The Seisracks project focuses on the seismic design of pallet racking systems, of the type frequently used in commercial areas open to the public.

These structures are made of cold-formed thin-walled open cross-section profiles, with holes and openings to allow mechanical connections between members and rapid reconfiguration.

In addition to usual local and global limit states, during an earthquake an additional limit state is represented by the sliding and fall of the pallets.

At present in Europe there is no officially accepted design code for racks in seismic areas, but only the 2005 version of FEM10.2.08 (not EN).

Beam-to-upright and base-plate connections were tested to characterise their behaviour.

Full-scale pushover, pseudo-dynamic and dynamic shaking-table tests were performed for assessment of the actual structural response and ductility, leading to definition of possible Q factor values.

Assessment of the static and dynamic friction factor was achieved through full-scale sliding tests considering different types of beams and pallets.

A warehouse was continuously monitored for two years, recording accelerations caused by forklifts’ accidental impacts on the structure.

A numerical model including pallet sliding simulation capability was set up, allowing numerical parametric analysis of racks under seismic loading.

The main outcome of the Seisracks project is a revised version of FEM10.2.08, which will lead to a more uniform quality standard in design of racks in seismic areas.
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Storage racks in seismic areas

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Despite their lightness, racking systems carry very high live load (many times larger than the dead load, opposite of what happens for usual civil engineering structures) and can raise a considerable height. Prediction of the structural behaviour of pallet racks is difficult because it is affected by the particular geometry of their structural components: members made by high slenderness thin-walled, open-section profiles (hence prone to global, local and distortional buckling problems), beam-to-upright and base-plate joints exhibiting a non-linear behaviour.

Due to their 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 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).

The design needs particular attention for storage racks installed in a seismic zone, where they must be able to withstand dynamic forces. Besides the usual global and local collapse mechanisms, an additional limit state for the system is represented by the fall of the pallets with subsequent damage to goods, people and to the structure itself.

In Europe, no official document is currently available for the seismic design of pallet racks, and the designers are compelled to operate with a total lack of references and of commonly accepted design rules. Very often they make reference to the Rack Manufacturers Institute (R.M.I.) Specifications, while the European Racking Federation (F.E.M.-ERF) is presently working in order to produce an official document.

Racks are widely adopted in warehouses where they are loaded with tons of (more or less) valuable goods. The loss of these goods 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. The falling of the pallets, in this case, may endanger the life of the clients as well as that 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.

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.

Hence, solution of the problems connected with safe and reliable design of steel storage racks in seismic areas has a very large economic impact.

At present, there are technical limitations in the field of safety and design of storage racks in seismic areas: lack of knowledge on actions challenging the structures, lack of knowledge on structural behaviour in terms of ductility and sliding conditions of the pallets on the racks and lack of Standard Design Codes in Europe.

To solve some of these limitations, the EU sponsored through the Research Fund for Coal and Steel an RTD project titled “Storage Racks in Seismic Areas” (acronym SEISRACKS, Contract Number: RFS-PR-03114).

The objectives of this project, initiated in December 2004 and terminated in June 2007, are:

- to increase knowledge on actual service conditions of storage racks,
- to increase knowledge on racks’ actual structural behaviour
- to assess design rules for racks under earthquake conditions.

The research team was composed by the following units: ACAI the Italian Association of Steel Constructors (Co-ordinator), Instituto Superior Tecnico of Lisbon (P), National Technical University of...
Athens (EL), Politecnico di Milano (I), University of Liege (B) and the European Laboratory for Structural Assessment (ELSA) of the Joint Research Center at Ispra (Subcontractor of Politecnico di Milano). This author acted as Scientific Coordinator of the project.

The research activities carried out in co-operation among the partners, in order to achieve the aforementioned objectives within this project, are subdivided in the following Work Packages:

WP 1 – Full scale dynamic tests of storage racks
WP 2 – Full scale pseudo-dynamic and pushover tests of storage racks
WP 3 – In-situ monitoring of storage racks
WP 4 – Cyclic testing of components
WP 5 – Assessment of seismic design rules for storage racks

The project focuses on steel selective pallet storage racks located in areas of retail warehouse stores and other facilities, eventually accessible to the general public.

Storage racks are composed of specially designed steel elements that permit easy installation and reconfiguration, consistent with the merchandising needs of a warehouse retail store. Except where adjacent to walls, storage racks normally are configured as two rows of racks that are interconnected. Pallets typically can have plan areas of approximately one square meter and can have a maximum loaded weight of approximately 10-15 kN. Storage rack bays are typically 1.0-1.1 meter deep and 1.8-2.7 meters wide and can accommodate two or three pallets. The overall height of pallet rack structural frames, found in retail warehouse stores, varies between 5 and 6 meters. In industrial warehouse facilities, racking system can reach considerable heights, such as 12-15 meters or more.

The rack industry calls the longitudinal direction the down-aisle direction, and the transverse direction the cross-aisle direction. Proprietary moment connections are typically used as the structural system in the down-aisle direction and braced frames are typically used as the structural system in the cross-aisle direction.

Results of the cyclic tests on full scale rack components (namely beam-to-upright connections and column bases) carried out at the Department of Civil Engineering and Architecture of Instituto Superior Tecnico of Lisbon are presented and discussed in chapter 2. In particular, an innovative cyclic testing procedure for structural elements and components, alternative to the classic ones recommended by ECCS (1986) and ATC(1992), has been proposed.

The seismic behaviour of beam-to-upright connections of racking systems (and more in general of beam-to-column connections showing an unsymmetrical response and damage accumulation) is determined by a hybrid loading consisting of vertical load and horizontal motion effects, and the beam-to-column connections bending moments are shifted in the negative, hogging, direction as a consequence of the vertical load effects. Failure occurs when the critical sections are no longer able to withstand vertical loads as a consequence of accumulated damage (induced by horizontal motion and vertical forces acting together).

However, general recommended testing procedures, encompassing only displacement controlled conditions, fail to address the unsymmetrical displacement histories experienced by critical beam-to-column connections when subjected to earthquake motion acting simultaneously with vertical (live and dead) loads.

Considering the former limitations of the commonly accepted testing procedures (particularly evident when testing is performed in the inverted T configuration, with the beam standing vertically and the column horizontally), an innovative testing procedure was developed which intends to capture the hybrid nature of loading imposed to beam-to-column connections when subjected to combined vertical and horizontal load effects. This testing procedure consists of "half-cycles" performed partially under "force-controlled" conditions (in order to impose to the specimen the "gravity" design load) and partially under "displacement-controlled" conditions (in order to impose to the specimens the displacements required by the ductility demand).
The innovative testing procedure can be considered a development of the commonly accepted testing procedures as it inherits some of their characteristics, such as the cycle repetition in the post-elastic range and the fact that the controlled-displacement part of the testing cycles is indexed to the yielding displacement (determined through monotonic tests).

In the same chapter 2, results of 30 tests carried out on two different types of beams (respectively 70 mm and 130 mm deep), connected to the same type of upright were presented together with those of 40 tests on column bases. Tests were carried out on the same type of column base, but different loading directions and axial load in the column, as well as different type of steel base-to-foundation connections were considered (concrete foundation or steel foundation).

The failure mode for 70 mm (hogging bending) and 130 mm (hogging and sagging bending) deep beams consisted of large deformations in the top zone of the beam end connector. In 70 mm deep beam subject to sagging bending, the failure mode was the fracture of the fillet weld between the beam and the end-plate connector. The connection behaviour for all tests was not influenced by safety bolt deformation.

The rotation capacity difference between the loading verse (sagging or hogging bending) was larger for the 70 mm deep beam due to different collapse occurred in the sagging loading.

According to Eurocode 3 (2005) the connection with a 70 mm deep beam can be considered as semi-rigid, for both beam lengths of 1.8 and 2.7 m. According to Eurocode 3 (2005) the connection with a 130 mm deep beam can be considered as semi-rigid for a beam lengths of 2.7 m, and as flexible for a beam lengths of 1.8 m (as in the case of the structures tested full scale within this research); hence, in this last case, the influence of this connection behaviour may be ignored in structural analysis.

The results of test performed with the innovative cyclic testing procedure were fundamentally different from those obtained through the application of the ECCS recommended testing procedure. In those tests performed according to the innovative cyclic testing procedure, the imposed displacement history is unsymmetrical. Displacements tend to systematically accumulate in the positive direction as a consequence of the absence of closure of the top part of the connection (the bottom part is generally closed throughout the tests). This connection behaviour also leads to a reduction of the pinching effects. Imposed forces are shifted in the positive (hogging) direction for the innovative testing procedure whereas, apart from the asymmetry that may result from unsymmetrical connection detailing, positive and negative force amplitudes for ECCS tests are not excessively different.

Failure of the connection is explicitly addressed by the innovative testing procedure since failure occurs when the connection is no longer able to withstand vertical load effects.

In the column base connections under cross aisle bending, the axial compression load is beneficial when the bolts are in the compression zone. In this case, an increase of the axial force results in an increase of resistance and stiffness but in a decrease of rotation capacity of the specimens. When the loading direction results in tension of the bolts, the axial force in the upright causes a reduction of resistance and stiffness of the column bases, because induces distortional buckling of the free edges of the cross-section profile.

The tests performed on column base connections in the down aisle direction proved that presence of an axial force of 17% and 34% of the yield load (25 and 50 kN respectively) increases the initial stiffness and the resistance but decreases the rotation capacity of the connection. In the cyclic tests, the higher the axial force, the bigger the difference between the resistance under positive and negative bending moments.

The collapse modes exhibited by the specimens were weld failure, base bending and distortional buckling of the cross section of the upright. In the tests with premature weld failure there was a reduction in the rotation capacity of the connection. Therefore, it is extremely important to control the welding process during the manufacturing of the components. Distortional buckling of the cross section of the upright occurred in the tests with an axial force equal to 34% or 52% of the yield load (respectively 50 and 75 kN), except for the monotonic tests when the
loading direction is such that bolts are in the compression zone. In the tests on column bases under bending in the cross aisle direction and axial force of 75 kN (52% of the yield load) distortional buckling occurred prematurely, drastically reducing the mechanical properties of the connection. The base welded and bolted to the steel deck exhibited an increase in the resistance and rotation capacity of the connection since there was no base plate bending and in the cyclic tests a higher capacity of energy dissipation was observed, when compared to simply bolted connections. The results proved that connecting the column base to a concrete slab or to a steel deck does not change the mechanical properties or the failure mode of the connection. In both cases pre-tension in the bolts should be provided.

Chapter 3 deals with the assessment of the friction factor between pallets and rack beams, which is governing the “pallet sliding” phenomenon. This turned out to be the most important effect governing the dynamic behavior of racking systems. These activities were part of WP1

Assessment of both the static and the dynamic sliding conditions of pallets stored on steel racking systems was carried out within the SEISRACKS research project, by means of static as well as dynamic tests performed at the Earthquake Engineering Laboratory of the National Technical University of Athens.

More than 1260 Static tests were carried out in both down and cross aisle direction, by means of an “inclined plane” device, by slowly increasing the inclination of the plane, and measuring the sliding of the pallet on the rack steel beams.

Influence of the following parameters was investigated:
Type of beam (namely type of surface finish of the beam)
Type of pallet
Geometry and weight of mass resting on the pallet

Influence of the type of beam was investigated by adopting six different types of beam specimens, produced by different manufactures from 3 different European countries, with different types of surface finish in combination with different types of pallet conditions: wooden and plastic Europallets as well as wooden American pallets, in new, old, dry or wet conditions. In particular, hot zinc, hot dip and powder coated steel beams were considered.
In both cross and down aisle direction, the surface finish influenced very much the static friction factor, with differences as large as 20-30% from one type to the other, in the case of wooden pallets.

Influence of the type of pallet was investigated by adopting three different types of pallets, namely: wooden Euro pallets, wooden American-pallet and plastic Euro pallet. In both cross and down aisle direction the plastic Euro pallet showed a very low friction factor (in the order of 0.2), practically being non-influenced by the type of beam surface finish. The wooden pallets show a very similar friction factor (in the order of 0.5), and similarly are influenced by the beam surface finish.
In both cross and down aisle direction, the mass weight didn’t affect much the results. However, its geometry (height of the c.o.g.) and its “placement” on the pallet (centered or eccentric) resulted in small variations of the measured friction factor.

More than 200 Dynamic tests were carried out on the shaking table facility of the National Technical University of Athens, on a simplified set-up, made of two uprights, connected by two horizontal beams, at approximately 0.30 m from the shaking table. On the beams three wooden Euro pallets were positioned, with concrete blocks rigidly fixed on top. Most tests were carried out with a sinusoidal excitation, with constant frequency and increasing acceleration. Some tests, in down aisle direction only, were carried out with sinusoidal excitation, with constant acceleration and increasing frequency, in order to verify independence of the obtained results on the type of adopted excitation. A lower bound of the acceleration exists, beyond which pallets start sliding on the steel beams. When acceleration of the mass is lower than such “lower bound”, the pallet “sticks” on the beams, and no sliding occurs. When the “lower bound” of acceleration is exceeded, increasing the acceleration of the input motion results in a lower increment in the mass acceleration, until an “upper bound” is reached of the mass
acceleration. Any further increase in the acceleration of the input motion doesn’t affect the acceleration of the mass, that is “free” to slide on the beams. “Sticktion” between pallet and beam is not resumed until a reduction of the acceleration occurs. The “upper bound” of the sliding acceleration is, in general, lower than the static friction factor.

In both cross and down aisle direction lateral pallets slide systematically earlier than the central one.

Dynamic behaviour in cross aisle direction is completely different to the one in down aisle direction. In cross aisle direction, the torsional stiffness as well as the flexural stiffness in the horizontal plane of the beams influence very much the results. In particular, such stiffnesses are affected by the out-of-plane and torsional behaviour of the beam-to-upright connections, whose stiffness rapidly deteriorates under cycling. Test results show, in general, a dependence of the sliding acceleration on the frequency of the input motion. Both the lower and the upper bound of the sliding acceleration seem to decrease when increasing the frequency of the excitation.

Lower bound sliding acceleration as low as 0.1 g was measured, for wooden pallets on hot dip coated steel beams. Upper bound values of the acceleration ranging from 0.3g to 0.5 g were measured depending on the type of beam surface finish as well as on the position of the pallet (lateral or central one).

In down aisle direction, the sliding acceleration is in general higher than the one measured in cross-aisle direction, under the same testing conditions, with a lower bound of the measured sliding acceleration of nearly 0.3 g, and an upper bound of nearly 0.6 g. Also in down aisle direction, test results show, in general, a dependence of the sliding acceleration on the frequency of the input motion. However, in this case, both the lower and the upper bound of the sliding acceleration seem to increase when increasing the frequency of the excitation.

Results of tests carried out with constant acceleration and increasing frequency are fully compatible with those obtained in tests with constant frequency and increasing acceleration.

Test result confirm that “sliding” is, under severe dynamic conditions, the main factor influencing the rack response. Hysteresis loops were obtained, showing the presence of an energy dissipation through sliding.

A few seismic tests were carried out, adopting three different input motions recorder in Greece during recent earthquakes, and characterized by different durations and frequency contents. Both monodirectional and bi-directional tests were carried out. The obtained results were compared with those of tests carried out with a sinusoidal excitation, showing full compatibility. Measured sliding accelerations range from 0.15g to 0.35 g in cross aisle direction and from 0.45g to 0.6g in the down aisle direction. Similar compatibility was also obtained for bi-directional tests, when comparing the resultants of the vector-compositions of the components of the sliding accelerations in the two orthogonal directions.

Chapter 4 and 5 deal respectively with the full scale pushover and pseudo-dynamic tests that were carried out at the European Laboratory for Structural Assessment of the Joint Research Center of Ispra. In particular, one pseudo-dynamic test and a pushover test in the down aisle direction, and one pushover test in the cross-aisle direction were carried out on two bays, three storeys full scale rack models. Re-analysis of the results allowed to draw interesting conclusions on the seismic behaviour of racking systems. These activities cover WP2.

The specimen under pushover test in down-aisle direction showed a progressive loss of stiffness associated to accumulation of plastic deformation in the column-base connections and to the large inter-storey drift of the first level. Inter-storey drifts of the upper level are much smaller than those of the first level; this is characteristic of a “soft-floor” type of collapse mechanism, that may lead to global instability due to second-order effects.

In order to reduce this type of problem, the deformability of the column-base connections should be
reduced and, somehow, limited. Eventually, adoption of a beam at the ground level might be considered. Despite the increment of cost, this solution would in any case allow use of the space at ground level for storage of goods, while the structure will behave as rotationally restrained at the base. Due to the bracing systems of the uprights, the specimens show a higher stiffness in cross-aisle direction than in the down-aisle one. Such bracing system is the most stressed structural component, and its failure leads to global collapse, accompanied by flexural-torsional buckling of the columns, consequent to the increment of the buckling length of the profile due to failure of the bolted connections with the lattice members.

For this reason, the solution with all diagonals inclined in the same direction should be reconsidered when the structure has to be erected in a seismic zone. Difference between the rotations of the uprights is due to deformation of the bracing system of the transverse frames, as well as to the different behaviour of the base-plate connections and of the beam-to-upright connections.

When loaded by horizontal loads applied transverse to the beam, the connections on one side can transfer the loads by means of both portions of the end-plate in bearing against the upright. On the contrary, the connections on the other side can transfer load only by means of the safety bolt in shear as well as of the hooks in tension; furthermore, bending of the beam in the horizontal plane induces bending in the end-plate. This different behaviour of the connections contributes to the difference in the global response of the uprights of the two sides. An evaluation of the behaviour factor has been carried out for both down-aisle and cross-aisle directions, with two possible definitions of the q-factor. One value can be identified based on ductility considerations as the ratio of the displacement \( v_{\text{max}} \) corresponding to the maximum load carrying capacity of the structure to the yield displacement \( v_y \), being \( q_{\mu_{\text{max}}} = \frac{v_{\text{max}}}{v_y} = 3.7 \) for the down aisle direction and \( q_{\mu_{\text{max}}} = 2.4 \) for the cross aisle direction.

With reference to the ductility factor theory, a value of the q-factor based on strength was also defined as the ratio of the ideal strength \( F_{\text{max,el}} \) (corresponding to \( v_{\text{max}} \) and evaluated on the basis of the initial elastic stiffness) to the maximum load carrying capacity \( F_{\text{max}} \), being \( q_{f_{\text{max}}} = \frac{F_{\text{max,el}}}{F_{\text{max}}} = 3.1 \) for the down aisle direction and \( q_{f_{\text{max}}} = 2.1 \) for the cross aisle direction.

The results of the pseudo-dynamic test on the rack specimen under down-aisle seismic loading previously presented are fully compatible with those obtained on similar specimens, tested under dynamic conditions on the shaking table of the Laboratory for Earthquake Engineering of the national technical University of Athens. Under pseudo-dynamic conditions the specimen could sustain the series of earthquake events although it didn’t collapse during the last test, performed with PGA = 1.4 g (ePGA=1.5g). Under dynamic conditions specimen A1 (having the masses fixed on the beams in order to prevent sliding, simulating the "quasi-static" conditions of the pseudo-dynamic tests) collapsed under an earthquake with a PGA=1.46g (ePGA=1.41g). Probably, the strain rate effect plays some role in this type of structure; the small movements allowed to the hooks in the holes, in fact, under dynamic conditions result in local impacts that, under increasing number of cycles, may cause cracking either in the hooks or at the edges of the holes.

In any case, comparing the deterioration of the second eigen-frequency (the most excited one) of the specimen tested under pseudo-dynamic conditions with the similar one tested in Athens on the shaking table it can be noticed that their trend of reduction is similar. This means that, in general, damage accumulated in the specimen during the two different types of test is similar.

Hence it can be concluded that, from the point of view of the assessment of the seismic resistance and of the damage accumulation of pallet racking systems, pseudo-dynamic tests and shaking table tests are fully compatible, although local damage due to local dynamic effects cannot be reproduced by the pseudo-dynamic testing methodology.
Of course, due to the intrinsic quasi-static nature of the pseudo-dynamic testing procedure, no information can be derived about the effects caused by the sliding of the pallets on the beams during a seismic event.

The values of acceleration that were reached during the pseudo-dynamic tests largely exceed the upper bound of the pallet sliding acceleration.

This means that only full scale dynamic testing will allow a clear assessment of the limit states of pallet racking systems under seismic loading.

Chapter 6 presents the results of the full scale dynamic tests carried out within the SEISRACKS project, covering activities related to WP1.

In addition to some palletised merchandise tests, earthquake simulation tests were carried out on six full scale rack models of three levels (total height 6.0 m) and 2 bays (total width 3.6 m). Five specimens were tested in down-aisle direction (two of which with base isolation systems) and one in cross-aisle direction.

Effects such as the beam size, presence/absence of pallet sliding as well as of a base isolation system were investigated.

The importance of small structural detailing, to be taken into account when designing pallet racks in seismic areas, was highlighted. Most of the observed failure, in fact, involved failure of bolted or welded connections.

An assessment of the q-factor was performed, based on the experimental results. q-factor values of 3.7 and 2.7 were obtained respectively for the down-aisle and the cross-aisle directions. In the case of pallets rigidly fixed on the beams (in order to prevent sliding) a q-factor of 2.8 was identified. This value, however, might be affected by the excessive stiffening of the structure, associated with the way in which the pallets were connected to the steel beams, generating a sort of “composite” beam. The estimated values are similar to those obtained by re-analysis of the push-over tests carried out on similar structures.

Positive effects of the presence of the base isolators were also highlighted. The two specimens with base-isolation systems resisted earthquakes with a PGA higher than 1.30g without any damage.

Assessment of the q-factor for specimen A5 lead to a value of 6.9.

Chapter 7 presents the main results of the monitoring activities carried out in a warehouse nearby Athens. Continuous monitoring has been carried out for a two year period, and relevant information related to accidental impacts as well as service conditions of a pallet rack installation during “everyday” working conditions. This activity covers WP3.

Different aspects of the numerical modelling and of the analysis of rack structures have also been investigated within the SEISRACKS research project, and are presented in Chapter 8. These activities lead to the proposal of a set of design rules for pallet racks in seismic areas, covering WP5.

In particular, two new features of prime importance for an efficient analysis of racks subjected to seismic action have been included in the FEM software FineLg: springs with hysteretic energy dissipation and sliding point-mass with coulomb friction law. These tools are fully operational, even if some future improvements have already been identified (convergence of the sliding model in the stick phase, test of other friction laws, sliding mass model with numerous contact points...).

Models have been validated versus a selection of test results obtained during the SEISRACKS research and during previous research programs. The validation has been carried out for cross- and down-aisle seismic excitation and for braced as well as non braced structures.

Some particular aspects have been emphasized during the calibration procedure: the need for a precise knowledge of the stiffness and resistance of the column bases, the horizontal bracing role played by the pallets as long as they are not sliding or the need for a future calibration of the behaviour of beam-to-column joints regarding rotation around the longitudinal axis of the upright.

Two complementary studies have also been performed in the perspective of normative prescriptions. The first one is a parameter study about the consequences of pallet sliding on the structural response. The study evidences clearly that the horizontal force reduction coefficient is depending on the intensity
of the ground motion, on the value of the friction coefficient and on the structural typology (i.e. the structural natural period and the number of loaded levels). The reduction coefficient ranges roughly from 0.2 to 1.0. Additional studies would be necessary to refine these conclusions and calibrate properly the reduction factor.

The second complementary study compared different types of seismic analyses (lateral force method, response spectrum analysis and pushover analysis) applied to a same structure exhibiting significant second order effects. The main conclusions are that, for the considered structure, all analyses provide similar results, provided that second order effects are really accounted for. To this purpose, the use of the approximate amplification factor \(1/(1-\theta)\) is found efficient. Moreover, a verification taking into account second order effects by using a sway buckling length appears strongly over-conservative.

Comments on FEM 10-2-08

General introduction

The whole research project SEISRACKS has been an opportunity to analyse the current draft of the normative document pr FEM 10-2-08 "Recommendations for the design of static steel pallet racks under seismic conditions". In particular, a series of items have been identified as questionable and are listed here with the corresponding sections of pr FEM 10-2-08 in its version of December 2005.

Determination of the period of the structure and of the seismic action, and in particular:

- Regularity criteria and consequences on the behaviour factor (2.2.5),
- Effect of the actual position of the gravity centre of the masses, vertical eccentricity with respect to the beams (2.3.6),
- Methods of analysis (2.4),
- Definition of regularity criteria (3.1.4),
- Modelling assumptions in the perspective of the structural analysis (3.3),
- Account for the different sources of energy dissipation (Viscous damping, friction of pallets, energy dissipation within the stored goods)
- Definition and values of parameters \(ED_1, ED_2\) and \(Rf\) (2.3.1, 2.3.2, 2.3.3, 2.3.4, 4.2.2, 4.2.3)
- Assessment of the structural ductility and associated behaviour factor
- Definition of ductility classes (3.1.1)
- Material properties and overstrength coefficient (3.1.2)
- Definition of the q-factor according to the structural typology (3.1.3, 3.4)
- Impact of (ir-)regularity (3.1.4, 3.4)
- Design rules for non dissipative vs. dissipative structures (3.1.5)
- Identification of the resisting system (3.2)
- Detailing of dissipative elements and overstrength criteria (5)

On the base of the knowledge gained during the research project and on engineering judgement, many of these items can be addressed. The final output will be a revised version of pr FEM 10-2-08.
Objectives of the project

Increase knowledge on actual structural behaviour of storage racks

a) by definition of the sliding properties of pallets on the racks, as a function of: i) type of pallet, ii) stored material, iii) acceleration, iv) frequency of the excitation

b) by identification of base isolation devices with characteristics suitable to storage racks in seismic areas in order to minimise the pallet sliding phenomenon, and verification by full scale testing of one full-scale base-isolated storage rack

c) by assessment of the actual lateral load carrying capacity and ductility of storage racks by means of pseudo-dynamic tests carried out up to failure of full-scale structures

Increase knowledge on actual service conditions of storage racks collecting data by continuous monitoring of a structure located in a warehouse in seismic area. These data refer, in particular to:

a) actions (actual live load distribution on the rack, occupancy ratio, vertical loads, accidental actions due to impacts, loading cycles, etc.)

b) structural response (vibrations, frequencies, settlements, permanent deformations, etc.)

Assessment of design rules for racks under earthquake conditions

a) by definition of a set of design actions for serviceability and ultimate state design for racks in seismic areas.

b) by definition of q-factors to be adopted in seismic design of racks

c) carrying out a revision of the most updated draft of FEM 10.2.08 Design Standard on the basis of the previous work and collected data, in order to incorporate into the document all the information relevant for a safe, although competitive, design of storage racks in seismic areas.
Comparison of initially planned activities and work accomplished

No major deviations from the initial plan occurred with exception of the requested (and granted) six-month extension of the project. This was due to the following reasons:

- some difficulties were encountered during the continuous monitoring activities of a warehouse (within WP3). Most probably the personnel, did not feel comfortable being photographed when making a mistake at work (impacting with the fork lift against the structure). However, activities had to be stopped and resumed in different occasions. An acquisition unit spoiled because of water dropped on it. The view angle of the camera allowing correlation of the recording with the cause was obstructed by cardboard boxes put in front of it.
- A serious car accident occurred to one of the main investigators, who was compelled to a long convalescence and absence from work.

More experimental activities were performed with the initially planned program. Namely

- In WP1 1260 static tests were performed for the assessment of the static friction factor (which were not encompassed in the initial proposal)
- In WP1 full scale shaking table tests were carried out on six specimens, while initial proposal considered only three specimens
- In WP2, instead of performing two pseudodynamic tests on full scale models, as initially planned, two push-over and one pseudodynamic tests on full scale models were carried out. The two pushover tests (one in down-aisle and the other in cross-aisle direction) allowed a clearer assessment of the q-factor in both directions.
- In WP3, two years continuous monitoring of an installation was carried out, with respect to the twelve months initially planned

No deviations occurred for activities carried out within WP4 and WP5 with respect to the initial program.

Description of activities and discussion

The research activities to be carried out in co-operation among the partners, in order to achieve the aforementioned objectives within this project, are subdivided in the following Work Packages:

WP 1 – DYNAMIC BEHAVIOUR OF STORAGE RACKS (Presented in Chapters 3 and 6)
WP 2 – PSEUDO-DYNAMIC TESTS OF STORAGE RACKS (Presented in Chapters 4 and 5)
WP 3 – IN SITU TESTING OF STORAGE RACKS (Presented in Chapter 7)
WP 4 – CYCLIC TESTING (Presented in Chapter 2)
WP 5 – ASSESSMENT OF SEISMIC DESIGN RULES FOR STORAGE RACKS (Presented in Chapter 8)
1 INTRODUCTION

1.1 BACKGROUND

Despite their lightness, racking systems carry very high live load (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). The behaviour of these systems is affected by the particular geometry of their structural components, made by high slenderness elements, the non-linear behaviour of both the beam-to-column and the base-plate joints.

Therefore, these structures cannot be considered as buildings, and reference cannot be made to usual Structural Design Recommendations and Standards.

The most recent Design Standards for steel storage racks (R.M.I. 2002 a and b, FEM 2001, RAL 1990, A.S. 1993, FEM 2005) 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).

The design needs particular attention for storage racks installed in a seismic zone, where they must be able to withstand dynamic forces. Besides the usual global and local collapse mechanisms, an additional limit state for the system is represented by the fall of the pallets with subsequent damage to goods, people and to the structure itself. In Europe, no official document is currently available for the seismic design of pallet racks, and the designers are compelled to operate with a total lack of references and of commonly accepted design rules. Very often they make reference to the Rack Manufacturers Institute (R.M.I.) Specifications (R.M.I. a and b, 2002), while the European Federation of Maintenance (F.E.M.) is presently working in order to produce an official document (FEM, 2005).

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.

Racks, in fact, are widely adopted in warehouses where they are loaded with tons of (more or less) valuable goods. The loss of these goods 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.

The 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.

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

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

This brief introduction shows 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.

1.2 THE INTERNATIONAL SITUATION

During the past few decades, the number of large public warehouse stores (often referred to as big-box stores) has grown significantly, changing both consumer buying habits and the public’s risk of injury during earthquakes. Whereas traditional retailers typically store goods and products outside the retail space in limited access storage rooms and warehouse facilities, big-box stores keep goods in close proximity to the consumer at all times. Typically, shoppers in these stores browse in aisles between steel storage racks, 5 to 6 meters in height, that hold pallets of inventory goods, some of which can be very heavy.

