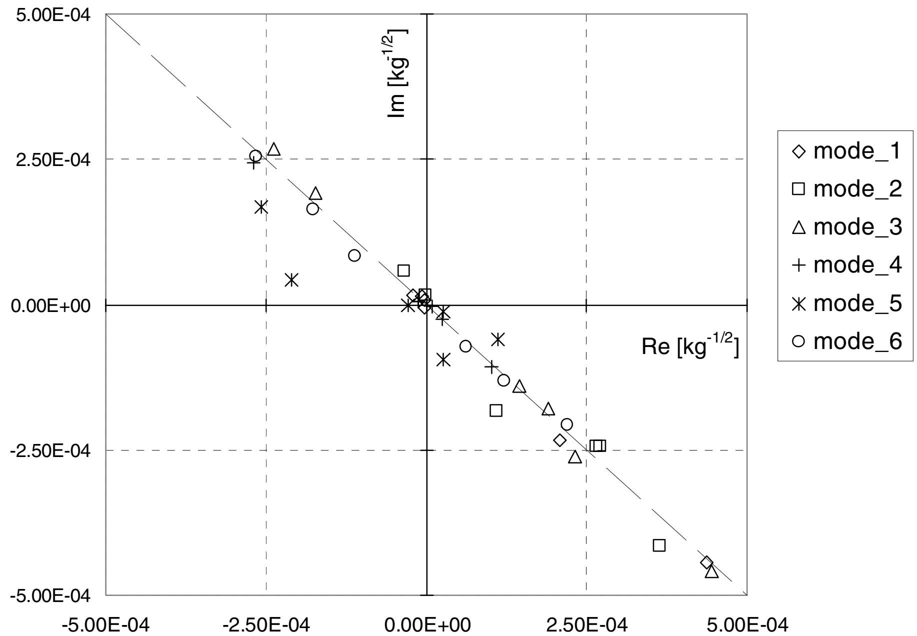

Fig. 13. Complex plan representation of the first six mode shapes in phase I, normalized with respect to the mass matrix.

