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PROVE SU TAVOLA VIBRANTE SULLA VALUTAZIONE DEL COMPORTAMENTO SISMICO DI TELAI IN ACCIAIO

SHAKING TABLE TESTS FOR SEISMIC PERFORMANCE EVALUATION OF STEEL FRAMES

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SOMMARIO

Questa nota propone uno studio sul comportamento dinamico di telai in acciaio con soletta in calcestruzzo gettata su lamiera grecata. L'analisi sperimentale, di seguito presentata in sintesi, ha previsto due differenti tipi di prove: prove sulla componente giunto trave-colonna in presenza di azioni cicliche alternate di tipo quasi statico e prove su tavola vibrante di sistemi intelaiati a grandezza reale. Il comportamento dei giunti ed i principali parametri che ne caratterizzano la risposta sono identificati e discussi. Viene inoltre analizzato il comportamento del giunto trave-colonna ed è proposto un approccio semplificato, utilizzabile a livello progettuale, per tenere in conto l' accumulo del danno in sistemi intelaiati in acciaio in presenza di azione sismica.

ABSTRACT

This paper reports on a study on the dynamic behaviour of steel moment resisting frames with a concrete slab on metal sheeting. The experimental analysis, which is herein summarized, consists of two different series of tests: component tests on beam-to-column joints under quasi-static cyclic reversal loading and dynamic tests of complete full scale framed structures on the shaking table. Performance of joints and main parameters affecting their response are identified and discussed. Experimental results are also considered with reference to low-cycle fatigue behaviour and the associated fatigue resistance line are presented. Moreover, the performance of the joints in the frame under dynamic loading is analysed and a simplified approach, which can be used for design purposes, is applied to monitor the damage accumulation process in steel framed structures under seismic action.

1. INTRODUCTION

Moment-resisting (MR) steel frames are usually designed in accordance with the capacity design philosophy, i.e., by assuming that the structural system has to provide sufficient strength, ductility and energy dissipation capabilities to resist severe earthquake, despite being severely damaged. Recent researches, which were carried out in the last decades on nodes of MR frames, Zandonini et al (1997) and Plumier (1994), permitted to develop a satisfactory knowledge on joint cyclic behaviour. As a consequence, modern seismic provisions require that dissipation occurs at beam-ends and, eventually, at base section of columns. Nodal zones should hence embody sufficient strength and rotational stiffness so as to allow yielding as well as strain hardening in the dissipative zones, without any brittle fracture of structural components. Therefore, during Northridge (1994) and Kobe (1995) earthquakes, a lot of MR frames suffered local damages in beam-to-column joints. Unprecedented combined phenomena (i.e., notch effect due to backing bars, lack of preheating for thick flange plates and inadequate workmanship and inspection) appeared as key factors responsible for the severe failure modes of joints. Moreover, high strain rate effects, associated with ground motion, generated material over strength and, as a consequence, joint ductility resulted remarkably reduced. Several studies were hence carried out to explain both fracture locations and failure modes, which were observed during the aforementioned earthquakes. This underlines the importance and the need of efficient design approaches, based on the possibility of predicting accurately the response of beam-to-column joints in terms of hysteretic behaviour as well as of cumulated damage and failure. As a consequence of these unexpected failures, a great number of research programs were undertaken, all over the world, in order to:

- *Investigate the behaviour of typical connections used in MR frames under seismic loading;*
- *Identify new connection typologies, developing a ductile behaviour under an earthquake;*
- *Set up safe design rules for new structures in seismic areas;*
- *Propose suitable damage assessment methods and repair procedures to be adopted in engineering practice.*

This paper deals with a part of the research, which is currently in progress at the Politecnico di Milano in co-operation with the National Technical University of Athens, aimed to assess the validity of the linear approach for seismic design based on the "component behaviour", which has been up-to-now presented in the literature with reference to isolated structural components (beams, columns, joints). Owing to the influence of joints in MR steel frames, the first part of the experimental analysis, executed at the Politecnico di Milano, comprised joint tests. Key aspects of the behaviour of this component are in the following presented and discussed, by considering also the influence on the joint performance due to the presence of a concrete slab on metal sheeting. Furthermore, the results associated with the second experimental phase of the study, i.e., shaking table tests of complete framed systems, carried out at the Laboratory of the National University of Athens, are proposed and joint action in the overall frame is analysed. Finally a simplified approach is shortly presented and applied, which has been developed for the study of the damage accumulation process and can be used for practical design purposes in the field of steel as well as steel-concrete composite frames.