During an earthquake, occupant safety in a big-box store depends on both the structural performance of the building and on the performance of the storage racks and their contents. Earthquake ground motions can cause storage racks to collapse or overturn if they are not properly designed, installed, maintained, and loaded. In addition, goods stored on the racks may spill or topple off. Both occurrences pose a life-safety risk to the exposed shopping public.

The seismic design for new warehouse stores, including both the buildings and storage racks, is governed by the building code in force in the jurisdiction where a store is built.

The seismic requirements for new stores and storage racks, however, do not stipulate how goods are to be stored on the racks. Thus, in virtually all jurisdictions, requirements for securing storage rack contents are self-imposed by store owners and operators.

The situation is further complicated by the fact that these types of storage racks can be easily reconfigured (i.e., shelf level changed) to meet changing merchandising needs. The reconfiguration work, generally performed many times over the life of the structure, is done by store employees who may not always understand required procedures. Further, fork lifts are used to load goods on the racks and the racks can easily be damaged in the process. Finally, heavy merchandise stored on the floor near storage racks can topple during an earthquake and damage rack columns and braces, initiating rack collapse.

During the 1994 Northridge earthquake (magnitude = 6.7), serious storage rack collapses occurred in several warehouse retail stores that would likely have resulted in injuries and possibly deaths if the earthquake had occurred during a time when the stores had significant public presence rather than at 4:30 a.m. on a holiday.

Many existing racks have been since voluntarily strengthened or replaced and stricter quality assurance programs for rack loading and reconfiguration have been implemented by some owners, in order to prevent a reoccurrence of the Northridge problems. In addition, the 1994 NEHRP Recommended Provisions included a 50 percent increase of seismic loads for storage racks in areas accessible to the public. FEMA also recognized that the design process must take into account earthquakes larger than those recently experienced.

In Europe, no official document is currently available for the seismic design of pallet racks and the designers are compelled to operate without references to commonly accepted European design rules. Present Eurocodes 1, 3 and 8 give insufficient information on many design issues for racking systems. Recently, rack manufacturers defined a set of conventional design criteria, based on engineering experience, and drafted a “code of good practice” for users, a strict application of which is intended to achieve a safe working environment (FEM 2001b). Also this Code gives insufficient information to some design aspects, so very often designers make reference to the Rack Manufacturers Institute (R.M.I.) Specifications (R.M.I. a and b, 2002). In the meanwhile, the Industry in Europe, under the guidance of the European Federation of Maintenance (F.E.M.) issued a Manufacturers’ Design Recommendations called FEM 10.2.02. (F.E.M., 2001a). These recommended the way in which components are brought together to provide the optimum strength and stability required to store specified pallet load size and maximum weight. An increasing number of European manufacturers are
presently able to design according to these recommendations, if required by their customers. Furthermore, CEN has recently activated a Technical Committee (CEN-TC344), with the aim of developing a set of Eurocodes dedicated to racking and shelving.

The cause of this lack of Design Standards and Codes of Practice is the short knowledge of the actual behaviour of these structures under earthquake. Very little information is presently available related to the actual global ductility of the racks, that is strongly influenced by the behaviour of the connections. Only a few experimental studies were carried out on this topic. Furthermore, only very limited research was carried out on the actual dynamic behaviour of pallet racks. Existing studies were performed mainly in the US where, after the Northridge Earthquake, the problem revealed all its economical impact, with enormous losses of stored goods. Only one study is presently available in Europe, carried out within the EC sponsored ECOLEADER program for Free Access to Large Scale Testing Facilities (Castiglioni, 2003).

Many times, after an earthquake event, loss of goods was reported, with or without contemporary failure of the steel rack structural system. Most probably, the structural failures were 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.

Moreover no clearly established data and statistics exist related to the actual loading conditions of storage racks, in particular related to the “occupancy rate” of the rack during normal service, to different kinds of storage equipment, to different environments where racking systems are located and to different goods to be stored. This is due to the practically infinite possibilities of different applications occurring in the real practice.

In this context it shall also be mentioned that any observed damage to a rack component due to lift truck collisions, in general means loss of store capacity and flexibility over a certain period, so it requires an immediate repair.

1.3 OVERVIEW OF DAMAGE TO STEEL PALLET STORAGE RACKS AND CONTENT SPILLAGE

In 2003, estimated pan–European sale value for the racking industry exceeded 1.2 Billion Euro. Racking systems operated by industrial trucks represent approximately 70% of the total yearly racking industry market. The current estimated yearly loss due to accidental impact is 600 million Euro. Moreover the losses due to consequent fires far exceeds this value. Economical losses are expected to continue to rise due to competitive pressure in the logistic industry, resulting in higher driving speeds of industrial trucks within the racking environment.

The warehouse workplace is a potentially dangerous working environment. Careless driving of trucks can cause impact on racking and the dislodgment of loaded pallets onto operatives and even the collapse of part of the racking with its loads. In order to give an idea of the potential economic damage related to a collapse of one of these structures, it is enough to mention as an example that in the last two years, only in the Netherlands, at least two major collapses occurred, with a consequent fire. This fact made things of public domain (which is not usually the case). In these two collapses, there was more than 100 million Euro damage to goods and warehouse. Fortunately no person killed or injured. In the same period, in Europe, a conservative estimate gives more than 500 million Euro of goods lost due to racking system failures. Moreover, after a failure, the warehouse is usually out of service for a long period, increasing the economic damage.

1.4 CODES AND STANDARDS FOR STORAGE RACKS-PREVIOUS RESEARCHES

Racking systems are not “buildings” but a very peculiar steel construction work. They are different from buildings for the use, for the loads to be supported, for the geometrical dimensions and for the
steel components, mainly made of thin gauge profiles and continuously perforated uprights, which ensure the typical functionality, adaptability and flexibility needed for the huge variability of requirements in storing goods. Only the clad warehouse, where racking systems support goods but also mezzanines, roof and walls, shall be considered as a very special “building”.

For this reason it is necessary to explain how to consider the peculiarities of such kind of construction work when they are to be designed for seismic actions, because these peculiarities influence significantly the response to earthquakes and don’t allow a designer to follow exactly the same approach for “ordinary steel structures”, which is stated in the various Building Regulations.

While the basic technical description of earthquakes is obviously the same as for buildings, for racking systems it is of great importance to define whether or not it is possible to apply the “general design rules” which are enforced for ordinary steel structures, and how to correctly modify general principles and technical requirements, in order to take into account those peculiarities and to achieve the requested safety level.

Many specific physical phenomena affect the structural behavior of a racking system during an earthquake, such as the energy dissipation in the deformation of stored goods, or the sliding effect that can occur between pallets (or other unit-loads) and their directly supporting components, like beams, when seismic forces exceed certain limits, depending both on the acceleration values and also on the actual friction between the contact surfaces. Furthermore, the variable loads, like pallets or other unit-loads, can result in more than 95% of the total mass, differently from buildings where dead weight and permanent loads sum generally in a significant percentage. Therefore the load presence and distribution on racking systems affect very much the response of the structure under seismic actions.

As far as the safety level is concerned, it is of great importance to consider the potential movements of the stored goods, which can fall down accidentally from the supporting beams, regardless of the strength of the racking systems against the earthquake. Therefore, proper designed accessories should be placed on the seismic resistant racks, in order to reduce as much as possible the risk of fall and the consequent risk of impacts, damages or even domino-collapse.

Methods of seismic isolation can be studied, to cut down the seismic forces and the rack oscillations, to prevent accidental movements of the stored goods.

At present, very few Codes are available, all over the world, dealing with the problem of the seismic design of racking systems.

In Europe, the Federation Europeenne de la Manutention (FEM) performed Standard development research activities for the European Union (EU). One result is the 2005 FEM seismic design standard, FEM10.2.08, *The Seismic Design of Static Steel Pallet Racks*. Current FEM work includes analytical research, static and dynamic element testing as well as shake-table testing. Stub-column tests and beam-to-column connection tests for moment-rotation characteristics and properties have been conducted, using test facilities at the University of Trento and Politecnico di Milano in Italy. At the National Technical University in Athens, full-scale steel pallet racks have been tested at ground accelerations up to failure. This research indicates that movement of merchandise within packaged unit loads, movement of unit loads or packages on a pallet, and movement of pallets on pallet beams within the rack occur even at relatively low ground accelerations. Specific sliding tests have been developed to improve the understanding of these phenomena and their influence on damping, period, and overall structural behaviour.

Since the early 1970s, in the U.S., RMI has sponsored many analytical and experimental storage rack research projects conducted at Cornell University. These studies have included full-scale, component, and element tests focusing on, hot-rolled and cold-formed structural elements, beams, columns, perforations, beam-to-column connectors and connections, base plates, flexural and torsional-flexural buckling, and testing and loading protocols.

During the late 1970s and early 1980s, major research projects were undertaken, including subassembly tests at Stanford University and full-scale shake-table testing at the University of California/Berkeley.
using El Centro 1940 records, by URS/Blume (see John A. Blume and Associates, 1973; and Chen, Scholl, and Blume, 1980a, 1980b, and 1981), with funding from the RMI membership and a large grant from the National Science Foundation (NSF). The results of that testing, along with analytical studies, provided important baseline information about storage rack seismic performance, helped identify topics for further research, and articulated issues needing further study.

Among the most important ongoing RMI initiatives there is the current testing program to determine the moment-rotation characteristics of the beam-to-column connectors of RMI members’ products. The testing protocol will give information on the role of connector properties in the seismic performance of rack structures including information on damping, drift, base shear, and natural frequencies. The protocol is designed to mimic accepted testing provisions for building connections. This testing program, being conducted for RMI by an independent testing laboratory, covers cold-formed and hot-rolled members as well as the linear elastic, nonlinear elastic, and inelastic behaviour of the connecting elements. The results of this beam-to-column connection testing program will yield proposals for changes in the RMI standard, the NEHRP Recommended Provisions, ASCE 7, the IBC, and NFPA 5000 and should contribute to a convergence of the seismic requirements in those documents.

1.5 RESEARCH NEEDS

Experimental and analytical studies of the seismic performance of storage racks are scarce and the results are often proprietary; and consequently, they have not significantly influenced the development of codes and regulations related to storage rack systems.

The current engineering knowledge base concerning the earthquake safety and vulnerability of storage racks is 20 to 30 years old and is limited to contents and racks unlike many modern applications. The retail industry and the state-of-the-art of the design of storage racks have changed considerably in the interim. Large chains of stores now routinely invite the public to shop in a physical environment that formerly was found only in a warehouse, racks have more complex configurations and are taller, and their contents have become heavier. These facts clearly pinpoint to urgent research needs related to the seismic behaviour of storage rack systems. In this section, experimental and analytical research that is perceived to be the most urgently needed is briefly listed.

Only two full-scale shake-table testing investigations of storage racks fully loaded have been performed in Europe (Castiglioni et al. 2003) and other three in the United States (Chen et al. 1980a, 1980b, 1981; Filiatrault 2001). There is an urgent need to increase the experimental database of the seismic response of complete storage rack systems through shake-table testing. The main variables that need to be investigated in such experimental programs are:

- The layout and types of storage racks representing current construction practices and innovative systems such as eccentric bracing.
- The layout and types of merchandise contents.
- The types of seismic restraints (e.g., plastic wraps, screens, ledges, etc.) for contents.
- The structural interaction between neighboring racks.
- The direction of the horizontal seismic input, relative to the rack’s orientation (transverse, longitudinal, or non-orthogonal).
- The characteristics of the input ground motions, including consideration of whether vertical accelerations must be characterized and near-field motions, and relating these input motions to seismic hazard mapping and codes.

As demonstrated by available experimental and analytical results, the seismic response of storage racks in their down-aisle direction is strongly affected by the non-linear response of the beam-to-upright and base plate connections. Since numerous variables enter in the design of these connections, an experimental parametric study on the cyclic response of beam-to-upright and base plate connections is urgently needed.
While the needs of the down-aisle direction are urgent, testing needs in the cross-aisle are even more urgent since the understanding of this directions behaviour is even less understood. Failures of racks in earthquakes are most commonly reported as cross-aisle failures.

The information on the seismic response of merchandise contents installed in storage racks is very limited. There is an urgent need to conduct shake-table studies of merchandise. For this purpose, shake-table testing could be used to simulate the motions experienced by various levels of storage racks during earthquakes. A robust numerical model would be required to develop these input motions. Various merchandise items could be mounted on the shake-table via a rigid assembly representative of the level on which they are mounted. Various types of merchandise contents would be investigated experimentally under a large number of input motions representative of several seismic hazard levels. Furthermore, these results could be compared with the ones obtained when various types of seismic restraints are introduced. With this information, clear recommendations could be provided on the types of seismic restraint to be used for a particular type of merchandise content.

There is a need to develop a general purpose computer-based numerical model for the prediction of the seismic response of storage racks and contents. The development of such a general-purpose model requires close coordination and interaction with the experimental work.
2 COMPONENT TESTS

2.1 OVERVIEW

Component tests were performed with the aim of characterizing the behaviour of both beam-to-upright and base connections, in order to allow a correct interpretation of the full scale tests as well as to calibrate numerical models. It has to be noticed that the behaviour of both components is strongly influenced by the following factors:

- Nature and geometry of the profiles (unsymmetrical cross section of the upright, thin walled sections of both beams and uprights, Figure 2)
- Asymmetry of the connections. In the case of the beam-to-upright connection, the asymmetry is caused by the inclined hooks and by the presence of the safety-bolt on the upper side of the beam end-plate connector only (Figure 3 a). In the case of the base connections, the asymmetry is due to the eccentric position of the upright on the base-plate, and by the asymmetrical disposition of the bolts (Figure 3 b).

Special attention was paid to component and connections testing procedures presented in Chapter 5 – Tests of FEM 10.2.02 (2000), The Design of Static Steel Pallet Racks. Based on such specifications the following tests were performed at the Laboratory for Structures and Strength of Materials of the Department of Civil Engineering and Architecture of Instituto Superior Técnico, Lisbon, during 2005 and 2006.

2.2 BEAM-TO-UPRIGHT CONNECTIONS

This study focused on the beam-to-upright connection type shown in Figure 4. A “hooked” end plate connector is welded to the beam at both ends. Connection is attained by introducing the hooks in the openings (punched during fabrication) on the uprights, and by adding a safety bolt connecting the upper part of the extended end-plate to the upright. As a general remark, it should be noticed that this proprietary beam-to-upright connection is strongly non-symmetric in both vertical and horizontal planes. In the vertical plane, non-symmetry is due to the presence of the safety bolt on the upper side of the beam only and by the fact that the beam is fillet welded to the end-plate on three sides only, leaving
the lower flange un-welded. In the horizontal plane, non-symmetry is due to the shape of the end-plate connector, which has hooks on one side only, and is obtained by forming of a thin plate, bent in shape of an L, so with a stiffened edge (the same edge where hooks are present). A non-symmetric response is hence to be expected under hogging and sagging bending.

Objectives of the tests were:
- Assessment of the moment-rotation curves;
- Assessment of the collapse modes of these connections under monotonic and cyclic loads.

Two different cross-section for beams (TG 70x45x1.5 and TG 130x45x1.5 mm) were adopted (Figure 5), with upright of identical cross-section (100/20b) shown in Figure 2 b. Consequently, beam height varied between 70 mm and 130 mm. The member geometrical properties supplied by SACMA are shown in Table 1.

<table>
<thead>
<tr>
<th>Member</th>
<th>Properties</th>
<th>Gross section</th>
<th>Net section</th>
</tr>
</thead>
<tbody>
<tr>
<td>100/20b</td>
<td>A (mm²)</td>
<td>588.8</td>
<td>525.7</td>
</tr>
<tr>
<td></td>
<td>t (mm)</td>
<td>2.0</td>
<td>2.0</td>
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<tr>
<td></td>
<td>Jx (mm⁴)</td>
<td>436020</td>
<td>406100</td>
</tr>
<tr>
<td></td>
<td>Jy (mm⁴)</td>
<td>812280</td>
<td>694680</td>
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<td></td>
<td>Wx (mm³)</td>
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<td>8330</td>
</tr>
<tr>
<td></td>
<td>Wy (mm³)</td>
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<td>14323</td>
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<td>TG 70x45x1.5 mm</td>
<td>A (mm²)</td>
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<td>514.06</td>
</tr>
<tr>
<td></td>
<td>t (mm)</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>Jx (mm⁴)</td>
<td>407500</td>
<td>407500</td>
</tr>
<tr>
<td></td>
<td>Jy (mm⁴)</td>
<td>136272</td>
<td>136272</td>
</tr>
<tr>
<td></td>
<td>Wx (mm³)</td>
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<td>11754</td>
</tr>
<tr>
<td></td>
<td>Wy (mm³)</td>
<td>4764</td>
<td>4764</td>
</tr>
<tr>
<td>TG 130x45x1.5 mm</td>
<td>A (mm²)</td>
<td>697.66</td>
<td>697.66</td>
</tr>
<tr>
<td></td>
<td>t (mm)</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>Jx (mm⁴)</td>
<td>1742220</td>
<td>1742220</td>
</tr>
<tr>
<td></td>
<td>Jy (mm⁴)</td>
<td>168862</td>
<td>168862</td>
</tr>
<tr>
<td></td>
<td>Wx (mm³)</td>
<td>26792</td>
<td>26792</td>
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<tr>
<td></td>
<td>Wy (mm³)</td>
<td>5171</td>
<td>5171</td>
</tr>
</tbody>
</table>

Table 1: Geometrical properties of the members

The material used for the beams and upright is S275 steel, with actual values of yield and ultimate stress showed in Table 2. The test set-up and instrumentation are shown in Figure 6 and Figure 7 Specimen size, configuration and instrumentation were adopted according to FEM 10.2.02 (2000) Recommendations. The moment-rotation curves were plotted for each test. The bending moment was defined as \( M = F \cdot a \) and the rotation of the connection by eq. (2.1).

\[
\phi = \frac{V}{a} - \left( \frac{\delta_1 - \delta_2}{d} \right)
\]

where (see Figure 8): \( V = \) displacement due to load \( F \); \( a = \) lever arm for the load \( F \); \( \delta_1 = \) deflection measured by transducer T1; \( \delta_2 = \) deflection measured by transducer T2; \( d = \) beam height.

<table>
<thead>
<tr>
<th>Member</th>
<th>( f_y ) (kN/cm²)</th>
<th>( f_u ) (kN/cm²)</th>
<th>( \varepsilon_u ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam 70x45x1.5</td>
<td>35.36</td>
<td>44.61</td>
<td>26.90</td>
</tr>
<tr>
<td>Beam 130x45x1.5</td>
<td>35.70</td>
<td>45.80</td>
<td>27.00</td>
</tr>
<tr>
<td>Upright 100x82x2.0</td>
<td>34.80</td>
<td>49.30</td>
<td>25.50</td>
</tr>
<tr>
<td>Beam end connector</td>
<td>26.30</td>
<td>38.50</td>
<td>41.10</td>
</tr>
</tbody>
</table>

Table 2: Material Characteristics
The 30 tests, summarized in Table 3, were carried out, on beam-to-upright specimens. In particular, monotonic tests under both hogging (MB) and sagging (MT) bending moments were performed. As the Standard cyclic testing procedure proposed by ECCS (1986), in particular in the case of unsymmetric behaviour (as in the case under exam), leads to a correct evaluation of the cyclic behaviour of components only for the condition of unloaded structure (vertical load F=0), an innovative cyclic testing procedure has been identified and applied. Hence, cyclic tests were carried out under different levels of vertical load (namely 0%, 25%, 50%, 66% and 75% of the yield load Fy of the connection) in order to simulate the presence of an accidental gravity load on the beam. The yield strength Fy, as well as other relevant parameters, such as the yield displacement vy, the ultimate strength Fu, as well as the ultimate displacement vu, and the initial elastic stiffness Si,ini, can be conventionally defined according to ECCS (1986), with reference to the results of a monotonic test (Figure 9).
Figure 7: Test set up  a) test set-up,  b) lateral restraint system,  c,d) instrumentation

Figure 8: Specimen dimensions and instrumentation

<table>
<thead>
<tr>
<th>TEST</th>
<th>Beam width - 70 mm</th>
<th>Beam width - 130 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monotonic – below (hogging bending)</td>
<td>70MB-1</td>
<td>130MB-2</td>
</tr>
<tr>
<td></td>
<td>70MB-3</td>
<td>130MB-3</td>
</tr>
<tr>
<td></td>
<td>70MB-4</td>
<td>130MB-4</td>
</tr>
<tr>
<td>Monotonic – top (sagging bending)</td>
<td>70MT-3</td>
<td>130MT-2</td>
</tr>
<tr>
<td></td>
<td>70MT-5</td>
<td>130MT-3</td>
</tr>
<tr>
<td></td>
<td>70MT-6</td>
<td>130MT-5</td>
</tr>
<tr>
<td>Cyclic – ECCS</td>
<td>70ECCS-1</td>
<td>130ECCS-2</td>
</tr>
<tr>
<td></td>
<td>70ECCS-2</td>
<td>130ECCS-3</td>
</tr>
<tr>
<td>Cyclic – Fy25</td>
<td>70Fy25-3</td>
<td>130Fy25-1</td>
</tr>
<tr>
<td></td>
<td>70Fy25-4</td>
<td>130Fy25-2</td>
</tr>
<tr>
<td>Cyclic – Fy50</td>
<td>70Fy50-1</td>
<td>130Fy50-1</td>
</tr>
<tr>
<td></td>
<td>70Fy50-2</td>
<td>130Fy50-2</td>
</tr>
<tr>
<td>Cyclic – Fy66</td>
<td>70Fy66-1</td>
<td>130Fy66-1</td>
</tr>
<tr>
<td></td>
<td>70Fy66-2</td>
<td>130Fy66-2</td>
</tr>
<tr>
<td>Cyclic – Fy75</td>
<td>70Fy75-1</td>
<td>130Fy75-1</td>
</tr>
</tbody>
</table>

Table 3: Tests on beam-to-upright connections
2.2.1 Comparison and analysis of test results

2.2.1.1 Monotonic tests

Effect of the beam size on the main parameters (yield and ultimate strength, initial stiffness, yield and ultimate rotation) is shown in Table 4, with reference to monotonic tests.

<table>
<thead>
<tr>
<th>Loading</th>
<th>Beam</th>
<th>$M_y$ (kNm)</th>
<th>$\phi_y$ (mrad)</th>
<th>$S_{y,ini}$ (kNm/rad)</th>
<th>$M_u$ (kNm)</th>
<th>$\phi_u$ (mrad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H hogging</td>
<td>130x45x1.5</td>
<td>2.53</td>
<td>26.0</td>
<td>95.8</td>
<td>3.14</td>
<td>129.7</td>
</tr>
<tr>
<td></td>
<td>70x45x1.5</td>
<td>1.23</td>
<td>27.8</td>
<td>47.9</td>
<td>1.60</td>
<td>178.7</td>
</tr>
<tr>
<td></td>
<td>130/70</td>
<td>2.06</td>
<td>0.94</td>
<td>2.00</td>
<td>1.96</td>
<td>0.73</td>
</tr>
<tr>
<td>S hogging</td>
<td>130x45x1.5</td>
<td>2.11</td>
<td>18.8</td>
<td>119.0</td>
<td>2.44</td>
<td>107.3</td>
</tr>
<tr>
<td></td>
<td>70x45x1.5</td>
<td>1.24</td>
<td>26.0</td>
<td>48.9</td>
<td>1.42</td>
<td>77.7</td>
</tr>
<tr>
<td></td>
<td>130/70</td>
<td>1.70</td>
<td>0.72</td>
<td>2.43</td>
<td>1.72</td>
<td>1.38</td>
</tr>
</tbody>
</table>

Table 4: Effect of the beam size – Monotonic tests

Figure 10 shows the comparison between the moment – rotation curves for both beam sizes, and loading conditions.

Examining Table 4 and Figure 10, it can be noticed that connections with 130x45x1.5 mm beam showed larger stiffness than those with 70x45x1.5 mm beam. Furthermore, stiffness of the connections with 130x45x1.5 mm beams under sagging bending is approximately 25% larger than that under hogging bending, because of the different collapse mechanisms. Under hogging bending (Figure 11.c), the safety bolt is in tension, and only one hook (that finally comes out from the hole in the upright) participates to the resistant mechanism, together with the end-plate in bending. Collapse mechanism, in this case, is due to deformation of the end-plate, to punching of the safety bolt through the end-plate and to deformation of one hook (a second one is only partially deformed). Under sagging bending (Figure 11.b), on the contrary, two hooks participate to the resistant mechanism, while the safety bolt doesn’t,
because it is in the compression zone. This differences explain both the higher stiffness and the smaller yield strength and rotation (as well as ultimate strength and rotation) exhibited by the 130x45x1.5 mm beam-to-upright connection under sagging bending with respect to the same connection under hogging bending.

The non symmetric geometry (and stiffness) of the end plate would cause a rotation of the beam in the horizontal plane (as shown in Figure 11.b). Such a rotation is however prevented by the test setup. Hence, all deformation concentrates in the end-plate connector and in the hooks. The elastic behaviour of the 70x45x1.5 mm. beam-to-upright connection was practically symmetric, under hogging and sagging bending, as shown in Table 4. On the contrary, the ultimate behaviour of this connection under sagging bending was very different from the one exhibited under hogging bending, as evidenced in the same table.

Under hogging bending, the safety bolt is in tension, and the center of rotation is located at the lower flange of the beam (in compression) as sketched in Figure 11.c. The end plate is extended, below the lower flange, in order to allow presence of a third hook. As evidenced during the tests, the central hook is located approximately in correspondence of the lower flange of the 70x45x1.5 mm beam (i.e. at the center of rotation), so it doesn’t participate to the resistant mechanism. Collapse mechanism, in this case, is due to deformation of the end-plate, punching of the safety bolt through the end-plate and to deformation of one hook.

Under sagging bending, on the contrary, the center of rotation is close to the upper flange of the beam, while the safety bolt is in the compression zone. The portion of the end-plate, extended below the lower flange, is restrained, on one side, by the lowest hook. Furthermore, the whole end-plate, is stiffened on the “hooked” side, by the presence of the edge stiffener represented by the bent portion of the L shaped plate. When the beam is bent upward, the deformation of the lower part of the end plate, should comply with the restraints represented by the lower flange of the beam, by the lowest hook and by the stiffened
edge. Its deformed shape is sketched in Figure 11 a). The presence of the hooks and of the stiffened edge, forces the end-plate to bend in both the vertical and the horizontal plane. Accordingly should do the beam, which is however restrained to out-of-plane bending by the test set-up.

This results in a concentration of deformations at the fillet weld connecting the beam to the end plate, on the side of the stiffened (hooked) edge, that fractured, with consequent collapse of the connection. Crack initiated at the end of the vertical leg of the fillet weld connecting the beam to the end-plate. This is the most stressed point of the connection, also because of the stress concentration due to the fact that the beam was fillet welded to the end plate on three sides only, and the fillet was not returned around the corner, despite of what is recommended by clause 4.3.2.1(4) of EN 1993-1-8:2005 (“Fillet welds finishing at the ends or sides of parts should be returned continuously, full size, around the corner for a distance of at least twice the leg length of the weld, unless access or the configuration of the joint renders this impracticable.”).

Crack initiated at the end of the vertical leg of the fillet weld connecting the beam to the end-plate. This is the most stressed point of the connection, also because of the stress concentration due to the fact that the beam was fillet welded to the end plate on three sides only, and the fillet was not returned around the corner, despite of what is recommended by clause 4.3.2.1(4) of EN 1993-1-8:2005 (“Fillet welds finishing at the ends or sides of parts should be returned continuously, full size, around the corner for a distance of at least twice the leg length of the weld, unless access or the configuration of the joint renders this impracticable.”).

![Graph a) Lb=1800 mm](image)

![Graph b) Lb=2700 mm](image)

**Figure 12:** Classification of the beam-to-upright connections according to EN 1993-1-8:2005

Figure 12 show a classification of both the connections, according to Eurocode 3 (2005) – Part 1.8, adopting respectively \( L_{beam} = 1.8 \) m, as for the structures tested full scale within this research, and \( L_{beam} = 2.7 \) m, as in the most common real applications of racks where, at each level and for each span, up to three pallets can be placed side-by-side on a couple of beams. It can be noticed that for a beam length of 1800 mm (Figure 12.a) the connection with beam 70x45x1.5 can be considered as semi-rigid while in the case of beam 130x45x1.5, the same end-plate connection behaves as flexible. When the beam length is increased to 2700 mm (Figure 12.b), both connections can be considered semi-rigid.

### 2.2.1.2 Cyclic Tests

Effect of the beam size and of the gravity loads in terms of hysteresis loops, is shown in Figure 13 and Figure 14, respectively with reference to ECCS and to cyclic tests, as well as in Table 5 that...
summarizes the mean values of the response parameters for both 70x45x1.5 mm (a) and 130x45x1.5 mm (b) beam, and for different values of the gravity loads.

Figure 13: Effect of the beam size – ECCS cyclic tests

Figure 14: Effect of the beam size and of the gravity loads – cyclic tests

In Table 5, in addition to the elastic parameters (i.e. initial stiffness \( k_y \), yield moment \( M_y \) and rotation \( \phi_y \)), the maximum positive moment \( (M_u^+) \) and corresponding rotation \( (\phi_u^+) \), as well as the minimum negative moment \( (M_u^-) \) and corresponding rotation \( (\phi_u^-) \) are reported, together with the number of cycles at failure \( N_c \), the failure rotation \( \phi_c \) and the bending moment for gravity loads \( (M_g) \).

It is evident that the response of the connections under symmetric cycles and in absence of gravity loads (ECCS tests, Figure 13) is very different from that of the same connections tested in presence of gravity loads (Figure 14).

