THE "SEISRACKS2" EU-RFCS RESEARCH PROJECT "SEISMIC BEHAVIOUR OF STEEL STORAGE PALLET RACKING SYSTEMS"

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This is a presentation of the EU research project SEISRACKS2: "Seismic Behaviour of Steel Storage Pallet Racking Systems", carried out in the period 2011-2014, with a Research Grant of RFCS - RFSR-CT-2011-00031. After a general overview of the project, the activities performed and the main results are summarized.

1. INTRODUCTION

CEN TC344 has recently issued a normative document EN 16681: 2016, "Steel static storage systems. Adjustable pallet racking systems. Principles for seismic design".
The background knowledge for the development of EN 16681 is mainly based on European Racking Federation (ERF) FEM 10.2.08 v 1.04: 2011 and on recent research works. FEM 10.2.08 (v.1.04: 2011) is fundamentally based on the experimental results obtained within the frame of the EU-RFCS project SEISRACKS 1. However, in 2011, such a document was still far from becoming a Euronorm (EN) due to remaining lacks of knowledge leading to conservative design rules and consequently to strong technical limitations when designing static steel pallet racks with respect to seismic safety.
The objective of the SEISRACKS2 project was to solve these limitations by increasing knowledge on actual structural behaviour and ductility and to assess design rules for earthquake conditions. The main expected outcomes of the research were:

1. Detailed reports on the different aspects investigated;
2. Validation or invalidation of the rules in the current draft of FEM 10.2.08, v 1.04: 2011;
3. Improvements and extension of the current rules in order to optimize the seismic behaviour of structures designed according to European rules;
4. Definition of standardized experimental procedures to qualify structural elements of rack structures to be used in seismic areas.
5. Development of a software tool for the design of rack structures under seismic loads

Partners of the project were: Politecnico di Milano, Architecture, Building and Construction Department (Coordinator) (I), University of Liege, Depatment ArGenCo (B), RWTH Aachen, Institute of Steel Construction (D), National Technical University of Athens, Department of Steel Construction (GR), SCL Ingegneria Strutturale (I), MODULBLOK S.p.A. (I), NEDCON Magazijninrichting B.V. (NL), FRITZ SCHAFER GmbH (D), STOW INTERNATIONAL N.V (B) and CCS COMPUTER CONTROL SYSTEMS S.A. (GR)

The project was organized in 8 work-packages, whose objectives were respectively the following:

2. WP1 – DEFINITION OF CASE STUDIES

The reanalysis of the FEM 10.2.08 pointed out the relevant weaknesses of the Norm summarized below, requiring the improvement of the knowledge of important aspects of the seismic behaviour of racks:

1) The assessment of the design spectrum modification factors, which are introduced to take into account the dynamic interaction between the rack and the supported loads
2) The applicability of low ductility and ductile design concepts for thin gauged cold formed profiles
3) The assessment of the damping of the rack structures
4) The design of racks with values of the interstorey drift sensitivity coefficient θ larger than 0.3, which is the maximum permitted by the Eurocode 8
5) The validation of the proposed formulas for the calculation of θ
6) The validation of the method of modal response spectrum analysis taking into account 2nd order effects
7) The assessment of the consequences of the use of bracing schemes for the upright frames, commonly adopted in constructions but not fully complying with the rules of Eurocode 8
8) The assessment of the behavior factors indicated for low ductility design concept
9) The lack of Normative prescriptions for floor fixings in seismic conditions; this item should be resolved by the next publication of an ETAG document on the subject.
10) The assessment of arrangements for the improvement of the performance of beam-end connectors (to be secured with a bolt)
11) The assessment of the pallet beam design for horizontal bending and in particular the quantification of the positive effect of friction between beam and pallet
12) The investigation of problems related to the construction of upright frames, assessment of their effective shear stiffness and strength to be adopted in design calculations
13) The evaluation of the effects of baseplates detailing on the behavior in cross aisle direction
14) The assessment of the effects of eccentricities of the schemes due to construction
15) The definition of testing procedures to derive the design parameters from tests, applicable for seismic design, or the assessment of correlations rules in order to use the data obtained from tests designed for static conditions
16) The reanalysis of the effects on the design of the mass distribution on the rack, and the determination of the relevant ones for the design
17) The need for definition of proper methodologies for the design of racks on suspended floors
18) The assessment of proper rules for the ductile design concept

The comparison with ANSI-RMI-2008 edition and FEMA 460 was carried out considering mainly the low ductile design concept of FEM 10.2.08, which is the most relevant, and was focused on the following aspects:

1) The definition of the design seismic action, that is established on very similar bases
2) The Importance factor, that is applied in different ways; it results that FEM 10.2.08 allows considerably greater seismic risk in some situations
3) The definition of the seismic mass, that in both cases is based on a specified weight of the unit load, which is corrected by coefficients in both cases taking into account the normal "filling" of the rack in operations and the interaction between rack and unit loads; regarding

the last one, FEM 10.2.08 uses a more analytical approach allowing to differentiate the effects based on the type of stored product, intensity of the earthquake, etc.

