Dynamic tests on steel pallet racks

Prove dinamiche su scaffalature porta-pallet

This paper describes the results of dynamic tests carried out on four full-scale steel storage pallet racks at the Laboratory of Earthquake Engineering of the National Technical University of Athens.
The experimental campaign is part of a research project sponsored by the EU within the ECOLEADER Research Programme for Free Access to Large Scale Testing Facilities by the F.E.M. (European Federation of Maintenance) and MIUR (Italian Ministry for Education, University and Research).
The four specimens were chosen from six structures produced by two different manufacturers, designed to sustain a ground acceleration of 0.075g, 0.15g and 0.35g respectively, on the basis of the EC-8 standard.
After determining the own frequencies of the structures, these were tested under dynamic conditions up to failure point or to a serviceability limit state.
The paper demonstrates that pallet sliding occurs at ground acceleration rates lower than the design rate. A sliding test was also carried out to determine and better investigate this phenomenon, which represents a limit state for racking systems.

In questo lavoro, si presentano i risultati di una serie di prove dinamiche eseguite, presso il Laboratorio di Ingegneria Sismica del Politecnico di Atene, su quattro scaffalature industriali in scala reale.
La campagna sperimentale è parte di un progetto di ricerca finanziato dalla U.E., nell'ambito del programma ECOLEADER di Libero Accesso alle Grandi Attrezzature di Prova, dalla F.E.M. (Fédération Européenne de la Manutention) e dal MIUR (il Ministero dell'Istruzione, Università e Ricerca). Sono stati utilizzati quattro campioni scelti tra sei strutture progettate da due Ditte costruttrici, per un'azione sismica di progetto rispettivamente di 0.075g, 0.15g and 0.35g, secondo le prescrizioni di Eurocodice-8.
Dopo averne caratterizzato le frequenze proprie, le quattro strutture sono state sottoposte a prove dinamiche fino a collasso o fino al raggiungimento di uno stato limite di servizio.
Si è riscontrato che lo slittamento dei pallet, posti sugli scaffali, può avvenire anche per accelerazioni al suolo inferiori rispetto a quelle di progetto. E' stata quindi effettuata una serie di prove di "slittamento" per meglio caratterizzare il fenomeno che rappresenta, di fatto, uno stato limite per le scaffalature industriali.

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1. INTRODUCTION

Despite their lightness, racking systems carry very high live load (that may be many times larger than the dead load, opposite of what happens for usual civil engineering structures) and can raise a considerable height. For these reasons they have to be properly designed.
Many difficulties arise in the prediction of the structural behaviour of pallet racks, like instability (global, local and distortional) or modelling problems (beam-upright connections stiffness, base plate anchoring) [1-7]. The behaviour of these systems is affected by the particular geometry of their structural components, by the non-linear behaviour of both the beam-to-column [1] and the base-plate joints [7] and by the high slenderness of the elements.
Due to such peculiarities, additional modelling and design rules are required for these non-traditional steel structures (not building structures, but load bearing civil construction work from an engineering point of view) and reference cannot be made to usual Structural Design Recommendations and Standards.
The most recent Design Standards for steel storage racks [8-13] provide a combined numerical-experimental approach in which the design structural analysis is supported by specific tests to evaluate the performance of the key components (members and joints).
Things are even more complicated when a storage rack is installed in a seismic zone where, subjected to an earthquake, it has to withstand dynamic forces and, in addition to the usual global and local collapse mechanisms, an additional limit state for the system is represented by the possibility of falling down of the pallets with subsequent damage to goods, people and to the structure itself. In Europe, no document (industry code or standard) is currently available for the seismic design of pallet racks, and the designers are compelled to operate with an absolute lack of references and of commonly accepted design rules. Very often they make reference to the Rack Manufacturers Institute (R.M.I.) Specifications [8-9], while the European Federation of Manufacturers of Racking and Shelving (F.E.M. X – R&S) is presently working in order to produce an official document [13].
Additional difficulties for a proper seismic engineering job in case of adjustable pallet racking are the facts that they are:

- In principle designed for a predominantly vertical loading while, depending on the seismic zone and given the first eigenvalue, horizontal seismic forces can be developed up to 5, 15 or even 30% of the vertical masses
- In general made of thin gauge cold rolled upright, beam and frame bracing sections as well as by hook-in connections with no or relatively small capacity of seismic energy dissipation

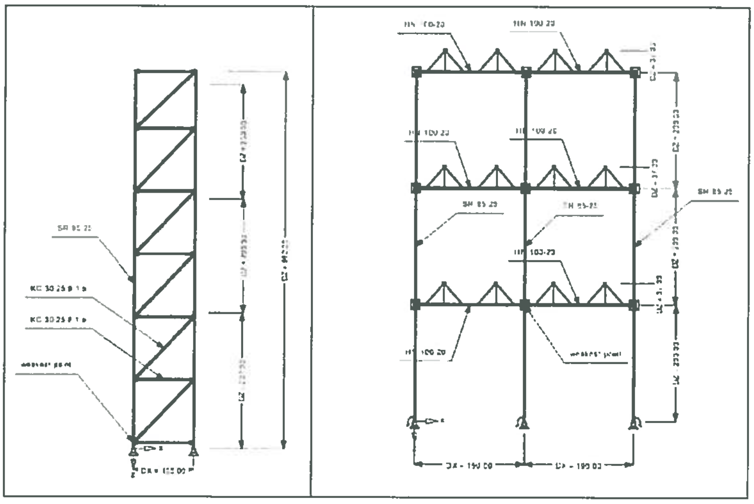

Fig. 1: bi-dimensional view of specimen A

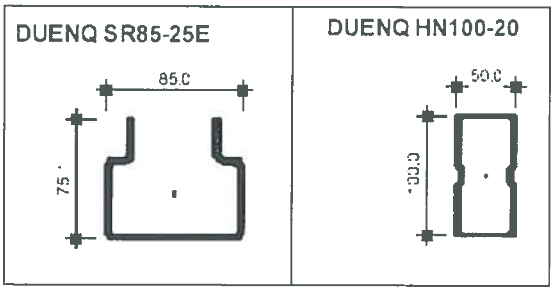

Fig. 2: cross sections of the uprights (SR85-25E) and of the beams (HN100-20) of specimen A.