<table>
<thead>
<tr>
<th></th>
<th>( M_k ) (kNm)</th>
<th>( K_y ) (kNm/rad)</th>
<th>( M_y ) (kNm)</th>
<th>( \phi_y ) (mrad)</th>
<th>( M_u^+ ) (kNm)</th>
<th>( \phi_u^+ ) (mrad)</th>
<th>( M_u^- ) (kNm)</th>
<th>( \phi_u^- ) (mrad)</th>
<th>( N_c )</th>
<th>( \phi_c ) (mrad)</th>
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<tbody>
<tr>
<td>70-ECCS</td>
<td>0.00</td>
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<td>1.19</td>
<td>28.10</td>
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<td>101.45</td>
<td>-1.37</td>
<td>-45.00</td>
<td>11.5</td>
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<tr>
<td>70-Fy25</td>
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<td>47.65</td>
<td>1.29</td>
<td>27.85</td>
<td>1.57</td>
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<td>105.10</td>
<td>10</td>
<td>365.80</td>
</tr>
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<td>70-Fy50</td>
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<td>204.85</td>
<td>-0.78</td>
<td>127.10</td>
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Table 5: Average values of response parameters for 70x45x1.5 mm (a) and 130x45x1.5 mm (b) beams

In particular, the response of the specimens tested in presence of gravity loads is characterized by cyclic creep (ratchetting), with a progressive accumulation of plastic deformation in the same direction of the
mean applied load. The measured ratchetting rate increases with the applied gravity load.

Collapse of the connection is attained approximately under the same rotation $\phi_c$ (360 mrad for 70x45x1.5 mm beams and 192 mrad for 130x45x1.5 mm beams), although some dependence of $\phi_c$ on the applied gravity load can be observed, at least for large values of the gravity load (66% and 75% of the yield load). Hence, connections of beams supporting a high gravity load collapse at a number of cycles ($N_c$) smaller than those supporting a smaller gravity load, due to the different creep rate. Hysteresis loops are clearly non symmetric, with similar maximum values of the hogging moment ($M_{u+}$) and of its corresponding rotation ($\phi_{u+}$), practically independent on the applied gravity load, but with the minimum values of the sagging moment ($M_{u-}$) and of its corresponding rotation ($\phi_{u-}$) becoming (in absolute value) smaller and smaller, increasing the gravity load. Dependence of the various inelastic response parameters on the applied gravity load ($M_g$) is shown in Figure 15. In absence of gravity load (ECCS tests), the response of the specimens is nearly symmetric; non symmetry is due to the different response of the connection under hogging and sagging bending, already highlighted in the case of monotonic tests. In particular, it should be noticed that under sagging bending the connections show a lower ductility than under hogging bending ($\phi_{u-} < \phi_{u+}$). The values of $M_{u+}$, $\phi_{u+}$, $M_{u-}$ and $\phi_{u-}$ are (in absolute value) smaller than those measured in monotonic tests (reported in Table 4), because obtained under cyclic loading. Differences can also be noticed in the failure modes of the different specimens. In absence of gravity loads (ECCS tests), failure occurred always under sagging bending, i.e. under the loading condition for which the connection showed the lower strength and ductility in monotonic tests. Collapse mechanism, in the case of 70x45x1.5 mm beams, with cracking of the fillet weld connecting the beam to the end-plate, deformation of the end plate and of the central hook and a small deformation of the lowest hook. In this case, however, due to the cyclic load reversals, deformation of the upper hook as well as partial punching of the safety bolt through the end-plate could be observed. Also in the case of 130x45x1.5 mm beams, the collapse mechanism is similar to the one described for monotonic
tests under sagging bending, with deformation of the two lowest hooks. Furthermore, also in this case, due to the cyclic load reversals, large deformation of the upper hook as well as partial punching of the safety bolt through the end-plate could be observed. In presence of gravity loads, on the contrary, failure occurred always under hogging bending. No difference could be noticed in the failure mechanisms under increasing values of the gravity loads, but only a shorter number of cycles to failure. Collapse mechanism, in the case of both 70x45x1.5 mm and 130x45x1.5 mm beams, involved complete punching of the safety bolt through the end-plate, with partial deformation of the central and of the lowest hooks, because of the cyclic reversal loading. Cracking of the hooks was also observed.

2.2.2 Conclusions

- The failure mode for 70 mm (hogging bending) and 130 mm (hogging and sagging bending) deep beams consisted of large deformations in the top zone of the beam end connector;
- In the 70 mm deep beam subject to sagging bending, the failure mode was the fracture of the fillet weld between the beam and the end-plate connector;
- The connection behaviour for all tests was not influenced by safety bolt deformation;
- The rotation capacity difference between the loading directions was larger for the 70 mm deep beam due to different collapse occurred in the sagging loading;
- According to Eurocode 3 (2005) the connection with a 70 mm deep beam can be considered as semirigid, for both beam lengths of 1.8 and 2.7 m;
- According to Eurocode 3 (2005) the connection with a 130 mm deep beam can be considered as semirigid for a beam lengths of 2.7 m, and as flexible for a beam lengths of 1.8 m (as in the case of the structures tested full scale within this research); hence, in this last case, the influence of this connection behaviour may be ignored in structural analysis;
- The results of test performed with the innovative cyclic testing procedure were fundamentally different from those obtained through the application of the ECCS recommended testing procedure;
- In those tests performed according to the innovative cyclic testing procedure, the imposed displacement history is unsymmetrical. Displacements tend to systematically accumulate in the positive direction as a consequence of the absence of closure of the top part of the connection (the bottom part is generally closed throughout the tests). This connection behaviour also leads to a reduction of the pinching effects;
- Imposed forces are shifted in the positive (hogging) direction for the innovative testing procedure, whereas – apart from the asymmetry that may result from unsymmetrical connection detailing – positive and negative force amplitudes for ECCS tests are not excessively different;
- Failure of the connection is explicitly addressed by the innovative testing procedure since failure occurs when the connection is no longer able to withstand vertical load effects.

2.3 COLUMN BASE CONNECTIONS

Proprietary moment connections are typically used also as column-base connections for steel selective pallet storage racks. This study focused on the beam-to-upright connection type shown in Figure 16. The column bases consist of two vertical steel plates fillet-welded to the base plate. The upright is connected to the base by bolting through the slotted holes using two M10x25 (grade 8.8) bolts for each vertical plate. The base plate is connected to the foundation surface (in this case either in concrete or in steel) by means of two M16 (grade 8.8) bolts. All bolts were preloaded.
As a general remark, it should be noticed that this proprietary column-base connection is strongly non-symmetric in cross aisle direction, where an eccentricity of 66.5 mm exists between the bolt line and the c.o.g. of the up-right profile. A non-symmetric response is hence to be expected in cross aisle direction under transverse load reversals, when the bolts may be either in tension or in compression.

Objectives of these tests on column base connections were:
- Assessment of the moment-rotation curves;
- Assessment of the collapse modes

under either monotonic or cyclic transversal loads, for different levels of axial load applied to the upright. The specimens were tested in the cross aisle and down aisle directions. In the cross aisle direction the specimens were subjected to a monotonic load in two different directions, in order to allow respectively tension and compression in the (anchor) bolts of the column base connection.

One type of upright cross-section (100/20b), whose geometrical properties are shown in Table 6 was studied under four values of axial load: 0, 25kN, 50kN and 75 kN. These correspond respectively to 0%, 17%, 34% and 52% of the yield resistance of the net cross section under axial load (Af_y), fy being the yield strength of 275MPa.

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Table 6 Geometrical properties of the upright 100/20b

Column bases connected to steel and concrete surfaces were examined. As in the full scale dynamic tests carried out at NTUA the base plates were welded to the steel deck of the shaking table, some tests
were also carried out with the steel base welded and bolted to a steel surface. These tests are indicated hereafter as “Athens base” tests. Table 7 summarizes the performed tests. The size and shape of the test specimen are shown in Figure 17, while the test set up and instrumentation are shown in figures from Figure 18 to Figure 19.

The tested specimens comprise a portion of upright 100/20b, 650 mm long, connected to a steel plate at the top and to the base plate at the bottom. The steel plate at the top, welded to the upright, allowed the coupling of the specimen to the actuator, by means of four M16 (grade 8.8) preloaded bolts. The specimens were tested in a system using a main reaction frame, a mechanical actuator and a secondary frame used to induce the axial force in the specimens.

In order to simulate different “floor conditions”, the specimens were connected either to a steel member or to a concrete block. These, in turn, where connected to the main frame using bolts. The lateral displacements of the specimens were restrained using a double angle. This angle was welded to the metallic member that was connected to the main frame, as shown in Figure 18. The axial force was set in the centre of gravity of the upright using two 36 mm dywidag bars. The system of application of the axial force comprised a secondary reaction frame, a beam, two hydraulic actuators and two loading cells, as shown in Figure 19.

The dywidag bars, 8 m long, were connected to the head of the actuator, which was bolted to the plate at the top of the upright. Tests were carried out under displacement controlled conditions. The actuator applied the transversal displacement at a distance of 1 m from the column base, while the imposed load was measured through a load cell. Four electric displacement transducers (LVDTs) were positioned in the column base to evaluate the relative bending of the base, as shown in Figure 18.

The moment-rotation curves were plotted for each test in which the moment was obtained at the column base \( (M = F.a) \) and the rotation of the connection by eq.2.2.

\[
\phi = \frac{V}{a} \quad (2.2)
\]

Where \( V \) is the displacement due to the applied transversal load \( F \) and \( a = 1.0 \) m is the lever arm of the applied transversal load \( F \).
Figure 18  Test set-up and instrumentation

Figure 19  Test set-up a) main frame, b) secondary reaction frame, c) axial load system connected to the secondary reaction frame
### Table 7: Column-base tests. Connected to a) a steel surface, b) a concrete surface

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### 2.3.1 Comparison and analysis of test results

#### 2.3.1.1 Monotonic tests

Figure 20 and Figure 21 show a comparison of the response in terms of moment-rotation curves for column-base connections for different directions of bending, under similar conditions of applied axial load and “deck connection”.

---

36
Figure 20: Comparison of the moment-rotation curves for column-base connections, without axial load and connected to a steel deck, for different directions of bending.

Figure 20 refers to specimens without axial load and connected to a steel deck. Figure 21 presents the results for specimens connected to a concrete deck with 50 kN axial load, while Figure 22 refers to specimens with 25 kN axial load bolted-and-welded to a steel deck.

Figure 21: Comparison of the moment-rotation curves for column-base connections, with 50 kN axial load and connected to a concrete deck, for different directions of bending.

Table 8 and Table 9 show a comparison of the mean values of the results of monotonic tests on column-base connections bent in cross-aisle direction, respectively with bolts in tension and in compression, while Table 10 refers to the case of down-aisle bending. Each table shows the comparison of the results obtained for different types of deck connection (namely bolted to a concrete deck (b) and either bolted (a) or bolted-and-welded (c) to a steel deck).

Figure 22: Comparison of the moment-rotation curves for column-base connections, with 25 kN axial load bolted-and-welded to a steel deck, for different directions of bending.
Table 8: Comparison of results of monotonic tests on column-bases bent in cross-aisle direction, with bolts in tension.

Table 11 shows a comparison of the mean values of the results obtained in monotonic tests on column-bases bent in cross-aisle direction with bolts in tension vs. bolts in compression zone. Similarly, Table 12 compares the results of cross-aisle bending with bolts in tension with those of down-aisle bending, while Table 13 compares the results of down-aisle bending with those of cross-aisle bending with bolts in compression.

Examining Table 8 it can be noticed that in monotonic bending tests with bolts in tension, increasing the axial load in the column always results in an increment of stiffness but in a reduction of strength, ductility and energy absorption capacity of the specimens bolted either to a steel or a concrete deck. Bolting and welding the specimens to a steel surface is always beneficial with respect to simply bolting to either concrete or steel deck, under the same axial load.

Table 9: Comparison of results of monotonic tests on column-bases bent in cross-aisle direction, with bolts in compression zone.

In monotonic bending tests with bolts in the compression zone (Table 9), increasing the axial load in the
column results in an increment of stiffness, strength, ductility and energy absorption capacity of the specimens bolted a concrete deck. On the contrary, for specimens bolted to a steel deck, increasing the axial load results in an increment of stiffness and strength, but in a reduction of ductility and energy absorption capacity. Bolting and welding the specimens to a steel surface is always beneficial with respect to strength, ductility and energy absorption capacity with respect to simply bolting to either concrete or steel deck, under the same axial load.

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<table>
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<th>$\phi_y$</th>
<th>$S_{jini}$</th>
<th>M_max</th>
<th>$\phi_{max}$</th>
<th>E_max</th>
<th>Result</th>
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<td></td>
<td></td>
<td></td>
<td>cbd bf25</td>
</tr>
</tbody>
</table>

| Table 10 : Comparison of results of monotonic tests on column-bases bent in down-aisle direction |

In the case of monotonic bending in down-aisle direction (Table 10) it can be noticed that increasing the axial load results in an increment of strength and stiffness, and in a reduction of ductility and energy absorption capacity for specimens bolted to a steel deck. When specimens are bolted to a concrete surface, increasing the axial load results in an increment of stiffness, a reduction of ductility and energy absorption capacity, while strength increases for low and intermediate values of the applied axial force (up to 50 kN), but reduces for higher values of the axial load (75 kN). Bolting and welding the specimens to a steel surface is, in this loading direction, always beneficial with respect to stiffness, strength, ductility and energy absorption capacity with respect to simply bolting to either concrete or steel deck, under the same axial load.
When comparing the main response parameters of monotonic tests in cross-aisle bending with bolts in tension with the corresponding parameters related to tests performed with bolts in compression zone (Table 11), it can be noticed that the yield and maximum bending moments as well as the maximum absorbed energy of tests performed with bolts in tension are always larger than those of tests performed with bolts in compression zone, when the axial load is less than 75kN.

<table>
<thead>
<tr>
<th>Steel base</th>
<th>My</th>
<th>$\phi_y$</th>
<th>$S_{j,ini}$</th>
<th>$M_{max}$</th>
<th>$\phi_{max}$</th>
<th>$E_{max}$</th>
</tr>
</thead>
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<tr>
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<td>2.91</td>
<td>1.05</td>
<td>1.87</td>
<td>sbcb/sbct All</td>
</tr>
<tr>
<td>3.52</td>
<td>1.32</td>
<td>2.52</td>
<td>3.25</td>
<td>1.50</td>
<td>2.85</td>
<td>sbcb/sbct Base bending</td>
</tr>
<tr>
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<td>1.15</td>
<td>2.13</td>
<td>2.49</td>
<td>1.42</td>
<td>3.37</td>
<td>sbcb25/sbct25</td>
</tr>
<tr>
<td>1.64</td>
<td>0.79</td>
<td>1.99</td>
<td>1.70</td>
<td>1.18</td>
<td>2.17</td>
<td>sbcb50/sbct50</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete base</th>
<th>My</th>
<th>$\phi_y$</th>
<th>$S_{j,ini}$</th>
<th>$M_{max}$</th>
<th>$\phi_{max}$</th>
<th>$E_{max}$</th>
</tr>
</thead>
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<td>2.01</td>
<td>1.71</td>
<td>3.07</td>
<td>cbcb25/cbct25</td>
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<tr>
<td>1.75</td>
<td>2.02</td>
<td>0.91</td>
<td>1.63</td>
<td>1.28</td>
<td>1.90</td>
<td>cbcb50/cbct50</td>
</tr>
<tr>
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<td>1.25</td>
<td>0.41</td>
<td>0.47</td>
<td>0.72</td>
<td>0.32</td>
<td>cbcb75/cbct75</td>
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</table>

<table>
<thead>
<tr>
<th>Athen base (bolted+welded)</th>
<th>My</th>
<th>$\phi_y$</th>
<th>$S_{j,ini}$</th>
<th>$M_{max}$</th>
<th>$\phi_{max}$</th>
<th>$E_{max}$</th>
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<tbody>
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<td>2.47</td>
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<td>2.28</td>
<td>2.16</td>
<td>0.61</td>
<td>1.07</td>
<td>asbcb25/asbct25</td>
</tr>
</tbody>
</table>

Table 11: Comparison of results of monotonic tests on column-bases bent in cross-aisle direction: bolts in tension vs. bolts in compression.

<table>
<thead>
<tr>
<th>Steel base</th>
<th>My</th>
<th>$\phi_y$</th>
<th>$S_{j,ini}$</th>
<th>$M_{max}$</th>
<th>$\phi_{max}$</th>
<th>$E_{max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.37</td>
<td>1.18</td>
<td>1.13</td>
<td>1.34</td>
<td>1.34</td>
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</tr>
<tr>
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<td>1.31</td>
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<td>0.86</td>
<td>sbcb/sbdb Weld failure</td>
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<td>0.99</td>
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<td>1.20</td>
<td>1.15</td>
<td>1.43</td>
<td>sbcb/sbdb Base bending</td>
</tr>
<tr>
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<td>1.62</td>
<td>0.74</td>
<td>1.22</td>
<td>1.71</td>
<td>1.92</td>
<td>sbcb25/sbdb25</td>
</tr>
<tr>
<td>0.85</td>
<td>1.29</td>
<td>0.67</td>
<td>0.86</td>
<td>1.32</td>
<td>1.07</td>
<td>sbcb50/sbdb50</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete base</th>
<th>My</th>
<th>$\phi_y$</th>
<th>$S_{j,ini}$</th>
<th>$M_{max}$</th>
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<th>$E_{max}$</th>
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<td>0.70</td>
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<td>1.18</td>
<td>cbcb50/cbdb50</td>
</tr>
<tr>
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<td>1.48</td>
<td>0.23</td>
<td>0.33</td>
<td>1.30</td>
<td>0.42</td>
<td>cbcb75/cbdb75</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Athen base (bolted+welded)</th>
<th>My</th>
<th>$\phi_y$</th>
<th>$S_{j,ini}$</th>
<th>$M_{max}$</th>
<th>$\phi_{max}$</th>
<th>$E_{max}$</th>
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</thead>
<tbody>
<tr>
<td>0.98</td>
<td>1.38</td>
<td>0.71</td>
<td>1.04</td>
<td>0.76</td>
<td>0.55</td>
<td>asbcb25/asbdb25</td>
</tr>
</tbody>
</table>

Table 12: Comparison of results of monotonic tests on column-bases in cross-aisle bending with bolts in tension vs. down-aisle bending.

The yield and maximum rotation of specimens tested with bolts in tension are always larger than those of tests performed with bolts in compression zone, with the exception of specimens with 50 kN axial load bolted to a steel deck ($\phi_y, CT > \phi_y, CB$) and of those bolted to a concrete surface and axial load 75kN ($\phi_{max}, CT > \phi_{max}, CB$). Stiffness of specimens connected to a steel deck (simply bolted or bolted-and-welded) tested with bolts in tension was always higher than that of specimens tested with bolts in compression zone.

The opposite is true for specimens bolted to a concrete deck, for which ($S_{j,ini}, CT > S_{j,ini}, CB$). This is probably due to some problems with the bond between the anchor bolts and the concrete or eventually to cracking of the concrete under tensile loads.
When comparing the main response parameters of monotonic tests in cross-aisle bending with bolts in compression with the corresponding parameters related to tests performed in down aisle direction, (Table 12) the following considerations can be drawn: stiffness $S_{j,ini}$ in down-aisle direction is always higher than in cross aisle direction, in presence of axial load in the column. Both the yield $M_y$ and maximum $M_{max}$ bending moments in down-aisle are larger than in cross-aisle, for axial load larger than 25 kN. The yield rotation $\phi_y$ in cross aisle direction, with bolts in tension is always larger than the one in down aisle direction. The same consideration is true also for the maximum rotation $\phi_{max}$, with the exception of the case when the base is bolted-and-welded to the steel base.

When comparing the main response parameters of monotonic tests in cross-aisle bending with bolts in compression zone, with the corresponding parameters related to tests performed in down aisle direction, (Table 13) the following considerations can be drawn: stiffness $S_{j,ini}$ as well as both the yield $M_y$ and maximum $M_{max}$ bending moments in down-aisle direction is always higher than in cross aisle direction. The yield rotation $\phi_y$ as well as the maximum rotation $\phi_{max}$, in cross aisle direction, with bolts in compression zone, are always larger than those in down aisle direction, in presence of an axial load in the column. The maximum absorbed energy in down-aisle direction is always higher than in cross aisle direction, for axial loads lower than 75kN.

2.3.1.2 Cyclic tests

The Figure 23 shows the comparison of the hysteretic behaviour of a column-base connected to a concrete deck, with 25kN axial load, for different bending directions. It should be pointed out that the hysteresis loops are plotted in such a way that positive bending moments and rotation correspond to tensile forces in the bolts, while negative bending moments and rotations correspond to situations in which bolts are in compression zone.
Response in cross-aisle direction of this type of connection, when bolts are in the compression zone, is better than the one evidenced by specimens connected to the steel deck, most probably due to the local deformability of the concrete surface, or in any case to the different behaviour at the base-deck interface.

It was observed during the tests that increasing the axial load up to 50 kN is beneficial for the specimen response in down-aisle direction. In cross-aisle direction, increasing the axial load up to 50 kN results in a reduction of rotation capacity. The load carrying capacity is also reduced when bolts are in tension (because of occurrence of buckling of the free edges of the cross section of the column profile which are in compression). On the contrary, the bending strength is improved by increasing the axial load, for those conditions in which the bolts are in the compression zone (as in this case the axial load reduces the bending deformation of the steel base plate). Further increasing of the axial load to 75 kN causes a loss of load-carrying as well as rotation capacity of the specimens in both bending directions.

During the tests it was observed that the beneficial effect of the bolted-and-welded type of connection is evident, when comparing the behaviour of specimens connected to a steel floor, particularly for cross aisle bending. When compared to specimens connected to a concrete floor, the response of bolted-and-welded base connections shows a small enhancement of both load-carrying and rotation capacity in down-aisle direction. In cross aisle direction, the response of the bolted-and-welded specimens is superior when bolts are in tension, but is comparable in terms of load carrying and rotation capacity with that of specimens connected to the concrete floor when bolts are in the compression zone, despite a
much higher energy dissipation capacity evidenced by the shape of the hysteresis loops. Figure 24 compare the response of specimens simply bolted to a steel and to a concrete deck, under 25kN axial load, respectively under bending in down-aisle direction.

<table>
<thead>
<tr>
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<th>M_y</th>
<th>(\phi_y)</th>
<th>S_{j,ini}</th>
<th>N_u</th>
<th>E_{tot}</th>
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</tr>
<tr>
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<td>0.76</td>
<td>0.96</td>
<td>0.67</td>
<td>0.38</td>
<td>sbccf25 / cbdcf25</td>
</tr>
<tr>
<td>1.11</td>
<td>0.72</td>
<td>1.53</td>
<td>2.50</td>
<td>4.29</td>
<td>sbdcf25 / cbdcf25</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete base</th>
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<th>(\phi_y)</th>
<th>S_{j,ini}</th>
<th>N_u</th>
<th>E_{tot}</th>
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<tr>
<td>0.88</td>
<td>0.79</td>
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<td>0.67</td>
<td>0.62</td>
<td>cbdcf25 / cbccf25</td>
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<td>1.89</td>
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<td>1.89</td>
<td>cbdcf50 / cbccf50</td>
</tr>
<tr>
<td>0.17</td>
<td>0.37</td>
<td>0.47</td>
<td>3.00</td>
<td>13.99</td>
<td>cbdcf75 / cbccf75</td>
</tr>
<tr>
<td>0.46</td>
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<td>1.18</td>
<td>0.75</td>
<td>0.49</td>
<td>cbccf50 / cbccf25</td>
</tr>
<tr>
<td>1.24</td>
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<td>1.38</td>
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<td>cbdcf50 / cbdcf25</td>
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<td>0.34</td>
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<td>0.22</td>
<td>0.04</td>
<td>cbccf75 / cbccf50</td>
</tr>
<tr>
<td>0.83</td>
<td>0.74</td>
<td>1.13</td>
<td>0.55</td>
<td>0.33</td>
<td>cbdcf75 / cbccf50</td>
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<tr>
<td>0.16</td>
<td>0.13</td>
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<td>0.17</td>
<td>0.02</td>
<td>cbccf75 / cbccf25</td>
</tr>
<tr>
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<td>0.46</td>
<td>2.25</td>
<td>0.75</td>
<td>0.48</td>
<td>cbdcf75 / cbdcf25</td>
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<th>M_y</th>
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<td>1.25</td>
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<td>asbdcf25 / sbdcf25</td>
</tr>
<tr>
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<td>1.29</td>
<td>0.70</td>
<td>0.61</td>
<td>asbdcf25 / sbdcf25</td>
</tr>
<tr>
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<td>1.25</td>
<td>1.13</td>
<td>0.83</td>
<td>1.35</td>
<td>asbdcf25 / cbdcf25</td>
</tr>
<tr>
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<td>0.74</td>
<td>1.97</td>
<td>1.75</td>
<td>2.63</td>
<td>asbdcf25 / cbdcf25</td>
</tr>
</tbody>
</table>

Table 14 Comparison of results of cyclic tests on column-bases

It was observed that the response of the specimens bolted to a steel deck can be compared to the one of specimens bolted to the concrete floor, in the case of cross-aisle bending, with bolts in tension. When the bending direction is reversed, and bolts are in compression zone, the specimens connected to the concrete deck show higher load-carrying and rotation capacity. In down-aisle direction, specimens connected to a steel deck show a larger rotation capacity than those connected to a concrete floor. However, when under bending with bolts in compression zone, the load-carrying capacity of specimens connected to a concrete floor is higher. Table 14 summarizes these type of comparisons, in terms of ratios of the main response parameters (yield moment M_y and rotation \(\phi_y\), initial elastic stiffness S_{j,ini} number of plastic cycles to failure N_u and total absorbed energy E_{tot} ) of the various specimens.

2.3.2 Conclusions

- In the column base connections under cross aisle bending, the axial compression load are beneficial when the bolts are in the compression zone. In this case, an increase of the axial force results in an increase of resistance and stiffness but in a decrease of rotation capacity of the specimens.
- When the loading direction results in tension of the bolts, the axial force in the upright causes a reduction of resistance and stiffness of the column bases, because induces distorsional buckling of the free edges of the cross-section profile.
• The tests performed on the column base in the down aisle direction proved that the axial force (25 and 50 kN) increases the initial stiffness and the resistance but decreases the rotation capacity of the connection.

• In the cyclic tests, the higher the axial force, the bigger the difference between the resistance under positive and negative bending moments.

• The collapse modes exhibited by the specimens were weld failure, base bending and distortional buckling of the cross section of the upright. In the tests with premature weld failure there was a reduction in the rotation capacity of the connection. Therefore, it is extremely important to control the welding process during the manufacturing of the components.

• Distortional buckling of the cross section of the upright occurred in the tests with an axial force of 50 and 75 kN, except for the monotonic test when the loading direction is such that bolts are in the compression zone. In the tests with the column bases in the cross aisle direction and axial force of 75 kN distortional buckling occurred prematurely, drastically reducing the mechanical properties of the connection.

• The base welded and bolted to the steel deck exhibited an increase in the resistance and rotation capacity of the connection since there was no base plate bending. In the cyclic tests a higher capacity of energy dissipation was observed, when compared to simply bolted connections.

• The results prove that connecting the column base to a concrete slab or to a steel deck does not change the mechanical properties or the failure mode of the connection.

• In both cases pre-tension in the bolts should be provided.
3 PALLETS SLIDING

3.1 OVERVIEW

3.1.1 Friction models

Friction is the tangential reaction force between two surfaces in contact. Physically these reaction forces are the results of many different mechanisms, which depend on contact geometry and topology, properties of the bulk and surface materials of the bodies, displacement and relative velocity of the bodies and presence of lubrication.

In dry sliding contacts between flat surfaces, friction can be modelled as elastic and plastic deformation forces of microscopical asperities in contact. For each asperity contact the tangential deformation is elastic until the applied shear pressure exceeds the shear strength $\tau_y$ of the surface materials, when it becomes plastic.

There are different models of friction that consider stationary condition (Coulomb friction model, Karnopp Model and Armstrong’s Model), e.g. constant velocity of the contact surfaces, and other, developed in the last century, that consider friction with a dynamic model (Dahl Model, Bliman and Sorine, LuGre Model). Numerical results for the sliding of pallets are also available.

3.1.2 Aims and scopes of the investigation

This study is developed within the SEISRACKS project, and presents the results of the static and dynamic experimental tests, aimed to an assessment of both the static and the dynamic friction factor developed between the pallet and the beam. Both static and dynamic tests carried out at the Laboratory for Earthquake Engineering of the National Technical University of Athens (LEE/NTUA).

First, the influence of different parameters (such as the type of pallet and beam, the stored mass and the mass eccentricity) on the static friction factor is analysed. Then, the dynamic tests carried out on a small portion of pallet rack are presented. The structure is excited with a sinusoidal input signal of the shaking table, in cross and in down aisle direction. The objective is to study the pallets behaviour under dynamic condition, and to identify the horizontal actions that are transferred to the rack structure by the live load (i.e. by the palletized goods) under dynamic conditions, as a function of the input motion characteristics (in terms of acceleration and frequency) as well as of the pallet-rack interface characteristics (i.e. beam type and surface finish). The results of some seismic tests carried out on the same structure, and a comparison with the results of the sinusoidal tests are also presented.

3.2 ASSESSMENT OF THE STATIC FRICTION FACTOR

3.2.1 Overview

A total of 1260 static friction tests were performed at the LEE/NTUA, Greece. The aim of this group of tests was to obtain the static friction factor for different combinations of beams and pallets, and to study the influence of the mass and of its eccentricity. The test set-up for static tests is shown in Figure 25. Two horizontal beams were fixed on a rigid steel frame with pinned support. The frame was free to rotate about the pinned axis. The axial distance between the pinned axis and the point of rotation was 1575mm. One pallet with a rigidly fixed mass was positioned on the beams. The system was gradually and slowly inclined with the use of a crane, measuring the vertical displacement and the relative displacement between pallet and beam, due to sliding of the pallet. Thirty repetitions of each test (combination of pallet and beam) were carried out. These tests were performed in down and cross aisle direction.
Different types of pallets and beams were used during the experimental tests. Three different values of the applied mass were considered (251 kg, 785 kg, 1036 kg) as well as the different position of the mass on the pallet (centered, eccentric downward, eccentric upward). Seven types of pallets and six types of beam were used in the tests (Table 15), with the following denomination:

<table>
<thead>
<tr>
<th>PALLETS</th>
<th>BEAMS</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1: Wooden Euro pallet 800x1200, new, dry</td>
<td>B1: Cold rolled, powder coated, new (Producer A)</td>
</tr>
<tr>
<td>P2: Wooden Euro pallet 800x1200, old, dry</td>
<td>B2: Cold rolled, hot dip coated, new (Producer A)</td>
</tr>
<tr>
<td>P3: Wooden Euro pallet 800x1200, old, wet</td>
<td>B3: Cold rolled, hot zinc coated, new (Producer A)</td>
</tr>
<tr>
<td>P4: Wooden American pallet, new, dry</td>
<td>B4: Cold rolled, hot dip coated, new (Producer B)</td>
</tr>
<tr>
<td>P5: Wooden American pallet, old, dry</td>
<td>B5: Cold rolled, hot dip coated, new (Producer C)</td>
</tr>
<tr>
<td>P6: Wooden American pallet, old, wet</td>
<td>B6: Cold rolled, hot dip coated, new (Producer C)</td>
</tr>
<tr>
<td>P7: Plastic Euro pallet</td>
<td></td>
</tr>
</tbody>
</table>

Table 15 Type of pallet and beam used in the different tests

Pallet P1 is a wooden europallet new, however, after being in use for a while, the lower faces of the pallet wear out. So, the normal situation is the one represented by pallet P2, a wooden Euro pallet old and dry. In order to investigate eventual environmental conditions, pallet P3 is a wooden Euro pallet, old, that was spread with water for a few minutes, before testing. The same conditions were reported for the American type of pallets, respectively P4, P5, P6. Pallet P7 is a plastic Euro pallet. This type of pallet is more and more adopted, as it is a more resistant and can be more easily cleaned than the wooden one. It is widely adopted, for example, for storage of food, in particular in refrigeration units.