4) The behavior factors q defined in FEM 10.2.08 are mainly related to the structure and don't take into account the interactions with the unit loads, an effect included in the response modification factor (R) approach of RMI-2008

5) Direct comparison between q and R is not correct; R should be compared to $q/(ED_1*ED_3)$. Generally FEM 10.2.08 values are lower, except for some cases with strong earthquakes

6) The reference method of analysis for RMI-2008 is the lateral force method of analysis (LFMA), while FEM 10.2.08 assumes the modal response spectrum of analysis (MRSA), allowing the LFMA as a simplified procedure under conditions ensuring that modes with lower periods have negligible relevance. RMI-2008 allows also a displacement-based method referenced in FEMA-460, and MRSA appears to be used in the design practice even if not mentioned in the Code.

7) Second order effects are considered by FEM 10.2.08 in all cases in which θ exceeds 0.1 either directly or with a simplified method, while RMI-2008 requires, in the reference design procedure, to consider the 2nd order amplification only for the evaluation of the rotational demand of the connections for unbraced racks in down aisle direction

8) The effects of the seismic action occurring in the 2 main directions is not required to be combined by RMI-2008, while it is mandatory for FEM 10.2.08

9) In addition to the prescriptions of FEM 10.2.08, the RMI-2008 requires to verify against overturning the rack fully loaded, with 67% of its rated load capacity.

10) Control of the rotational capacity of the connections: it is required by FEM 10.2.08 only in ductile design concept, while it is always requested by RMI-2008 when the flexibility of the structure is considered to reduce the seismic action in the unbraced directions and in any case in seismic design category D

11) In low ductility design approach FEM 10.2.08 allows using the test protocols of the EN 15512, while cyclic tests are required to assess the rotational capacity of beam-end and floor connectors for ductile design; the testing protocol is not well specified and difficult to apply. RMI-2008 uses a quite simple testing protocol for the beam-end connector, but nothing is specified for the baseplates.

12) FEM 10.2.08 provides a detailed procedure for the design of pallet beams under seismic actions, taking into account the effects of friction between unit loads and beams; no specifications are provided by RMI-2008 for seismic conditions

13) Both FEM 10.2.08 and RMI-2008 consider concentric bracing schemes; FEM 10.2.08 differentiates the behavior factor and the design rules depending on the pattern of the bracings, while there are no requirements in RMI-2008

14) FEM 10.2.08 requires, in low ductility design approach, bolt's strength to be 1.2 times larger than the bearing strength, while Eurocode 8 design rules need to be applied in ductile design; requirements are not prescribed by RMI-2008

15) For the beam-end connector stiffness, FEM 10.2.08 in low ductility design approach allows using the values obtained from tests according EN 15512; RMI-2008 requires using the connection secant stiffness derived from the moment-rotation curve obtained from static test consistent with the applied base shear and resulting displacements (this implies iterative design procedure).

Eight models, covering a wide range of constructive typologies and different static schemes, were identified by the IPs (Industrial Partners), focusing on technical solutions for which current FEM10.2.08 Recommendations of were felt less efficient.
Design parameters and geometrical properties to be used in the tests and in the numerical analysis are defined in Table 1; 4 types of upright frames as well as 4 type of longitudinal frames were identified, respectively based on the geometrical pattern of the diagonals and on the diagonals' position and connection (Figure 1). Table 2 shows the configurations tested (made by the composition of frame's type and diagonal's type) and the IP that provided them.
The system of longitudinal bracings is composed of vertical and horizontal bracings, at each load level. Three types of horizontal schemes are identified, whose components and connection details are shown in Figure 2, while for the vertical bracings the following options are identified:
- X bracings with extra uprights and extra horizontal elements; diagonals working only in tension (made by flats or rounded bars with turnbuckle); diagonals working in tension and compression made either by tubes or channels;
- Bracings made by cables with turnbuckle and post element.

Table 1 Geometrical configuration of the racks and design parameters

Height of the frame	8 m
Number and height of the beam levels	4 levels @ 2 m nearly
Length of the rack	6 bays; 3 pallets per beam
Max Acceleration (Low Seismicity)	0.12g-0.15g (Soil C, Type 2)
Max Acceleration (High Seismicity)	0.25g-0.30g (Soil C, Type 1)
Pallet mass	800 kg
Ed2	1.0
β (spectrum cut-off)	0.2
Friction coefficient	0.375 (wooden pallet on steel beams – normal warehouse conditions)
CμL	0.67
CμH	1.5
Importance Class	2
Design life	30 years $\rightarrow$ γ_1=0.84 (normal use of the racks)
Ed3	0.67

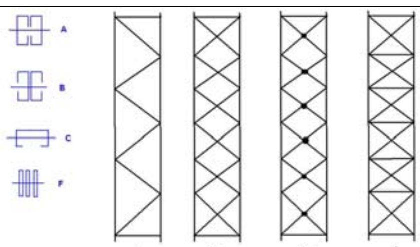

Fig. 1 Upright frames to be used in the tests and numerical analyses.