2. JOINT RESEARCH PROGRAM AND TEST RESULTS

The specimens considered for the joint experimental analysis consist of an IPE 160 beam attached to a HEB180 column by a welded connection. Two types of welds were considered: type C1 are representative of a typical site welding, with single bevel V groove welds and a backing bar; type C4 are representative of a typical shop welding, with double bevel K groove welds (Fig. 1). Two series of 6 specimens each were fabricated, representative of an external beam-to-column connection in a MR frame. The first set of specimens (named as "ns") consists of the sole steel beam-to-column connection; the second one, (identified as "s") encompassing the presence of a 100 mm r.c. slab, cast in a corrugated steel sheeting 55 mm deep (Fig. 2). The slab was connected to the steel beam only by means of three studs positioned near the free end of the beam, used only to prevent separation and detachment of the concrete slab from the beam end at the load application zone. Hence, the specimens cannot be considered as acting as composite, but the presence of the slab is considered only in order to simulate real structural conditions and to prevent local buckling of the upper flange of the beam. Of the 6 specimens of each series, three were welded according to the C1 detail, the other three according to the C4 detail. Only 5 specimens of each series have been tested at present; the sixth one with a C4 type connection was left intact, for eventual future experimental verifications. During the test, the column is laying horizontally while the beam is standing in the vertical position, and subjected to a horizontal displacement imposed at the beam end by a jack governed by an electric engine.

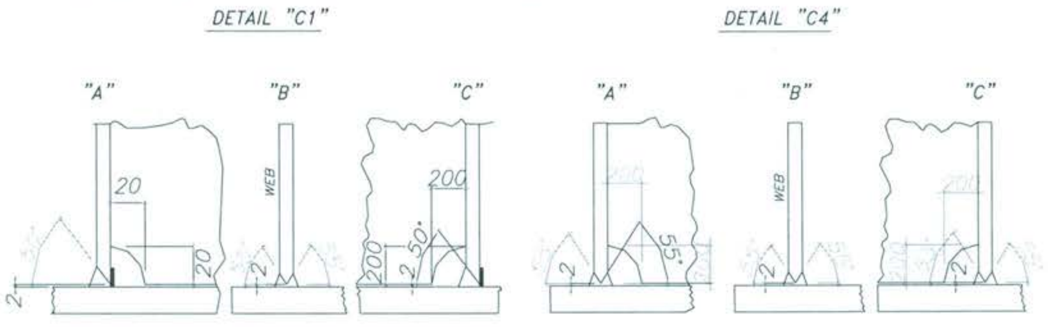


Figure 1: Welding details of the steel beam-to-column joints.