Six different types of beams, manufactured by three different companies were considered. Beams B1, B2 and B3 were manufactured by the same company (A); the cross section was the same for all the three types, but surface treatment was different. B1 was powder coated beam, B2 was a hot dip coated beam and B3 a hot zinc coated beam. Beam B4 was manufactured by a different company (B), with a different cross section and surface treatment than the previous ones.

Beam B5 and B6 were manufactured by another company (C) and differ for their geometry, although surface treatment was the same for both beams.

Static tests are based on the principle of the inclined plane shown in Figure 26, in which when the pallet starts to slide on the beam the component of the gravity force along the beam ($F_{\parallel}$) is equal to its perpendicular component ($F_{\perp}$) multiplied by a static friction factor, that can be obtained in this way:

$$F_{\parallel} = F_{\text{grav}} \cdot \sin(\alpha) = F_{\text{frict}} = \mu \cdot F_{\text{grav}} \cdot \cos(\alpha)$$

hence:

$$\mu = \tan(\alpha)$$

Figure 26 Principle of inclined plane
3.2.2 Friction in cross aisle direction

Figure 27 shows the setup for tests in cross aisle direction. Sliding in this direction is very dangerous because the pallet width is 1200 mm while the rack width is 1100 mm. Hence, a few mm of displacement, eventually correlated to a small eccentricity of positioning, can result in a loss of support of the pallet. In the next paragraphs all the figures show the mean values of the static friction factor, with the indication of the standard deviation, for every test type. All the tests are repeated 30 times in the same conditions.

![Figure 27 Set up for cross aisle tests](image)

3.2.2.1 Influence of the pallet type

Figure 28 shows the influence of the pallet type on the static friction factor. All the tests are carried out with a mass of 785 kg centred on the pallet.

![Figure 28 Influence of the pallet type on the friction factor in cross aisle direction, for different types of beam](image)

It can be observed that:

- Pallet 4 (American Wooden Pallet) has the highest mean value of the friction factor (0.51), while pallet 7 (Plastic Euro Pallet) has the lowest. Pallet 1 (Wooden Euro Pallet) has an intermediate value, very close to the one measured for Pallet 4.

3.2.2.2 Influence of the beam type

Similar considerations can be drawn with regards to Figure 29 that shows the influence of the beam type. Tests are carried out with a centred mass of 785 kg. It can be observed that:

- Pallet 4 (American wooden Pallet) shows the highest value of the friction factor, pallet 1 (Wooden Euro Pallet) an intermediate one and pallet 7 (Plastic Euro Pallet) the lowest one.
- Friction factor for pallet 7 is quite constant, while for the other two types of pallet the friction factor shows a strong dependence on the beam type. In particular the lowest values are obtained for beam type 3 and 4, while in the other cases the friction factor is similar.
- For pallet 1 and 4 the highest friction factor is obtained with beam 5.

Related to the repetition of tests it can be noticed that in the first tests, the friction factor shows an increasing trend while after 5-10 tests; the obtained value is practically constant. This is most probably due to the “wearing” of the surface of the beam.
3.2.2.3 Influence of the applied mass

The influence of this factor is measured for a mass of 251 kg, 785 kg and 1036 kg, for pallet type P1, for different types of beam. The mass is fixed on the pallet so that there is no relative displacement. Figure 29 shows the influence of the applied mass on the friction factor in cross aisle direction, for different beam type. It can be observed that:

- Applied mass strongly influences the friction factor only for beam B1, B2 and B3, while in the other cases it is quite constant.
- The highest value of the friction factor is obtained with the mass of 251 kg independently of the beam type, while the lowest one with the intermediate mass.

As expected, the value of the applied mass influences the response of the system less than the other analysed parameters. The mean values of the static friction factor are practically non influenced by the value of the applied mass.

3.2.2.4 Friction in down aisle direction

Sliding in this direction is less dangerous than in cross aisle direction, because fall of the pallet can occur only if a rotation around the vertical axes is associated with the pallet displacement. The same parameters are analysed as in cross aisle direction. Figure 31 presents the instrumentation set-up for tests in down aisle direction.

3.2.2.5 Influence of the pallet type

Figure 32 shows the influence of the pallet type on the friction factor in down aisle direction, for different types of beam. It can be observed that:

- Trend of friction factor is similar for every considered beam with the exception of beam type 4. In all cases the highest value is obtained for pallet 1 (Wooden Euro Pallet). The lowest one is obtained...
for pallet 7 (Plastic Euro Pallet) in the most of cases, except for pallet 4 (Wooden American Pallet) that shows a minimum corresponding to beam type 4.

3.2.2.6 Influence of the beam type
The same results are plotted in Figure 33 showing the influence of the beam type on the friction factor, for a mass of 785 kg centred on the pallet. It can be observed that:

- Friction factor for pallet 1 (wooden Euro pallet) is the highest. Pallet 7 (plastic Euro pallet) shows the lowest friction factor. Pallet P4 (wooden American pallet) has an intermediate behaviour.
- Plastic pallet (P7) shows practically the same friction factor independently on the beam type.
- Behaviour of the friction factor for pallet P1 and P4 is quite similar: the lowest value is obtained for beam type 4; beam types B1, B2, B5 and B6 have more or less the same value.

3.2.2.7 Influence of the applied mass
The influence of the mass is measured on the same pallet type P1 (Wooden Euro pallet) positioning the mass on the pallet without eccentricity and considering different types of beams. In Figure 34 the influence of beam type on friction factor for different types of applied masses and the influence of applied mass on friction factor for different types of beams are presented.

It can be concluded that variation of the mass has a limited influence on the value of the friction factor. Such an influence is lower than the one of other parameters like pallet and beam types.
3.2.2.8 Influence of the mass eccentricity

The influence of the mass eccentricity was investigated only in the down aisle direction. The position of the mass on the pallet determines a different distribution of the weight force on the beam, that can influences the value of the friction factor.

Figure 35 shows the results of the tests, carried out with the combination of Pallet type P4 (Wooden American Pallet) with a beam type B6. When the mass is positioned with an upward eccentricity, the measured friction factor is larger than in the case with the downward eccentricity. The centred mass develops a friction factor larger than in the case of downward eccentricity. The variations of the friction coefficient due to differences in the eccentricity of the mass are in any case very small.

![Figure 35](image)

**Figure 35** Influence of the mass eccentricity on the friction factor in down aisle direction

3.3 ASSESSMENT OF THE SLIDING CONDITIONS OF THE PALLETS UNDER DYNAMIC ACTIONS

Within the “SEISRACKS” research Project 188 tests were carried out in order to investigate the dynamic sliding of pallets on racks. Dynamic tests were performed at LEE/NTUA. Different types of “sliding” test were performed, considering different combinations of beams and pallets, both in cross aisle and down aisle direction. The following types of excitations were considered:

- Sinusoidal with constant frequency and increasing acceleration (133 tests)
- Sinusoidal with constant acceleration and increasing frequency (27 tests)
- Seismic tests with recorded input motions (22 tests)

3.3.1 Test in cross aisle direction

3.3.1.1 Test set up

In cross aisle direction tests were carried out only with a sinusoidal excitation of the table with constant frequency. The test set up is shown in the Figure 36.

![Figure 36](image)

**Figure 36** Experimental and instrumentation set up for cross aisle tests

3.3.1.2 Results of the cross aisle tests

3.3.1.2.1 Test with Pallet type P2 – Beam type B1

All the results of the cross aisle tests carried out with a wooden Euro Pallet and a cold rolled, powder coated, new beam are analysed. From the analysis of the results obtained with the use of the beam type B1 (cold rolled, powder coated, new beam) some observations can be drawn. When the frequency of the table acceleration is very low (e.g. 1.0 Hz) sliding occurs only on the lateral
pallets, while no relative displacement occurred between the central pallet and the beam. This behaviour is due to the fact that the lateral pallets reach a higher value of the sliding acceleration earlier than the central one. On the contrary, for the central pallet sliding starts with higher values of the table acceleration.

A similar response is expected for both the lateral pallets. In some cases, however, they show different values of sliding acceleration and upper bound acceleration.

In a lot of tests the dispersion of the data is very low, with a c.o.v.% lower than 13%. The highest scatter of 12.7% is obtained for the upper bound of the acceleration for pallet 1 at 4.0 Hz, but in the most of cases the c.o.v.% value is lower than 10%. This is probably achieved by frequent replacement of the beams, as soon as deterioration of the surface paint was detected. Unfortunately, this was not possible for the whole set of tests, as the number of the available beams of the same type was limited.

Analysing the global results of this group of tests, that can be observed that the acceleration of sliding initiation shows a decreasing trend when the frequency increases. This behaviour is evident for the two lateral pallets; the central pallet starts to slide at higher values of acceleration. Figure 37 shows, for every pallet, the mean values of the lower and upper bound of the sliding acceleration plotted vs. the frequency of the excitation. It can be noticed that the general behaviour of this combination of beam and pallet is a decreasing trend of the sliding acceleration when the frequency increases.

![Figure 37](image1)

3.3.1.2.2 Test with Pallet type P2 – Beam type B3

Also in this group of tests the pallet used is the wooden Euro Pallet, while the beam is a cold rolled, hot zinc coated, new. From the analysis of the results obtained with the use of the beam type B31 (cold rolled, powder coated, new beam) some observations can be drawn.

The acceleration of sliding initiation (lower bound) shows a more or less constant value for the two lateral pallets while, for the central one, it shows a decreasing trend when the frequency increases. At lower frequency, e.g. 1.0 Hz, only the two lateral pallets slide on the beam.

For the same reasons shown in the previous tests carried out with the powder coated beam (B1), also in this case sliding starts first for the lateral pallets. Acceleration of the central mass should be much higher when sliding occurs. In some cases sliding of the central mass couldn’t be developed because the test was stopped before the attainment of the limit conditions.

The upper bound value of the acceleration shows a strongly decreasing trend, with the exception of frequency of 1.0Hz. In particular the upper bound of the sliding acceleration is similar for the three pallets in the case of f=1.5Hz.

The behaviour of pallet 1 is similar to the pallet 3.

In Figure 38 the lower and the upper bound of the sliding acceleration, are shown for every pallet with respect to the frequency.

![Figure 38](image2)
3.3.2 Tests in down aisle direction

3.3.2.1 Test set up
In down aisle direction tests were carried out with two different loading histories:
1. sinusoidal excitation of the table with constant frequency and increasing acceleration;
2. sinusoidal excitation with constant acceleration and increasing frequency.

The test set up is similar to the previous series of tests in cross aisle direction, but with a different position of the instrumentation (Figure 39).

3.3.2.2 Results of the down aisle tests

3.3.2.2.1 Constant frequency and increasing acceleration tests
These tests are obtained with the use of an old, dry, wooden Euro pallet (pallet type 2), combined in most of cases with a new, hot dip coated, cold rolled beam (Beam type 2). Only few tests are carried out with a new, hot zinc coated, cold rolled beam (Beam type 3), but were neglected in the following analysis, due to the low number of available beams during the experimental tests.

Tests in down aisle direction show some characteristics similar to those of the cross aisle tests. The data confirms that the first pallet to slide is one of the two lateral ones.

In the tests at frequency 1.0 Hz sliding of the central pallet never occurred, as the test had to be terminated before the acceleration reached a value large enough to cause its sliding. From tests at frequency 2.0 Hz the values of sliding initiation obtained for the three pallets are very close to each other. The difference that was evident in the cross aisle tests between the sliding acceleration of the central pallet and the two lateral ones is not evident in this case. In down aisle tests, the effect of the deformation in the X-Y plane is limited, and doesn’t influence much the pallet behaviour. Also the upper bound of the acceleration shows similar values for all pallets.

In order to evaluate the effect of the deterioration of the beam surface, also in this case tests were carried out for the same frequency, on the same beam, in two different groups, spaced of a number of tests from each other. This means that after performing a first group of tests at 2.0 Hz, the frequencies of 3.0 Hz and 4.0 Hz were investigated (running the first group of tests for each frequency). Later, a second group of tests was carried out for the frequencies of 2.0 Hz and 3.0 Hz.

From the analysis of these results it can be noticed that deterioration of the beam surface influences very much the sliding behaviour of the pallets, and that the mean values of both the lower and upper bound of the acceleration of all pallets increase when passing from the first to the second group of tests. Figure 40 shows for each investigated frequency the mean values of the results obtained in terms of
lower bound and upper bound of the sliding acceleration.

3.3.2.2 Constant acceleration and increasing frequency tests

These tests were carried out on the combination pallet type 2 (wooden Euro pallet) and beam type 2 (cold rolled, hot dip coated, new beam). The purpose of this kind of tests is to analyse the behaviour of a storage rack with three pallets when the shaking table is excited with an increasing value of the frequency under a constant acceleration, and then compare the data with those obtained from the tests with constant frequency for the same type of pallet and beam. In this group of tests useful data could be obtained only with regard to the lower bound of the sliding acceleration.

The risk with this type of testing procedure is that, increasing the frequency of the excitation, the own frequency of the structure can be reached during the test, causing resonance and consequent serious damage. In order to avoid this problem most of tests were terminated before reaching resonance. Hence, the results take into account only the response of the structure before the resonance peak, and are obtained with the same procedure used in the tests in down aisle direction with constant frequency.

Figure 41 show the values of acceleration and frequency when sliding starts, for each pallet. In the case of pallet 1 the measured values of the sliding acceleration are concentrated at the frequency of about 2.5 Hz and 5 Hz. No relevant value is present between these two frequencies. For pallet 2 and 3, values could be measured also for intermediate frequencies. Sliding acceleration of the central pallet is slightly higher than that of the two lateral ones. It seems that sliding occurs at the same value of acceleration independently to the frequency. No pallet shows sliding for input motions with a low frequency.

![Figure 41](image)

**Figure 41** Lower bound of sliding acceleration for pallet 1, 2 and 3 in the tests with constant acceleration and variable frequency

3.4 COMPARISON BETWEEN THE DIFFERENT TESTS

3.4.1 Cross aisle tests

Tests in cross aisle direction have been carried out with a sinusoidal excitation of the shaking table with constant frequency and an increasing acceleration. Only one type of pallet was considered, the wooden Euro pallet, old (type P2). Two different types of beams were adopted, beam B1, hot rolled, powder coated, and beam B3, hot rolled, hot zinc coated, both manufactured by the same producer.

![Figure 42](image)

**Figure 42** Sliding tests in cross aisle direction for beam type B1 and B3.

Figure 42 summarizes the results of all tests carried out in cross aisle direction, respectively on beam type B1 and type B3. It can be observed that, in the case of beam B1 (powder coated), the minimum measured value of the acceleration causing sliding of at least one of the pallets decreases when increasing the frequency of the excitation, ranging from approximately 0.17g for a frequency of 1.0 Hz, to 0.1g for a frequency of 4.0 Hz. On the contrary, in the case of beam B3 (hot zinc coated) the minimum measured value of the acceleration causing sliding of at least one of the pallets is approximately 0.1g and this value is practically independent on the frequency.

Figure 43 summarizes the mean values of the sliding accelerations for the set of both pallet 1 and 3, for
the two types of beams B1 and B3. As already evidenced, the set of data is rather homogeneous, and seems to be not much influenced by the frequency of the excitation, at least up to 3.0Hz. Also, it can be stated that no sliding of the lateral pallets occurs if the acceleration is lower than 0.12g.

![Figure 43](image1.png)

**Figure 43** Sliding of the lateral pallets in cross aisle tests

The behaviour of the lateral pallets (pallet 1 and 3) is similar for all tests. Sliding of the central, pallet 2, starts under values of acceleration higher than for the two lateral pallets. When the structure will be subjected to a seismic event (or to a dynamic excitation), the first pallets to slide will be the lateral ones. Figure 44 shows, respectively for the two lateral pallets P1+P3, and for the central pallet P2, the mean values of the “upper bound” of sliding acceleration, obtained for different frequencies of the excitation, and for the two types of beam B1 and B3.

![Figure 44](image2.png)

**Figure 44** Upper bound of sliding acceleration vs frequency in cross aisle direction for the lateral pallets and central pallet.

Upper bound of sliding acceleration for the central pallet is larger than the one of the two lateral ones. There is a slightly decreasing trend of the upper bound of the sliding acceleration with the frequency. It can be noticed that the lower bound of the sliding acceleration for beam type B3 is lower than the one of beam type B1. On the contrary, the upper bound of the sliding acceleration for beam B3 is higher than the one of beam B1. The maximum of the mean values of the acceleration that could be measured on a mass in cross aisle direction is 0.45g for the lateral pallets, and 0.5g for the central one.

### 3.4.2 Down aisle tests

Tests in down aisle direction are carried out with two different types of beams: type B2 (hot dip coated) and type B3 (hot zinc coated) both manufactured by the same producer. In the first case it is possible to compare a large batch of data of tests performed both at constant frequency and at constant acceleration. In the second case the low number of tests doesn’t allow a significant re-analysis. The Figure 45 shows a comparison of the mean values of acceleration causing sliding initiation of the pallets.

![Figure 45](image3.png)

**Figure 45** Response of the down aisle tests for pallet 1, 2 and 3 (mean values)

The response seems to be independent on the frequency. There is no pallet sliding for acceleration lower than 0.4g, for the lateral pallets. The central pallet (pallet 2) slides for accelerations larger than 0.5g, higher than those of the two lateral ones.
Figure 46 shows, respectively for the two lateral pallets P1+P3 and for the central pallet P2, the mean values of the “upper bound” of the sliding acceleration, obtained for different frequencies of the excitation.

![Figure 46](image1)

**Figure 46** Upper bound of sliding acceleration vs frequency in down aisle direction for the lateral pallets and central pallet.

Also in this case the upper bound of the sliding acceleration for the central pallet is slightly higher than the value obtained from the two lateral ones. It seems that there is a sort of increasing trend of the upper bound of the sliding acceleration with the frequency. This behaviour is more evident for the lateral pallets. It can be noticed that the maximum of the mean values of the acceleration measured on a mass in down aisle direction is 0.55 g for the lateral pallets, and 0.6 g for the central one, higher than the values obtained in cross aisle tests.

### 3.5 Seismic Tests

Seismic tests were carried out both in down aisle and cross aisle direction adopting different types of input motions. Tests set-up was the same used in the tests with sinusoidal excitation, in cross and down aisle direction, using respectively beams type B1 and B2, always with a wooden Euro pallets. The structure was excited with three different types of seismic signal, samples of Greek earthquakes, scaled appropriately until sliding of pallets. The seismic input motions were EDESSA, KALAMATA and ARGOSTOLI earthquake. Seismic tests have been carried out in cross aisle direction for beam B1 type, and in down aisle direction for beam B2 type. In some cases the excitation of the table is bidirectional, both in X and Y direction.

#### 3.5.1 Seismic test – Cross aisle direction

The sliding acceleration is the one corresponding to occurrence of a relative displacement between pallet and beam. Figure 47 shows the acceleration of the pallet when sliding occurs for all types of test.

![Figure 47](image2)

**Figure 47** Acceleration of the lateral pallets and central pallet when sliding occurs in cross aisle seismic tests with the beam type B1.

These results can be compared with the sliding domain in cross aisle direction. It can be noticed that in the seismic tests sliding occurs first for ARGOSTOLI signal at around 0.15g. For the other types of seismic test the acceleration of sliding initiation is higher. The central pallet starts sliding with acceleration higher than the two lateral ones, for the same reason previously described with regard to sinusoidal cross aisle tests. If sliding occurs on the central pallet, its final displacement can be large.

#### 3.5.2 Seismic test – Down aisle direction

Seismic tests in down aisle direction were carried out with the beam type B2 (hot dip coated beam). Only two tests were carried out with Edessa time history, and one test with Argostoli time history. Sliding during these tests starts for acceleration of the pallet higher than 0.4g. This is in good agreement with the results of down aisle tests carried out under sinusoidal excitation, previously presented. In both tests the three pallets start sliding nearly contemporary. The final displacement of the pallets is larger in
the case of Argostoli earthquake (nearly 9mm for pallet 2) than in the case of Edessa earthquake (nearly 4.5mm for pallet 2). The values of acceleration of sliding initiation for the down aisle seismic tests can be compared with the sliding domain in down aisle direction. It can be noticed that, also in this case, sliding occurs first for ARGOSTOLI signal, at around 0.45g. For the other types of seismic test the acceleration of sliding initiation is higher. The central pallet starts sliding more or less at the same value of acceleration of the two lateral ones, as previously described with regard to sinusoidal down aisle tests.

3.5.3 Bidirectional Seismic test

Bidirectional seismic tests were carried out with Edessa and Argostoli time history. From experimental results, it can be noticed that in all tests sliding occurs with acceleration in cross aisle direction higher than that obtained as “lower bound” in the cross aisle tests with constant frequency excitation, considering the beam type B1, very similar to the type B2. The Figure 48 shows the values of the absolute acceleration of the three pallets when sliding occurs.

3.6 CONCLUSIONS

Assessment of both the static and the dynamic sliding conditions of pallets stored on steel racking systems was carried out within the SEISRACKS research project, by means of static as well as dynamic tests performed at the Earthquake Engineering Laboratory of the National Technical University of Athens.

Static tests were carried out in both down and cross aisle direction, by means of an “inclined plane” device, by slowly increasing the inclination of the plane, and measuring the sliding of the pallet on the rack steel beams. Influence of the following parameters was investigated:

- Type of beam (namely type of surface finish of the beam)
- Type of pallet
- Geometry and weight of mass resting on the pallet

Influence of the type of beam was investigated by adopting six different types of beam specimens, manufactured by three different producers, with different types of surface finish. In particular, hot zinc, hot dip and powder coated steel beams were considered. In both cross and down aisle direction, the surface finish influenced very much the static friction factor, with differences as large as 20-30% from one type to the other, in the case of wooden pallets.

Influence of the type of pallet was investigated by adopting three different types of pallets, namely: wooden Euro pallets, wooden American-pallet and plastic Euro pallet. In both cross and down aisle direction the plastic Euro pallet showed a very low friction factor (of the order of 0.2), practically being non-influenced by the type of beam surface finish. The wooden pallets show a very similar friction factor (of the order of 0.5), and similarly are influenced by the beam surface finish.

In both cross and down aisle direction, the mass weight didn’t affect much the results. However, its geometry (height of the c.o.g.) and its “placement” on the pallet (centered or eccentric) resulted in small variations of the measured friction factor.

Figure 48 Acceleration of the pallets when sliding occurs in bidirectional seismic tests - type P2-B2
Dynamic tests were carried out on the shaking table facility of the National Technical University of Athens, on a simplified set-up, made of two uprights, connected by two horizontal beams, at approximately 0.30 m from the shaking table. On the beams three wooden Euro pallets were positioned, with concrete blocks rigidly fixed on top. Most tests were carried out with a sinusoidal excitation, and with constant frequency and increasing acceleration. Some tests, in down aisle direction only, were carried out with sinusoidal excitation, with constant acceleration and increasing frequency, in order to verify independence of the obtained results on the type of adopted excitation. A lower bound of the acceleration exists, beyond which pallets start sliding on the steel beams. When acceleration of the mass is lower than such “lower bound”, the pallet “sticks” on the beams, and no sliding occurs. When the “lower bound” of acceleration is exceeded, increasing the acceleration of the input motion results in a lower increment in the mass acceleration, until an “upper bound” is reached of the mass acceleration. Any further increase in the acceleration of the input motion doesn’t affect the acceleration of the mass that is “free” to slide on the beams. “Sticktion” between pallet and beam is not resumed until a reduction of the acceleration occurs. The “upper bound” of the sliding acceleration is, in general, lower than the static friction factor. In both cross and down aisle direction lateral pallets slide systematically earlier than the central one.

Dynamic behaviour in cross aisle direction is completely different to the one in down aisle direction.

In cross aisle direction, the torsional stiffness as well as the flexural stiffness in the horizontal plane of the beams influence very much the results. In particular, such stiffness is affected by the out-of-plane and torsional behaviour of the beam-to-upright connections, whose stiffness rapidly deteriorates under cycling. Test results show, in general, a dependence of the sliding acceleration on the frequency of the input motion. Both the lower and the upper bound of the sliding acceleration seem to decrease when increasing the frequency of the excitation. Lower bound sliding acceleration as low as 0.1 g was measured, for wooden pallets on hot dip coated steel beams. Upper bound values of the acceleration ranging from 0.3g to 0.5 g were measured depending on the type of beam surface finish as well as on the position of the pallet (lateral or central one).

In down aisle direction, the sliding acceleration is in general higher than the one measured in cross-aisle direction, under the same testing conditions, with a lower bound of the measured sliding acceleration of nearly 0.3g, and an upper bound of nearly 0.6g. Also in down aisle direction, test results show, in general, a dependence of the sliding acceleration on the frequency of the input motion. However, in this case, both the lower and the upper bound of the sliding acceleration seem to increase when increasing the frequency of the excitation. Results of tests carried out with constant acceleration and increasing frequency are fully compatible with those obtained in tests with constant frequency and increasing acceleration.

A few seismic tests were carried out, adopting three different input motions recorder in Greece during recent earthquake, and characterized by different durations and frequency contents. Both mono-directional and bi-directional tests were performed. The obtained results were compared with those of tests carried out with a sinusoidal excitation, showing full compatibility. Measured sliding accelerations range from 0.15g to 0.35g in cross aisle direction and from 0.45g to 0.6g in the down aisle direction. Similar compatibility was also obtained for bi-directional tests, when comparing the resultants of the vector-compositions of the components of the sliding accelerations in the two orthogonal directions.
4 PUSHOVER TESTS

4.1 OVERVIEW

Two push-over tests (one in down aisle, the other in cross aisle direction) were carried out, in order to evaluate the possibility to propose in a Standard Design Code for Racks in Seismic Areas, static push-over analyses (currently available in many commercially available software packages for structural analysis) as alternative to dynamic (linear or non-linear) analyses, as it seems to be the current trend in many codes for seismic design of building structures.

The tests were carried out at the European Laboratory for Structural Assessment of the Joint Research Centre at Ispra, on three-levels, two-bays full-scale specimens, 6.0 m high and 3.6 m wide, with 130x45x1.5 mm beams and 100/20b uprights. No bracing system was present in the down-aisle direction (Figure 49). In the cross-aisle directions the frames were made of the uprights braced with a system of diagonals and struts. Diagonals were all positioned in the same direction (Figure 50).

Details of the beam-to-upright connections as well as of column bases were the same as in the tests conducted at Instituto Superior Tecnico of Lisbon (see chapter 2).

Four pallets were placed at each level, with an 8.5 kN load on each pallet, for a total vertical load on the structure of 102 kN (34 kN at each floor).

In order to avoid possible falling of the pallets/masses in the event of collapse of the specimen, these were hanged to a safety frame by means of steel bars connected to the (steel) masses by means of eye-
plates (Figure 51). In order to avoid friction at the eye-plates in case of large deformation of the specimen, teflon was placed around the bars.

Figure 51 “Safety” mass support system.

4.2 PUSH-OVER TEST IN DOWN-AISLE DIRECTION

4.2.1 Test set-up

An inverted triangular force distribution was adopted for the pushover test in down aisle direction, as shown in Figure 52.

Figure 52 Force distribution for the down-aisle pushover test

Displacements in the down aisle direction were imposed to the various levels of the specimen by means of three actuators (one per level), and by means of load distributors, the forces of the actuators were transferred to the uprights directly at the beam-to-upright joints.

In order to avoid local buckling of the uprights due to direct application of the forces, the load distributors were connected to the uprights by means of 200 mm long portions of beams, with the standard endplates and safety bolts (Figure 53).
The actuators were connected to the reaction wall by means of spherical hinges and were hanged to the safety frame, in order to avoid influence of their load on the specimen (Figure 54).

It was decided to perform the test with steps of loading up to a given top displacement, followed by unloading and by a subsequent reloading up to a larger top displacement value, as shown in the following Figure 55.

The specimen was instrumented (Figure 56) in order to measure:
- Displacements at all levels
- Forces at actuators
- Beam-to-upright rotations
4.2.2 Structural behaviour

After the cycles shown in Figure 55, the specimen was driven to collapse under monotonic loading. Specimen collapse was achieved corresponding to a top level transversal displacement of approximately 650 mm.

![Figure 56] Summary of the instrumentation applied to the specimen

Figure 57 shows the deformed shapes at various stages during the test, whereas Figure 58 shows the collapse mechanism, due to subsequent failure of various components. Initially, failure occurred in the base plate connections, that lost stiffness, and behaved like hinges. Immediately afterwards, plastic hinges formed in the uprights, at the top of the lower level, just below the beam-to-upright connection. The structural collapse was hence due to a soft-floor mechanism.

Finally, local buckling occurred in the uprights of the W-side (to which the actuators were connected), between the first and the second level. This mechanism was caused by some local effect connected with the concentrated-load transfer between actuators and structure. In the beam-to-upright connection zones, in fact, short portions of SHS were introduced inside the upright section, in order to avoid distortion of the cross section, due to the concentrated load. Local buckling occurred in the unstiffened
portion of the upright.