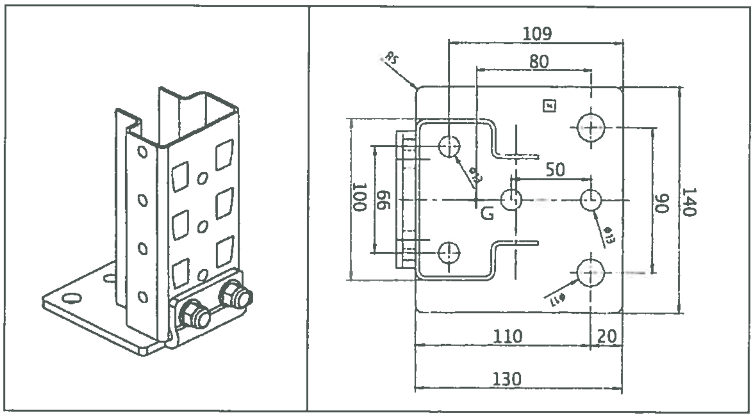

Fig. 3:details of the base-plate connections of specimens A and B

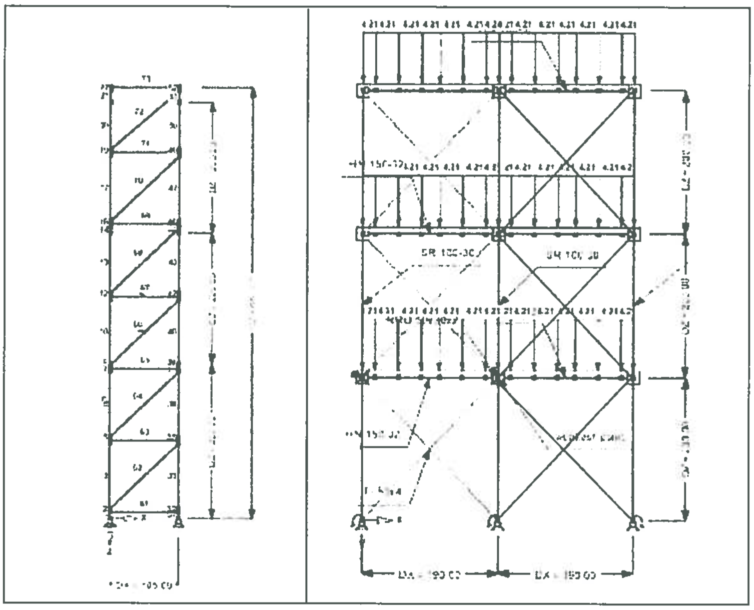

Fig. 4: bi-dimensional view of specimen B.

It must be pointed out that the seismic behaviour of steel storage racks is not only a very interesting and challenging problem from a scientific point of view, but it has also a very large economic impact for the whole society.

Racks, in fact, are widely adopted in warehouses where they are loaded with tons of (more or less) valuable goods. Loss of these goods (due to fall during an earthquake) may represent for the owner a very large economic loss, much larger than the cost of the whole rack on which the goods are stored, or of the cost for its seismic upgrade.

Racks are also more and more frequently adopted in supermarkets and shopping centres, in areas open to the public. Falling of the pallets, in this case, may endanger the life of the clients as well as of the workmen and employees, involving not only Civil and Penal Right considerations about the liability of the owners, but also economic considerations related to the insurance coverage.

In fact, sliding of the pallets on the racks and their consequent fall represents a serviceability limit state i.e. a situation that might occur during a seismic event also in the case of a well designed storage rack, the phenomenon depending only on the dynamic friction coefficient between the pallet and the steel beam of the rack and the support conditions of the pallet. If sliding occurs and a pair of beams only directly supports the pallet, the chance of pallets falling down during an earthquake is very large. Only in case additional depth beams (or cross bars) are used, the time of the earthquake of approximately 5 to 15 seconds might be sufficiently small that sliding will not lead to falling down, but only to an overloading of one of the beams.

Many times, after an earthquake event, loss of goods was reported, with or without contemporary failure of the steel rack structural system. Most probably, these structural failures might be a consequence of the fall of the pallets and of the impact of the goods on the beams at the lower levels, creating a progressive dynamic collapse.

The uncertainties associated with a clear assessment of the causes of such failures (if they were due to structural design faults or if they were caused by fall of the pallets) may result in long quarrels among engineers, constructors, users and insurance companies.

By this brief introduction it appears that although these structures, made by thin-walled (and many times cold-formed) steel profiles, are very light and represent only a small percentage of the annual sales of steel profiles in the world, very large economic interests, as well as Civil and Penal Right liability problems might arise as a consequence of an earthquake event striking them.

In order to study the structural aspects of the problem, with the aim to contribute to a clarification of this complex topic, within the ECOLEADER Research Program for Free Access to Large Scale Testing Facilities, the research project "Seismic Behaviour of Pallet Rack Systems" has been set up, planning both a theoretical and experimental study of racks subject to dynamic forces [14, 15].

Two manufacturers, referenced in this paper as manufacturer M1 and manufacturer M2, provided six different specimens to be dynamically tested in full

scale on the shaking table facility of the Labora-tory for Earthquake Engineering at the National Technical University of Athens.
Of these six structures only four were tested: they are labelled A and B for the manufacturer M1 and C and D for the manufacturer M2. For space's sake the results reported in the paper refer only to specimens B and D, designed for a ground acceleration of 0.35g.
In addition an investigation was performed in order to study the sliding of the pallets supported by the rack beams. The obtained results allowed the definition of the sliding conditions to be assumed in the assessment of the seismic behaviour of these structures.

2. THE SPECIMENS

The following pallet-racks' characteristics have been required to the manufacturers:

- two bays of 1800 mm;
- 3 levels;
- width of the upright frame: 1100 mm;
- total height of the structure about 6000 mm;
- load for pair of beams: 16 kN, for a total of 96 kN;
- Seismic action: 0.075g, 0.15g and 0.35g.

All the structures complied the requests, except specimen D that was designed for a load of 8 kN for each pair of beams.
In order to reproduce real service conditions, six concrete blocks (reproducing the concrete slab of a warehouse foundation) were fixed to the shaking table by means of four anchor bolts. The racks were clamped to these concrete blocks by means of two chemical anchorages for each base-plate. As a consequence, the base of the racks was at about 50 cm above the shaking table level.
Furthermore, like in usual commercial installations in seismic zones, all the beam-upright connections were equipped with a pair of locking pins whose effect is to prevent the unintentional release of the beams.
Hereafter, a brief description is given of the main characteristics of the four specimens.

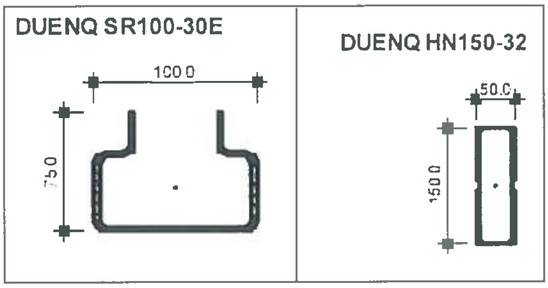

Fig. 5: cross sections of the uprights (SR100-30E) and of the beams (HN150-32) of specimen B