	A	B	C	F
1	IP C (3)	IP D (3)	IP B (3)	
2a		IP C (3)		
2b		IP D (3)		
4				IP A (2)

Table 2 Configuration of upright frames provided by the different Industrial Partners.

Table 3 summarizes the configurations to be tested (made by the composition of vertical and horizontal bracing type), and the IP that provided them.
Each partner was asked to design, according to daily practice and following FEM recommendations, two configurations: one unbraced for low/medium seismicity, and one braced for high seismicity.
The number of cases was determined by the number of possible component tests and full scale tests. Nevertheless, some IPs studied more numerical cases, to compare braced and unbraced solutions either in low or in high seismicity and to compare more configurations of upright frames; in total the number of numerical case studies is 11:

- 4 cases unbraced for low seismicity
- 1 case braced for low seismicity
- 1 case unbraced for medium seismicity
- 4 cases braced for high seismicity
- 1 case unbraced for high seismicity

The solutions designed for the upright frames resemble either the normal practice or new proposal adapted just for seismic conditions. The aim was to compare the different ways of connection of the diagonals and to confirm which was the most effective one in seismic conditions; to confirm the q factors actually permitted by the Norms; to identify the weaknesses or skills of commonly used constructive typologies.
The solution designed for vertical bracing resemble common practice (although the solution with cables is less used); a comparison of different design possibility and different stiffness of the frame - braced and unbraced – allows for identification of the most effective solution with respect to the seismicity level.
The design was based on low ductility rules; the checks of components was performed according to EN15512 and EC3.
The reference seismic analysis method was the modal response spectrum analysis with SRSS superposition of the modes (or multimodal analysis). Second order effects were considered when the interstorey drift sensitivity coefficient was $\theta > 0.1$

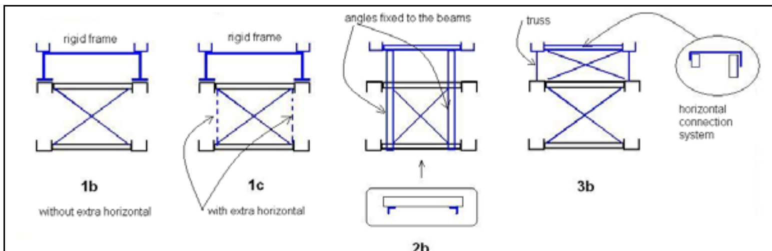

Fig. 2 Horizontal bracing system configuration.

	Cables	X Bracings Tension only	X Bracings Tension & Compression
1b			IP D
1c		IP A	
2b	IP C		
3b		IP B	

Table 3 Bracing configuration provided by the different Industrial Partners.

3. WP2 – COMPONENT TESTING

3.1 Tests on beam-to-upright connections

Tests were performed aimed to the assessment of the moment rotation characteristic of the beam to upright connections as well as of the influence of loading conditions.
The setup of the down aisle tests is different from the one proposed in EN 15512 (2009) for beam end connector tests, and was developed within the scope of this research project to provide information about the plastic deformation capacity under realistic support and loading conditions.
The test setup represents one shelf of a rack system with a bay width of 2.70 m, commonly used for storage of 3 pallets. The height of the frame is 1.00 m and the beams are installed at half height. To load the rack with pallets two frames – a front and a back frame - are needed. The front frame is made of the rack parts (uprights and beams) to be tested. The uprights are perfectly hinged at the supports and at the top so that the sway of the frame is constrained by the beam to upright connectors only. The back frame is a kinematic frame made of hollow sections with the beam perfectly hinged to the columns (Figure 3).