With further increasing the transversal displacement, collapse of the beam-to-upright connection was also achieved.

4.2.3 Assessment of the q-factor

Assessment of the q-factor was carried out by analysing the structural response in terms of base shear plotted vs. the horizontal top displacement, as shown in Figure 59.

The yield force $F_y$ and the yield displacement $v_y$ are conventionally defined according to the ECCS Recommendations.

Two values can be identified for the behaviour factor, based on ductility considerations: a value

$$ q_{\mu_{max}} = \frac{v_{max}}{v_y} = \frac{193.9}{52.1} = 3.7 $$

defined as the ratio of the displacement $v_{max}$ corresponding to the maximum load carrying capacity of the structure to the yield displacement ($v_y$), and a value

$$ q_{\mu_u} = \frac{v_u}{v_y} = \frac{363.8}{52.1} = 6.98 $$

defined as the ratio of the displacement $v_u$ (the maximum displacement bearable by the structure with a load carrying capacity larger than the yield force $F_y$) to the yield displacement ($v_y$).
With reference to the ductility factor theory, it is also possible to define a value of the q-factor based on strength, as the ratio of the ideal strength $F_{\text{max,el}}$ (corresponding to $v_{\text{max}}$ and evaluated on the basis of the initial elastic stiffness) to the maximum load carrying capacity $F_{\text{max}}$:

$$q_{\text{max}} = \frac{F_{\text{max,el}}}{F_{\text{max}}} = \frac{27.6}{8.9} = 3.1$$

### 4.3 Push-over Test in Cross-aisle Direction

#### 4.3.1 Test set-up

In the test in cross-aisle direction the base plates were subjected to bending in a direction perpendicular to the line of the bolts. The diagonal members of the upright were positioned (according to erection common practice) all in the same direction. The structure was hence positioned in such a way to resist to the applied loads with diagonals in compressions.

The following Figure 60 shows the instrumentation as well as the test set up, characterised by two jacks on the top level, and by one jack at each of the remaining floors. This configuration was due to the need to avoid possible torsion effects of the structure, at least by controlling the rotation of the top level. Loads were applied directly to the beams; thus allowing the real loading condition being simulated, with loads transferred from the pallets to the beams, and from the beams to the uprights. Furthermore, as shown in Figure 60, the actuators were positioned in such a way to obtain a uniform redistribution of the applied horizontal forces among the transversal frames.

In addition to LVDT’s measuring the transversal displacements at each level, inclinometers were positioned on the lower part of the uprights and on the column bases, in order to measure their rotations. The horizontal load distribution among the different levels was assumed according to the first
eigenvector, on-line identified during the test by means of the well-known Stodola method. The loading history adopted is shown in Figure 61. As in the case of the down-aisle direction, the test was performed with a sequence of loadings and unloadings. In particular, the first two loading-unloading sequences were performed symmetrical to the origin, and with a small absolute top displacement (approximately +/- 35 mm).

This allowed an assessment of the elastic stiffness and of the structural response to be made in the elastic range, when diagonal of the upright were respectively in tension or in compression. From the third loading-unloading sequence, however, the test was carried out under increasing values of the maximum displacement, always pushing in such a way to have diagonals in compression.

In each loading-unloading sequence, once the maximum imposed displacement was reached, the structure was unloaded until zero value of the applied force, and then reloaded up to a larger imposed displacement. Being the structural behaviour in the plastic range, this resulted in permanent deformations of the unloaded structure, as evident in Figure 61.

![Figure 61](image)

**Figure 61** Applied loading history: a) displacements; b) forces

### 4.3.2 Structural behaviour

The pushover test in the cross-aisle direction was carried out up to collapse of the specimen. Failure occurred because of buckling of the diagonal members of the transversal frames. These members were all positioned in the same direction (as from common erection practice), and loads were
applied in order to stress the diagonals in compression. Buckling started in the diagonals of the lower level, where the inter-storey drift is larger. The test was however continued until buckling of the diagonal members of all levels occurred. Figure 62 shows the deformed shape of the structure during the test. Buckling of the diagonal members of the uprights is clearly evident. Other local phenomena were observed, such as base-plate bending and upright buckling near to the diagonal-to-upright connection, as well as failure of the bolts connecting the diagonal to the column.

![Figure 62 Deformed shapes of the specimen in the pushover test](image)

Buckling of the diagonal members occurred around mid-test, arising not simultaneously in the three transversal frames. Afterwards, the structure was subjected to torsional effects, due to differences in the stiffness of the transversal frames with and without buckled diagonals. In any case, no relevant difference could be noticed among the three frames, as the horizontal displacements of the central one were similar to those of the lateral ones.

Buckling of the diagonals started in a lateral frame, most probably because the member geometric imperfections were larger than in the members of the other frames. When the diagonal buckled, the whole structure was subjected to torsion, most evident at the third level, where the presence of two actuators allowed the structure to continue to respond according to the first modal shape.

The structural response in the cross-aisle direction was strongly influenced by the orientation of the diagonal members of the transversal frames, by the behaviour of the base-plate connections as well as by the out-of-plane behaviour of the beam-to-upright connections.

### 4.3.3 Assessment of the q-factor

Assessment of the q-factor was carried out by analysing the structural response in terms of base shear plotted vs. the horizontal top displacement, as shown in Figure 63.

The yield force $F_y$ and the yield displacement $v_y$ were conventionally defined according to the ECCS (1986) Recommendations.

One value can be identified for the behaviour factor, based on ductility considerations:

$$ q_{\mu_{\text{max}}} = \frac{v_{\text{max}}}{v_y} = \frac{178.5}{75.1} = 2.4 $$

defined as the ratio of the displacement $v_{\text{max}}$ corresponding to the maximum load carrying capacity of the structure to the yield displacement ($v_y$).
As the test was stopped before complete structural collapse, the ultimate displacement $v_u$ (the maximum displacement bearable by the structure with a load carrying capacity larger than the yield force $F_y$) could not be assessed, as well as $v_q = \mu v_u / v_y$.

With reference to the ductility factor theory, it is possible to define a value of the $q$-factor based on strength, as the ratio of the ideal strength $F_{max,el}$ (corresponding to $v_{max}$ and evaluated on the basis of the initial elastic stiffness) to the maximum load carrying capacity $F_{max}$:

$$q_{fmax} = \frac{F_{max,el}}{F_{max}} = \frac{49.5}{23.6} = 2.1$$

### 4.4 DOWN-AISLE CROSS-AISLE COMPARISON

The tests carried out in down-aisle and in cross-aisle directions are different for the loading histories and force distribution among the vertical levels. An inverted triangular force distribution was adopted for the pushover test in down aisle direction, as shown in Figure 52, and the test was performed with steps of loading up to a given top displacement, followed by unloading to the undeformed condition (characterized by zero horizontal displacements at all levels) and by a subsequent reloading up to a larger top displacement value. This procedure did not allow for cumulative plastic deformation of the structure.

On the contrary, the test in the cross-aisle direction was carried out assuming a horizontal load distribution among the different levels according to the first eigenvector. The test was performed with a sequence of loadings and unloadings. The first two loading-unloading sequences were performed symmetrical to the origin. This allowed an assessment of the elastic stiffness and of the structural response in the elastic range, when diagonal of the upright are respectively in tension or in compression. From the third loading-unloading sequence, however, the test was carried out under increasing values of the maximum displacement, always pushing in such a way to have diagonals in compression. In each loading-unloading sequence, once reached the maximum imposed displacement, the structure was unloaded until zero value of the applied force, and then reloaded up to a larger imposed displacement, resulting in permanent deformations. Despite these differences, a comparison of the structural response in the two tests is very interesting in order to draw general conclusions about the behaviour of the racking systems in the two directions.

Figure 64 shows a comparison of the global response of the structure in cross and down aisle directions. It can be observed that in cross-aisle direction the structure is stiffer and stronger, but more fragile than in down-aisle direction. Both in down-aisle and in cross-aisle direction, the structural response was satisfactory. In down-aisle direction formation of plastic hinges in all the joints could be observed. In
cross-aisle direction most of the structural components remained in the elastic range, excluding the diagonal members of the bracing system of the vertical frames that, beyond a certain level, could not withstand the applied compressive forces and buckled.

In the cross-aisle direction loss of stiffness can be noticed only after the first pushing cycle, whereas the first two symmetric cycles did not cause any deterioration. On the contrary, the structure subjected to down-aisle loading shows a stiffness deterioration since the beginning of the test. It can also be noticed how the stiffness in the cross-aisle direction is larger than in the down-aisle one.

![Figure 64](image_url)

Comparison of the hysteresis loops in terms of base shear vs. the top displacement.

Comparison of the absorbed energy in the two tests confirmed that the behaviour in the down-aisle direction is more dissipative, thanks to the plastic deformation of the beam-to-upright as well as of the base plate connections. In the cross-aisle direction, on the contrary, dissipation is practically due only to the plastic deformation of the bracing system of the vertical (transversal) frames, while most other components remain in the elastic range.

4.5 CONCLUSIONS

The specimen under pushover test in down-aisle direction showed a progressive loss of stiffness associated to accumulation of plastic deformation in the column-base connections and to the large inter-storey drift of the first level. Inter-storey drifts of the upper level are much smaller than the first level one; this is characteristic of a soft-floor type of collapse mechanism, that may lead to global instability due to second-order effects.

In order to reduce this type of problem, the deformability of the column-base connections should be reduced and, somehow limited. Eventually, adoption of a beam at the ground level might be considered. Due to the bracing systems of the uprights, the specimens show a larger stiffness in cross-aisle direction that in the down-aisle one. Such bracing system is the most stressed structural component, and its failure leads to global collapse, accompanied by flexural-torsional buckling of the columns, consequent to the increment of the buckling length of the profile due to failure of the bolted connections with the lattice members.

For this reason, the solution with all diagonals inclined in the same direction should be reconsidered when the structure has to be erected in a seismic zone.

Difference between the rotations of the uprights is due to deformation of the bracing system of the transverse frames, as well as to the different behaviour of the base-plate connections and of the beam-to-upright connections.

When loaded by horizontal loads applied transverse to the beam, the connections on one side can transfer the loads by means of both portions of end-plate in bearing against the upright. On the contrary, the connections on the other side can transfer load only by means of the safety bolt in shear as well as of...
the hooks in tension; furthermore, bending of the beam in the horizontal plane induces bending in the end-plate. This different behaviour of the connections contributes to the difference in the global response of the uprights of the two sides.

An evaluation of the behaviour factor has been carried out, with two possible definitions of the $q$-factor for both down-aisle and cross-aisle directions.

One value can be identified based on ductility considerations as the ratio of the displacement $v_{\text{max}}$ corresponding to the maximum load carrying capacity of the structure to the yield displacement ($v_y$), being $q_{\mu_{\text{max}}} = \frac{v_{\text{max}}}{v_y} = 3.7$ for the down aisle direction and $q_{\mu_{\text{max}}} = 2.4$ for the cross aisle direction.

With reference to the ductility factor theory, a value of the $q$-factor based on strength was also defined as the ratio of the ideal strength $F_{\text{max,el}}$ (corresponding to $v_{\text{max}}$ and evaluated on the basis of the initial elastic stiffness) to the maximum load carrying capacity $F_{\text{max}}$, being $q_{f_{\text{max}}} = \frac{F_{\text{max,el}}}{F_{\text{max}}} = 3.1$ for the down aisle direction and $q_{f_{\text{max}}} = 2.1$ for the cross aisle direction.
5 PSEUDODYNAMIC TESTS

5.1 TEST SET-UP

The structure tested under pseudo-dynamic conditions was similar to the one adopted for the push-over test. Only tests in the down-aisle direction were performed. Due to the quasi-static loading conditions, no sliding was expected to occur during the test.

The accelerogram adopted for the test (shown in Figure 65) is an artificial signal, compatible with spectrum type 1, soil D of EC-8.

The tests summarized in Table 16 were carried out on the same structure, with different values of the PGA which was increased from one test to the other.

Of course, for all tests after the first one, the initial structural conditions were affected by the damage accumulated during the previous tests. This is introducing an approximation in the analysis; however, as the plastic deformations depend on the maximum deformation reached in the previous tests, the approximation was not expected to affect the structural behaviour for deformations larger than the maximum previously reached.

The structure resisted an earthquake with PGA of 1.4 g, without collapsing. Tests were then stopped, considering that further increments of PGA would have no physical meaning from an engineering point of view, and would in any case be beyond the scope of this research.
5.2 TEST RESULTS

Hereafter, only the results of the last test are presented.

Damage interested the beam-to-upright connections, where both hooks and openings in the uprights suffered localized deformations as shown in Figure 66. Figure 67, Figure 68 and Figure 69 show respectively the absolute displacements, the inter-storey drifts and the applied forces at the three levels of the specimen. The structural response did not evidence any torsional effect. The absolute displacements are much smaller than those measured at collapse during the push-over test on a similar specimen (see chapter 4). The largest displacements and inter-storey drifts were measured at the third level of the specimen; during most part of the test the second floor was characterized by the lowest inter-storey drifts.

The displacements of the first and second levels were all the time in phase. The absolute displacements of the second level were always larger or equal to those of the first level. The maximum displacements of the three levels were in-phase and increased from bottom to top.

Figure 70 shows the relative rotations and the hysteresis loops (in terms of shear force vs. rotation) at the various levels of the specimen. It can be noticed that the external joints (on both W and E sides of the specimen) experienced larger rotation than the central joints. Furthermore, the relative beam-to-upright rotations (both positive and negative) of the third level were the largest, while those of the first level are the smallest (Figure 71).

As shown in Figure 72, the largest energy amount was absorbed at the first level, and the minimum at the second level.

The largest energy amount absorbed by the first level indicates the largest plastic deformation of the components. Plastic deformation of the base-plate connection, behaving as a semi-rigid connection, contributes to the energy absorption at the first level. The second level, evidencing the smallest relative displacements (hence the smallest plastic deformations) dissipated the smallest energy amount.
Examining Figure 72 and Figure 73, showing the energy absorbed at the different levels, it can be noticed the non-linear behaviour of the first level since the earlier cycles. Absence of a linear behaviour since the first cycle can be explained with the kinematics of the beam-to-upright connections allowing energy dissipation due to friction among the components in association with the low testing velocity.

The restoring forces at the different levels are plotted vs. the absolute displacements and vs. the inter-storey drifts of the various levels respectively in Figure 74 and Figure 75. The restoring force, developed by the structure at the first level, was the largest because it was influenced by the stiffness of the base-plate connection, and was associated to the smallest displacements.
Figure 70  Relative beam-to-upright rotation and shear vs. rotation hysteresis loops at the various levels

Figure 71  Comparison of shear vs. rotation hysteresis loops at the various levels
The total energy input can be evaluated as the sum of the absorbed and of the kinetic energy. Figure 76 shows that the kinetic energy of the system was very low, with a peak of 5660 J, around 10
sec., corresponding to a peak in the input accelerogram. The total energy input strongly depends on the input accelerogram: increasing the acceleration results in an increment of the total energy input. Figure 77 compares the total energy input with the structural response. It can be noticed that there is a perfect balance between input and dissipated energy.

![Figure 76 System energy input.](image)

![Figure 77 System energy balance](image)

In Figure 78 the first three eigen-frequencies are plotted vs. the test duration and compared with the displacements at the various levels plotted vs. the test duration. It can be noticed that:

- In the time-interval 0-11.5 sec. the eigen-frequencies progressively reduced, while the amplitude of the oscillations at the various levels increased.
- In the time-interval 11.5-20 sec. The eigen-frequencies remained practically constant, while the amplitude of the oscillations never exceeded the maximum values previously attained.

Increasing the amplitude of the oscillations resulted in plastic deformations and in a reduction of the
specimen stiffness, hence in a reduction of the eigen-frequencies. When the oscillations do not exceed
the maximum value previously attained, the stiffness of the specimen remains constant as well as its
eigen-frequencies.

\[
\begin{align*}
y &= -0.0217x + 0.6075 \\
y &= 0.0053x + 0.317 \\
y &= -0.057x + 4.4579 \\
y &= 0.0167x + 3.6863 \\
y &= -0.0419x + 2.1782 \\
y &= 0.0072x + 1.6729
\end{align*}
\]

![Figure 78 Trend of eigen-frequencies and of displacements at various levels.](image)

Additional information about the inelastic behaviour of the specimen can be obtained by examining
Figure 79, that shows the damping factor for the first three eigen-modes as well as the displacements at
the various levels of the specimen plotted vs. the test duration.

Since the viscous damping had been disregarded in the test (i.e. all coefficients of viscous damping
matrix in the equation of motion were equal to 0), only the hysteretic and friction damping
contributions are considered.

Observing Figure 79 it can be noticed that the damping factor is strongly influenced by the amplitude of the oscillations of the various levels. In fact, the hysteretic damping contribution increases with the amplitude of the oscillations. This effect is particularly evident by observing the displacements of the third level.

Figure 80 shows typical deformed shapes of the structure during the test. The specimen responded mainly according to its second fundamental mode. This fact can be explained by considering the predominant period \( T_p = 0.52 \text{ sec} \) and the mean period \( T_m = 0.40 \text{ sec} \) of the input accelerogram; these values correspond respectively to frequencies of 1.92 Hz and 2.5 Hz, that are close to the second eigen-frequency of the (undamaged) specimen, equal to 2.44 Hz.

### 5.3 CONCLUSIONS

The results of the pseudo-dynamic test on the rack specimen under down-aisle seismic loading previously presented are fully compatible with those obtained on similar specimens, tested under dynamic conditions on the shaking table of the Laboratory for Earthquake Engineering of the national technical University of Athens (that will be presented in Chapter 6).

Under pseudo-dynamic conditions, in fact, the specimen could sustain the series of earthquake events summarised in Table 16 although it did not collapse during the last test, performed with PGA = 1.4 g (ePGA=1.5g); under dynamic conditions, it will be shown that specimen A1 collapsed under an earthquake with a PGA=1.46g (ePGA=1.41g).

Probably, the strain rate effect plays some role in this type of structure. The small movements allowed to the hooks in the holes, in fact, under dynamic conditions result in local impacts that, under increasing number of cycles, may cause cracking either in the hooks or at the edges of the holes.

In any case, comparing the deterioration of the second eigen-frequency of the specimen tested under pseudo-dynamic conditions with the similar one tested in Athens on the shaking table (Table 17 and Figure 81) it can be noticed that the trend of reduction of the second eigen-frequency (the most excited one) is similar.

<table>
<thead>
<tr>
<th></th>
<th>f(Hz) Initial</th>
<th>f(Hz) Final</th>
<th>Δ%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic</td>
<td>2.34</td>
<td>1.76</td>
<td>-24.79</td>
</tr>
<tr>
<td>Pseudo-dynamic</td>
<td>2.39</td>
<td>1.79</td>
<td>-25</td>
</tr>
</tbody>
</table>

**Table 17** Variation of the second eigen-frequency of specimens tested under dynamic and pseudo-dynamic conditions
This means that, in general, damage accumulated in the specimen during the two different types of test was similar.

Hence it can be concluded that, from the point of view of the assessment of the seismic resistance and of the damage accumulation of the pallet racking systems, pseudo-dynamic tests and shaking table tests are fully compatible, although local damage due to local dynamic effects cannot be reproduced by the pseudo-dynamic testing methodology.

Of course, due to the intrinsic quasi-static nature of the pseudo-dynamic testing procedure, no information can be derived about the effects caused by the sliding of the pallets on the beams during a seismic event.

The values of acceleration that were reached during the pseudo-dynamic tests largely exceeded the upper bound of the pallet sliding acceleration, that has been derived in Chapter 3.

By using the pseudo-dynamic method it was possible to explore the mechanical properties of the racking system beyond the sliding threshold. On the other hand, only full scale dynamic testing can fully account for the dynamic effects due to the pallets.
6 DYNAMIC FULL-SCALE TESTS

6.1 OVERVIEW
Shake-table testing on full-scale storage rack systems loaded with real merchandise represents the most direct experimental procedure to assess their seismic behaviour. Unfortunately, this type of testing is expensive compared to other testing procedures and only a very limited number of shake-table studies on storage racks have been performed to date in the world and are available in the literature. The SEISRACKS Project, whose dynamic tests main results will be summarized in this chapter, was conceived in order to try to give some information to the seismic response of storage racks and merchandise contents installed in storage racks.

6.1.1 Previous studies
The current engineering knowledge-base concerning the earthquake safety and vulnerability of storage racks is 20 to 30 years old and is limited to contents and racks unlike many modern real applications. The first published in-situ dynamic investigation of storage racks was performed in the mid 1970s (John A. Blume & Associates 1973, Krawinkler et al. 1979). The first published shake-table studies on storage racks in the United States, was performed in the late seventies on the shake-table at the University of California, Berkeley (Chen et al. 1980a, 1980b, 1981). This study provided the background information for the seismic design provisions for storage racks in the U.S. More recently, studies were performed by Filiatrault et al. (2001, 2004, 2006).

The only European research project on full scale steel pallet racks under dynamic conditions has been carried out with the financial support of E.U. (within the ECOLEADER Research Program for Free Access to Large Scale Testing Facilities), of Section X of the European Federation of Maintenance (F.E.M.) (now European Racking Federation), and of the Italian Ministry of University and Research (M.I.U.R) in 2002 (Castiglioni et al).

6.2 DYNAMIC TESTS ON MERCHANDISE
Within the SEISRACKS Research Project, a number of tests were carried out on three different types of palletised goods. The tests were performed using the shaking table facility of LEE/NTUA. For the definition of their frequency and damping ratio, a sine logarithmic sweep signal was applied on three types of “palletised” goods: Good Type 1 contains baby diapers, good Type 2 contains boxes with powder clothes detergent and good Type 3 contains boxes with liquid clothes detergent. These palletized goods, were lent, free-of-charge, to LEE/NTUA by Procter&Gamble.

The frequency range of sine logarithmic test was 1-32 Hz at an increment rate of one octave per minute. The tests were executed along each one of the global axes X and Y separately with a constant acceleration of 0.50 m/sec². Since sine sweep rate is one octave per minute and start frequency is 1 Hz, the frequency is given by the following expression:

\[ f(Hz) = 1.0 \times 2^{\frac{\text{Time(sec)}}{60}} \]

The pallets were fixed directly on the shaking table.

The fundamental natural frequencies and the damping of each type of “palletised” good are shown in Table 18. These frequencies were determined from acceleration time history of the sine logarithmic sweep response of each type of good. Damping ratio was calculated by using the half power bandwidth method.

Examining the results presented in Table 18, it can be stated that the dynamic characteristics of the “palletized” goods are depending on the nature of goods and the way of packing. Further research is needed in this field for an exhaustive assessment of the problem.
6.3 SHAKE-TABLE TESTS ON FULL-SCALE PALLET-TYPE STEEL STORAGE RACKS

6.3.1 Test infrastructure
The tests were performed at the Laboratory of Earthquake Engineering at National Technical University of Athens where a shaking simulator is installed. The simulator consists of a rigid platform with dimension 4.0 m x 4.0m with 6 degrees of freedom and of a system controlling the input motion and the response of the specimen tested on the platform. The facility has a full active control along the 6 degrees of freedom. The maximum acceleration to each horizontal direction (X, Y) is 2.0 g, while the maximum acceleration to vertical direction (Z) is 4.0 g.

6.3.2 Specimens
Shake-table tests were carried out on six full-scale rack models, produced by the same manufacturer, affiliated to ACAI, who provided the specimens free-of-charge.

All the rack structural models tested on the shake-table consisted in two bays of 1800 mm each, three levels, for a total height of the structure about 6000 mm, width of the upright frames 1100 mm, and were positioned on the table and loaded as shown in Figure 82. Neither vertical (down-aisle) nor horizontal (in-plane) bracing systems are present in the specimens that were mainly designed for static loading conditions, without any specific requirement for earthquake loading. Live loads consisting of concrete blocks (7.7 kN each) were positioned on pallets, two pallets per bay. Hence, at each level, a live load of 30.8 kN was present (15.4 kN for pair of beams), for a total live load of 92.4 kN. At the base, the specimens were bolted and welded on stiff steel plates, which were directly fixed on the shaking table.

Instrumentation varied from specimen to specimen, according to the direction of the excitation and the type of test. The characteristics of each specimen are summarized in Table 19. In addition, however, a safety frame was provided capable to sustain both masses and structure in case of collapse or fall of the pallets from the rack as shown in Figure 83. In this way it was possible to perform tests up to collapse, without risk of damaging the testing facility.

6.3.3 Testing procedure
The test procedure was consisted from two different types of tests: random vibration test and earthquake tests.

6.3.3.1 Random vibration test
The specimen was excited by a random acceleration signal along Y or X axes. The frequency range of random test was from 0.5Hz to 50Hz. The amplitude of vibration along both X and Y direction was 0.04g (Figure 84 a). These tests were performed, as first test for each specimen, in order to determine its natural frequencies and damping ratios.
6.3.3.2 Earthquake Tests
A series of uniaxial earthquake tests along Y or X direction was performed for each specimen. The time history was an artificial time history which was generated to match the elastic response spectrum Type 1 of EC8 with peak ground acceleration 0.35g, subsoil glass D and damping 2% (Figure 84 b). In order to adjust to the available displacement capacity of the shaking table, the artificial accelerograms were filtered with a high pass filter 1Hz. Several earthquake tests were performed for each specimen with the acceleration of shaking table, along the tested axis, being increased step-wise.

6.3.3.3 Specimen A1
Specimen A1 is made with beams type 130x45x1.5 mm and uprights type 100/20b. Pallets have been rigidly connected to the steel beams, in order to avoid any possible sliding. Specimen A1 was submitted along the down-aisle direction (the Y main direction of shake table) to the test history summarized in Table 20, where for each test both the Peak Ground Acceleration (PGA) as well as the Effective Design Acceleration (EDA) are reported. Totally 14 tests were carried out.
Figure 84: Acceleration time histories adopted for the dynamic tests: a) random vibration tests, b) earthquake tests: shake table input time-history and elastic response spectrum of input signal.

<table>
<thead>
<tr>
<th>Test No</th>
<th>Test type</th>
<th>Direction</th>
<th>PGA[g]</th>
<th>EDA[g]</th>
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<td>Random test</td>
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<td></td>
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<td>2</td>
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<td>Y</td>
<td>1.31</td>
<td>1.175</td>
</tr>
<tr>
<td>14</td>
<td>Earthquake</td>
<td>Y</td>
<td>1.46</td>
<td>1.313</td>
</tr>
</tbody>
</table>

Table 20: Tests performed on specimen A1.

6.3.3.4 Specimen A2
Specimen A2 is similar to specimen A1, but the pallets are free to slide on the steel beams, and it was submitted along the down-aisle direction to the test history summarized in Table 21. Totally 18 tests were carried out. After the seismic tests, a random test was executed in order to check any variation of the dynamic characteristics.

6.3.3.5 Specimen A3
Specimen A3 is similar to specimen A2, but with beams type 70X45X1.5 mm. This reduced beam size was chosen with the aim of testing a specimen with beams working at stress ratios similar to those usual in design practice. The stress ratio is 67% of the one usually assumed in design practice for the 130x45x1.5 mm beams. The pallets are free to slide on the steel beams. The specimen was submitted, along the down-aisle direction to the test history summarized in Table 22. A total of 13 tests were carried out.
### Table 21: Tests performed on specimen A2.

<table>
<thead>
<tr>
<th>Test No</th>
<th>Test type</th>
<th>Direction</th>
<th>PGA[g]</th>
<th>EDA[g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Random test</td>
<td>Y</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.05</td>
<td>0.043</td>
</tr>
<tr>
<td>3</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.09</td>
<td>0.082</td>
</tr>
<tr>
<td>4</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.15</td>
<td>0.121</td>
</tr>
<tr>
<td>5</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.20</td>
<td>0.167</td>
</tr>
<tr>
<td>6</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.25</td>
<td>0.207</td>
</tr>
<tr>
<td>7</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.30</td>
<td>0.250</td>
</tr>
<tr>
<td>8</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.35</td>
<td>0.289</td>
</tr>
<tr>
<td>9</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.40</td>
<td>0.330</td>
</tr>
<tr>
<td>10</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.43</td>
<td>0.370</td>
</tr>
<tr>
<td>11</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.47</td>
<td>0.411</td>
</tr>
<tr>
<td>12</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.46</td>
<td>0.411</td>
</tr>
<tr>
<td>13</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.54</td>
<td>0.516</td>
</tr>
<tr>
<td>14</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.63</td>
<td>0.600</td>
</tr>
<tr>
<td>15</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.75</td>
<td>0.668</td>
</tr>
<tr>
<td>16</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.86</td>
<td>0.778</td>
</tr>
<tr>
<td>17</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.92</td>
<td>0.881</td>
</tr>
<tr>
<td>18</td>
<td>Random test</td>
<td>Y</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 22: Tests performed on specimen A3.

<table>
<thead>
<tr>
<th>Test No</th>
<th>Test type</th>
<th>Direction</th>
<th>PGA[g]</th>
<th>EDA[g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Random test</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Earthquake</td>
<td>X</td>
<td>0.12</td>
<td>0.089</td>
</tr>
<tr>
<td>3</td>
<td>Earthquake</td>
<td>X</td>
<td>0.22</td>
<td>0.176</td>
</tr>
<tr>
<td>4</td>
<td>Earthquake</td>
<td>X</td>
<td>0.33</td>
<td>0.264</td>
</tr>
<tr>
<td>5</td>
<td>Earthquake</td>
<td>X</td>
<td>0.45</td>
<td>0.350</td>
</tr>
<tr>
<td>6</td>
<td>Earthquake</td>
<td>X</td>
<td>0.58</td>
<td>0.440</td>
</tr>
<tr>
<td>7</td>
<td>Earthquake</td>
<td>X</td>
<td>0.67</td>
<td>0.533</td>
</tr>
<tr>
<td>8</td>
<td>Earthquake</td>
<td>X</td>
<td>0.85</td>
<td>0.627</td>
</tr>
<tr>
<td>9</td>
<td>Earthquake</td>
<td>X</td>
<td>0.95</td>
<td>0.717</td>
</tr>
<tr>
<td>10</td>
<td>Earthquake</td>
<td>X</td>
<td>1.08</td>
<td>0.806</td>
</tr>
<tr>
<td>11</td>
<td>Earthquake</td>
<td>X</td>
<td>1.24</td>
<td>0.894</td>
</tr>
</tbody>
</table>

### Table 23: Tests performed on specimen A4.

<table>
<thead>
<tr>
<th>Test No</th>
<th>Test type</th>
<th>Direction</th>
<th>PGA[g]</th>
<th>EDA[g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Random test</td>
<td>X</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 6.3.3.6 Specimen A4

Specimen A4 is similar to specimen A2 with the pallets free to slide and was submitted, along the cross-aisle direction, to the test history summarized in Table 23. Totally 11 tests were carried out.