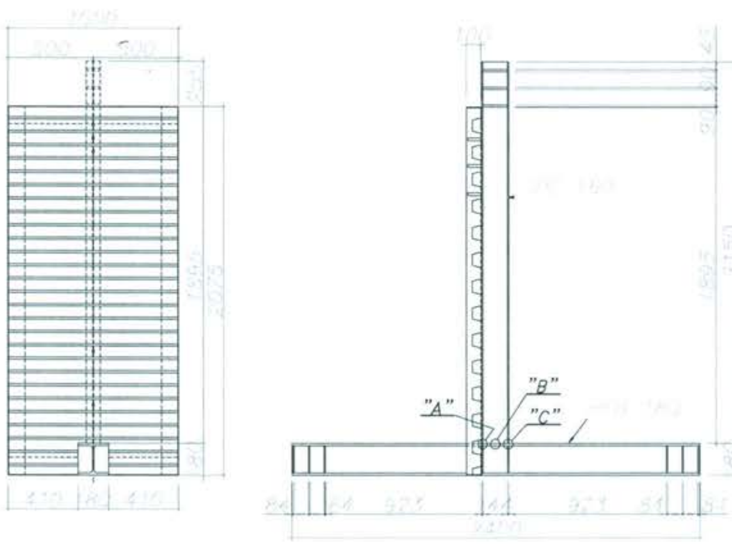


Figure 2: Specimen of steel beam-to-column joint with the concrete slab on metal sheeting

In Tables 1 and 2 key features of the joint tests are summarized, for specimens with and without concrete slab, respectively. Term Δv represents the displacement range imposed at the beam end, i.e. the assumed control test parameter, v_y^s and F_y^s are respectively the yield displacement and the yield strength in the case of hogging moment (concrete in tension), while v_y^c and F_y^c are respectively the yield displacement and the yield strength in the case of sagging moment (concrete in compression).

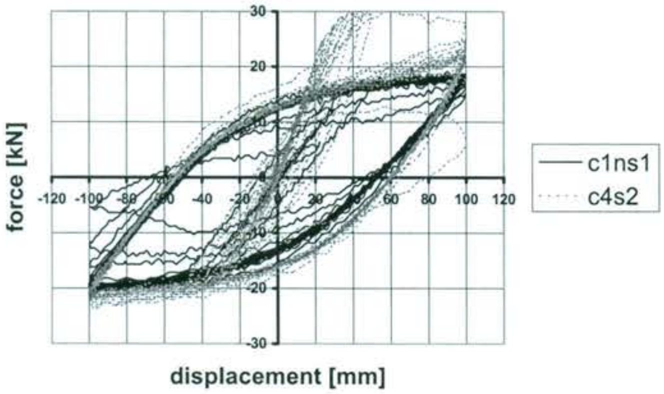
Table 1: Test results for specimens with the concrete slab on metal sheeting

Specimen	Δv [mm]	v_y^s [mm]	F_y^s [kN]	v_y^c [mm]	F_y^c [kN]	N_{tot}.
C4-s-3	350	38.8	19.1	35.6	34.1	4
C4-s-2	200	36.2	22.9	30.0	29.1	17
C1-s-1	160	37.6	22.2	31.3	29.9	37
C1-s-2	140	35.8	22.9	30.6	29.2	56
C1-s-3	120	39.1	20.7	32.4	33.9	57

Table 2: Test results for specimens without the slab

Specimen	Δv [mm]	v_y^s [mm]	F_y^s [kN]	N_{tot}.
C1-ns-2	300	41.4	18.6	5
C1-ns-3	240	43.3	18.8	22
C1-ns-1	200	43.0	19.1	25
C4-ns-1	160	39.9	18.4	51
C4-ns-2	120	41.6	18.8	73

As a general remark, it can be said that specimens with the concrete slab, in the first cycles, when the concrete in compression is undamaged, show a larger strength and stiffness than the corresponding ones made of steel only, owing to the influence both of adhesion and chemical bond and of the interface friction. As an example, Figure 3 can be considered, where the comparison between the hysteresis loops for the two specimens C4-s-2 and C1-ns-1, that were tested under cycles of the same amplitude (200 mm) is presented.