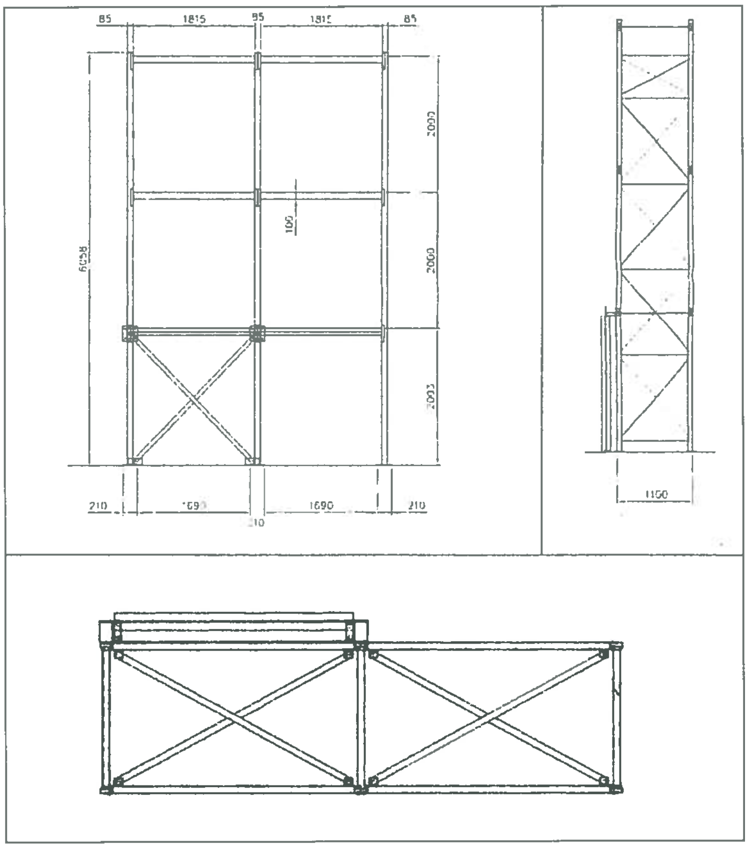

Fig. 6: bi-dimensional view of specimen C

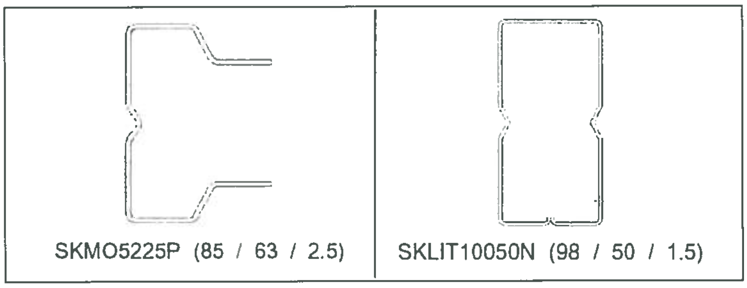

Fig. 7: cross sections of the uprights (SKMO5225P) and of the beams (SKLIT10050N) of specimen C

2.1 Specimen A

Specimen A is designed by Manufacturer M1 to withstand an earthquake with maximum ground acceleration of 0.075g. This model presents only the transversal bracing system, therefore the longitudinal forces are absorbed by the upright-beam connections only. A bi-dimensional view of the structure is reported in Fig. 1, while the cross sections of the uprights and beams are shown in Fig. 2. Being the design earthquake of low intensity, it has been possible to use open sections both for the uprights and for the braces, while the beams have a much stiffer rectangular hollow cross section.
The rectangular base-plate (130 x 140 mm) is connected to the uprights by means of two bolts positioned on the external side. The other two bolts, positioned in the internal side, are used to fix the base-plate to the concrete blocks simulating the floor. The eccentricity of the anchorage from the upright axis is of about 80 mm (Fig. 3).

2.2 Specimen B

Specimen B is designed by Manufacturer M1 to withstand an earthquake with maximum ground acceleration of 0.35g.
Due to the relevant dynamic design forces acting on the specimen, bracings at each level are provided both in the down-aisle direction and in plane, made of thin bars with a compact rectangular section. In the cross-aisle direction bracings are made of rectangular hollow section (Fig.4). In order to allow the positioning of the pallets, the down-aisle bracing system has an eccentricity of about 0.20 m from the rear of the rack: this produces a considerable asymmetry in the behaviour of the structure.

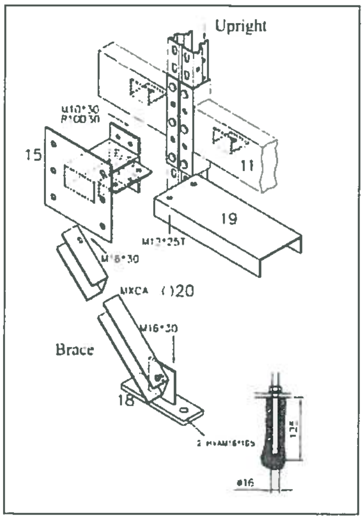

Fig. 8: detail of the anchorage system of the bracings of specimens C and D

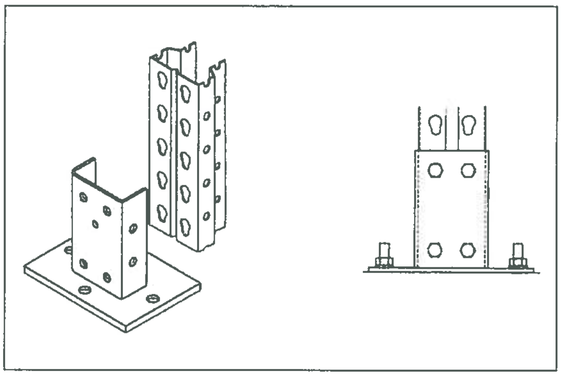

Fig 9: details of the base plate connections of specimens C and D

The beams have rectangular hollow section 150x50 mm, while the uprights are made of an open section with 100 mm width (Fig.5). The rectangular base-plate (130 x 140 mm) is connected to the uprights by means of two bolts M10x25 positioned on the external side. The other two bolts M12x130, positioned in the internal side, are used to fix the base-plate to the concrete blocks. As for specimen A, the eccentricity of the anchorage from the upright axis is of about 80 mm (Fig. 3).

2.3 Specimen C

The specimen C is designed to withstand an earthquake with maximum ground acceleration of 0.15g. A bi-dimensional view of the structure is reported in Fig. 6 while the cross sections of the uprights and beams are reported in Fig. 7. Different solutions were adopted for the bracing systems and the base-plate with respect to specimens A and B. In this specimen the cross-aisle bracing system extends over all the rack height and is made of thin drilled bars, much weaker than the bars used by the first manufacturer. The goal of this different conception is to dissipate the energy by the local plasticization that should occur near the holes. On the contrary the bracing system in down-aisle direction is made of rigid beams and is provided only for the first level of one bay. A detail of the anchoring system of the down-aisle bracing is drawn in Fig. 8. As in the previous specimens, the down-aisle bracing system introduces relevant asymmetries in the behaviour of the structure. The base plates are rectangular, (210 x 130 mm), with four holes, but only two are used in order to fix the rack to the floor by means of bolts M16x165. The upright is fixed to the base plate by means of four bolts M16x30, so that the eccentricity between its axis and the anchor bolts is negligible (Fig. 9).

2.4 Specimen D

The specimen D is designed to withstand an earthquake with maximum ground acceleration of 0.35g. A bi-dimensional view of the structure is reported in Fig.10 while the cross sections of the uprights and beams are reported in Fig.11.

As in specimen C the cross-aisle bracing system is made of thin drilled bars and extends over all the height of the upright, while the other bracing systems (both in down-aisle direction and in plane) are made of rigid beams. The down-aisle bracing system extends only for the first two levels of the rack and is connected to the main structure by means of the same detail adopted for specimen C (fig. 8), hence introducing relevant asymmetries in the global structural behaviour. As for specimen C, the base plates are rectangular, (210 x 130 mm), with four holes, but only two are used in order to fix the rack to the floor by means of bolts M16x165. The upright is fixed to the base plate by means of four bolts M16x30, so that the eccentricity between its axis and the anchor bolts is negligible (Fig. 9).