### Table 23: Tests performed on specimen A4.

<table>
<thead>
<tr>
<th>Test No</th>
<th>Test type</th>
<th>Direction</th>
<th>PGA[g]</th>
<th>EDA[g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Random test</td>
<td>X</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 6.3.3.7 Specimen A5

Specimen A5 is similar to specimen A2, with the pallets are free to slide on the steel beams, but each upright base is mounted on an isolation system type1, as shown in Figure 85 a
As this type of isolator allows independent in-plane movement of each base of the uprights, it was specified by the designer to introduce beams at the base level, as well as an in-plane bracing system at the same level in order to avoid relative movements of the column bases. The specimen was submitted, along the down-aisle direction, to the test history summarized in Table 24. Totally 17 tests were carried out.

<table>
<thead>
<tr>
<th>Test No</th>
<th>Test type</th>
<th>Direction</th>
<th>PGA[g]</th>
<th>EDA[g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Random test</td>
<td>Y</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.05</td>
<td>0.042</td>
</tr>
<tr>
<td>3</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.09</td>
<td>0.083</td>
</tr>
<tr>
<td>4</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.19</td>
<td>0.172</td>
</tr>
<tr>
<td>5</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.30</td>
<td>0.263</td>
</tr>
<tr>
<td>6</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.40</td>
<td>0.352</td>
</tr>
<tr>
<td>7</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.51</td>
<td>0.441</td>
</tr>
<tr>
<td>8</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.62</td>
<td>0.534</td>
</tr>
<tr>
<td>9</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.74</td>
<td>0.629</td>
</tr>
<tr>
<td>10</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.86</td>
<td>0.722</td>
</tr>
<tr>
<td>11</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.99</td>
<td>0.812</td>
</tr>
<tr>
<td>12</td>
<td>Earthquake</td>
<td>Y</td>
<td>1.06</td>
<td>0.895</td>
</tr>
<tr>
<td>13</td>
<td>Earthquake</td>
<td>Y</td>
<td>1.12</td>
<td>0.991</td>
</tr>
<tr>
<td>14</td>
<td>Earthquake</td>
<td>Y</td>
<td>1.15</td>
<td>1.097</td>
</tr>
<tr>
<td>15</td>
<td>Earthquake</td>
<td>Y</td>
<td>1.28</td>
<td>1.211</td>
</tr>
<tr>
<td>16</td>
<td>Earthquake</td>
<td>Y</td>
<td>1.37</td>
<td>1.284</td>
</tr>
<tr>
<td>17</td>
<td>Random test</td>
<td>Y</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 24: Tests performed on specimen A5.

6.3.3.8 Specimen A6
Specimen A6 Specimen A5 is similar to specimen A, where the pallets are free to slide on the steel beams, but is mounted on a base isolation system type 2, as shown in Figure 85. Also this type of base isolation system allows in-plane movement, but the device comes already with a system of beams and braces, at the base level, so that the specimen could be mounted without any additional “restraining” member. The specimen was submitted, along the down-aisle direction to the test history summarized in Table 25. In total 14 tests were carried out.

6.4 TEST RESULTS
A summary of the main results obtained from the experimental campaign is presented with the aim of allowing understanding of the actual dynamic behaviour of steel pallet racks under dynamic conditions.

6.4.1 Specimen eigen-frequencies
The natural frequencies were directly measured from the peak values of the transfer functions between the base acceleration and response acceleration of each specimen at the top level. The results obtained
for the six specimens are summarized in Table 26.

<table>
<thead>
<tr>
<th>Test No</th>
<th>Test type</th>
<th>Direction</th>
<th>PGA[g]</th>
<th>EDA[g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Random test</td>
<td>Y</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.05</td>
<td>0.042</td>
</tr>
<tr>
<td>3</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.10</td>
<td>0.083</td>
</tr>
<tr>
<td>4</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.21</td>
<td>0.173</td>
</tr>
<tr>
<td>5</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.30</td>
<td>0.263</td>
</tr>
<tr>
<td>6</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.41</td>
<td>0.361</td>
</tr>
<tr>
<td>7</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.54</td>
<td>0.454</td>
</tr>
<tr>
<td>8</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.64</td>
<td>0.533</td>
</tr>
<tr>
<td>9</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.75</td>
<td>0.641</td>
</tr>
<tr>
<td>10</td>
<td>Earthquake</td>
<td>Y</td>
<td>0.88</td>
<td>0.732</td>
</tr>
<tr>
<td>11</td>
<td>Earthquake</td>
<td>Y</td>
<td>1.00</td>
<td>0.832</td>
</tr>
<tr>
<td>12</td>
<td>Earthquake</td>
<td>Y</td>
<td>1.11</td>
<td>0.942</td>
</tr>
<tr>
<td>13</td>
<td>Earthquake</td>
<td>Y</td>
<td>1.26</td>
<td>1.074</td>
</tr>
<tr>
<td>14</td>
<td>Earthquake</td>
<td>Y</td>
<td>1.42</td>
<td>1.200</td>
</tr>
</tbody>
</table>

Table 25: Tests performed on specimen A4.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Direction</th>
<th>Frequency [Hz]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>Longitudinal</td>
<td>0.702</td>
</tr>
<tr>
<td>A2</td>
<td>Longitudinal</td>
<td>0.696</td>
</tr>
<tr>
<td>A3</td>
<td>Longitudinal</td>
<td>0.513</td>
</tr>
<tr>
<td>A4</td>
<td>Transversal</td>
<td>1.720</td>
</tr>
<tr>
<td>A5</td>
<td>Longitudinal</td>
<td>0.684</td>
</tr>
<tr>
<td>A6</td>
<td>Longitudinal</td>
<td>0.630</td>
</tr>
</tbody>
</table>

Table 26: Natural frequencies of the tested specimens

Examining the transfer functions, it can be noticed that the response of specimens A1 and A2 is very similar, although specimen A1 is slightly more rigid, most probably due to the fixed masses. Specimen A3 is more flexible, and the first eigenfrequency is shorter than those of the previous two specimens. A number of possible eigenfrequencies can be identified between the two main (longitudinal) ones, for all three specimens. These eigenfrequencies are associated to transversal and torsional vibration modes.

### 6.4.2 Down-aisle tests

#### 6.4.2.1 Specimens without base isolation

Figure 86 show the maximum deformed shapes assumed by the non base-isolated specimens (namely A1, A2 and A3) tested in down aisle direction during the test series at increasing maximum acceleration of the input motion.

It can be observed that, in down-aisle (longitudinal) direction, the three specimens respond mainly according to the second mode. First mode type of response is evident only for specimen A1 up to test 3 (performed with PGA of 0.21 g) and for specimen A2 up to test 5 (characterized by a PGA of 0.20 g). The response of specimen A3, that is more flexible because of the smaller beams (70x45x1.5 mm type), is according to the second mode already from test 3 (performed with PGA of 0.21 g).

Figure 87 present, for specimens A1, A2 and A3: a) the trend of the maximum values of the inter-storey drift at the three levels (indicated as $\Delta$ I-T for the I level, $\Delta$ II-I for the second and $\Delta$ III-II for the third) as well as b) the trend of the maximum absolute displacements of the same levels together with those of the shaking table, measured during the test sequence. It is evident that all the three specimens show a similar dynamic behaviour. Inter-storey drifts of the first level are always opposite in sign (i.e. in counter-phase) and smaller than those of the upper levels. Specimen A3 is more flexible than specimens A1 and A2.

The trend of accumulation of the residual displacements measured at the various levels of the specimens at the end of each test is also plotted vs. the sustained PGA. In all three specimens the largest residual displacements occurred at the upper (third) level, and they decrease toward the bottom. No significant
transversal residual displacement could be observed.

Figure 86: Maximum deformed shapes of specimen A1, A2 and A3, during various tests.

Figure 87: Trend of the maximum inter-storey drifts and of the absolute displacements at the various levels of specimen A1, A2 and A3, during the test sequence.

6.4.2.2 Specimens with base isolation
Presence of a base isolation system strongly influences the structural response. Both the two proprietary systems adopted in this testing campaign, behave as “sliders”, allowing movement of the column base in the plane. Hence, under seismic input motion, the structure responds, at least for small base displacements, as a rigid body. Under strong input motions, the stiffness of the elastomeric “spring”
(that increases non-linearly with the displacements), restrains the structure. This effect, combined with the inertial forces on the masses, results in deformation of the structure that, in any case, responds according to the first mode. This behaviour is shown in Figure 88.

Effects of the presence of base isolation systems on the structural response are well evident in Figure 89 reporting the trend of the maximum inter-storey drifts and of the absolute displacements at the various levels of the specimens, during the test sequence. The three levels respond in phase, and their absolute displacements are always smaller than those of the shake table (i.e. of the input motion). A comparison with the corresponding values presented for non base-isolated specimens shows that both inter-storey drifts and absolute displacements of specimens A5 and A6 are much smaller than those measured on the specimens without base-isolation, under corresponding values of PGA.

Figure 90 shows the trend of accumulation of the residual displacements at the various levels of specimen A5, during the test sequence at increasing PGA. Residual displacements, smaller than 5 mm, are evident confirming that the rack is behaving as a rigid body. The accumulation of residual displacements, in this case, is not due to permanent plastic deformations accumulating in the structure, but to “friction” problems in the base-isolator, that do not allow a complete and perfect “re-centering” of the structure after the earthquake.

It should also be noticed that, in down aisle direction, specimen A5 sustained a total of 16 earthquake tests (plus two random tests), the strongest with a PGA of 1.37 g. Similarly, specimen A6 sustained a total of 13 earthquake tests (plus one random test), the strongest with a PGA of 1.42 g. Under such high accelerations, the base-isolators nearly reached their ultimate working conditions. Hence, it was considered useless to further increase the base acceleration, and testing was terminated, with both specimens practically undamaged.
6.4.3 Cross-aisle tests
Specimen A4, was submitted in cross-aisle direction. Figure 91 a and b shows the maximum deformed shapes assumed by the central upright frames during the test series at increasing maximum acceleration of the input motion and the trend of accumulation of the residual displacements at various levels during test sequence.
Accumulation of the residual displacements at the various levels of specimen A4, during the test sequence at increasing PGA, shown in Figure 91 b confirms that plastic deformation at the third level occurs already under values of PGA of the order of magnitude of 0.2-0.3g. Beyond values of the PGA of 0.6g, the central upright frame accumulates large residual displacements at all three levels, increase when passing from the first to the third level, the amplitude of these residual displacements, as well as their trend of increment with the PGA, increase.

6.4.4 Failure modes
In the following some images and comments will be presented, related to the failure modes of the three specimens (A1, A3 and A4) that were tested up to complete failure. In fact, as already said, in the case of specimens A5 and A6, base-isolated, testing was ended due to attainment of the ultimate design conditions of the base-isolators, when the structures were still in good conditions, and practically undamaged. In the case of specimen A2, testing was ended after an earthquake simulation test with a PGA of 0.92g, when the structure was starting to accumulate plastic deformations in the down aisle direction, but was not completely collapsed.

6.4.4.1 Specimen A1
Figure 92 shows the deformed shape of specimen A1 after collapse occurred during test 14. It can be noticed that the structure is hanging on the safety frame. Evident plastic deformations occurred at the column bases.

6.4.4.2 Specimen A3
Figure 93 shows the deformed shape of specimen A3 after collapse occurred during test 13, an earthquake simulation test with PGA of 1.46g. It can be noticed that the structure is hanging on the

---

**Figure 90**: Trend of accumulation of the residual displacements at the various levels of specimen A5, during the test sequence at increasing PGA.

**Figure 91**: a) Maximum deformed shapes during various tests in cross-aisle direction at central upright frame. b): Trend of accumulation of the residual displacements at the various levels, during the test sequence at increasing PGA.
safety frame. Also in this case, as for specimen A1, evident plastic deformations occurred at the column bases, together with fracture of the bolts connecting the upright profiles to the gusset plates welded to the base plate. Figure 94 shows also an evident damage in the beam-to-upright connection, with plastic deformation of the end-plate connector as well as local buckling in the upright profile.

Figure 92: Collapse of specimen A1, detail of the failure of the upright at the beam-to-column connection at the third level, detail of the column-base failure.

6.4.4.3 Specimen A4.
Specimen A4 was tested in cross-aisle direction. Collapse was associated with failure of the central upright frame, that is carrying horizontal forces double than the two lateral uprights. The failure mechanism is shown in Figure 94. Collapse interested the column-base connections as well as the beam-to-upright connections that suffered large plastic deformations in the horizontal plane. Failure occurred for accumulation of deformation in the direction in which diagonal members of the upright frames are in compression.

Figure 93: Collapse of specimen A3: global collapse and details of the column base and of the beam-to-column connection failure.
6.5 ASSESSMENT OF THE BEHAVIOUR FACTOR (Q FACTOR).

Eurocode-8 (2005) suggests the reduction of the design seismic forces through the behaviour factor, generally called “q-factor”. The q-factor takes implicitly into account the global and local ductility resources of the structure (which depend on the structural typology, the ductility of the material, the P-∆ effects and possible brittle fracture mechanisms) and depends on a wide number of factors related to the structure and the seismic input.

In general, the q-factor is defined as the ratio of the peak ground acceleration producing collapse of the structure $a_{\text{max}}$ to that of design $a_d$: $q = \frac{a_{\text{max}}}{a_d}$

In this definition the peak ground acceleration $a_{\text{max}}$ at collapse depends on the type of collapse mechanism that is considered.

Hereafter it is tried to assess the q-factor for steel pallet racks, based on re-analysis of the experimental shake-table test data previously presented and discussed.

Starting from the definition of the q-factor as “the ratio of the elastic structural response (in terms of accelerations) to the inelastic one”, the following procedure was identified. The procedure allows assessment of the q-factor as the ratio of the actual maximum base shear experienced by the structure, to the one that might be derived considering an indefinitely linear elastic structural behaviour.

The results obtained for specimens A1, A2, A3 and A4 are presented in Table 27, where $a_{\text{max}}$ (positive or negative) is the maximum (or minimum) acceleration corresponding to the assessment of both the actual base shear ($V$) and the elastic base shear ($V_e$).

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>$a_{\text{max}}$ [g]</th>
<th>$V_e$ [kN]</th>
<th>$V$ [kN]</th>
<th>q</th>
<th>q average</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>1.31 39.11 14.82 2.64</td>
<td>2.8</td>
<td></td>
<td>(2.77)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-1.07 -31.95 -11.08 2.89</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A2</td>
<td>0.92 62.39 17.88 3.49</td>
<td>3.7</td>
<td></td>
<td>(3.66)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.88 -59.67 -15.57 3.83</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A3</td>
<td>0.82 60.43 16.55 3.65</td>
<td>3.7</td>
<td></td>
<td>(3.68)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.61 -44.96 -12.15 3.70</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A4</td>
<td>0.78 36.77 11.40 3.23</td>
<td>2.7</td>
<td></td>
<td>(2.73)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.63 -29.70 -13.38 2.22</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 27: q-factor values of steel racks

Adopting the same procedure, an evaluation of the q-factor was attempted also for the other specimens.
presented in this study although none of them was tested up to ultimate conditions.

On the contrary, it is interesting to underline the result obtained for specimen A5. Although also this specimen could not be tested up to failure, corresponding to a PGA of 1.1g in the down-aisle direction a q-factor of 6.9 could be estimated, nearly two times larger than those obtained for similar specimens, non base-isolated.

6.6 CONCLUSIONS.
Earthquake simulation tests on six full scale rack models and same palletised merchandise tests were carried out with the use of shaking table facility of LEE/NTUA under the SEISRACKS research project. Five specimens were tested in down-aisle direction (two of which with base isolation systems) and one in cross-aisle direction. Effects such as the beam size, presence/absence of pallet sliding as well as of a base isolation system were investigated.

The importance of small structural detailing was also highlighted. Most of the observed failure was caused by failure of bolted or welded connections.

An assessment of the q-factor was also performed, based on the experimental results. q-factor values of 3.7 and 2.7 were obtained respectively for the down-aisle and the cross-aisle directions. In the case of pallets rigidly fixed on the beams (in order to prevent sliding) a q-factor of 2.8 was identified. This value, however, might be affected by the excessive stiffening of the structure, associated with the way in which the pallets were connected to the steel beams, generating a sort of “composite” beam. Positive effects of the presence of the base isolators were also highlighted. The two specimens resisted without any damage earthquakes with a PGA higher than 1.30g. Assessment of the q-factor for specimen A5 leads to a value of 6.9.
7 MONITORING SYSTEM

7.1 OVERVIEW

7.1.1 Aims and scopes of the investigation
This study is developed within the SEISRACKS project, and presents the results of the monitoring system developed by Laboratory for Earthquake Engineering of the National Technical University of Athens (LEE/NTUA). The system was developed and installed for the continuous monitoring of vibrations and earthquakes of a steel pallet rack.

7.2 MONITORING SYSTEM

7.2.1 Specimen
The selected steel pallet rack is located at Voyatzoglou Company at Inofita (close to Athens). Three bays of that rack were instrumented as shown in Figure 95.

![Figure 95 Selected steel pallet rack at Vogiatzoglou Company](image)

7.2.2 Instrumentation set-up
Totally, twenty-one accelerometers were placed on the selected rack in order to record in-plane and out-of-plane acceleration of the rack, while a triaxial accelerometer was placed on the floor (base acceleration). A camera was additionally installed in order to:
- Identify the source of the vibration (earthquake, passby of forklift, loading and unloading of the rack with pallets, other operations).
- Document the arrangement of masses on the rack when the system is triggered.

In Figure 96, the position of accelerometers is shown, while in Figure 97, accelerometers at measurement points 1 and 2 are presented. The accelerometers were made by Kistler Instrument Corporation. In Table 28 the technical specifications of accelerometers are given.
Figure 96 Instrumentation of selected rack

Figure 97 Acceleration at measurement points 1 and 2

<table>
<thead>
<tr>
<th>Specifications of accelerometers</th>
</tr>
</thead>
<tbody>
<tr>
<td>Range (g)</td>
</tr>
<tr>
<td>Max. Range</td>
</tr>
<tr>
<td>Temperature Range, operating</td>
</tr>
<tr>
<td>Output Impedance</td>
</tr>
</tbody>
</table>

Table 28

The monitoring system is completed with a data acquisition system (Figure 98) of 16bits for recording and saving the data with sampling rate 200 Hz and a DC power supply system (Figure 99).

Figure 98 Data acquisition system

Figure 99 DC power supply system

7.2.3 Difficulties during monitoring system operation

The monitoring system faced some difficulties during its operation. The personnel of the company didn’t feel comfortable with the presence of the camera. At the beginning of installation the pictures after the triggering of system were clear (Figure 100 a). After some time, boxes were placed close to the camera, which is leading to the reduction of optical angle of pictures (Figure 100 b). Due to this fact, a lot of events, more than 300, where recorded but the source of the vibration cannot be recognised. Another problem was that operations that performed after 6.00 PM cannot be identified as the illumination was not suitable; the brightness of pictures was not satisfactory (Figure 101).
7.3 ANALYSIS OF TEST DATA
Continuous data recording was carried out for more than two years, and is still ongoing. All events recorded up to now concern actions of fork lift. Up to this time, none earthquake motion have been recorded. For each event, the captured pictures and the time histories plots of recorded acceleration at all measurement points are presented. The frequencies and the corresponding damping ratio of vibration are extracted. The spatial direction of motion is also illustrated for each event. Table 29 presents a list of typical recorded events that will be presented and analysed hereafter.

<table>
<thead>
<tr>
<th>Event</th>
<th>Date</th>
<th>Time</th>
<th>Duration</th>
<th>Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10/11/2005</td>
<td>9_42</td>
<td>3 secs</td>
<td>Loading – unloading procedure</td>
</tr>
<tr>
<td>2</td>
<td>10/11/2005</td>
<td>10_3_57 to 10_3_4</td>
<td>12 secs</td>
<td>Passing of fork lift</td>
</tr>
<tr>
<td>3</td>
<td>10/11/2005</td>
<td>11_31_56 to 11_31_57</td>
<td>6 secs</td>
<td>Passing of fork lift</td>
</tr>
<tr>
<td>4</td>
<td>10/11/2005</td>
<td>14_5_49</td>
<td>3 secs</td>
<td>Loading-moving of fork lift</td>
</tr>
<tr>
<td>5</td>
<td>10/12/2005</td>
<td>13_18_16</td>
<td>3 secs</td>
<td>Operation of fork lift</td>
</tr>
<tr>
<td>6</td>
<td>10/01/2006</td>
<td>7_45_25 to 7_45_27</td>
<td>9 secs</td>
<td>Fork lift is leaving the area</td>
</tr>
</tbody>
</table>

Table 29 Discretion of analysis data

7.3.1 Recording event 10_11_2005_9_42: duration 3secs
In Figure 102, the captured picture is presented. In this picture, a loading-unloading procedure is shown. In figures from Figure 103 to Figure 108 the recorded accelerations signals at measurement points Ch1 to Ch21 are shown.

Figure 102 Recording event 10_11_2005_9_42
The maximum acceleration was observed on Ch1 and Ch2 located at the top of rack and is 0.049g.
Down aisle acceleration is lower than the cross aisle Ch1-2/Ch5=2.67. The triaxial acceleration on floor
didn’t record any vibration. High frequency vibrations are absorbed far away from the source.
The recorded data were analysed in order the frequencies and damping of vibration to be determined. In
Figure 109 and Figure 110 the spectrum corresponding to cross aisle recorded acceleration at Ch1 and
down aisle recorded acceleration at Ch5 are depicted respectively. In Table 30 the main frequencies and
corresponding damping are given. In Figure 111 the spatial direction of motion is shown.

Figure 103 Acceleration time histories at measurement points Ch1 – Ch4
Figure 104  Acceleration time histories at measurement points Ch5 – Ch8

Figure 105  Acceleration time histories at measurement points Ch9 – Ch12
Figure 106 Acceleration time histories at measurement points Ch13 – Ch16.

Figure 107 Acceleration time histories at measurement points Ch17 – Ch20
**Figure 108** Acceleration time history at measurement point Ch21.

**Figure 109** Spectrum of Ch1

**Figure 110** Spectrum of Ch5

**Figure 111** Spatial direction of motion.

<table>
<thead>
<tr>
<th></th>
<th>Ch1- Cross aisle</th>
<th>Ch5- Down aisle</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Frequency</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Hz)</td>
<td>22.00</td>
<td>22.00</td>
</tr>
<tr>
<td><strong>Damping</strong></td>
<td>0.80</td>
<td>0.90</td>
</tr>
<tr>
<td>(%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>23.33</td>
<td>1.70</td>
<td>23.33</td>
</tr>
<tr>
<td>25.00</td>
<td>3.60</td>
<td>25.00</td>
</tr>
<tr>
<td><strong>Damping</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(%)</td>
<td>0.90</td>
<td>1.90</td>
</tr>
</tbody>
</table>

**Table 30** Frequencies and damping of vibration

### 7.4 CONCLUSIONS

The main conclusions derived from the analysis of recorded data which are concerns operations of forklift are summarized as follows:

1. High frequency vibration is occurred from operations of forklift.
2. The signal is absorbed far away from the source of vibration.
3. The maximum acceleration was recorded on the top of the rack (ch1 to ch4).
4. The down aisle acceleration was lower than the cross aisle.
5. The triaxial acceleration on floor didn’t record any vibration.
In Table 31 the maximum recorded cross aisle and down aisle acceleration for each event are depicted.

<table>
<thead>
<tr>
<th>Event</th>
<th>Action</th>
<th>Acceleration (g)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Cross Aisle</td>
<td>Down Aisle</td>
</tr>
<tr>
<td>1</td>
<td>Loading – unloading procedure</td>
<td>0.049</td>
<td>0.018</td>
</tr>
<tr>
<td>2</td>
<td>Passing of fork lift</td>
<td>0.083</td>
<td>0.021</td>
</tr>
<tr>
<td>3</td>
<td>Passing of fork lift</td>
<td>0.059</td>
<td>0.016</td>
</tr>
<tr>
<td>4</td>
<td>Loading-moving of fork lift</td>
<td>0.046</td>
<td>0.016</td>
</tr>
<tr>
<td>5</td>
<td>Operation of fork lift</td>
<td>0.071</td>
<td>0.032</td>
</tr>
<tr>
<td>6</td>
<td>Fork lift is leaving the area</td>
<td>0.056</td>
<td>0.023</td>
</tr>
</tbody>
</table>

**Table 31** Maximum absolute cross aisle and down aisle acceleration
8 STRUCTURAL MODELLING AND NORMATIVE ASSESSMENT

8.1 NUMERICAL MODELLING OF RACKS – BASIC CONSIDERATIONS

8.1.1 Specific aspects of the numerical modelling of racks
As it has been evidenced in the previous chapters presenting the experimental results obtained on storage racks subjected to seismic action, rack structures are very peculiar structures. Some of these peculiarities can be emphasised in the perspective of numerical modelling:
- Racks are very flexible and are therefore very sensitive to second order geometrical effects;
- The ratio between live and dead loads is very high;
- The shape of the hysteretic loops is rather unusual referred to classical steel structures, due to the specific types of devices used for connections;
- The flexural stiffness of column-bases is significantly influenced by the compression level;
- The pallets are likely to move with respect to their support.
Effectiven numerical simulations should therefore include all these features.

8.1.2 Choice of the numerical tool for the study
The software tool used for simulation purpose within the Seisracks project is the finite element program FINELG, developed in collaboration by the University of Liège (Department ArGEnCo) and the Design office Greisch. The computer program FineLg is a finite element program to solve geometrically and materially non linear solid or structural problems under static or dynamic loading. The software presents abilities that are of prime importance in the Seisracks context:
- Computation of eigenfrequencies and eigenmodes (buckling and vibration);
- Step-by-step dynamic analysis;
- Nonlinear geometrical effects (member and global buckling, second order effects);
- Dynamic response of structures subjected to spectrum compatible artificially generated accelerograms or to user-defined accelerograms;
- Nonlinear springs (for joints);
- Efficient 3D beam elements;
- Powerful graphical outputs.

During the project, two main lacks have been identified. The first one is related to the available constitutive laws for the connection springs. A wide series of constitutive laws are defined in the FineLg library (from simple linear law to sophisticated concrete law including damage), but all of them are essentially dedicated to monotonic loading. This was obviously too limited to allow for a good representation of the energy dissipation in the joints, according to the complicated hysteretic behaviour described previously in the report. Therefore two new models have been developed in the frame of the project and are presented in section 8.1.3.1.

The second problem was the impossibility to account for the sliding of pallets. A "sliding mass model" has thus been developed in the software. Its development and validation is presented in section 8.1.3.2.

Finally, the project also allowed identifying other possible developments that could be very useful for future refined numerical investigations of rack structures. These "open doors" are summarized in section 8.1.3.3.

8.1.3 New developments in FineLg

8.1.3.1 Hysteretic spring
Two different hysteretic spring models have been implemented in FineLg in order to represent the joint behaviour. The first one is a Takeda model (see Figure 112), usually dedicated to the modelling of plastic hinges in concrete structures, but whose shape is very similar to what has been observed in the component tests. The model is defined by 5 parameters:
- Initial stiffness \( K_0 \);
- Yield limit \( F_y \);
- Post-yield ratio \( r = K/K_0 \);
- Unloading stiffness parameter \( \alpha \), such as \( K_u = K_0(dy/dm)^\alpha \), \( dy \) being the elastic displacement and \( dm \) the maximum displacement;
- Reloading parameter $\beta$, defining the position of the connection point for the reloading phase. The abscissa of this point is given by $(dm – \beta dp)$, $dp$ being the total amount of plastic deformation.

![Diagram](image)

**Figure 112** Two examples of Takeda hysteretic models

The second model is a hysteretic model with pinching, such as depicted on Figure 113. In this model, the governing parameters are:
- Initial stiffness $K_0$;
- Yield limit $F_y$;
- Post-yield ratio $r = K_t/K_0$;
- Unloading stiffness parameter $\alpha$, such as $K_u = K_0(dy/dm)^\alpha$, $dy$ being the elastic displacement and $dm$ the maximum displacement;
- Reloading parameter $\beta$, defining the position of the connection point for the reloading phase. The abscissa of this point is given by $(\beta dy)$.

![Diagram](image)

**Figure 113** Pinched model

Since the behaviour of column-base and beam-to-column connections are likely to be unsymmetrical (in particular for column-bases in cross-aisle direction and for beam-to-column connections), hysteretic models are actually defined by 10 parameters. Each of the five parameters can indeed be defined separately for positive and negative behaviour.

The models have been compared to component test results. Figure 114 compares both models and test results for one particular case of beam-to-column connection. In practice, one or the other model can be seen as more accurate according to the type of connection. The best indicator for the choice of the right model should be the estimate of the total amount of dissipated energy for the whole test series. This comprehensive comparison has not been made within this project.
For the following numerical simulations, it has been chosen to use mainly the Takeda model, since its behaviour has been identified as more robust, and to use reasonable sets of parameters:
- $K_0$ and $F_y$ are taken from the corresponding cyclic component test ($K_0$ is the stiffness and $F_y$ the elastic moment resistance);
- $r$ is taken very low, but not zero to avoid numerical problems (in general a ratio of 1 or 2% is considered);
- No unloading stiffness degradation is considered ($\alpha = 1$)
- The reloading connection point is taken close to the very end of the yield plateau ($\beta = 0.2$)

8.1.3.2 Sliding mass model – Principles and basic validation examples

In order to be able to account for sliding of pallets, a sliding point-mass model has been implemented. The starting point of the development of the sliding-mass model is to use the concept of “mathematical deck” already available in FineLg. The mathematical deck was originally elaborated to study the dynamic behaviour of structures subjected to moving loads or vehicles and particularly to study the bridge-vehicles interaction.