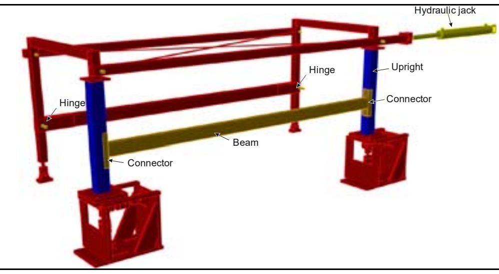

Fig. 3 3D Test setup

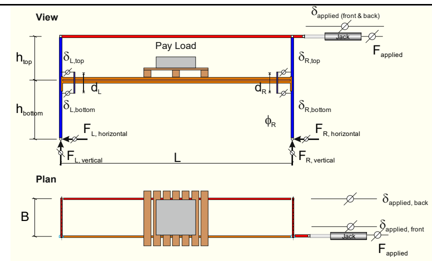

Fig. 4 Sketch of test setup

The applied force $F_{applied}$ was imposed by an hydraulic jack. Along with the applied force the reaction forces at the supports of the front frame, the sway of the frame and the rotation of the beam ends were measured during the tests (Figure 4). The payload was applied by loaded pallets, as in a realistic loading situation. Influence of loading on the connector behaviour is investigated by testing the racks with different pay load (0%, 50% and 100% of service load). For each load case and for each producer (4 producers) one monotonic and one cyclic test were performed. Under deformation controlled conditions by means of monotonic push over tests a load deformation curve was generated to derive the control values for the cyclic tests. The cyclic tests were also deformation controlled where the applied deformation amplitudes were related to the reference deformation e_y from the monotonic tests in accordance to the ECCS cyclic testing procedure. One cycle with the amplitude factor 0.25, 0.50, 0.75 and 1.0 and 3 cycles with the amplitude factor 2, 3, 4,.. were carried out until failure. Tests results provided information related to Failure modes, Influence of payload, Differences between cyclic and monotonic behaviour, Moment rotation characteristic, Compatibility of these test results with those derived from EN 15512 standard tests, Effectiveness of safety bolts.

Tests were carried out for the assessment of the moment rotation characteristic of the beam to upright connectors and of the influence of pallet loads on the cross aisle deformation behaviour of the beams. It was expected that the influence of pallet loads was mainly governed by the friction between pallet and beam and the stiffness of the pallet. The influence of shear forces transferred from the beams on the behaviour of the beam to upright connection was expected to be low or negligible.

In addition to the test program initially included in the proposal, frictions tests were performed to allow determining the influence of pallets on the cross aisle deflection resistance. The friction tests were not performed in accordance with FEM-rules (FEM 10.2.08, 2010) where the pallet beams are inclined until sliding of the pallet. In the tests performed here the beams remain in horizontal position while the pallet is moved by a measured external force. This test setup allows for the distinction of adhesive and sliding friction (Figure 5).

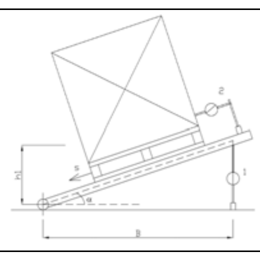

Fig. 5 Setup of frictions tests (left: FEM tests; right: performed tests)

The adhesive friction coefficient is the maximum friction value when the pallet starts sliding while the sliding friction coefficient is the mean value during sliding (Figure 6).

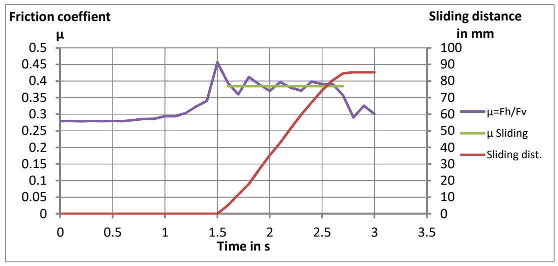

Fig. 6 Example for derivation of friction coefficients from tests

In total, 27 friction tests were performed on the pallets representing the 50% of the maximum service load (approx. 400kg). The tests started on beams with untouched surface (producer IP C) and were continued without changing the beams during testing. Pallets were placed on the beams and pulled in longitudinal direction. When the maximum displacement of the displacement transducers was reached the pallet was lifted up and moved backward to the starting position of the next test. It was observed that the friction coefficient on the beams with untouched surface is significantly lower than the coefficient on the scratched surface. Testing started with the 412kg pallet (tests 412kg-1 and 412kg-2). After testing the other pallets the tests on the 412kg pallet were repeated (tests 412kg-3 and 412kg-4). The sliding coefficient of friction increased from 0,34 to 0,49.

Figure 7 summarises the results of the friction tests on pallets with 50% of the maximum service load.

The test setup for the cross-aisle specimens represents a typical rack detail with a bay width of 2.70 m: Two pairs of uprights allow the installation of beams that can be loaded by standard pallets.

The end frames are detailed such that one frame is movable while the other end frame is fixed and the reaction forces *R* are measured. In the tests a horizontal displacement δ is applied to the movable end frame and the corresponding force *F* is measured (Figure 8). Additional measures are the transverse rotation of the beam in the connectors, the shear forces *H* transferred from the uprights to the beams and the lateral displacements *d* of the beams between the pallets.

The measurement of the rotation angles and the global displacements allow determining the moments in the connectors by application of mechanical rules (Figure 9). The result may be cross checked by comparison with the measured shear forces in the case of unloaded tests. In case of loaded pallets, friction of pallets may influence the deflection of the beams such that mechanical rules for determining the moments do not apply. The change of the deflection line of the beams is determined by the displacement measurement d_i in the gaps between the pallets. Together with the shear forces H, the moments and rotation in the connections it is possible to determine friction effects of the pallets.