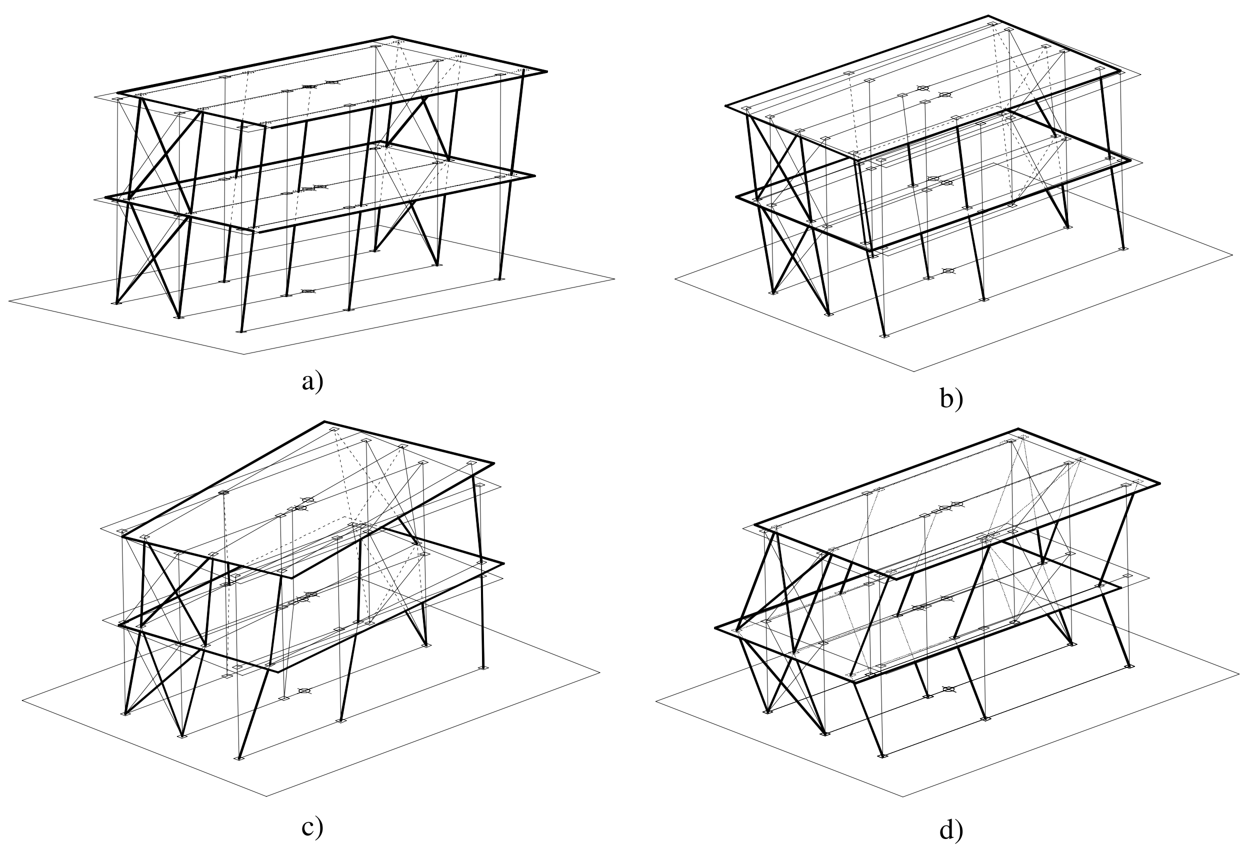

Fig. 14. Mode shapes of the frame structure relevant to phase I: a) first flexural mode in the main frame direction; b) first flexural mode in the orthogonal direction; c) first torsional mode; d) second flexural mode in the main frame direction

A PRELIMINARY ANALYSIS OF DAMAGE

The results of dynamic tests carried out in the three phases of the structure life allow preliminary information on the damage undergone by the structure owing to PsD and cyclic tests to be acquired. A qualitative comparison between FRFs is reported in fig. 15. It shows that moving from the undamaged to the medium and strongly damaged phases, a substantial shift of resonance peaks towards the lower frequencies zone happened. A detailed analysis of the identified natural frequency values, reported in table 3, shows that a general reduction of the natural frequencies has occurred: this phenomenon appears particularly pronounced for the flexural modes in the main frame direction, i.e. for modes 1 and 4. A reduction of nearly 40% between the undamaged and the strongly damaged state, e.g. between test I and III for the first frequency has been observed.

Another comparison between the three phases moving from the undamaged to the damaged state, can be performed evaluating the modal shapes variation via the Modal Assurance Criterion (MAC) [16, 17, 29 and 31]. The MAC index estimates the correspondence level between the modal shape vectors extracted from different phases results. It assumes values comprised between 0 and 1 in the case in which the modes are perfectly uncorrelated and practically identical, respectively.

Tables 4, 5 and 6 reports the values of the MAC index obtained from the comparison of modes extracted in tests I and II, I and III and II and III, respectively. An attentive reader can observe that correlations between the modal shapes identified in the three phases are generally good as diagonal terms of the MAC matrices assume values greater than 0.9 for nearly all modes [31]. The representation on the complex plan of single modes identified in the three phases, and reported in fig. 14, shows that nearly all modal shapes lie along a straight line inclined of -45°; together with the high MAC diagonal coefficients. This trend indicates that, contrarily to resonance frequencies, modal shapes do not vary with the damage level.

The only exceptions seem to be modes 2 and 5, that exhibit some anomalies. In detail, mode 5 is characterized by lower MAC values, approximately equal to 0.6 in the test comparisons I and II and I and III, but it reaches MAC values equal to 0.98 in the comparison of the tests II and III. This could indicate that a relevant variation of the 5th modal shape happened in the second phase but that any significant change did not occurred in the third phase. In fact, during the execution of PsD tests, therefore between dynamic tests I and II, cracking of the slab happened owing to the application of aforementioned static loads. This phenomenon did not modify the static behaviour of the collaborating slab in the direction of the main frames and, therefore, it did not affect both PsD and cyclic tests. However, it probably altered the dynamic properties of the structure in the orthogonal direction.