In this approach, the interactive behaviour is obtained by solving two uncoupled sets of equations, respectively for the structure and for the vehicles, and then by ensuring compatibility and equilibrium at the contact points between the structure and the vehicles with an iterative procedure. In this scheme, the so-called mathematical deck acts as an interface element to evaluate the position of the vehicles with respect to the physical deck and to perform the iterative compatibility process (Figure 115 a). Regarding the possible motion of the vehicles, the horizontal displacement is imposed according to the own speed of the vehicle and to its traffic lane. The vertical displacement, velocity and acceleration are on the contrary the result of a dynamic computation and are derived from the behaviour of the vehicle itself, from the underlying structure and from their possible interaction.

The idea in elaborating the "sliding mass" model is to start from a "single moving mass" vehicle without any user-imposed speed and to derive the horizontal behaviour of the mass through a dynamic computation according to a stick/slip model (Figure 115 b).

![Figure 115 General scheme of the mathematical deck: (a) Original formulation – (b) Evolution for the sliding mass model](image)
"Stick" behaviour:
The procedure for solving the global system when the masses are assumed to be fixed on the structure is the following. For each time-step:

1. Solve the structure assumed to be empty and subjected to the imposed ground acceleration. This is done by a classical Newmark procedure.
2. Thank to the mathematical deck, calculate the acceleration of the structure at the location of the contact points between the structure and the pallets. The acceleration is computed in both horizontal and vertical directions.
3. Estimate the inertial forces on the pallets corresponding to the level of acceleration computed in step 2. From these inertial forces, evaluate the contact force (horizontal and vertical) to be transferred from the pallets to the structure.
4. Solve the structure subjected to the ground motion and to the estimated contact forces. Update the acceleration of the contact points.
5. Go back to step 3 and loop until stabilization of the structural displacement. Figure 116 presents a schematic picture of the final converged situation.
6. From the converged value of the horizontal component of the contact force, define for each pallet if the next time-step has to be treated as "stick" or "slip".

\[
F_h - F_{h_{st}} = U_{str} = U_{pallet}
\]

Figure 116 Sliding mass model in "stick" phase at the end of the iterative procedure (equal displacements and contact forces)

"Slip" behaviour
As soon as the horizontal contact force computed in step 6 exceeds the static friction resistance \(R_{h_{st}}\), the mass starts sliding. The dynamic response of the two sub-systems (pallets and structure) are then evaluated separately under the combined effect of the imposed ground acceleration and of a constant contact force equal to the dynamic friction resistance \(R_{h_{dyn}}\) (Figure 117). During this stage, the pallet moves on the mathematical deck and its position, velocity and acceleration \(= R_{h_{dyn}}/M\) can be evaluated at any time step. The sliding behaviour lasts until the relative velocity between the pallet and the structure becomes equal to zero. So by estimating the relative velocity at the end of each time-step, it is possible to define if the next time-step has to be treated as "stick" or "slip".

\[
R_{h_{dyn}} - F_{h_{dyn}} = U_{str} \neq U_{pallet}
\]

Figure 117 Sliding mass model in "slip" phase

Convergence of the iterative procedure used for the "stick" behaviour
A strict application of the procedure described above may lead to strong convergence problems. This can be illustrated on the simple example of Figure 118, where \(k_s\) represents the stiffness of the structure and where \(m_s\) and \(m_p\) are respectively the mass of the structure and the mass of one pallet. \(a_g\) is the imposed ground acceleration, \(\alpha\) is a parameter of the Newmark method and \(\Delta t\) is the time-step.

\[
k_s = \infty
\]

Figure 118 Simple example to illustrate the convergence problem

For this situation, the steps of the iterative procedure are:

1. Calculation of the motion of the empty structure with a Newmark procedure:
\[ x_i^0 = -\frac{m_i \alpha}{k_F} \] with \[ k^{\text{NM}}_F = k_s + \frac{m_i}{\alpha \Delta t^2} \] (1)

2. Structural acceleration:
\[ \dot{x}_i^0 = \frac{x_i^0}{\alpha \Delta t^2} \] (2)

3. Contact force applied by the pallet on the structure:
\[ f_{p-s}^1 = -m_p \left( \dot{x}_i^0 + a_g \right) = -m_p \left( \frac{x_i^0}{\alpha \Delta t^2} + a_g \right) \] (3)

4. Update of the estimated structural displacement:
\[ x_i^1 = -\frac{1}{k^{\text{NM}}_p} \left( m_s a_g + a_p + \frac{x_i^0}{\alpha \Delta t^2} \right) \] (4)

5. Resulting global iterative process:
\[
x_i^{k+1} = \frac{-m_p}{m_i + k_s \alpha \Delta t^2} x_i^k + \frac{-\alpha \Delta t^2 (m_i + m_p) a_g}{m_i + k_s \alpha \Delta t^2} \\
= -e x_i^k - C
\] (5)

The time-step being assumed small, the convergence is thus ensured if and only if:
\[ e = \frac{m_p}{m_i + k_s \alpha \Delta t^2} = \frac{m_p}{m_i} < 1 \] (6)

This is obviously not possible for pallets on a rack structure, as the weight of the pallets is usually around 40 to 50 times the self-weight of the structure.

Therefore, the procedure is adapted and a relaxation parameter is introduced. The displacement at iteration \( k+1 \) is defined as a linear combination of the displacement at iteration \( k \) and of the results of equation (5). The new iterative process is thus:
\[ x_i^{k+1} = (1-\eta) x_i^k + \eta (-e x_i^k - C) \]
\[ = \left[ 1 - \eta \left( 1 + e \right) \right] x_i^k - \eta C \] (7)

Convergence is now ensured provided that the relaxation parameter \( \eta \) is less than \( \eta_{\text{max}} \) defined by:
\[ \eta_{\text{max}} = \frac{2}{1 + e} = \frac{2 m_i}{m_i + m_p} \] (8)

For the situation considered in this report, \( \eta_{\text{max}} \) can reasonably be taken equal to 0.05. The main consequent problem is that, with such a small value of the relaxation parameter, the convergence of the iterative process is relatively slow. For a fully-loaded structure, the number of iterations required to reach a precision of \( 10^{-5} \) on the structural displacement, which is necessary to manage adequately the situation of a structure with many pallets, can go up to 100 iterations in the worst situations.

Nevertheless, the main advantage of the proposed approach is to separate completely the resolution of the equations of motion for the structure and for the pallets. A same model can be used for both the "stick" and the "slip" behaviour, without any modification of the stiffness, mass and damping matrices characterizing the structure and the pallets. The only information required for computing the coupled effect is the relation between the acceleration imposed at the base of the pallet and the reaction force applied by the pallet on its support [i.e. \( f_{p-s} = f(t) \text{acc}_{\text{palle}} \)]. This also allows an easy account of the structural non-linearities, since these latter only implies a modification of the structural stiffness matrix without additional consequences on the resolution procedure.

**Simple validation examples**

In order to validate the sliding mass model, a set of very simple systems has been studied with FineLg and compared to equivalent MDOF systems solved with a semi-analytical approach. Some of the considered examples are presented at Figure 119.

The results obtained with FineLg and with the reference semi-analytical procedure are found in very good agreement. As illustration, results obtained with FineLg for case (c) are plotted in Figure 120 for \( \mu/\alpha = 1.00 \) (no sliding) and \( \mu/\alpha = 0.5 \) (\( \mu \) is the friction coefficient and \( \alpha \) is the maximum imposed
acceleration referred to gravity). In this second configuration, four sliding phases are observed, during which the relative displacement between M2 and M3 varies (see the green curve in Figure 120).

\[ u(t) = \alpha g \sin(\omega t) \]

\[ a(t) = \alpha g \sin(\omega t) \]

\[ P(t) = P^0 \sin(\omega t) \]

Figure 119 Validation examples (a) 1DOF – (b) 2DOF – (c) 3DOF

Figure 120 Time-history of the displacements obtained for case (c)

8.1.3.3 Possible future improvements of the two new tools in FineLg

Coming at the end of the research project, some possible future improvements of the new features developed in FineLg specifically for the modelling of racks can be identified.

- Combination in a single spring model of both Takeda and pinched behaviour, with a comprehensive assessment of the model properties with respect to the test results;
- Improving the reliability and robustness of the pinched model for dynamic computation;
- Improving the convergence of the point-mass model for the stick phase;
- Developing a full pallet model with six contact points and with the possibility to include the flexibility and damping of the pallet and stored goods;
- For the modelling of high pallets, including the rocking behaviour of the goods in the numerical model. This aspect was not felt as important in this project, since all tests have been realised with pallets loaded by concrete blocks or steel plates. The gravity centre of the "merchandise" was thus rather low and the rocking not relevant.

8.2 Calibration of the numerical models

In order to assess the performances of the numerical model before carrying out some parameter studies, a selection of typical configurations is made among the big amount of experimental results obtained during the research project. The main objective of this section is to use some well-chosen examples with the aim of evidencing the abilities and limitations of the model developed during the research project.
8.2.1 Dynamic sliding tests (2 beams – 3 pallets)
The configuration considered for this calibration is presented at Figure 121(a- test specimen; b-numerical model). The reference case for the comparison and calibration is test P2-B1 in cross-aisle direction, with a frequency of the sinusoidal table excitation equal to 1.0 Hz (sliding tests aa52 to aa61) and a linear variation of the amplitude. The table acceleration is given by:

\[ a_y(t) = 0.07t \sin(2\pi t) \]  \hspace{1cm} (9)

Connections between elements are realised by springs with properties based on component test results, and summarized in Table 32. Rigidities in the table corresponds to the rotation around the considered axis: x = down-aisle direction, y = cross-aisle direction, z = vertical direction

<table>
<thead>
<tr>
<th>Column bases</th>
<th>Beam-to-column connections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kx</td>
<td>100 kNm</td>
</tr>
<tr>
<td>Ky</td>
<td>120 kNm</td>
</tr>
<tr>
<td>Kz</td>
<td>Rigid</td>
</tr>
</tbody>
</table>

Table 32 Assumed rigidities of connections

8.2.1.1 Modal analysis
The flexural stiffness of beam-to-column connections around the vertical axis has not been identified during the component tests. It has however a very significant impact on the transverse vibration of the beam activated by the cross-aisle excitation of the structure. In order to calibrate this stiffness, an identification of vibration modes is carried out. Although no explicit modal computation has been performed, eigenfrequency and eigenmodes are obtained using transfer functions. Figure 122 a shows the DSP of acceleration of the 3 pallets, while Figure 122 b shows the transfer function between the table acceleration and the pallet acceleration. The main peak on the left figure corresponds obviously to the frequency of the excitation (1 Hz). A smaller peak can however also be identify around 3 Hz. This second peak is much clearer on the transfer functions. The frequency is equal to 2.93 Hz and the associated normalized mode shape is given by \[ \Delta_p = [0.54 \hspace{0.5cm} 1.00 \hspace{0.5cm} 0.59] \].

Figure 123 a and b shows the evolution of the frequency and modes (expressed in terms of modal displacement of the lateral pallets for a modal displacement of the central pallet equal to 1.00) when the joint stiffness increases. On these graphs, the two red lines correspond to the limit cases of pinned and
fully rigid joints. It can be clearly seen that it is impossible to find any agreement between numerical and test model: the only possibility to get a correct modal shape is to reduce the joint stiffness, which would lead to a significantly too low frequency. This clearly emphasizes the bracing effect of the pallets. Figure 124 shows a model that considers this effect with properties obtained by trial and error. The joint stiffness is equal to 20 kNm (rather close to a pinned configuration) with a small bracing section (the cross sectional area of the bracing bars is equal to 1.5 mm²). The resulting normalized modal shape is given by $\Delta_p = [0.55 \; 1.00 \; 0.55]$, with a frequency equal to 3.10 Hz.

![Figure 122](image)

**Figure 122** (a) DSP of pallet accelerations – (b) Transfer functions table-pallets

![Figure 123](image)

**Figure 123** Effect of joint stiffness on the numerical modal properties – (a) Frequency – (b) Mode shape

8.2.1.2 Dynamic analysis

From the corresponding test measurements, the initiation of sliding for the 1 Hz testing (aa51 to aa61) occurs between 18 and 20 s. Therefore, a first computation is made with classical mass representation (i.e. without using the sliding mass model presented in 8.1.3.2) using the table acceleration defined by (9) as input data for the first 20 seconds. The mechanical properties of the model are those obtained in the previous section. Figure 125 a and b compares the test measurements and the numerical results respectively for what regards the transverse displacement of the beam at the pallet location and the acceleration recorded on the pallet for the first 20 s of excitation. Results can be found in fairly acceptable agreement.

![Figure 124](image)

**Figure 124** Braced numerical model
In a second stage, a comparison is made between the numerical model with fixed masses and with likely-to-slide masses. For the sliding model, the choice of the friction coefficient is made according to the test results. The friction coefficient is equal to 0.15 for the lateral pallets and equal to 0.25 for the central pallet.

Figure 126 a shows a comparison of the transverse beam displacements at the location of the pallets if sliding is accounted for or not. Figure 126 b presents a similar comparison for the pallet acceleration. Figure 127 shows the same results obtained during one particular sliding test. Finally Figure 128 shows the evolution of the sliding displacement of the lateral pallets (the central pallets is actually not moving with respect to the beam, which is in agreement with the test observations). Figure 128 a is dealing with the test results and Figure 128 b with the numerical results.
8.2.1.3 Observations

- The trend of the sliding motion of the pallets is reproduced correctly by the model (i.e. global shift in one direction). However the amplitude of this movement is strongly overestimated. This can be explained by three aspects:
  
  o The friction coefficient considered in the model is the lowest possible value of the lower bound for the whole series of tests, while it is compared to one particular test;

  o The lower bound value derived from the sliding tests corresponds to the initiation of the relative motion between pallet and beam, which has been identified as an essentially torsional behaviour of the pallet. On the other hand, the numerical model is considering the pallets as point-masses and can thus obviously not reproduce any torsional effect, but only translational sliding;

  o The friction model is a pure Coulomb law with constant contact force during the sliding motion. It is likely that a model with a friction force varying with the pallet velocity could produce more accurate results.

- Analysis of the pallet acceleration confirms the first comment. The cut-off value of the transverse horizontal acceleration of the pallets is actually more in agreement with the upper bound value identified from the sliding tests (corresponding to a friction coefficient equal to 0.25 for the considered case), while the cut-off value obtained with the model is related to the lower bound value, due to the choice made. Further it is obvious that the cut-off of acceleration is much more stable for the model than in the tests.

- Regarding the beam displacements measured during the tests, a significant variation of the system stiffness can be observed as soon as the very first sliding occurs, i.e. as soon as the relative torsional motion between the pallets and the beams occurs. This could be explained by a loss of the bracing effect brought to the beam by the pallets as soon as these latter are moving. The numerical model is intrinsically unable to catch this behaviour since pallets are modelled by point-masses and bracings by additional truss bars (see Figure 124). A solution to get a better agreement between test and model displacements would be to remove the additional bracing bars, but this would also lead to an overestimation of the displacements (and thus of the internal forces) for the behaviour before sliding. Moreover, the very best modelling solution would be a complete tri-dimensional model of the pallets with 6 contact points. Such an advanced model would indeed be able to account for the torsional...
sliding. The programming environment for such a model has been prepared in FineLg during this project, but the development has not been taken to an end.

8.2.1.4 Conclusions
The main conclusions of this study on sliding are the following:
- The sliding mass model is behaving as expected according to the assumption of a point-mass approximation of the pallets;
- This approximation can be considered as reasonable if the torsional behaviour is not too significant. In particular, the model would be much more convenient for down-aisle excitation than for cross-aisle;
- The main possible improvements of the model are the development of a 3D pallet model with 6 contact points and the implementation of a more refined contact law;
- If a point-mass model is used, the most reasonable approximation in the perspective of a safe estimation of the structural behaviour consists in not considering the bracing effects of the pallet, which leads to higher values of the displacements and accelerations, and to use the upper-bound value of the friction coefficient, which leads to higher values of the force transferred from the mass to the rack.

8.2.2 Dynamic full-scale shaking table test (2 bays – 3 levels) – Down-aisle behaviour (2D model)

8.2.2.1 General considerations
The configuration considered for this numerical analysis corresponds to the down-aisle test realised on the shaking table of NTUA referred to as "Dynamic test A2" in the related section of the report (beams 130/15, 3 levels, fixed bases, possible sliding of the pallets).
Since neither structural torsional effect nor any other kind of tri-dimensional behaviour is likely to be significant in this situation, the modelling is made with a 2D model. Figure 129 compares the test specimen and the corresponding numerical model.
Joints (column-bases and beam-to-column) are modelled by equivalent springs with properties obtained from the component tests:
- Column-base: $K = 170 \text{kNm}$ $M_{\text{max}} = 2.6 \text{kNm}$
- Beam-to-column: $K = 120 \text{kNm}$ $M_{\text{max}} = 2.5 \text{kNm}$
Members are assumed to behave elastically between connections:
- Uprights: $A = 588.8 \text{mm}^2$ $I = 81.2 \text{cm}^4$
- Beams: $A = 607.7 \text{mm}^2$ $I = 174.2 \text{cm}^4$

8.2.2.2 Modal analysis
Modal analysis is performed according to two assumptions. In the first case, vibration modes are estimated on the base of the initial stiffness matrix, while in the second case, the calculation accounts for the loss of stiffness in uprights due to the vertical loading (geometric matrix is added to the initial stiffness matrix). Results are given in Table 33 and can be put in relation with the results of the evaluation of the transfer function of the test specimen:
Numerical period is slightly overestimated for the first mode, but in very good agreement for the second mode. It is also worth noting that the first natural period of the test specimen obtained with a different methodology (i.e. stabilization charts, see Seisracks intermediate report) has been estimated equal to 1.60 s, which would lead to a better agreement between test and model. Further, stabilization charts also allowed providing an estimation of the elastic viscous damping of the system equal to 3.5 %.
8.2.2.3 Time-history dynamic analysis

Following the modal analysis, a full time-history analysis has been performed using the input accelerogram recorded on the shaking table. Comparison of test and model is made for three levels of accelerations corresponding to tests A2-07, A2-13 and A2-17, with PGA respectively equal to 0.30 g, 0.54 g and 0.92 g.

The numerical analysis is made with increasing level of complexity:

- Linear dynamic analysis;
- Geometrically non linear dynamic analysis with elastic joints;
- Geometrically and materially non linear dynamic analysis.

For each case, the computation is done with fixed masses and with masses that are likely to slide. In case of sliding, a friction coefficient of 0.4 is used. This is considered as a reasonable value for the case P2-B4 (P2 and B4 being respectively the pallet and beam types used for the shaking table test). The damping ratio is taken equal to 3.5 % according to the results of the modal analysis.

Figure 130 a compares the evolution of the top displacement obtained for the lowest value of PGA (A2-07) with the three levels of assumption. These curves are also compared with test measurements. All three models are producing very close results, which translates the fact that, for the considered level of acceleration, geometrically non linear effects are not playing an important role and connections are behaving elastically. Further, the general shape of the numerical time-history response is in good agreement with test results, even if slight differences are observed on the extreme displacements. Similar conclusions can be drawn from the acceleration curves Figure 130 b. Finally, no sliding is predicted by the model with sliding, in agreement with the test measurements. Concerning the free vibration phase, the damping is much lower for the numerical model. This is most probably due to the
braking system of the shaking table that induces an over-damping in the structure. This effect is not included in the model.

Figure 131 a and b present similar results for the intermediate level of PGA. In this case, the difference between the linear and non linear computations becomes more significant, highlighting the increased importance of the second order effects. Further, a small residual displacement of the system is observed due the yielding of connections. A similar trend is observed for the materially non linear model, even if the stabilization of the free-vibration phase is slower, for the reasons already explained above. For this level of acceleration, no sliding is predicted by the model, which can be compared with the very small sliding (almost negligible) values measured during the test, i.e. between 0.1 and 0.2 mm.

Figure 130 (a – b) – Top displacement/acceleration for case A2-07

Figure 131 (a – b) – Top displacement/acceleration for case A2-13

Figure 132 (a – b) – Top displacement/acceleration for case A2-17 (without sliding)
Figure 132 a and b are showing a comparison between test and numerical results for the model with fixed masses for the highest PGA level. In this case, the global behaviour obtained with the model starts deriving significantly from the test results. However, once sliding is taken into consideration, the results are in better agreement, as shown on Figure 133, in particular regarding the final value of the residual displacement (once again with the restriction previously mentioned on the effective damping level of the free-vibration phase). Moreover a comparison of the predicted sliding motion with the measured sliding motion shows that the order of magnitude of the measurements and of the prediction are similar and that the moments at which sliding is occurring are in good agreement. Figure 134.

In order to make the comparison clearer, the extreme values obtained with the different models are compared in Table 34, Table 35 and Table 36. From these values, it can be observed that:

- For a low table acceleration (A2-07), the estimation of displacement and acceleration are acceptable except for what regards the first level.
- For A2-07, no sliding is predicted by the model. However a non negligible difference is found for the non linear case whether the masses are considered fixed or modelled by the potentially sliding model. This is due to the slow convergence of the model in the stick phase. In order to be able to get solutions within a reasonable computation time, the strictness of the convergence criterion was rather moderate. In the studied configuration, the problem is particularly significant for the 1st level, with an error of 15%. This should be solved by improving the convergence procedure of the model.
- The more the behaviour is becoming non linear (i.e. when important differences are found between the various modelling assumptions), the less the results are accurate with respect to the test results. This could however significantly be improved by further calibration of the resistance of the joints. Moreover, additional sources of dissipation could also be investigated.

In conclusion, it can be stated that the numerical model including the new tools developed during the Seisracks project (i.e. hysteretic spring and sliding mass model) is able to reproduce with an acceptable level of accuracy the down-aisle behaviour of a rack structure, even if some complementary investigations would be necessary to assess in a more precise way the actual behaviour of the connections. It seems indeed that using values of stiffness and resistance of connections different from the component tests could lead to more accurate results, as soon as strong motion is applied to the structure. Moreover, the sliding mass model could be improved in order to get a better accuracy of the results while conserving a reasonable computation time.
8.2.3 Dynamic full-scale shaking table test (2 bays – 3 levels) – Cross-aisle behaviour (3D model)

8.2.3.1 General considerations

The previous section has presented numerical results related to the down-aisle behaviour. This could be realised by using a 2D plane model. As soon as it comes to the cross-aisle behaviour, a full tri-dimensional model becomes necessary. Indeed, a differential motion of the central upright frame with respect to the two lateral frames must be accounted for. Moreover some other aspects like the restraining effect of the beams bent around their weak axis require also the use of a 3D model.

It has been shown on the planar down-aisle model that the direct use of joint mechanical properties obtained from the component tests was leading to a model response that could diverge from the real test specimen response. The problem is even more pronounced for the cross-aisle behaviour, since no experimental tests have been realised to identify the properties of the beam-to-column connections in bending around the longitudinal axis of the upright. In the present context, a stiffness equal to the value obtained from the modal calibration of the two-beams model (section 8.2.1.1 – K = 20 kNm/rad) is used. As absolutely no information on the resistance and on the non linear behaviour of such joints is available, the calculations made in the frame of this research are limited to linear dynamic analysis, with the aim of evaluating trends, more than to obtain reliable quantitative results.
8.2.3.2 Modal analysis

Figure 135 shows the first 4 modes obtained with the 3D model. Table 37 summarizes the natural frequencies of these modes and makes a parallel with the values obtained from the transfer functions derived in the dynamic tests. For the frequencies that could be identified by the tests, tests and model are in good agreement. Down-aisle frequencies are also similar to those obtained with the 2D model.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Mode type</th>
<th>Numerical frequency [Hz]</th>
<th>Test frequency [Hz]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>First mode – down-aisle</td>
<td>0.58</td>
<td>0.70</td>
</tr>
<tr>
<td>2</td>
<td>First mode – cross-aisle</td>
<td>1.73</td>
<td>1.72</td>
</tr>
<tr>
<td>3</td>
<td>Second mode – down-aisle</td>
<td>2.30</td>
<td>2.31</td>
</tr>
<tr>
<td>4</td>
<td>Second mode – cross-aisle</td>
<td>2.98</td>
<td>Not identified</td>
</tr>
</tbody>
</table>

Table 37 Natural frequencies of the 3D model

8.2.3.3 Time-history analysis

Linear time-history analyses of the cross-aisle behaviour of the complete 3D model have been realised in the frame of the project. However the results obtained with the model were significantly diverging from what had been observed during the tests. These results are thus not reported here. Additional investigations are still needed to identify correctly the different parameters participating in the 3D cross-aisle modelling of a rack structure.

8.2.4 Modal identification of previous tests on braced or partially braced structures (Ecoleader tests)

All configurations tested in the Seisracks project were structures without bracings in the down-aisle direction and without horizontal bracings. Therefore, in order to validate the numerical model for braced configurations, a comparison has been made with previous test results obtained in the context of the Ecoleader program. In this report, only modal analyses are considered.

The two specimens used for this analysis are models C and D (Polypal 0.15g and Polypal 0.35g, respectively with 1 and 4 bays braced in the down-aisle direction – see Figure 136). The shear stiffness of the cross-aisle upright frames had been identified by appropriate standardized tests. The resulting effective area of the diagonals is equal to 0.17 A_b. The same reduction will thus be used for estimating the periods. On the contrary, the down-aisle bracings are realised by eccentric strong bars. In this case, no reduction of the area is taken into account.

For model D, 5 modelling assumptions are used. The connections of the system had been characterised for what regards the down-aisle behaviour. The various modelling assumptions are therefore dealing with the rotational behaviour for the other two directions, i.e. torsion and weak axis bending. Table 38 summarizes the periods obtained according to these assumptions.

It can be seen that the support conditions related to the torsional behaviour have no influence on the calculated period, for both column base and beam-to-column connections. Thus there is no real need to characterize this behaviour. On the contrary, the bending stiffness of the column base connection in the plane of the upright frame and the bending stiffness of the beam-to-column connection regarding weak axis bending of the beam are likely to influence more significantly the model behaviour. A hinged
assumption appears to produce results in good agreement with the experimental observations, which confirms the conclusions obtained on the Seisracks specimens presented in the previous section. In order to confirm these conclusions, the same hinged assumptions are used for estimating the first four natural periods of model C. Results are given in Table 39 and shows a good agreement for the first modes respectively in the cross- and down-aisle direction. Second cross-aisle mode had not been identified on the test specimen. No comparison is thus possible. Strong discrepancies on the second down-aisle mode are found and should require complementary investigations.

![Figure 136 Polypal models C and D (Ecoleader)](image)

<table>
<thead>
<tr>
<th>Beam-to-column</th>
<th>hinged</th>
<th>fixed</th>
<th>hinged</th>
<th>fixed</th>
<th>fixed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Torsion</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam-to-column</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Torsion</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weak axis bending</td>
<td>hinged</td>
<td>hinged</td>
<td>fixed</td>
<td>hinged</td>
<td>hinged</td>
</tr>
<tr>
<td>Column-base</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Torsion</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weak axis bending</td>
<td>hinged</td>
<td>hinged</td>
<td>hinged</td>
<td>fixed</td>
<td>fixed</td>
</tr>
</tbody>
</table>

| Period 1               | 1.284 s | 1.285 s | 1.318 s | 1.285 s | 1.384 s |
| Period 2               | 1.484 s | 1.485 s | 1.497 s | 1.485 s | 1.572 s |
| Period 3               | 2.418 s | 2.418 s | 2.452 s | 2.418 s | 2.451 s |
| Period 4               | 2.898 s | 2.898 s | 3.285 s | 2.898 s | 3.020 s |

Table 38 Numerical assessment of the periods for braced structures (Polypal D)

<table>
<thead>
<tr>
<th>Period</th>
<th>Model</th>
<th>Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (Down)</td>
<td>1.32 s</td>
<td>1.22 s</td>
</tr>
<tr>
<td>2 (Down)</td>
<td>1.21 s</td>
<td>0.56 s</td>
</tr>
<tr>
<td>3 (Cross)</td>
<td>1.01 s</td>
<td>1.00 s</td>
</tr>
<tr>
<td>4 (Cross)</td>
<td>0.46 s</td>
<td>not identified</td>
</tr>
</tbody>
</table>

Table 39 Numerical assessment of the periods for braced structures (Polypal C)

8.3 ADDITIONAL STUDIES
This section presents complementary analyses not directly related to the assessment of numerical models with respect to test results but aiming at evidencing some particular problems of the seismic behaviour of rack structures.
8.3.1 Cross-effect of sliding and number of levels

In order to investigate the actual beneficial impact of a possible sliding of pallets on the structural response of racks, a simple parameter study is performed. It consists in analysing 3 structures using the same element properties with respectively 1, 2 and 3 levels. This can also alternatively be seen as a same 3-levels structure loaded on the first, on the first 2 or on all 3 levels. The basic geometry is the geometry of the dynamic test specimens, the study is a 2D analysis in the down-aisle direction and the structure is assumed to behave linearly. The main practical objective is to evidence the consequences of sliding on the global seismic behaviour of the structure.

The structure is subjected to 7 artificial accelerograms with spectrum compatible with a reference spectrum having the following characteristics (see also Figure 137 for reference accelerograms used in this study). All accelerograms are obtained by scaling the reference time-histories.

- EC8 type I spectrum;
- PGA equal to 0.05 g, 0.10 g or 0.15 g;
- Soil type C;
- Duration = 15 s.

The friction coefficient of the pallets is varied from 2 (which is not physically relevant but corresponds to pallets fully fixed on the beams) to 0.25.

![Figure 137](image)

Figure 137 : accelerograms and spectrum used for parameter study

Figure 138 a shows the evolution of the maximum transverse displacement of the 1-level structure when the friction coefficient decreases. The plotted displacement is the average of the maximum displacements obtained from the 7 considered ground motions, in agreement with the methodology proposed by Eurocode 8. Figure 138 b shows in parallel the maximum relative displacement of the pallets with respect to the supporting beam. Figure 139 and Figure 140 presents similar information respectively for the 2-levels and 3-levels structures.