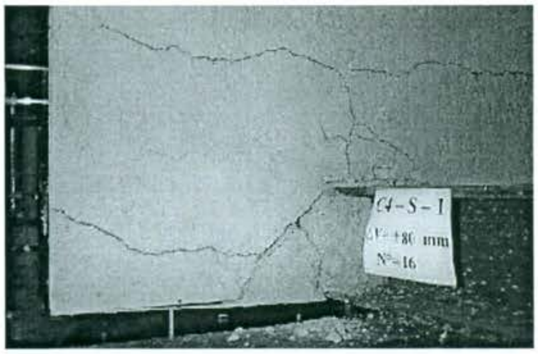


Figure 3: Comparison of hysteresis loops for specimens with and without concrete slab

Figure 4: Damage in the concrete slab

Furthermore, in case of joints with slab there is also an evident difference in stiffness between the case of sagging and hogging bending moment, due to action of concrete in compression. Immediately after the first large cycle beyond the elastic limit, the concrete slab cracked in compression. This fact was noticed also in specimens tested under smaller cycle amplitudes, as shown in Figure 4, in the case of specimen C4-s-1. After cracking of the concrete, an evident reduction is observed in the specimen strength that becomes comparable with that of corresponding specimen without slab. It is also evident (Fig. 5) that practically all the specimens tested collapsed in a "brittle" failure mode, after a relatively long period in which the capacity of the specimen to dissipate energy remained nearly constant. Independently on the presence or absence of the concrete slab, cracking at the welded joint always attained failure, with limited buckling evidence in the beam flange. This confirms the already mentioned "brittleness" of the failure mode.

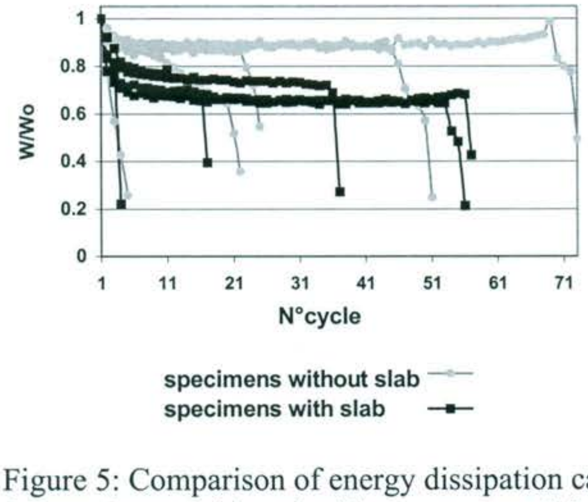


Figure 5: Comparison of energy dissipation capacity for specimens with and without concrete slab

Tables 3 and 4 summarise the tests results in terms of values for N_{tot}, the total number of cycles experimentally imposed to the specimen, and N_f is the number of cycles to failure according to the *"Relative Energy Drop"*, Bernuzzi et al (1997)

Table 3: Specimens without slab

Specimen	Δv[mm]	$N_{tot.}$	N_f
C1-ns-2	300	5	5
C1-ns-3	240	22	20
C1-ns-1	200	25	22
C4-ns-1	160	51	45
C4-ns-2	120	73	69

Table 4: Specimens with slab

Specimen	Δv[mm]	$N_{tot.}$	N_f
C4-s-3	350	4	3
C4-s-2	200	17	15
C1-s-1	160	37	35
C1-s-2	140	56	53
C1-s-3	120	57	56

The S-N lines for the specimens were obtained assuming as parameter S the displacement range Δv and are presented in Figure 6, which reports the best fit S-N curve, Ballio & Castiglioni (1995). It can be noticed that the low-cycle fatigue strength of the specimens with the reinforced concrete slab is lower than the one of the specimens made of the steel profiles only.