Hence, we can affirm that natural frequency values have been strongly influenced by the increasing of damage induced by PsD and cyclic tests. A similar trend has happened to the values of damping ratios of which only values relevant to the first and fourth modes are reported in table 7. Conversely, the modal shapes of the structure were not considerably affected by damage, as found in recent experimental analyses [21].

CONCLUDING REMARKS AND FUTURE DEVELOPMENTS

The preliminary results relevant to an experimental dynamic analysis carried out at the European Laboratory for Structural Assessment of the Joint Research Centre on a full-scale steel-concrete composite frame structure, designed in accordance to high ductility class requirements of Eurocode 8 have been illustrated. The structure was subjected to pseudo-dynamic tests of increasing peak ground acceleration and to a final cyclic test.

The analysis has allowed some preliminary conclusions about the damage undergone by the structure, showing as modal shapes do not exhibit significant changes owing to the presence of damage; while instead both frequency values and damping ratios can experience significant variations.

The damage undergone by beam-to-column joints owing to inelastic phenomena occurred in beam-to-column connections, web panels in shear and column base joints will be estimated with the following procedure. By using the local configurations B and C, it will be possible to estimate both modal rotations and modal displacements in correspondence of the instrumented beam sections, column sections and joints. Then, assuming that composite beams and columns will remain in the elastic range, it will be possible to estimate the characteristics of modal actions. Finally, by evaluating local stiffness values of connection and web panel suitable damage indices will be defined on the basis of rigidity ratios relevant to undamaged and damaged conditions.

ACKNOWLEDGEMENTS

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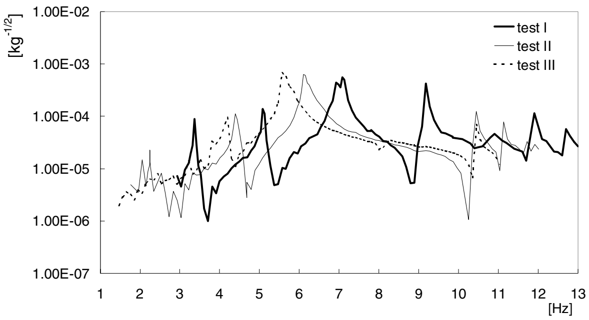
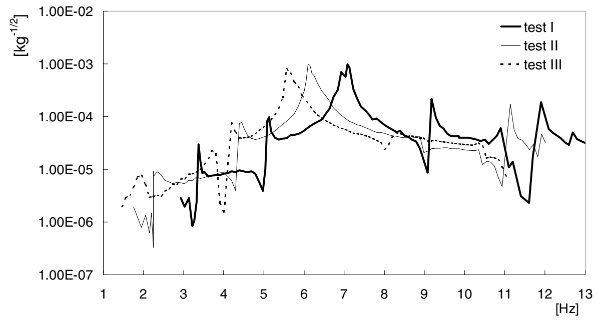
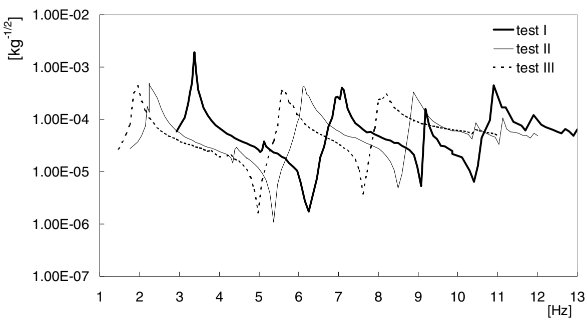