![Figure 138](image)

Figure 138 : Effect of sliding on the down-aisle behaviour of the 1-level rack – (a) maximum global displacement – (b) maximum local sliding displacement
The main observations that can be drawn from these results are the following:
- Non-braced rack structures are very flexible. The maximum global transverse displacement can go up to 1/15th of the total height for a PGA equal to 0.15 g. This implies that geometrical second-order effects should normally be taken into account.
- Sliding of the masses and subsequent limitation of the inertial forces to the friction force between pallets and beams is likely to significantly reduce the global displacements of the structure and hence the internal forces in the structure and the support reactions. For low friction coefficients, these displacements can be reduced to 20% of the values obtained if the pallets are supposed to be fully fixed on the beams.
- However, this reduction cannot be considered as a general rule. Indeed it is strongly related to the PGA level and to the structural typology (i.e. the number of levels). For example, no reduction is observed for a 3-level structure subjected to an earthquake with PGA equal to 0.05 g, even for a friction coefficient equal to 0.25, while the reduction is about 50% for the same structure but with a PGA equal to 0.15 g. Further, for a same PGA of 0.15 g and a same friction coefficient of 0.5, global displacements are reduced to about 35% of their fixed value for a one-level structure, while they are only reduced to 80% for a 3-level structure.
- Moreover the local sliding displacement of the pallets in the case of low friction coefficients can be very important and therefore non compatible with real conditions. For example, for a 3-level structure subjected to an earthquake with a PGA equal to 0.15 g, the local displacement of the pallets with respect to the structure can go up to 30 cm for a friction coefficient equal to 0.25. Such a value of the displacement is obviously non admissible, since it corresponds either to a fall of the pallet or at least to an impact of the pallet against an upright except if some specific devices are used to prevent fall or impact. It is important to remind that a friction coefficient as small as 0.25 can be fairly common in usual practical situations.

8.3.2 Numerical modelling of a 6 bays/7 levels structure
The 3D model developed to investigate the behaviour of the test specimen has been extended to a structure with 6 bays and 7 levels. Figure 141 shows the first 4 modes of vibration of the structure with their associated period. Time-history dynamic analyses have also been carried out with the same model. However since the calibration of the 3D model versus the test results was not really concluding, it is felt
that the results obtained on the full structure are also likely to be affected by the same kind of problem. They are thus not reported here.

\[
T_1 = 4.02 \text{ s} \\
T_2 = 1.98 \text{ s} \\
T_3 = 1.19 \text{ s} \\
T_4 = 1.16 \text{ s}
\]

**Figure 141** Modes of vibration of a 6 bays/7 levels rack structure

8.3.3 Code assessment of the 2 bays/3 levels structure

With the aim of comparing the different methodologies proposed by pr FEM 10-2-08 for the structural analysis, essentially based on EN 1998-1 recommendations, an assessment of the maximum ground acceleration that can be sustained by the 2 bays/3 levels test structure shaken in down-aisle direction has been performed. This comparison produces interesting information about the methods even if they can however not be directly put in correlation with the test results, since the assessment is based on the assumption of a type 1 – soil EC8 spectrum, while the tests have been realised with a table acceleration filtered by a high-pass filter with a cut-off period equal to 1.0 s. The first natural period of the structure is higher than 1.0 s (i.e. 1.5 s) and the structural response on the first mode is thus essentially different for both approaches (test and code assessment).

The code assessment is based on the following hypotheses:

- Behaviour factor \( q = 1.5 \);
- Seismic weight of the pallets \( W_E = E_d \times R_f \times Q_d = 0.8 \times 1.0 \times 750 \text{ kg} = 600 \text{ kg} \).

Eight different analyses are realised. For each type of analysis, the reference ground acceleration is increased until reaching a normative collapse. The normative collapse is defined by a situation in which the maximum resistance ratio (= Action / Resistance) is equal to 1.0 for at least one of the five following structural elements

- Lateral upright
- Central upright
- Beam
- Column-base connection
- Beam-column connection
The eight methodologies are:
1. Lateral force method using the initial stiffness of the system – second order effects taken into account in the element verification by the use of the sway buckling length of the columns;
2. Lateral force method using the initial stiffness of the system – second order effects taken into account through amplification coefficient \(1/(1-\theta)\);
3. Lateral force method applied on a system with reduced stiffness, because of the pre-compression of the uprights (geometric matrix included);
4. Lateral force method based on the initial stiffness – second order effects accounted for by using an amplification estimated on the base of an equivalent SDOF system;
5. Response spectrum analysis – second order effects taken into account through amplification coefficient \(1/(1-\theta)\);
6. Response spectrum analysis based on the reduced stiffness;
7. Lateral force method – second order effects explicitly accounted for by an elastic static non linear analysis;
8. Pushover analysis.

For the two methods for which the estimation of the coefficient \(\theta\) measuring the sensitivity of the structures to second order effects is required, the corresponding values are respectively equal to 0.316 (method 2) and 0.309 (method 5). This means that the structures are at the limit of what is allowed by EN 1998, that second order effects absolutely need to be accounted for and that the simplified approach using the amplification factor \(1/(1-\theta)\) is normally not allowed.

The modal mass associated with the first mode is equal to 89 %. According to EN 1998, it is the lower limit from which a lateral force method is permitted to replace the default methodology (default methodology is "response spectrum analysis").

Table 40 summarizes the maximum allowable acceleration and the collapse criterion activated at the limit state.

<table>
<thead>
<tr>
<th>Analysis</th>
<th>(a_{g,\text{max}})</th>
<th>Collapse criterion</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.14 m/s²</td>
<td>Buckling of the central column</td>
</tr>
<tr>
<td>2</td>
<td>0.64 m/s²</td>
<td>Failure of column base connection in bending</td>
</tr>
<tr>
<td>3</td>
<td>0.65 m/s²</td>
<td>Buckling of the central column</td>
</tr>
<tr>
<td>4</td>
<td>0.70 m/s²</td>
<td>Failure of column base connection in bending</td>
</tr>
<tr>
<td>5</td>
<td>0.68 m/s²</td>
<td>Buckling of the central column</td>
</tr>
<tr>
<td>6</td>
<td>0.87 m/s²</td>
<td>Buckling of the central column</td>
</tr>
<tr>
<td>7</td>
<td>0.74 m/s²</td>
<td>Failure of column base connection in bending</td>
</tr>
<tr>
<td>8</td>
<td>0.74 m/s²</td>
<td>Failure of column base connection in bending</td>
</tr>
</tbody>
</table>

Table 40

Some comments can be made on these results:
- Whatever the method used, the maximum allowable acceleration is rather small (less than 0.1 g);
- For all 8 cases, both "buckling of the central column" and "Failure of the column base connection in bending" resistance ratios are actually very close (less than 1 % of difference). This explains the fact that the collapse criterion activated at the limit state is shifting from one to the other according to the type of analysis;
- An analysis considering second order effects by using a sway buckling length for the verification of columns leads to very safe results compared to all other types of analyses;
- If the pushover method is considered as a reliable reference, the only unsafe approach consists in using the reduced stiffness in a modal analysis. This method is indeed not really accounting for the second order effects, since no amplification of the sway moments is included.
- Even if EN 1998 is limiting the use of the simplified amplification method based on the \(\theta\) factor to values of \(\theta\) lower than 0.2, it is shown here that this simplified approach remains safe compared to a pushover method, even if \(\theta\) is higher than 0.3.

8.4 CONCLUSIONS

Different aspects of the numerical modelling and of the analysis of rack structures have been investigated within the SEISRACKS research project.

Two new features of prime importance for an efficient analysis of racks subjected to seismic action have been included in the FEM software FineLg: springs with hysteretic energy dissipation and sliding
point-mass with coulomb friction law. These tools are fully operational, even if some future improvements have already been identified (convergence of the sliding model in the stick phase, test of other friction laws, sliding mass model with numerous contact points…).

Models have been validated versus a selection of test results obtained during the SEISRACKS research and during previous research programs. The validation has been carried out for cross- and down-aisle seismic excitation and for braced as well as non braced structures. Some particular aspects have been emphasized during the calibration procedure: the need for a precise knowledge of the stiffness and resistance of the column bases, the horizontal bracing role played by the pallets as long as they are not sliding or the need for a future calibration of the behaviour of beam-to-column joints regarding rotation around the longitudinal axis of the upright.

Two complementary studies have also been performed in the perspective of normative prescriptions. The first one is a parameter study about the consequences of pallet sliding on the structural response. The study evidences clearly that the reduction coefficient is depending on the intensity of the ground motion, on the value of the friction coefficient and on the structural typology (i.e. the structural natural period and the number of loaded levels). The reduction coefficient ranges roughly from 0.2 to 1.0. Additional studies would be necessary to refine these conclusions and calibrate properly the reduction factor.

The second complementary study is comparing different types of seismic analyses (lateral force method, response spectrum analysis and pushover analysis) applied to a same structure exhibiting significant second order effects. The main conclusions are that, for the considered structure, all analyses provide similar results, provided that second order effects are really accounted for. To this purpose, the use of the approximate amplification factor $1/(1-\theta)$ is found efficient. Moreover, a verification taking into account second order effects by using a sway buckling length appears strongly over-conservative.

8.5 COMMENTS ON FEM 10-2-08

8.5.1 General introduction

The whole research project SEISRACKS has been an opportunity to analyse the current draft of the normative document pr FEM 10-2-08 "Recommendations for the design of static steel pallet racks under seismic conditions". In particular, a series of items have been identified as questionable and are listed here with the corresponding sections of pr FEM 10-2-08 in its version of December 2005.

- Determination of the period of the structure and of the seismic action, and in particular:
  o Regularity criteria and consequences on the behaviour factor (2.2.5),
  o Effect of the actual position of the gravity centre of the masses, vertical eccentricity with respect to the beams (2.3.6),
  o Methods of analysis (2.4),
  o Definition of regularity criteria (3.1.4),
  o Modelling assumptions in the perspective of the structural analysis (3.3),
- Account for the different sources of energy dissipation (Viscous damping, friction of pallets, energy dissipation within the stored goods)
  o Definition and values of parameters $E_{D,1}$, $E_{D,2}$ and $R_F$ (2.3.1, 2.3.2, 2.3.3, 2.3.4, 4.2.2, 4.2.3)
- Assessment of the structural ductility and associated behaviour factor
  o Definition of ductility classes (3.1.1)
  o Material properties and overstrength coefficient (3.1.2)
  o Definition of the q-factor according to the structural typology (3.1.3, 3.4)
  o Impact of (ir-)regularity (3.1.4, 3.4)
  o Design rules for non dissipative resp. dissipative structures (3.1.5)
  o Identification of the resisting system (3.2)
- Detailing of dissipative elements and overstrength criteria (5)

On the base of the knowledge gained during the research project and on engineering judgement, many of these items can be addressed. The final output will be a revised version of pr FEM 10-2-08, but some background comments are given in the following.
8.5.2 Regularity criteria
In the current draft of pr FEM 10-2-08, all rules are taken directly from EN 1998-1. No contradictory information has been developed on this aspect within the project SEISRACKS and it is thus proposed to confirm the current draft.

8.5.3 Position and height of the masses
Pr FEM 10-2-08 proposes rules to account for the fact that the gravity centre of the pallets may be at a certain level above the beams, which induces an overloading of the pallet beams due to the overturning of the palletised goods subjected to horizontal inertial forces acting in the cross-aisle direction. This effect is particularly important for palletized goods having a significant height. These rules are based on a reasonable engineering judgement and no contradictory information has been developed within the project SEISRACKS, essentially because the considered structural test specimens were loaded by concrete blocks or steel plates with a very low position of the gravity centre with respect to the pallet beams. It is proposed to confirm the current draft.

8.5.4 Methods of analysis
Methods of analysis proposed in pr FEM 10-2-08 are derived from EN 1998. The reference method is modal response spectrum analysis and can be replaced by a lateral force method analysis if the structure can be classified as regular. Section 2.4 of the pre-normative document is confirmed.

A comment must however be made about the way to deal with second order effects (section 4.1.2.4). In EN 1998-1, the rule is the following.
- If the coefficient $\theta$ measuring the sensitivity of the structure to second order effects is lower than 0.1, these second order effects can be disregarded.
- If $\theta$ is between 0.1 and 0.2, second order effects can be accounted for in an approximate way by multiplying the results of a linear analysis by an amplification factor equal to 1/(1-$\theta$).
- If $\theta$ is between 0.2 and 0.3, second order effects must be accounted for explicitly.
- $\theta$ higher than 0.3 is not allowed.

In the case of racks, on the base of the conclusions of section 8.3.3 of the present report, the proposal of pr FEM 10-2-08 is confirmed:
- If the coefficient $\theta$ is lower than 0.1, second order effects can be disregarded.
- If $\theta$ is between 0.1 and 0.3, second order effects can be accounted for in an approximate way by multiplying the results of a linear analysis (obtained either modal response spectrum or lateral force method according to the regularity of the structure) by an amplification factor equal to 1/(1-$\theta$).
- If $\theta$ is between 0.3 and 0.5, second order effects must be accounted for explicitly, which can be done either through a non linear dynamic time-history analysis or through a push-over analysis.
- $\theta$ higher than 0.5 is not allowed.

In addition, as soon as $\theta$ is higher than 0.1, it is recommended to consider a stiffness of the structure reduced by the pre-compression of the uprights (by using the geometric matrix) for the calculation of the period. However, the use of the initial non reduced stiffness would normally lead to safe results, since the corresponding periods are higher.

8.5.5 Pallet weight modification factor $E_{D2}$
In pr FEM 10-2-08, a pallet weight modification factor is proposed to account for damping inside the palletized goods. Such a source of energy dissipation would be possible only if the frequencies of the stored goods are tuned on the frequency of the structure. This is very unlikely according to the respective range of frequencies observed during the SEISRACKS project. Natural frequencies of the racks are rather low (between 0.25 and 1 Hz), while the few values of frequencies obtained for merchandise are significantly higher (between 1.5 and 24 Hz). As long as extensive measurements are not performed to assess the actual dynamic behaviour of a wide range of merchandise, it is recommended not to account for any dissipation in the goods and thus to set $E_{D2} = 1.0$ in section 2.3.3 of pr FEM 10-2-08.
8.5.6 Design spectrum modification factor $E_{D1}$

Design spectrum modification factor $E_{D1}$ is introduced in the design code to account for possible energy dissipation by friction of the pallets on their supporting beams. A constant value equal to 0.8 is proposed in the current draft if no specific devices are present to avoid pallet sliding. Section 8.3.1 of the present report has shown that this reduction factor is actually depending on the design acceleration, on the friction coefficient and on the structural typology. With this perspective, the most advantageous situation would be a structure with only one loaded level, with a high design acceleration and a very low friction coefficient. For example, if the ground acceleration $a_g$ is equal to 0.5g, the design acceleration of the structure may rise up to 1.25g. If only one level is loaded and if the friction coefficient $\mu$ is equal to 0.25, the force acting on the structure is exactly limited to the friction force, which corresponds to an equivalent acceleration of 0.25g. The reduction factor is thus equal to $0.25g/1.25g = 0.2$. On the other hand, for structures with many loaded levels and for low values of $a_g$, it can happen that no sliding occurs. The reduction factor is consequently equal to 1.0. The practical range of $E_{D}$ is therefore rather wide, between 0.2 and 1.0. It is interesting to note that the value proposed by the American RMI is equal to 0.67, which is somewhere in the middle of this interval.

Some additional studies are suggested to calibrate an expression of $E_{D}$ that would depend on the structural typology and the ratio $a_g/\mu$. These additional investigations should be a priority. Complementary comments can also be done on section 2.3.4, 4.2.2 and 4.2.3 dealing with the "no sliding limit state". Indeed, as soon as sliding is used to justify a reduction of forces acting on the structure, it is contradictory to check that pallets are not sliding. Section 2.3.4, 4.2.2 and 4.2.3 should consequently be removed. Moreover, it has been shown during the research that the amplitude of the sliding motion is highly random and can go up to values like 30 cm. Therefore, clauses 2.3.4 (4) and (5) imposing specific devices to avoid the fall of goods (like for example a 3rd beam) should be mandatory. In short, the situation regarding sliding can be summarized in two points.

- If specific devices are used to avoid sliding, a value of $E_{D}$ equal to 1.0 should be used.
- Otherwise, value of $E_{D}$ smaller than 1.0 can be used but provisions must be taken to avoid the fall of pallets. Reduced values of $E_{D}$ should be varying between 0.2 and 1.0 according to the structural typology and to the ratio $a_g/\mu$.

8.5.7 Rack filling factor $R_F$

Even if a long-term monitoring of a typical structure has been realised during the research project, this was obviously a particular case. The proposal of pr FEM 10-2-08 regarding the rack filling ratio $R_F$ to be considered for the seismic assessment is confirmed, namely to use a value equal to 1.0 (structure fully loaded) if nothing else is specified by the Specifier/User.

8.5.8 Ductility and behaviour factor $q$

Various comments can be made on the behaviour factor to be used for the analysis.

- For down-aisle direction,
  - A $q$-factor equal to 1.5 can be considered in any case;
  - A $q$-factor equal to 2.0 can be considered for non braced structures, provided that members working in compression and/or bending are at least of class 3. This is supported by the pushover, pseudo-dynamic and dynamic tests realised during the SEISRACKS project (q-factor of about 3.5), affected by a reasonable safety factor.
  - Higher values can be used if properly demonstrated and if the rules of capacity design are followed.

- For cross-aisle direction,
  - For configurations in which the horizontal stability involves at least one bar in compression (configurations partially braced or configurations in Z, D or K), no dissipation should normally be accounted for. This implies that the $q$-factor must be taken equal to 1.0. A value of 1.5 could however be considered if properly demonstrated by appropriate test results.
  - For concentric X-bracing configurations, a minimum value of 1.5 can be used. If the structure is designed as dissipative following all principles of capacity design, a behaviour factor equal to 4.0 can be used provided that the condition of uniform dissipation all over the height of the upright frame is checked. If this condition is not fulfilled, the structure is actually behaving as an inverted pendulum with an equivalent
plastic hinge at the bottom of the frame (only one cell of the upright frame is behaving in the plastic range at ULS). In that case, the behaviour factor should be limited to 2.0.

8.5.9 Characterization of joints for the analysis
The proposal of pr FEM 10-2-08 consisting in using the stiffness of beam-to-column connections obtained from static tests according to FEM 10-2-02 for the structural analysis under seismic conditions is confirmed (Section 8.3.3). A similar proposal can also be made for column bases, provided that the actual level of compression is duly accounted for.

8.5.10 Detailing rules for moment resisting frames
In the case of down-aisle non braced configurations resisting by frame effect and designed with dissipative semi-rigid connections, the energy dissipation in the beam-to-column connections and in column bases should be properly assessed. To this purpose, the following procedure is proposed. A non linear pushover analysis is carried out considering joint properties obtained from a monotonic static test. From this analysis, the seismic rotation demand of the joints can be evaluated. A cyclic component test must then be realised following the new testing procedure proposed in the corresponding section of the present report with cycles up to the value of the rotation obtained from the pushover analysis. The joint should be able to keep its strength during a sufficient number of cycles, this number still being to be defined.

8.5.11 Ductility classes
Even if a distinction needs obviously to be kept between "low dissipative" and "dissipative" structures, the additional sub-classification made within the "dissipative" category between DCM and DCH (ductility classes medium and high) could be removed (see for example table 3.2 of pr FEM 10-2-08). This would have a very limited impact on the code and could make it much clearer. At the few locations where a distinction is nevertheless needed, the criterion could be based on the section class.

8.5.12 Vertical component of the seismic action
Section 2.2.4 of pr FEM 10-2-08 dealing with the vertical component of the seismic action is confirmed. However, it is felt that additional investigations are necessary to clarify its actual impact on the joint behaviour (beam-to-column joints and column bases) and on the sliding.
Conclusions

In Europe, no official document is currently available for the seismic design of pallet racks, and the designers are compelled to operate with a total lack of references and of commonly accepted design rules. Very often they make reference to the Rack Manufacturers Institute (R.M.I.) Specifications, while the European Racking Federation (F.E.M.-ERF) is presently working in order to produce an official document.

Racks are widely adopted in warehouses where they are loaded with tons of (more or less) valuable goods. The loss of these goods 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. The falling of the pallets, in this case, may endanger the life of the clients as well as that 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.

Sliding of the pallets on the racks and their consequent fall represents a serviceability limit state 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. Hence, solution of the problems connected with safe and reliable design of steel storage racks in seismic areas has a very large economic impact.

The project focused on steel selective pallet storage racks located in areas of retail warehouse stores and other facilities, eventually accessible to the general public.

Assessment of the component behaviour was achieved by means of monotonic as well as cyclic tests on beam-to-upright as well as base-plate connections.

The friction factor between pallets and rack beams is governing the “pallet sliding” phenomenon. This turned out to be the most important effect governing the dynamic behavior of racking systems. Assessment of both the static and the dynamic sliding conditions of pallets stored on steel racking systems was carried out by means of static as well as dynamic tests. More than 1260 Static tests were carried out in both down and cross aisle direction, by means of an “inclined plane” device, by slowly increasing the inclination of the plane, and measuring the sliding of the pallet on the rack steel beams. Influence of the type of beam (namely type of surface finish of the beam), of the type of pallet, of geometry and weight of mass resting on the pallet was investigated.

More than 200 Dynamic shakin- table tests were carried out on a simplified set-up, made of two uprights, connected by two horizontal beams, at approximately 0.30 m from the shaking table. On the beams three wooden Euro pallets were positioned, with concrete blocks rigidly fixed on top. Most tests were carried out with a sinusoidal excitation. A lower bound of the acceleration exists, beyond which pallets start sliding on the steel beams. When acceleration of the mass is lower than such “lower bound”, the pallet “sticks” on the beams, and no sliding occurs. When the “lower bound” of acceleration is exceeded, increasing the acceleration of the input motion results in a lower increment in the mass acceleration, until an “upper bound” is reached of the mass acceleration. Any further increase in the acceleration of the input motion doesn’t affect the acceleration of the mass, that is “free” to slide on the beams. “Sticktion” between pallet and beam is not resumed until a reduction of the acceleration occurs. The “upper bound” of the sliding acceleration is, in general, lower than the static friction factor.

Dynamic behaviour in cross aisle direction is completely different to the one in down aisle direction.

In cross aisle direction, the torsional stiffness as well as the flexural stiffness in the horizontal plane of the beams influence very much the results. Lower bound sliding acceleration as low as 0.1 g was measured, for wooden pallets on hot dip coated steel beams. Upper bound values of the acceleration ranging from 0.3g to 0.5 g were measured depending on the type of beam surface finish as well as on the position of the pallet (lateral or central one).

In down aisle direction, the sliding acceleration is in general higher than the one measured in cross-aisle direction, under the same testing conditions, with a lower bound of the measured sliding acceleration of nearly 0.3 g, and an upper bound of nearly 0.6 g. Results of tests carried out with constant acceleration
and increasing frequency are fully compatible with those obtained in tests with constant frequency and increasing acceleration. Test results confirm that “sliding” is, under severe dynamic conditions, the main factor influencing the rack response. Hysteresis loops were obtained, showing the presence of an energy dissipation through sliding.

A few seismic tests were carried out, adopting three different input motions recorded in Greece during recent earthquakes, and characterized by different durations and frequency contents. Both monodirectional and bi-directional tests were carried out. The obtained results were compared with those of tests carried out with a sinusoidal excitation, showing full compatibility.

One pseudo-dynamic test and a pushover test in the down aisle direction, and one pushover test in the cross-aisle direction were carried out on two bays, three storeys full scale rack models. Re-analysis of the results allowed to draw interesting conclusions on the seismic behaviour of racking systems.

An evaluation of the behaviour factor has been carried out for both down-aisle and cross-aisle directions, with two possible definitions of the q-factor. One value can be identified based on ductility considerations as the ratio of the displacement $v_{\text{max}}$ corresponding to the maximum load carrying capacity of the structure to the yield displacement ($v_y$), being $q_{\mu_{\text{max}}} = \frac{v_{\text{max}}}{v_y} = 3.7$ for the down aisle direction and $q_{\mu_{\text{max}}} = 2.4$ for the cross aisle direction.

With reference to the ductility factor theory, a value of the q-factor based on strength was also defined as the ratio of the ideal strength $F_{\text{max,el}}$ (corresponding to $v_{\text{max}}$ and evaluated on the basis of the initial elastic stiffness) to the maximum load carrying capacity $F_{\text{max}}$, being $q_{f_{\text{max}}} = \frac{F_{\text{max,el}}}{F_{\text{max}}} = 3.1$ for the down aisle direction and $q_{f_{\text{max}}} = 2.1$ for the cross aisle direction.

The results of the pseudo-dynamic test on the rack specimen under down-aisle seismic loading are fully compatible with those obtained on similar specimens, tested under dynamic conditions on the shaking table of the Laboratory for Earthquake Engineering of the national technical University of Athens.

Under pseudo-dynamic conditions the specimen could sustain the series of earthquake events although it didn’t collapse during the last test, performed with PGA = 1.4 g (ePGA=1.5g). Under dynamic conditions specimen A1 (having the masses fixed on the beams in order to prevent sliding, simulating the "quasi-static" conditions of the pseudo-dynamic tests) collapsed under an earthquake with a PGA=1.46g (ePGA=1.41g).

The strain rate effect plays some role in this type of structure; the small movements allowed to the hooks in the holes, in fact, under dynamic conditions result in local impacts that, under increasing number of cycles, may cause cracking either in the hooks or at the edges of the holes.

In any case, comparing the deterioration of the second eigen-frequency (the most excited one) of the specimen tested under pseudo-dynamic conditions with the similar one tested in Athens on the shaking table it was noticed that their trend of reduction is similar. This means that, in general, damage accumulated in the specimen during the two different types of test is similar. From the point of view of the assessment of the seismic resistance and of the damage accumulation of pallet racking systems, pseudo-dynamic tests and shaking table tests are fully compatible, although local damage due to local dynamic effects cannot be reproduced by the pseudo-dynamic testing methodology. Of course, due to the intrinsic quasi-static nature of the pseudo-dynamic testing procedure, no information can be derived about the effects caused by the sliding of the pallets on the beams during a seismic event.

The values of acceleration that were reached during the pseudo-dynamic tests largely exceed the upper bound of the pallet sliding acceleration. So, only full scale dynamic testing will allow a clear assessment of the limit states of pallet racking systems under seismic loading.

In addition to some palletised merchandise tests, earthquake simulation tests were carried out on six full scale rack models of three levels (total height 6.0 m) and 2 bays (total width 3.6 m). Five specimens were tested in down-aisle direction (two of which with base isolation systems) and one in cross-aisle direction.

Effects such as the beam size, presence/absence of pallet sliding as well as of a base isolation system were investigated.
The importance of small structural detailing, to be taken into account when designing pallet racks in seismic areas, was highlighted. Most of the observed failure, in fact, involved failure of bolted or welded connections.

An assessment of the q-factor was performed, based on the experimental results. q-factor values of 3.7 and 2.7 were obtained respectively for the down-aisle and the cross-aisle directions. In the case of pallets rigidly fixed on the beams (in order to prevent sliding) a q-factor of 2.8 was identified. This value, however, might be affected by the excessive stiffening of the structure, associated with the way in which the pallets were connected to the steel beams, generating a sort of “composite” beam. The estimated values are similar to those obtained by re-analysis of the push-over tests carried out on similar structures.

Positive effects of the presence of the base isolators were also highlighted. The two specimens with base-isolation systems resisted earthquakes with a PGA higher than 1.30g without any damage. Assessment of the q-factor for specimen A5 lead to a value of 6.9.

Continuous monitoring has been carried out for a two year period in a warehouse nearby Athens, and relevant information related to accidental impacts as well as service conditions of a pallet rack installation during “everyday” working conditions.

Different aspects of the numerical modelling and of the analysis of rack structures have also been investigated. These activities lead to the proposal of a set of design rules for pallet racks in seismic areas.

In particular, two new features of prime importance for an efficient analysis of racks subjected to seismic action have been included in the FEM software FineLg: springs with hysteretic energy dissipation and sliding point-mass with coulomb friction law. These tools are fully operational, even if some future improvements have already been identified (convergence of the sliding model in the stick phase, test of other friction laws, sliding mass model with numerous contact points). Validation of the models has been carried out for cross- and down-aisle seismic excitation and for braced as well as non braced structures.

Some particular aspects have been emphasized during the calibration procedure: the need for a precise knowledge of the stiffness and resistance of the column bases, the horizontal bracing role played by the pallets as long as they are not sliding or the need for a future calibration of the behaviour of beam-to-column joints regarding rotation around the longitudinal axis of the upright.

Two complementary studies have also been performed in the perspective of normative prescriptions. The first one is a parameter study about the consequences of pallet sliding on the structural response. The study evidences clearly that the horizontal force reduction coefficient is depending on the intensity of the ground motion, on the value of the friction coefficient and on the structural typology (i.e. the structural natural period and the number of loaded levels). The reduction coefficient ranges roughly from 0.2 to 1.0. Additional studies would be necessary to refine these conclusions and calibrate properly the reduction factor.

The second complementary study compared different types of seismic analyses (lateral force method, response spectrum analysis and pushover analysis) applied to a same structure exhibiting significant second order effects. The main conclusions are that, for the considered structure, all analyses provide similar results, provided that second order effects are really accounted for. To this purpose, the use of the approximate amplification factor $1/(1-\gamma)$ is found efficient. Moreover, a verification taking into account second order effects by using a sway buckling length appears strongly over-conservative.

The whole research project SEISRACKS has been an opportunity to analyse the current draft of the normative document pr FEM 10-2-08 "Recommendations for the design of static steel pallet racks under seismic conditions". In particular, a series of items have been identified as questionable. On the base of the knowledge gained during the research project and on engineering judgement, many of these items could be addressed. The final output will be a revised version of pr FEM 10-2-08.
Exploitation and impact of the research results

Actual applications

The main outcome of the SEISRACKS project is a revised version of FEM10.2.08, which will lead to a more uniform quality standard in design of racks in seismic areas. Technical and economic potential for the use of the results

Technical and economic potential for the use of the results

The new FEM10.2.08 Code will allow engineers designing pallet racks in Europe to make reference to a document where issues like ductility, q-factor, pallet sliding, second-order effects are included and addressed on the base of extensive experimental evidence and numerical simulations, and not only on conservative considerations derived from "good design practice".

Patent filing

The SEISRACKS project didn't lead to any patent filing

Publications/conference presentations resulting from the project


Castiglioni G., Comportamento sismico dei beam pallet racks, Tesi di laurea, Politecnico di Milano, October 2006

Ciceri S., Sliding of pallets on storage racks under dynamic conditions, Politecnico di Milano, October 2007


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