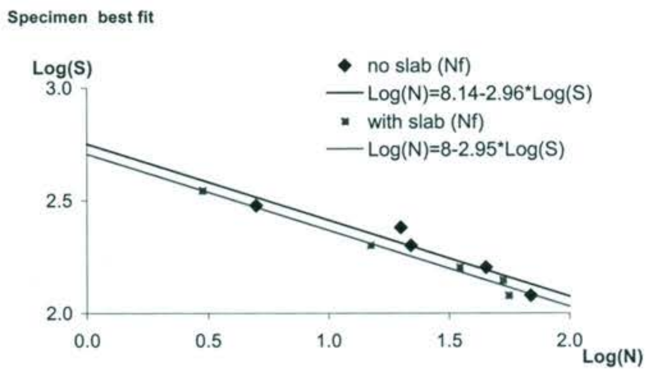


Figure 6: Best fit S-N lines assuming S=Δv

3 SHAKING TABLE TESTS OF COMPOSITE STEEL FRAMES

The dynamic tests were carried out at the shaking table facility of the Laboratory for Earthquake Engineering of the National Technical University of Athens. Although hereafter only the results of two dynamic tests on frames with welded beam-to-column connections are presented in order to carry out the comparisons with the cyclic tests performed in Milano, the research carried out in Athens is much wider in scope, encompassing tests on two sets of 5 frames each, characterised by different structural details for the beam-to-column connections and the r.c. slab connection to the steel beam. Tests were carried out on two similar frames (Fig. 7), with HE180B columns in Fe 510 Steel and IPE 160 beams in Fe 360; the two frames, named STF3 and STF4 respectively were characterised by having different connection details of the r.c. slab to the IPE160 beam: specimen

STF3 was constructed with 1 stud ϕ10mm per rib of the steel sheeting, while in specimen STF4 1 stud ϕ10mm every two ribs was adopted.

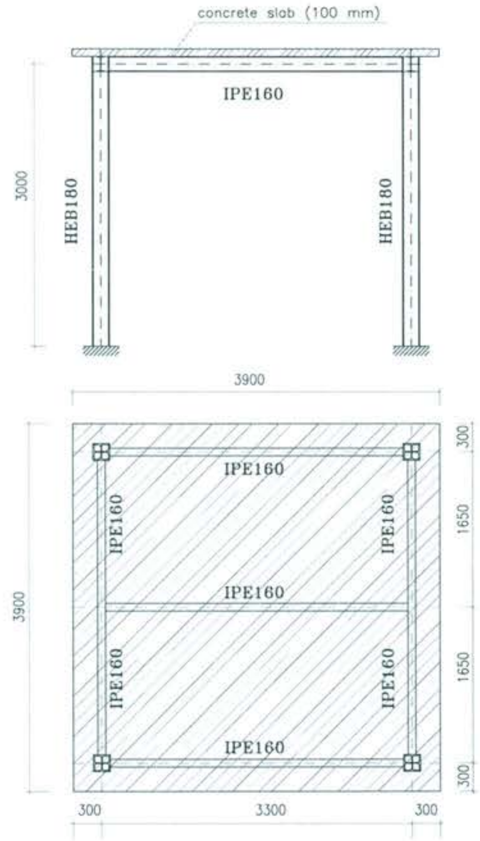


Figure 7: The frame specimen for shaking table tests

Table 5 summarises periods and damping factors computed by re-analysis of dynamic test results for the two specimens. The identification of these dynamic characteristics was performed by means of a sine logarithmic sweep base excitation, in a frequency range of 1-6 Hz, at the rate of one octave per minute. The damping factor was computed by using the half power bandwidth method. Earthquake simulation tests were performed using ramped sinusoidal excitations, with a frequency of 5 Hz, which is approximately equal to 80% of the own frequency of the specimens.

Table 5: Period and damping ratio of the tested frames

SPECIMEN	Period (sec)	Frequency (Hz)	Damping (%)
STF3 - 1stud/1 rib	0.144	6.93	2.10
STF4 - 1stud/2 ribs	0.147	6.80	2.50

The Figures 8 and 9 show, respectively for specimens STF3 and STF4, the acceleration of the shaking table compared to that recorded on the top of the r.c. slab of the specimen, the relative rotations recorded between the beams and the columns at the beam-to-column connections A of the specimens and the moment-rotations diagrams for the same connection.