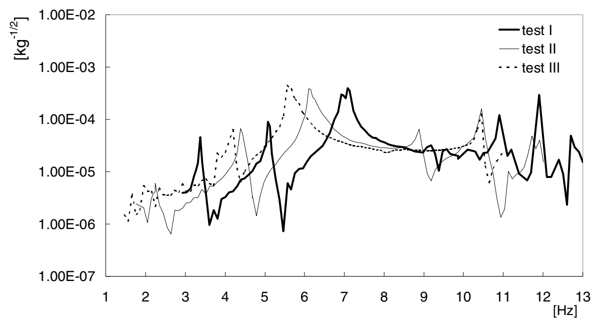
SST FRF – A 1



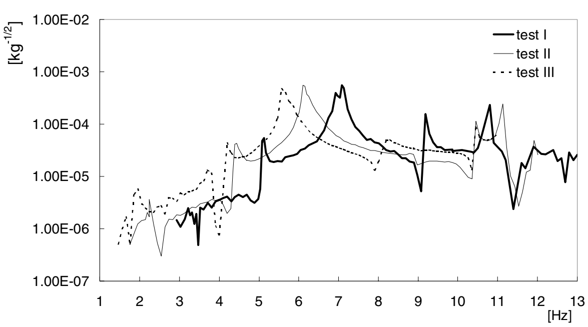
SST FRF – A 2



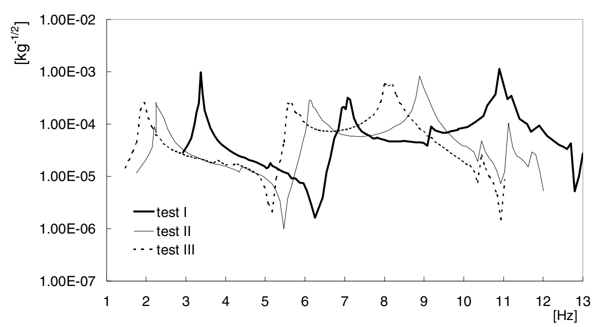
SST FRF – A 3



SST FRF – A 4



SST FRF – A 5



SST FRF – A 6

Fig. 15: FRF obtained from the SST at the maximum force of the shaker applied to the structure in test phases I, II and III. Accelerometers A1 - A6 of the global A configuration.

Table 3. Frequencies of the first six mode shapes and percentage variation between tests I, II and III

		Test I	Test II	Test III	Δf_{II-I} [%]	Δf_{III-I} [%]
		f [Hz]	f [Hz]	f [Hz]		
Modes	1	3.41	2.45	2.07	-28.15	-39.30
	2	5.13	4.49	4.24	-12.48	-17.35
	3	7.02	6.29	5.77	-10.40	-17.81
	4	11.00	9.06	8.18	-17.64	-25.64
	5	16.50	13.87	13.71	-15.94	-16.91
	6	22.60	19.90	19.10	-11.95	-15.49

Table 4. MAC matrix obtained from the comparison of mode shapes extracted in tests I and II

MAC I-II		Modes					
		1	2	3	4	5	6
Modes	1	**0.975130**	0.019381	0.120500	0.006079	0.026073	0.000460
	2	0.000826	**0.987480**	0.009752	0.000149	0.033251	0.005144
	3	0.079795	0.009457	**0.987970**	0.011615	0.081012	0.025326
	4	0.007026	0.003861	0.006506	**0.962920**	0.006824	0.199240
	5	0.002641	0.025550	0.038683	0.000977	**0.603680**	0.025151
	6	0.001184	0.016798	0.010413	0.136980	0.118730	**0.992890**

Table 5. MAC matrix obtained from the comparison of modes extracted in tests I and III

MAC I-III		Modes					
		1	2	3	4	5	6
Modes	1	**0.990820**	0.001969	0.126410	0.001763	0.024909	0.000421
	2	0.003864	**0.919050**	0.023882	2.28E-05	0.018983	0.006035
	3	0.119850	0.071204	**0.975900**	0.000975	0.076672	0.018078
	4	0.021166	0.001373	0.013532	**0.983900**	0.036113	0.212020
	5	0.000976	0.040165	0.037197	0.015449	**0.602640**	0.017063
	6	0.005991	0.004409	0.018431	0.170920	0.112670	**0.994840**