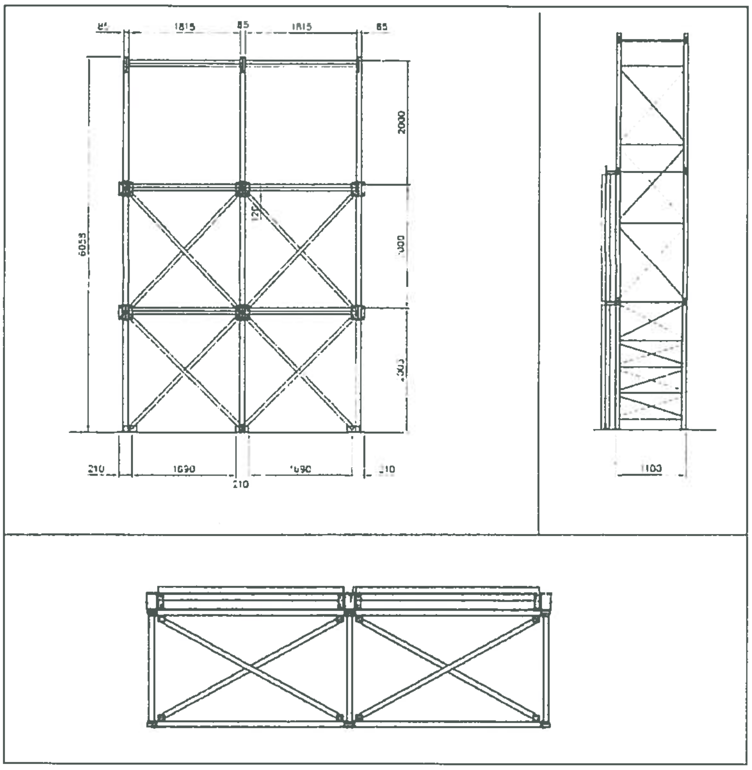

Fig. 10: bi-dimensional view of specimen D

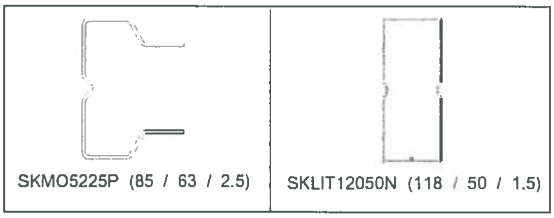

Fig. 11: cross sections of the uprights (SKMO5225P) and of the beams (SKLIT12050N) of specimen D

3. DYNAMIC TESTS

The dynamic tests on racking systems were performed at the Laboratory for Earthquake Engineering of the National Technical University of Athens. The laboratory facilities include a square shaking table, 4 m per side, with 6 degree of freedom controlled either by means of an analogical or digital unit with maximum accelerations of 1.5g in horizontal direction and 2.9g in vertical direction, while the maximum displacement is of ±0.10 m in any direction.

The load was applied to the racks by means of concrete blocks, with a weight of 8 kN each, placed on euro-pallets.

The specimens were subject to different excitations in order to identify the eigenfrequencies and, if possible, the level where damage occurs.

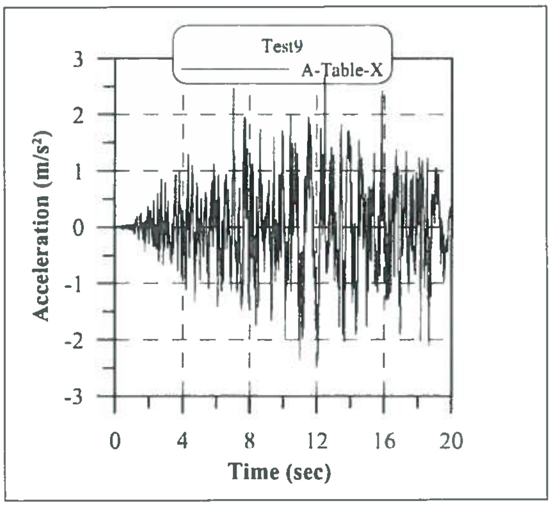

Fig. 12: acceleration time history adopted for the earthquake tests

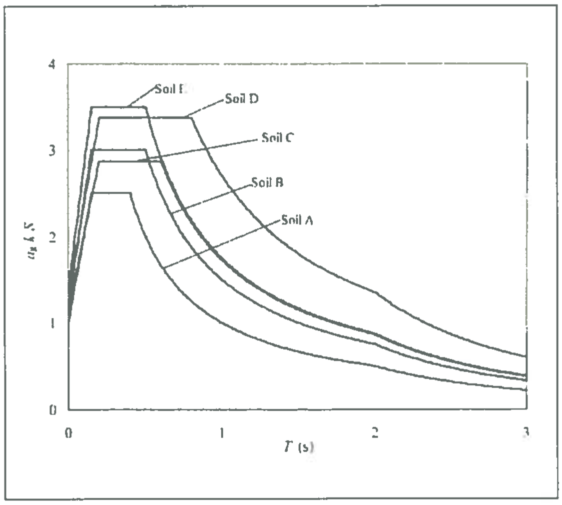

Fig.13: elastic response spectrum type 1 from EC8

The different types of test performed are:

- sine sweep: a sinusoidal vibration with variable frequency in order to obtain the first eigenmodes of the structure. The frequency of the vibration doubles every minute;
- single impulse: an initial perturbation that provides free oscillations to the racks;
- earthquake: this is an artificial earthquake (Fig. 12), numerically generated fitting the EC8 acceleration spectrum type 1, for subsoil type "D" (Fig. 13);

The earthquake tests were performed increasing the maximum ground acceleration until either the design value was reached, or sliding of the pallets occurred. Each earthquake test was followed by a sine sweep test, in order to monitor the damage suffered by the structure during the earthquake test, through evaluation of the changes in both the eigenfrequency and the damping factor.

During the tests, accelerations and displacements were recorded in the most significant points. Furthermore, six instruments were applied to the upright-beam nodes in order to evaluate the relative rotations (Fig. 14).

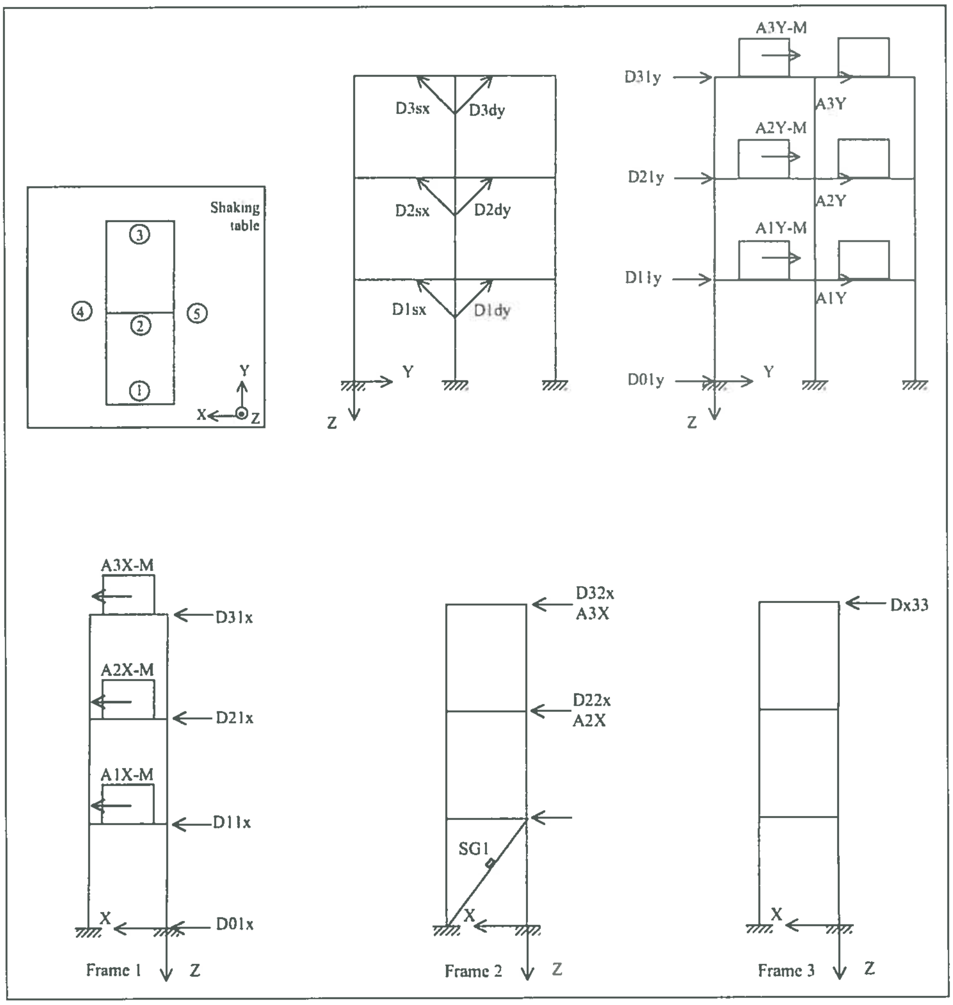

Fig. 14. Instrumentation of the specimens

In the following, the test history as well as the experimental results will be reported only for specimens B and D and for the sliding study.