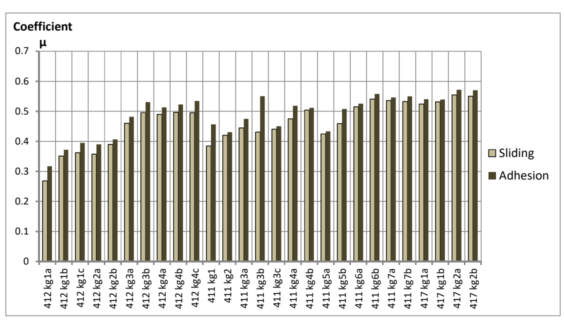

Fig. 7 Test results pallets with 50% max service load

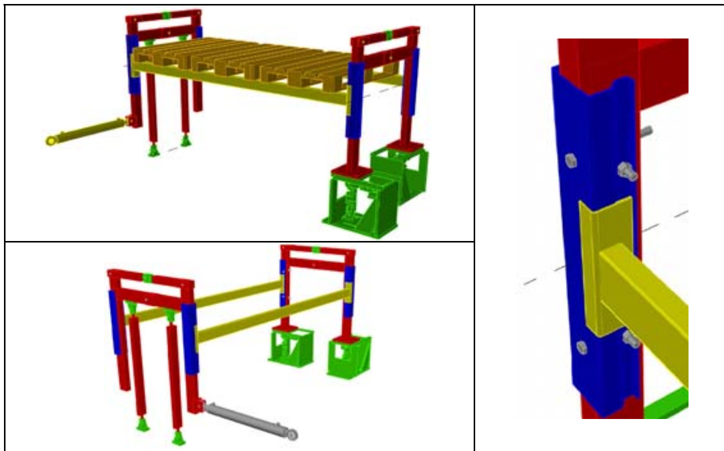

Fig. 8: Drawing of setup

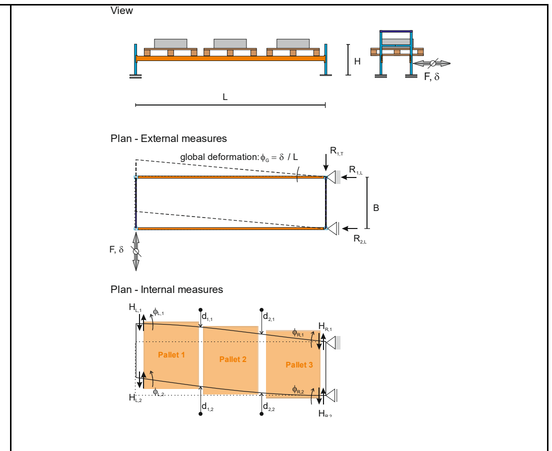

Fig. 9 Sketch of test setup

For the tested products it can be stated that the moment resistance in the connectors for cross aisle bending was negligible and pallet friction mainly controls the resistance in the cross aisle direction.

3.2 Column-base tests

The column base tests were carried out for the assessment of the moment rotation characteristic of the connection between the upright and the column base.
The connection of the base plate to the ground (concrete slab) was not within the scope of these tests to prevent the dowels to be tested instead of the upright to base plate connection. Furthermore, the possible variations of concrete slabs (thickness, reinforcement and strength) on the compression side of the connection and variation of applicable dowels (type and producer) on the tension side of the connection was too large to obtain results that could be transferred to real projects. The characteristics of this part of the connection can be obtained for a specific case application on the basis of the dowel characteristics (provided by dowel producer) and concrete characteristics and easily added to the moment-rotation characteristic from the tests presented here. Figure 10 shows the stiffness and resistance component of the complete base plate connection. The test setup shown in figure 11 represents a column base that is rigidly connected to the floor. On top of the upright there is a steel plate where the horizontal load is applied. To allow for horizontal deflection of the top steel plate a pendulum is installed between the top plate and hydraulic jack that applied the vertical force. The vertical force is controlled to be constant over the duration of the test while horizontal displacements are superimposed by a horizontal hydraulic jack measuring the applied force. Horizontal forces from second order effects are also measured by the horizontal devices. In the project proposal it was intended to test the column bases in down aisle and in cross aisle direction. Figure 12 shows the difference of the loading of the upright for the different directions: while the down aisle loading causes bending in the base plate connections, loading in cross aisle direction cause mainly normal forces at the column bases as the columns are connected by a framework to one section and the uprights act as flange. Bending in the baseplate connection due to cross aisle bending seems thus to be negligible.