Table 6. MAC matrix obtained from the comparison of modes extracted in tests II and III

MAC II-III		Modes					
		1	2	3	4	5	6
Modes	1	**0.987310**	0.002321	0.064647	0.001872	0.019172	0.000375
	2	0.006927	**0.917350**	0.032832	0.001623	0.016896	0.011060
	3	0.100510	0.031813	**0.995060**	4.45E-05	0.068853	0.013332
	4	0.020040	0.000519	0.011634	**0.982120**	0.011813	0.133700
	5	0.022076	0.005537	0.078208	0.003594	**0.98045**	0.137500
	6	0.000940	0.001576	0.029067	0.150130	0.11643	**0.997710**

Table 7. Percentage damping ratio and variation between tests I, II and III for modes I and IV

Modes	Test I	Test II	Test III	$\Delta\xi_{II-I}$ [%]	$\Delta\xi_{III-I}$ [%]
	ξ [%]	ξ [%]	ξ [%]		
1	0.47	0.55	1.00	17.02	112.77
4	0.42	0.46	0.61	9.52	45.24

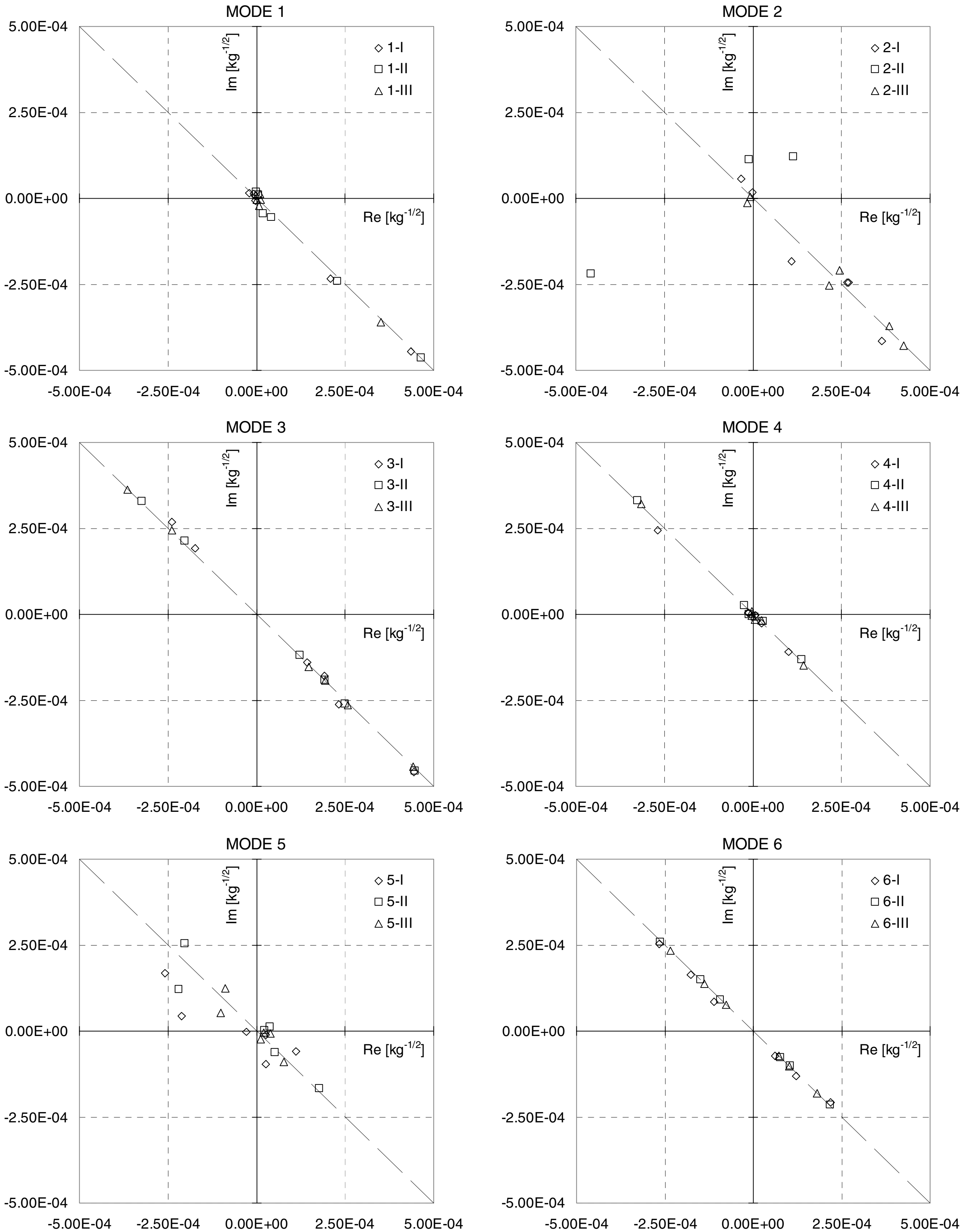


Fig. 16. Comparison of first six mode shapes identified in tests I, II and III

REFERENCES

1. Doebling SW, Farrar CR, Prime MB. “A summary review of vibration-based damage identification methods”. The Shock and Vibration Digest, 30(2):pp. 91–105, 1998.
2. Doebling SW, Farrar CR, Prime MB, Shevitz DW. “Damage Identification and Health Monitoring of Structural and Mechanical Systems From Changes in Their Vibration Characteristics: A Literature Review.” MS 13070, Los Alamos National Laboratory, LA, May 1996.
3. Peeter B. System Identification and Damage Detection in Civil Engineering. Thesis for the fulfilment of the PhD Degree, Katholieke Universiteit Leuven, 2000.
4. Maeck J, Wahab MA, Peeters N, De Roeck G, De Visscher J, De Wilde WP, Ndambi JM, Vantomme J. “Damage identification in reinforced concrete structures by dynamic stiffness determination”. Engineering Structures, 22:pp. 1339–1349, 2000.
5. Krawczuk M. “Application of spectral beam finite element with a crack and iterative search technique for damage detection”. Finite Elements in Analysis and Design, 38:pp. 537–548, 2002.
6. Ren WX, De Roeck G. “Structural Damage Identification using Modal Data. I: Simulation Verification”. Journal of Structural Engineering, 128(1):pp. 87–95, 2002.
7. Ren WX, De Roeck G. Structural Damage Identification using Modal Data. II: Test Verification. Journal of Structural Engineering, 128(1):pp. 96–104, 2002.
8. Bolchini GC, Antonacci E. “Il comportamento dinamico di S. Maria di Collemaggio: previsioni numeriche ed osservazioni sperimentali”. X Congresso Nazionale de L’ingegneria Sismica in Italia, Potenza., 9-13 settembre 2001. (in Italian)