The list of tests performed on specimen B is reported with their description in Table 1, while Table 2 shows the list of tests performed on specimen D.

Although both structures were designed to withstand an earthquake with maximum ground acceleration of 0.35g, it has not been possible, in both cases, to exceed a base acceleration of 0.20g both in down-aisle and in cross-aisle direction because sliding of the pallets occurred.

During the tests it was also noticed that, as expected, the presence of the bracing system, eccentric with re-

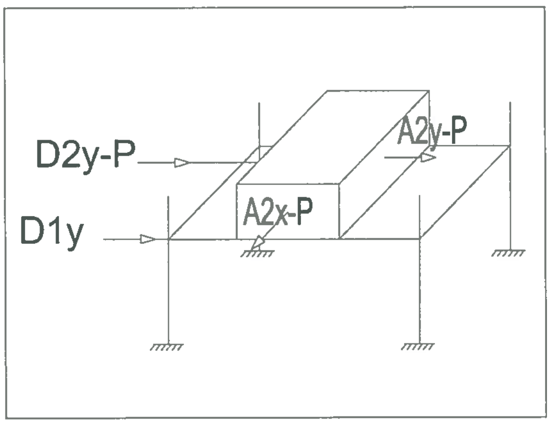

Fig. 15: instrumentation on the specimen for the sliding tests

Test n°	Type	Direction	notes
1	Sine-sweep	Down-aisle	From 1 to 8 Hz
2	Sine-sweep	Cross-aisle	From 1 to 8 Hz
3	Earthquake	Down-aisle	$a_{max} = 0.10g$
4	Sine-sweep	Down-aisle	From 0.5 to 4 Hz
5	Earthquake	Cross-aisle	$a_{max} = 0.10g$
6	Sine-sweep	Cross-aisle	From 0.5 to 4 Hz
7	Earthquake	Down-aisle	$a_{max} = 0.20g$
8	Sine-sweep	Down-aisle	From 0.5 to 4 Hz
9	Earthquake	Cross-aisle	$a_{max} = 0.15g$
10	Sine-sweep	Cross-aisle	From 0.5 to 4 Hz

Table 1: test history for specimen B

Test n°	Type	direction	notes
1	Sine-sweep	Down-aisle	From 0.5 to 4 Hz
2	Sine-sweep	Cross-aisle	From 0.5 to 4 Hz
3	Earthquake	Down-aisle	$a_{max} = 0.10g$
4	Free vibration	Down-aisle	
5	Earthquake	Cross-aisle	$a_{max} = 0.10g$
6	Free vibration	Cross-aisle	
7	Earthquake	Down-aisle	$a_{max} = 0.20g$
8	Free vibration	Down-aisle	
9	Earthquake	Cross-aisle	$a_{max} = 0.20g$
10	Free vibration	Cross-aisle	
11	Earthquake	Random	$a_{max} = 0.20g$

Table 2: test history for specimen D

Test n°	Frequency (Hz)	Base acceleration (g)
1	1.00	0.190
2	1.00	0.210
3	1.00	0.231
4	1.00	0.244
5	1.00	0.269
6	1.00	0.286
7	2.00	0.222
8	2.00	0.239
9	2.00	0.256
10	2.00	0.271
11	3.00	0.183
12	3.00	0.197
13	3.00	0.211
14	3.00	0.225
15	3.00	0.239
16	3.00	0.251
17	3.00	0.265
18	4.00	0.201
19	4.00	0.205
20	5.00	0.179
22	6.00	0.123
21	6.00	0.139

Table 3: test history for the sliding study

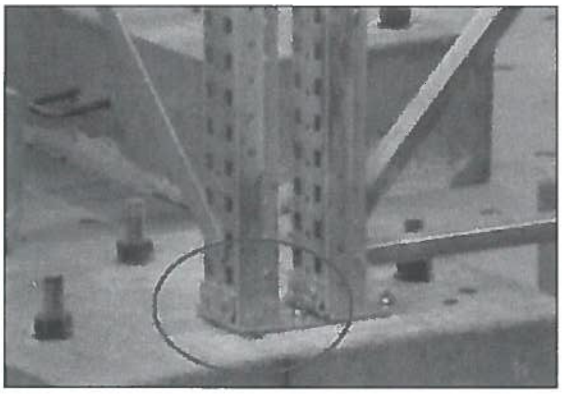

Fig. 16: detail of the bending at the base-plates of specimen B during the tests.

Test n°	1^{st} freq. (Hz)	2^{nd} freq. (Hz)	Damping factor
1	1.34	3.00	0.00734
2	1.83	6.01	0.00763
4	1.29	3.05	0.0085
6	1.554	---	0.00905
8	1.084	3.04	0.011
10	1.27	---	0.0093

Table 4: eigenfrequencies and damping factors of specimen B

spect to the centre of gravity of the masses applied to the structure, is responsible for the asymmetric behaviour of the racks, having a second eigenmode of torsional type. In fact, during the tests with a down-aisle excitation, an evident global torsion of the structure was observed while, during the tests with an excitation in the cross aisle direction, no torsion occurred. A sliding study was also performed positioning one pallet over a pair of beams at about 0.30 m over the base plate level, and instrumenting this simplified test set-up with two accelerometers recording both longitudinal and transversal acceleration of the pallet and two displacement transducers recording the relative displacements of the structure and of the pallet with respect to the shaking table (Fig 15). Various tests were performed with a sinusoidal excitation, with different combinations of acceleration and frequency (Table 3) in order to evaluate the conditions that lead to sliding.

4. RESULTS

4.1 Specimen B

In all the tests performed the base-plates (having an eccentricity of 80 mm with respect to the upright axis) showed relevant bending due to plastic deformations (Fig. 16).
The down-aisle bracing system, at the first level, had to be checked after every test and bolts were re-tightened if necessary. The pallets slide considerably during the earthquake test in down-aisle direction with $a_{max} = 0.20g$ (test 7). The bolts of the base-plates loosened after the earthquake test in cross-aisle with $a_{max} = 0.15g$ (test 9) and had to be re-tightened before performing the next test.
The high stiffness of this specimen allowed performing various earth-

quake tests, each of them followed by sine-sweep test. Table 4 shows the eigenfrequencies and the damping factors obtained from re-analysis of the acceleration data recorded at the third level.
The sequences of tests 1-4-8 and of tests 2-6-10 show a reduction of the first eigenfrequency due to a possible damage of the rack. Fig. 17 shows the residual displacements recorded at the end of each test. An evident damage occurred during test n. 7 as highlighted from the residual displacement (Fig. 17) and from the different eigenfrequencies recorded between test 4 and 8 (Table 4). Test 7 corresponds to the earthquake simulation in down-aisle direction with maximum acceleration of 0.20g. During the same test, sliding of the pallets occurred and it was considered unsafe to perform further tests with increased base acceleration. The limit state of the racking system was therefore reached not for "structural" but "functional" reasons (see sliding graphs in Fig. 19). Fig. 18 shows the deformed shapes of an upright, recorded at three different moments of test n. 1. Although the excitation was in down-aisle direction, the upright shows lateral deflections also in cross-aisle direction, that are at every level larger than the one in the down-aisle direction. This is due to the presence of the bracing system at the rear of the rack that, as already mentioned, is eccentric with respect to the centre of gravity of the applied masses and induces torsional effects in the behaviour of the structure. The pallets stood on the beams, but were not fixed. For high accelerations, sliding of the pallets occurred and it can be detected observing the sliding accelerations obtained as difference of the accelerations of the masses and of the structure at the same level. Fig.19 shows that the acceleration of sliding of the pallets, already at the second level, is greater than 0.5 m/s² at the time t = 24 sec., corresponding to a frequency of 1.31 Hz, to a base acceleration of 0.5 m/s² and to an acceleration of the racks of 1.5÷1.6 m/s².