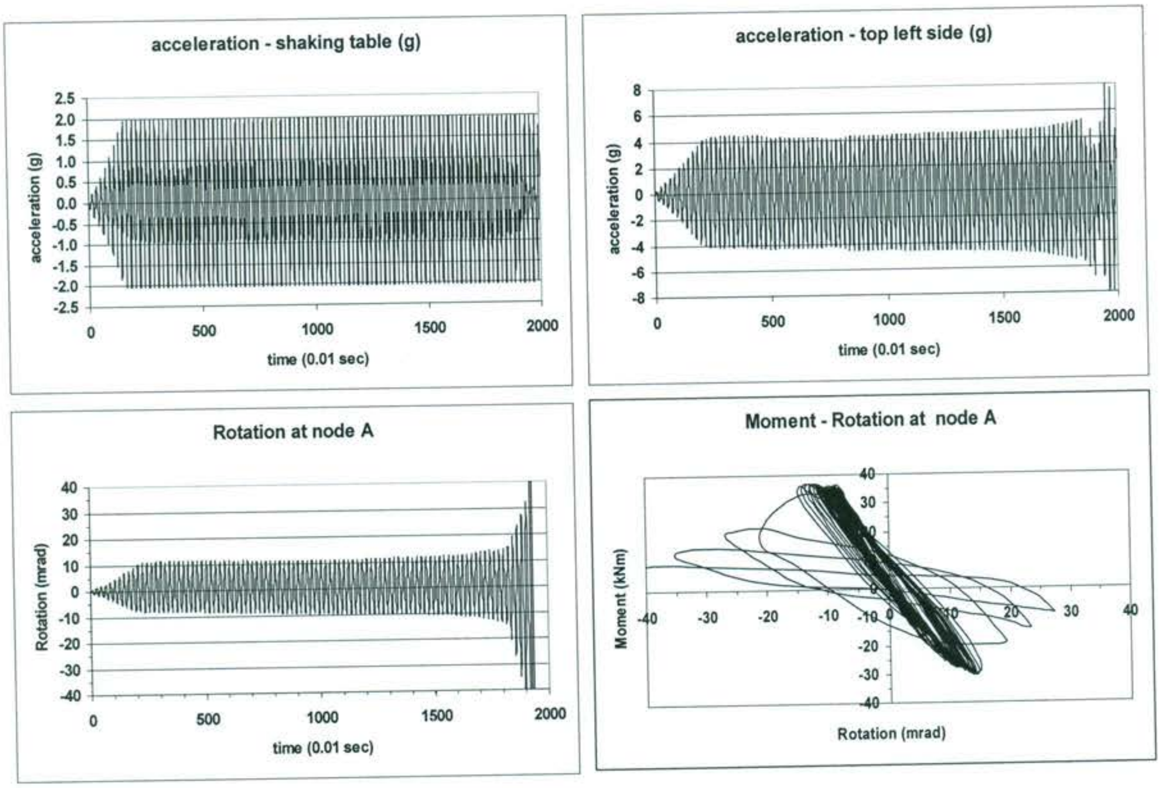


Figure 8: Diagrams for specimen STF3

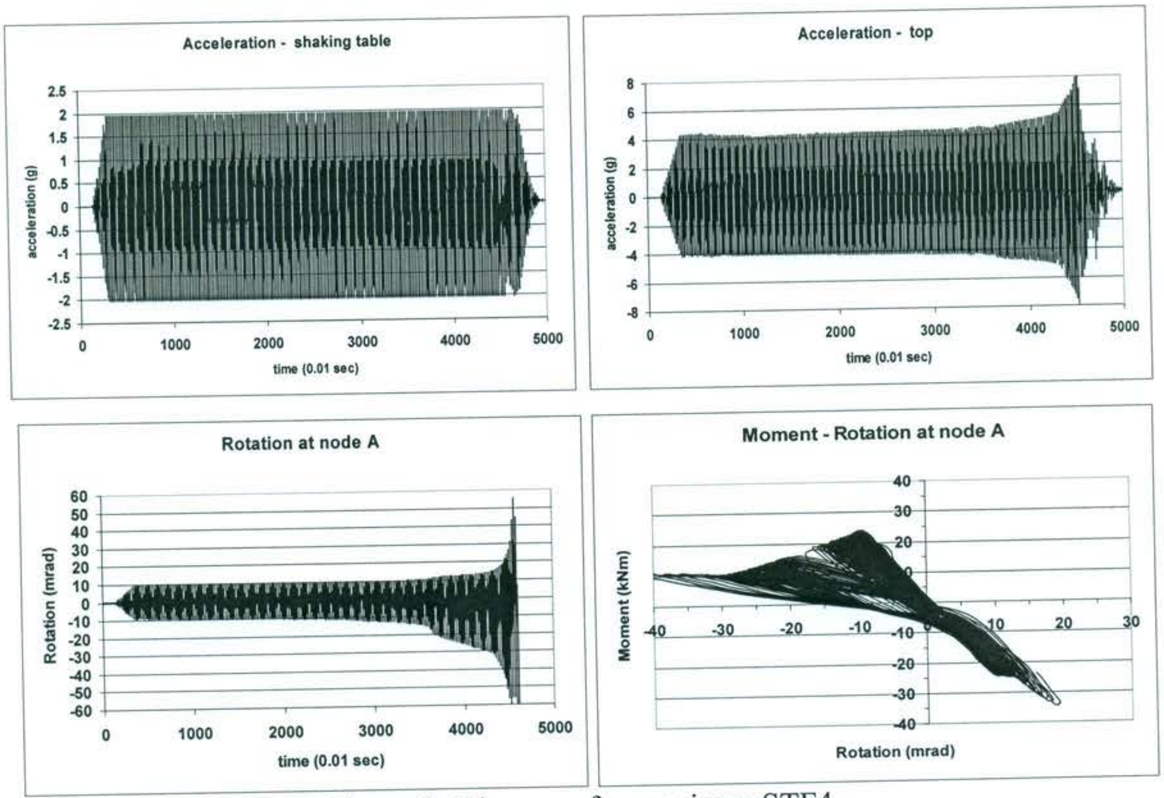


Figure 9: Diagrams for specimen STF4

The bending moments plotted in these figures are to be considered as effective elastic bending moments and were obtained as a linear extrapolation of the values obtained by strain gauge recording in the instrumented cross sections of the beams and the columns. It can be noticed that after nearly 18 seconds for specimen STF3 (Fig. 8) and nearly 35 seconds for specimens STF4 (Fig. 9), the acceleration recorded at the top of the r.c. slab begins to increase, while the input motion (i.e. the acceleration of the shaking table) remains constant. This fact indicates that some damage occurred in the structure. For both specimens collapse occurred due to failure of all the four welded beam-to-column connections. Cracks always started at the lower flange of the beam and propagated upward, in some cases completely severing the beam web from the column flange. Such type of failures are similar to those reported in many MR steel frames in California after the Northridge earthquake, despite the size of the members that in this case are very small, due to the need of keeping the size of the model to an acceptable dimension for the shaking table facility.

4 LOW-CYCLE FATIGUE DAMAGE ASSESSMENT FOR DYNAMIC TEST

Low cycle fatigue damage assessment was performed for the various beam-to-column connections of both specimens STF3 and STF4, based on Miner's rule and the S-N lines previously obtained for the joints tested in Milano, under cyclic quasi-static conditions (Fig. 6). The moment-rotation time history for each node was re-analysed by means of the Rainflow cycle counting method in order to obtain the histogram giving for each cycle amplitude the number of occurrences in the joint time-history. Then the damage index was computed according to Miner's rule, making reference to the S-N lines derived in terms of rotations, i.e. assuming as parameter S the relative rotation of the beam to the column, Ballio & Castiglioni (1995).