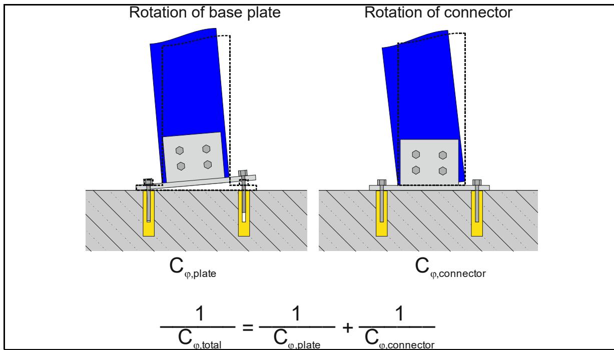

Fig. 10 Components of stiffness and resistance of the fixing upright to slab

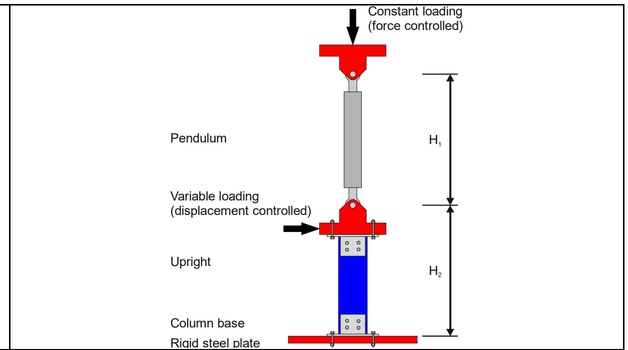

Fig. 11 Sketch of principle test setup

3.3 Substructure tests

Substructure testing was carried out on cross-frames and braced longitudinal frames under horizontal loading, aimed to the definition of standardized procedures for the assessment of the local ductility of cross-frames and of the longitudinal frame bracing properties.

4 substructure types have been identified based on the geometrical pattern of the diagonals as well as and 4 types of diagonals' position and connection (figures 1 and 2). 9 case-studies have been prepared (2 by each IP plus one extra for one Partner) with the objective of getting a wide range of situations (design for low/moderate/high seismicity, D/Z/X type of cross bracing).

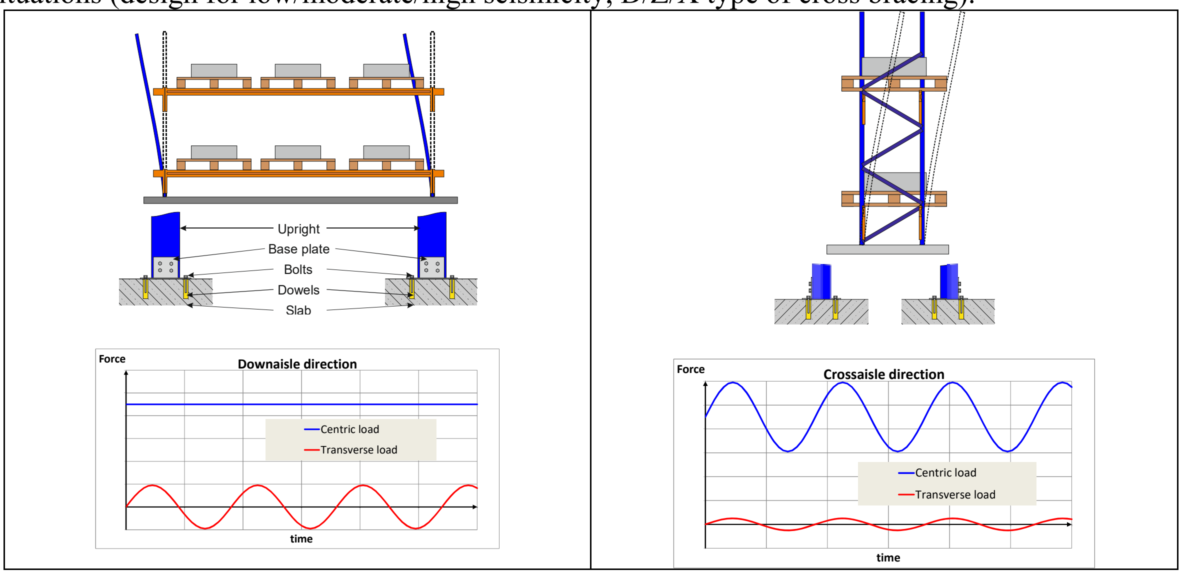

Fig. 12: Directions of rotation and loads on the base plate connection

The total number of tests on each substructure type is:

- for unsymmetrical frames: 3 tests = 2 pushover + 1 cyclic;
- for symmetrical frames: 2 tests = 1 pushover + 1 cyclic;

4. WP 3: WAREHOUSE TESTING

This work package included the activities for operational monitoring of a real warehouse, identification of the linear dynamic properties of racks, and identification of dynamic properties of pallets/merchandize.

4.1 Continuous monitoring of a warehouse during daily activities

During the previous Seisracks1 project, an installation near Athens was continuously monitored to record rate of occupancy, operations and accelerations. As the measurement system was still on the site, despite a number of accidents (more or less fortuitous) occurred in the meantime, it was reactivated in order to obtain data from continuous monitoring during the whole Sesiracks2 research. Figure 13 shows the positions of the accelerometers on the rack. Data were recorded continuously on site, and transmitted to the remote server of NTUA via wire-less. Because of the economic crisis that in the period 2011-2014 was causing a drastic reduction in the commercial activities in Greece, also the activities within the warehouse were reduced. It was hence to be expected that less goods were present on the racks, and that, due to the smaller request, also the picking activities were reduced. Despite all this, a large number of data related to everyday activities was recorded at a 200 Hz sampling rate, and re-analysed. Because of the enormous amount of recorded data, it was decided to re-analyze only the data related to "significant" events, in which the absolute value of the peak acceleration exceeded 0,05*g. With reference to the available "significant" events only, the frequency of occurrence of acceleration peaks was obtained (as shown in figure 14). Through this type of re-analysis performed on the global set of all recorded data, it was possible to derive indications on design values of acceleration to be considered, in order to keep into account the storage and retrieval (S/R) activities.