4.2 Specimen D

Large out-of-plane deflections of the thin drilled bars in the cross-aisle bracing system could be observed, at the end of the tests; this fact means that inelastic elongations occurred under loading, most probably due to plastic deformations in the net sections around the holes. Hence, the bracing system lost part of its effectiveness during the tests.
As the bracing system in the down aisle direction stops at the second level, the third level of this specimen behaved as a sort of "soft floor", where most of the deformability of the structure is concentrated both in the down-aisle and in the cross aisle direction (because of the torsional effects induced by eccentricity of the bracing system).
Differently from specimen B a free vibration test was performed after every earthquake test.
Table 5 shows the eigenfrequencies and the damping factors obtained from re-analysis of the acceleration data recorded at the third level.
The sequence of tests 1-4-8 shows a reduction of the first eigenfrequency due to a possible damage of the rack. Fig. 20 shows the residual displacements recorded at the end of each test. An evident damage occurred during tests n. 4 and n. 8 as highlighted from the residual displacement (Fig. 20) although Table 5 shows a reduction in the corresponding estimated

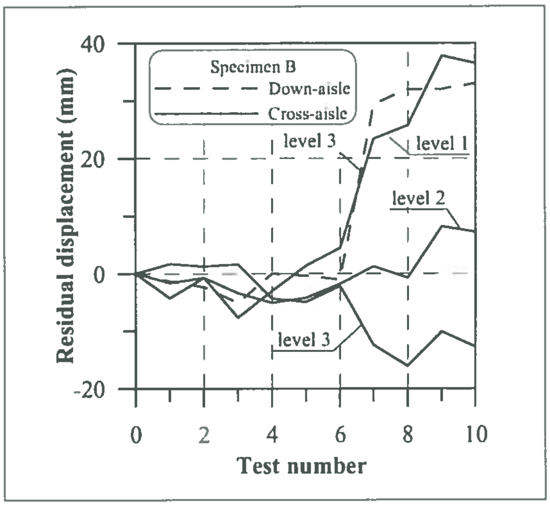

Fig. 17: residual displacements of specimen B recorded after every test

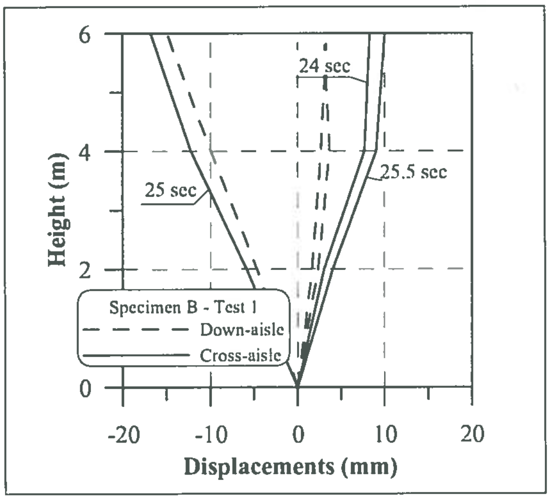

Fig. 18: shapes assumed by the upright of the specimen B at three different moments during the test 1: sine-sweep in down-aisle direction

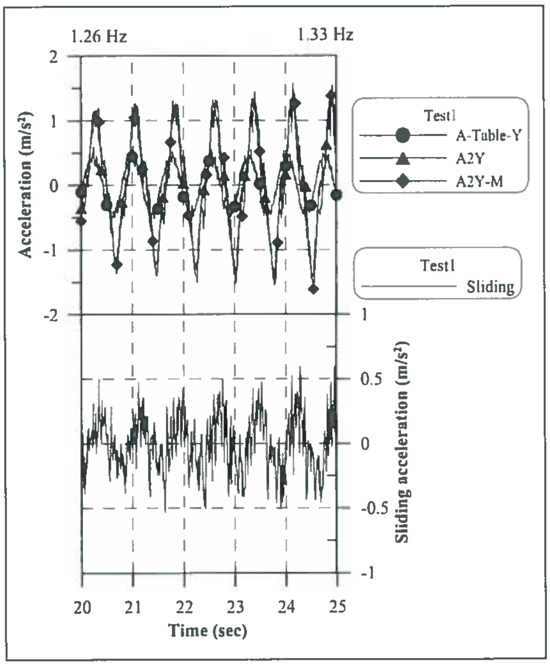

Fig. 19: sliding of the pallets at the second level of specimen B, during the test 1

Test n°	1st freq. (Hz)	2nd freq. (Hz)	Damping factor
1	1.58	2.40	0.0144
2	1.28	2.62	0.04
4	1.31	---	0.08
8	1.13	---	0.0417

Table 5: eigenfrequencies and damping factors of specimen D

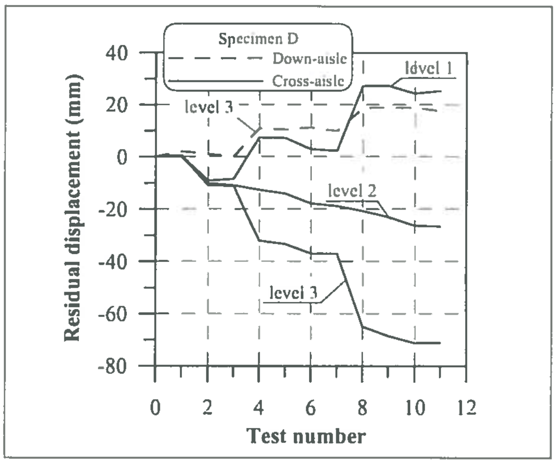

Fig. 20: residual displacements of specimen D recorded after every test

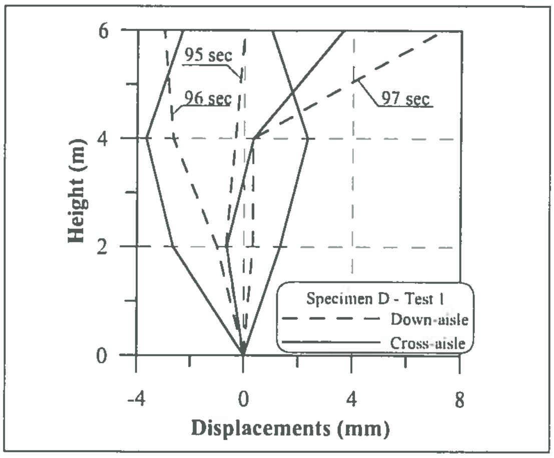

Fig. 21: shapes assumed by the upright of the specimen D at three different moments during the test 1: sine-sweep in down-aisle direction