9. Bonato P, Ceravolo R, De Stefano A, Molinari F. Identificazione strutturale a input incognito mediante estimatori di ampiezza e fase istantanea. IX Congresso Nazionale de L’ingegneria Sismica in Italia, Torino, 20-23 Settembre, 1999. (in Italian)
10. Zonta D. “Structural damage detection and localization by using vibrational measurements”. Thesis for the fulfilment of the PhD Degree, University of Bologna, DISTART. Bologna, 2000.
11. Bonato P, Ceravolo R, De Stefano A, Molinari F. “Identificazione diretta e indiretta a input incognito di edifici in muratura”. IX Congresso Nazionale de L’ingegneria Sismica in Italia, Torino, 20-23 Settembre, 1999. (in Italian)
12. Koh, CG, See, LM, Balendra, T. Determination of storey stiffness of three-dimensional frame buildings. Engineering Structures, 17(3):pp. 179-186, 1995.
13. Wong, CW, Mak, WH, Ko, JM. “System and parametric identification of flexibile connections in steel framed structures”. Engineering Structures, 17(8):pp. 581-595, 1995.
14. Lam, HF, Ko, JM, Wong, CW. “Localization of damaged structural connections based on experimental modal and sensitivity analysis”. Journal of Sound and Vibration, 210(1):pp. 91-115, 1998.
15. Brownjohn, JMW, Xia, PQ, Hao, H, Xia, Y. “Civil structure condition assessment by FE model updating: methodology and case studies”. Finite Elements in Analysis and Design, 37:pp. 761-775, 2001.
16. Bicanic, N, Chen, HP. “Damage identification in framed structures using natural frequencies”. International Journal for Numerical Methods in Engineering, 40:pp.4451-4468, 1997.
17. Lam, HF, Ko, JM, Wong, CW. “Localization of damaged structural connections based on experimental modal and sensitivity analysis”. Journal of Sound and Vibration, 210(1):pp. 91-115, 1998.
18. Molina, FJ, Pascual, R, Golinval, JC. “Description of steelquake benchmark”. Mechanical Systems and Signal Processing, 17(1):pp. 77-82, 2003.
19. Priestley MJN, Sritharan S, Conley JR, Pampanini S. “Preliminary result from the PRESSS Five-Storey Precast Concrete test building”. PCI Journal, 2(159):pp. 42–67, 1999.
20. Nakaki SD, Stanton JF, Sritharan S. “An overview of the PRESSS five-storey precast test building”. PCI Journal, 2(42):pp. 26–39, 1999.