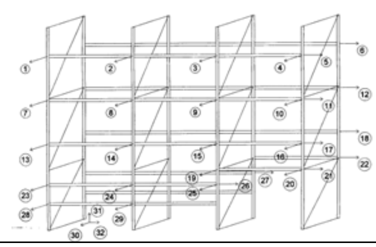	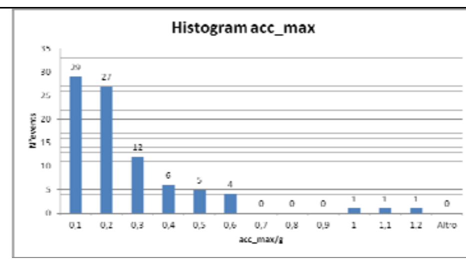
Fig. 13 Position of accelerometers on the rack	Fig. 14 Frequency of occurrence of acceleration peaks

For each event, by re-analyzing the acceleration time histories, indications were also derived on the deformation of the structure under impact loading due to the S/R actions, and its modal shapes.

4.2 Warehouse testing

The warehouse testing aimed at identifying the linear dynamic properties of racks on the basis of free response tests. The objective consisted in defining the range of periods and damping of a real structure and to calibrate numerical models. These tests were performed on site on existing structures in active warehouses chosen by the IPs (one warehouse for each IP). The geometrical and material properties of the structures to be monitored were chosen very similar to the case studies of the research.

The signals were decoupled in longitudinal and transversal vibrations. It can be assumed that the rack will deform mainly in shear following the vibration modes shown in figure 15.

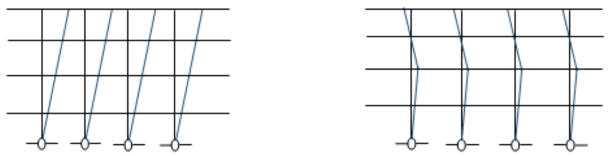

Fig. 15 Vibration modes.

To distinguish and measure both vibration modes it was necessary to place the sensors at 2 levels; one at mid-height (level 4250mm) and one on top of the structure (level 8540mm). The warehouse chosen by partner D is divided into 2 parts; one with a lot of fork-lift traffic and another part with goods that were stored for a longer period and where the traffic was reduced. In order not to disturb the activity of the store, it was decided to test a single-entry rack of this second part of the warehouse.
The transversal vibrations were measured in 3 different sections on the length of the structure. The longitudinal measurements were taken at the same place as for the transversal tests. An additional biaxial sensor was placed on top of the rack at the same place as the middle mono-axial sensor.
The 12th upright was pulled and pushed several times transversally before releasing the impulse and let the structure find its position after absorption of the movement. Several of these free response tests were carried out first at the 2^{nd} level (4.0 m from ground floor) and then at 4^{th} level (8.0 m from ground). Natural frequencies and mode shapes were obtained by stochastic subspace identification method. The results provided by the identification procedure were therefore scrutinized, case-by-case, for the specific results of each identification. In particular, 3 possible natural frequencies were identified (Table 4) with the corresponding mode shapes. For both types of excitation (2^{nd} level or 4^{th} level), the obtained results are in good agreement (Table 4).

Table 4 Transversal measurements

Shaking at level	Mode 1 Freq. (Hz)	Mode 2 Freq. (Hz)	Mode 3 Freq. (Hz)
2 (4.0 m)	1.49	2.07	2.34
4 (8.0 m)	1.59	2.01	2.42

The first mode corresponds to an in-phase movement of the 3 measured cross-frames in the same transverse direction. The second mode corresponds to a kind of local torsion in the rack, which can be termed as a "snaking mode". This mode cannot be understood as a global torsion otherwise it would have resulted in excessive amplitudes at both ends of the rack. The third mode shape is similar to the snaking mode mentioned for the second mode.

Table 5 Longitudinal measurements

Shaking at level	Mode 1 Freq. (Hz)	Mode 2 Freq. (Hz)
2 (4.0 m)	0.62	1.87
4 (8.0 m)	0.60	1.80

Two tests were performed, by shaking (with a fork lift truck) the rack longitudinally at the 2nd and at the 4th level. The forklift pushed the rack away from its rest position in the longitudinal direction before releasing it suddenly. This way of exciting the structure mainly provided a response in the lowest mode. For each level, multiple loading repetitions were performed. The natural frequencies, obtained by stochastic subspace identification method, are shown in Table 5.