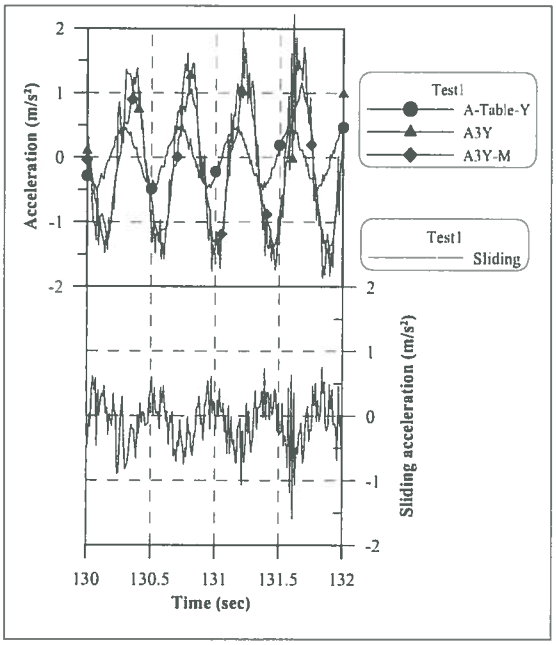

Fig. 22: sliding of the pallets at the third level of specimen D, during the test 1

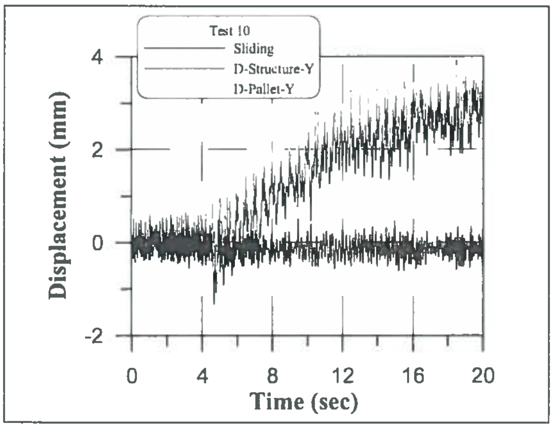

Fig. 23: sliding test displacements recorded during test n. 10 (f = 2.0 Hz, a = 0.27g)

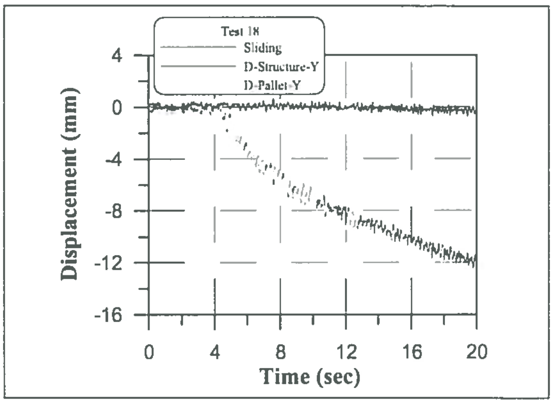

Fig. 24: sliding test displacements recorded during test n. 18 (f = 4.0 Hz, a = 0.20g)

damping factor at test n. 8.

The largest residual displacement was recorded in cross-aisle direction, although tests 4 and 8 correspond to free vibration tests in down-aisle direction. This effect is the consequence both of the high eccentricity of the structure in down-aisle direction and of the weak bracing in cross-aisle direction.

Fig. 21 shows the deformed shapes of an upright, recorded at three different moments of test n. 1, a sine-sweep in the down-aisle direction. It is clear that the structural behaviour is very different from that of the specimen B, due to the different down-aisle bracing system. In particular, the upper level suffers the absence of the bracing system and its displacement is much larger than the displacements of the other two levels. Furthermore, while at the two lower levels, as observed for specimen B, the cross-aisle deflections are larger than the down-aisle ones, an inversion occurs at the third level. As in the case of specimen B, for a ground acceleration of 0.20g the pallets slid over the rack beams, and it was considered unsafe to perform further tests with increased base acceleration. The limit state of the racking system was therefore, also in this case, not "structural" but "functional" (see sliding graphs in Fig. 22). The sliding acceleration at the third level is reported in Fig. 22, and it is greater than 1.0 m/s^2 at the time t = 131.5 sec., corresponding to a frequency of 2.28 Hz, to a base acceleration of 0.5 m/s^2 and to an acceleration at the third level of the rack of about 2.0 m/s^2.

4.3 Sliding tests

Sliding of the pallet, at different combinations of frequencies and acceleration, corresponding to tests 10 (frequency 2.0 Hz, and acceleration 0.27g) and test 18 (frequency 4.0 Hz, and acceleration 0.20g), is reported respectively in Fig. 23 and in Fig. 24. Due to

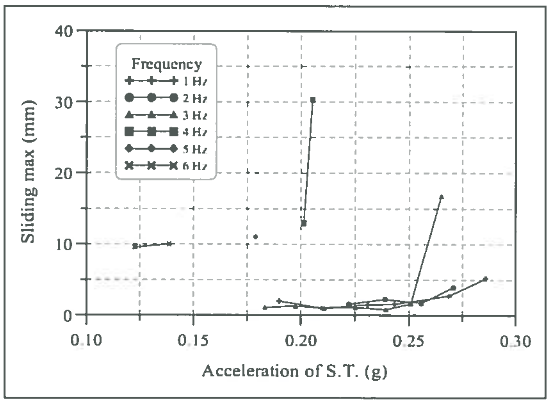

Fig. 25: correlation of the sliding of the pallets to both acceleration and frequency of the excitation