21. Zonta, D, Elgamal, A, Fraser, M, Priestley, MJN. "A vibrational based methodotology for after-earthquake damage assessment of multistory frame-resistant buildings". Proceedings of 12th European Conference on Earthquake Engineering, London, 2002.
22. Betti, R, De Angelis, M, Imbimbo, M, Yu, J." Identificazione del danno in sistemi lineari a numero ridotto di misurazioni". Atti del XI ANIDIS Congresso Nazionale "L'Ingegneria Sismica in Italia", Genova, 25-29 Gennaio 2004, 2004.
23. University of Pisa et al. "Applicability of Composite Structure to Sway Frames". ECSC Project n. 7210-PR-250, Eur Report, European Community, 2004.
24. Bursi OS, Caramelli S, Fabbrocino G, Molina J, Salvatore W, Taucer F, Zandonini R, "3D Full Scale Seismic Testing of a Steel-Concrete Composite Building at ELSA". Eur Report, Joint research Centre, European Community, 2004.
25. Bursi OS, Caramelli S, Fabbrocino G, Pinto AV, Salvatore W, Taucer F, Tremblay R, Zandonini R. "Pseudo-dynamic testing of a 3d full-scale high ductile steel-concrete composite MR frame structure at ELSA". Proceedings of the 13th World Conference on Earthquake Engineering. Vancouver, B.C., Canada, Paper no. 507. 2004.
26. Salvatore W, Bursi OS, Caramelli S, Haller M, Zandonini R. "Seismic design and pseudo-dynamic characterization of a high ductile steel-concrete composite moment resisting frame". Proceedings of the Composite Construction in Steel and Concrete V, The Kruger National Park Conference Centre Berg-en-Dal, Mpumalanga, South Africa, 2004.
27. prEN 1998-1. "Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings". CEN, European Committee for Standardization. Brussels, Belgium, 2002.
28. Bursi, OS, Molina, J, Salvatore, W, Taucer, F. "Dynamic characterization of a 3-D full scale steel-concrete composite building at ELSA". Eur Report, European Community, 2004.
29. Maia, N.M.M., Silva, J.M.M. "Theoretical and Experimental Modal Analysis". Research Studies Press LTD, Baldock, Hertfoordshire, England, 1997.
30. Balmes, E. "New results on the identification of normal modes from experimental complex modes". Mechanical Systems and Signal Processing, 11(2):pp. 229-243, 1997.
31. Ewins, DJ. "Modal Testing: Theory, Practice and Application". Research Studies Press Ltd, 2000.