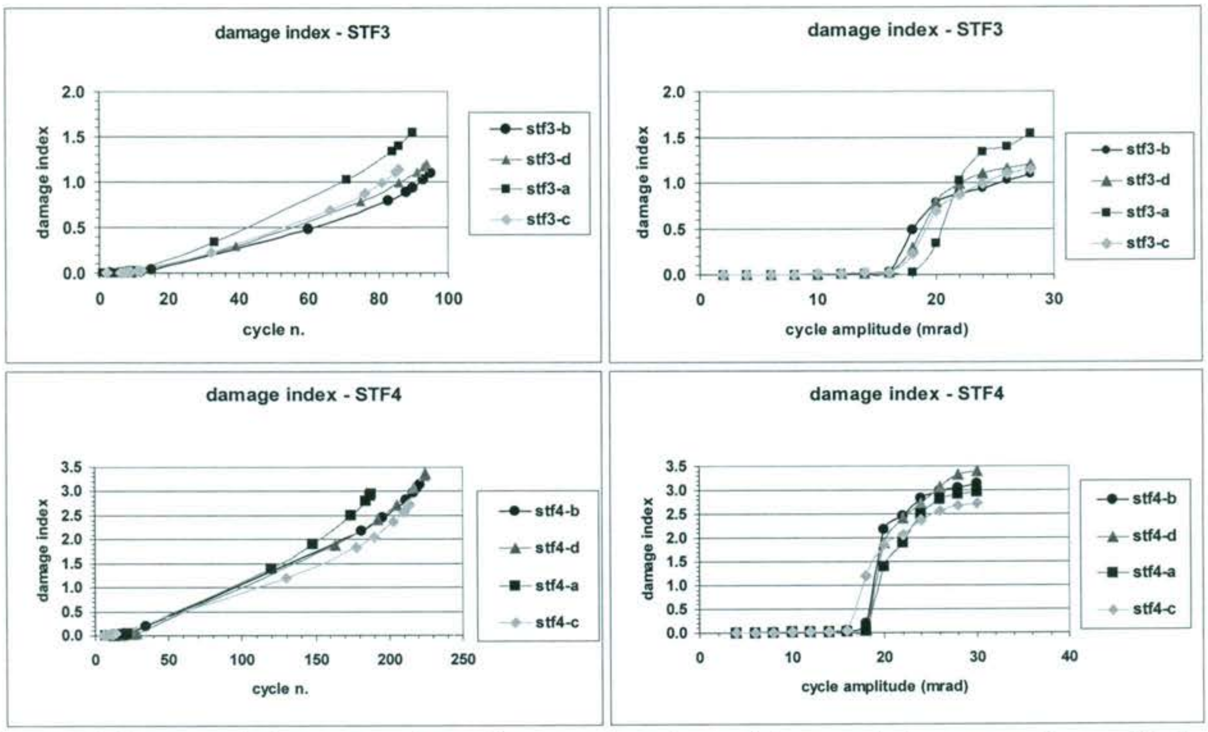


Figure 10: Damage index vs. number of executed cycles and damage index vs. cycle amplitude

Figure 10 shows for specimens STF3 and STF4 how the damage cumulates in the various nodes of each specimen. For both specimens, it can be noticed that collapse is caused by those cycles with amplitudes ranging from 15 to 22 mrad, i.e. by those cycles having amplitudes nearly 3 to 4 times the yield rotation. It fact, it is corresponding to cycles having this amplitude that the cumulative damage index attains a value of unity. Examining these figures, however, it can be concluded that the proposed damage assessment procedure allows a correct interpretation of the physical reality, in fact the results obtained for both specimens in terms of damage index are absolutely compatible with a definition of failure of the connection in good agreement with the experimental evidence.

5 CONCLUSIONS

Results of dynamic tests on full-scale frame specimens are in good agreement with those of cyclic quasi-static tests on joints. In particular, they confirmed the validity of a seismic damage assessment procedure based on S-N lines (expressed in terms of global displacement components) and of a linear damage accumulation model (e.g. Miner's Rule). Despite the size of the specimens, that had to be kept limited due to the shaking table capacity, the experimental results showed failure modes for the beam-to-column joints similar to those reported after Northridge and Kobe earthquakes, in structures with large size members and joints.

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