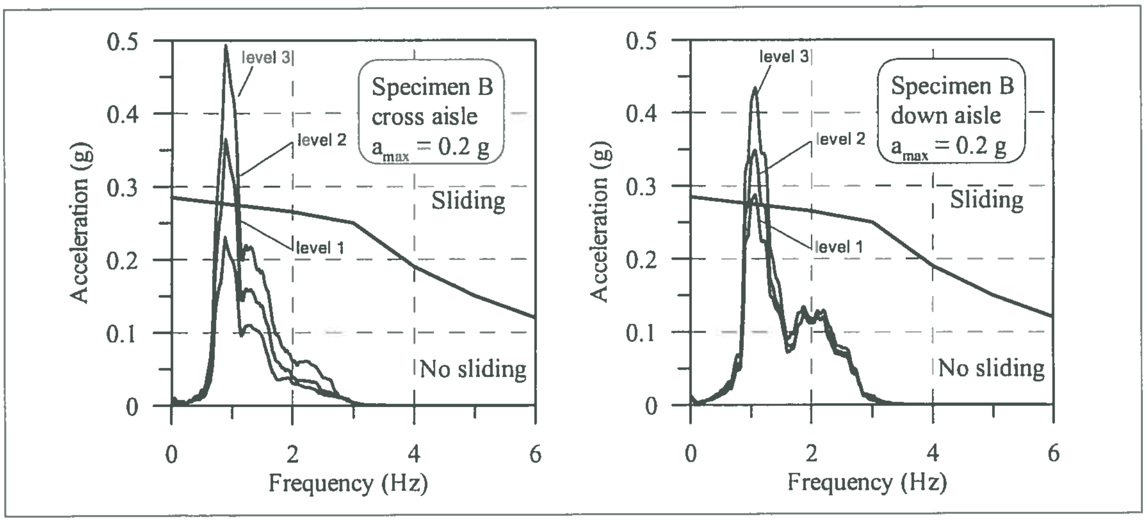

Fig. 26: frequency content of specimen B response – test n. 9 and test n. 7

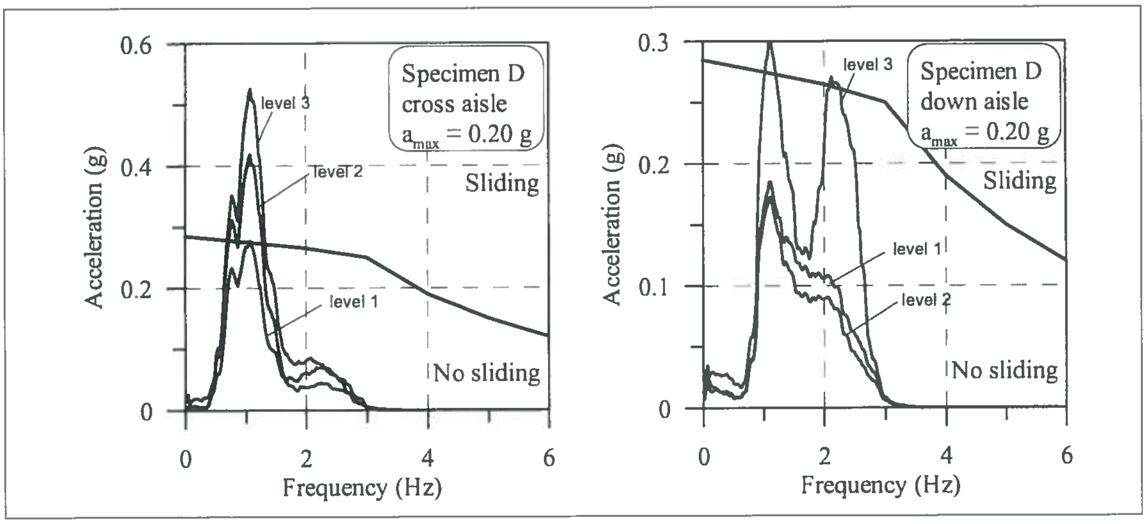

Fig. 27: frequency content of specimen D response – test n. 9 and test n. 7

the small distance between the shaking table and the beam where the pallet rests, it is considered that the structure has the same acceleration of the shaking table, disregarding any amplification factor. It can be noticed that increasing the frequency of the excitation results in larger sliding at lower accelerations.
This seems to be a general consideration, as it can be observed in Fig.25 that shows the correlation of the sliding of the pallet with the acceleration, for different frequencies of the excitation. There is a very strong dependence of the phenomenon on the frequency. For a frequency of 1.0 Hz, sliding occurs at an acceleration of nearly 0.28g, while increasing the frequency the sliding acceleration (i.e. the dynamic friction coefficient) reduces, becoming nearly 0.24g at 3.0 Hz, and at higher frequencies seems to become lower than 0.10g. At the same time, the displacement of the pallet becomes larger and larger, increasing the frequency, under the same acceleration. Through a re-analysis of these data, it was possible to derive the domain, in terms of frequency and acceleration, defining the conditions for which the pallet will or will not slide. It has to be noticed that the data reported in fig. 25, and hence the sliding conditions, refer and are valid for the case of wooden pallet loaded by a rigid block, considered in this study. If different material will be used, either for the pallet (e.g. plastic instead of wood) or for the mass set on the pallet (e.g. a barrel full or half full of liquid, a case of cans, boxes of granular or loose material, etc.), the sliding conditions will most probably be different.
In the following Figs. 26 end 27 the "sliding-no sliding" domain is compared to the data recorded during the earthquake tests performed both in cross-aisle and down-aisle directions with base acceleration of 0.20g, respectively for specimen B and D, and re-analysed in terms of frequencies and accelerations.
It can be seen that, for specimen B (Fig. 26) sliding should occur at all three levels during the earthquake in the down-aisle direction, while in the cross-aisle direction, the pallets could not slide at the first level only.
For specimen D (Fig. 27), sliding could occur at all the three levels in the cross-aisle direction, while in the down aisle direction, sliding occurred only at the third level. This fact may be due either to the lower load applied to the structure or to the bracing system (stopped at the second level). It is also interesting to notice the peculiar shape of the response of the third level, showing two peaks (Fig. 27). This is probably due to the already discussed shape of the down-aisle bracing system that is absent at the third level.

5. CONCLUSIONS

At the end of this experimental research on the seismic behaviour of steel storage racks, the following conclusions can be drawn:

- The steel structures designed for low seismicity area (0.075g and 0.15g) did not suffer any serious damage;
- The steel structures designed for medium-high seismicity area (0.35g) have been tested up to a maximum ground acceleration of 0.20g because of the sliding of the pallets and did not show any serious damage although permanent deformations were observed in some structural details;
- Sliding of the pallets represents a "serviceability" limit state for storage racks, occurring at base accelerations lower than the seismic design ones. In fact, as the pallet is only a few centimetres larger than the structure, the sliding values recorded during the tests show that fall of the pallets can occur at accelerations much lower than the design one, with the following consequences: loss of the goods; injuries to people; damage to the structure eventually resulting in a global collapse due to the impact.
- The sliding of the pallet can be verified by comparing the sliding conditions obtained from a sliding test with the frequency content of the response of the structure at each level.

Further research is needed in order to achieve a clear assessment of the sliding conditions of the pallets during service situations. Sliding tests should be carried out with different types of pallets and stored goods, and with a full grid of acceleration-frequency combinations, in order to derive "sliding-no sliding" domains for various cases, relevant in the common practice of commercial installations. This seems to represent a key point in the development of a comprehensive strategy for the seismic design of steel storage racks. These structures, in fact, seem to be "smart" enough to protect themselves from the seismic actions, through sliding of the pallets, that works as a "fuse", limiting the upper value of the seismic forces challenging the structure itself. In fact, sliding of the pallets and deformations of the palletised goods will probably contribute to the dissipation of the seismic energy and therefore will lead to a limitation of the horizontal seismic forces (that cannot be larger than the horizontal friction forces).If sliding of the pallets were somehow prevented, the full seismic action should be resisted by the structure that, as a consequence, should be strengthened resulting in a much heavier design than the actual one. On the other side, allowing sliding of the pallets represents a choice with many implications related not only to the structural design, but also to economic, safety as well as liability considerations. In case sliding of load units may occur, additional support to the "straight forward2 pair of beams should be provided (e.g. depth beams or cross bars), and a certain "overload" of one of the beams (which one cannot be known) should be taken into account. It is the author's opinion that a very promising, and probably the only, technique to approach the seismic design of steel storage racking systems is the base isolation. Of course, the base isolation devices should be chosen with characteristics allowing elimination, from the structural response, of all those combinations of acceleration and frequency that might cause sliding of the pallets.
This may be a rather easy task for new structures, but represents a very complex task in the case of the seismic upgrading of existing installations.

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AKNOWLEDGEMENTS

The research has been carried out with the financial support of E.U., ECOLEADER Research Program for Free Access to Large Scale Testing Facilities, from the European Federation of Maintenance (F.E.M.), and from M.I.U.R.
The contribution of the two manufacturers that designed and provided, free of charge, the specimens to be tested is also deeply acknowledged.
The author wishes to thank all the personnel of the Laboratory for Earthquake Engineering of the National Technical University of Athens, which all contributed to the success of this research; a particular mention to prof. P.Carydis, Director of the Laboratory, colleague and friend, for his invaluable advices and guidance and to dr. H.Mouzakis, for his skilfulness and useful advices during the testing phases.
The contribution of dr. ing. N. Panzeri and of arch. J.C.Brescianini, of the Structural Engineering Department of Politecnico di Milano is also deeply appreciated.