The dynamic properties (frequency, damping) of a large range of stored merchandizes were identified on the basis of a push-by-hand excitation on top of the stored good with a quick release or an impact given by the human waist. Several such shocks have been given in both directions of the pallets and the vibration and damping have been recorded.
For each of the chosen pallet a tri-axial sensor was fixed on top of the goods in order to measure the vibrations along both axes (x and y) parallel to the pallet edges. From each signal, natural period and damping ratio were identified for each reference axe, and then averaged to give the "global" property of the pallet.
The natural frequency of the stored goods on pallets varies from 3.25 Hz to 6.21 Hz; the damping ratio varies from 3% to 7%.

5. WP 4: FULL SCALE TESTING

This work package aimed to assess the global behaviour of full scale racks in down aisle and cross aisle directions. Specimens for longitudinal tests have 4 levels (8.0m) and 2 bays (6.0 m). For each producer 1 braced frame, and 1 unbraced have been tested in down-aisle direction and 1 in cross aisle direction, for a total of 12 full scale push-over tests. Tests were carried out under force controlled conditions, and forces were introduced at each level by means of hydraulic jacks having a 100 kN capacity and a 1.0 m stroke.
All the tests were performed outdoor, in a testing facility owned by Marcegaglia Buildtech and managed by Politecnico di Milano where full scale racks could be easily mounted and loaded (Figure 16).

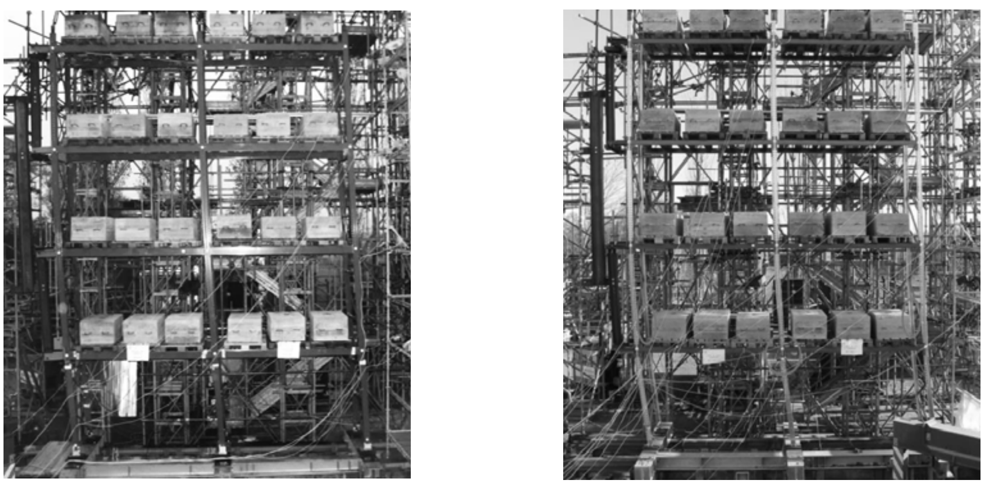

Figure 16 Deformed shapes at the last loading increment of IP A (left) and IP C (right) structures

5.1 Unbraced specimens

Global behaviour of specimens IPA1 and IPB1 was moderately ductile. These racks did not have a sudden local failure, thanks to progressive lateral bending of their uprights, and accumulation of plastic deformation in their upright bases and beam-end connectors. Ultimate deformation of both specimens IPA1 and IPB1 was greater than their yield displacement values; 5 times greater in case of IPA1, 2.5 times greater in case of IPB1. When specimens IPA1 and IPB1 were unloaded after tests, their residual permanent plastic deformations were greater than the elastic one; 3.5 times greater in case of IPA1, 1.5 times greater in case of IPB1.

Specimens IPC1 and IPD1 both had global collapse due to soft storey mechanism formed at their first levels, since their base-plate connections lost their initial stiffness and behaved like hinges after reaching a certain level of horizontal load. This loss of stiffness caused the formation of plastic hinges on the uprights and beam end connectors just below the first level, and after this point, the specimens could not sustain any more horizontal load. Specimen IPD1 had a soft story mechanism because of a plastic hinge formed in its central upright, and distortional buckling occurred in the lateral ones. Distortional buckling, also known as "stiffener buckling" or "local-torsional buckling", was caused by the rotation of the flange at the flange/web junction, in members with edge stiffened elements. At the end of the test, this phenomena arose along the first and second level upright's flange; in fact the distortion in the uprights was favoured by the absence of rigid constraints that could have been given by the bracing system joints in cross aisle direction.

Specimen IPC1 had a soft story mechanism, due to the distortional buckling centralized at its first level, while its upper stories remained undamaged with little drifts. During the test, this phenomena arose locally on the upright's flanges in direct contact with beam-end connectors surface; in fact the progressive increase of deflection in the upright, favoured by non-rigid constraints (base-plate behaving as a hinge), was not accompanied by a consequent deformation of the Beam-end connectors. The resulting transmission of strong localized pressures led to a distortion of upright section in correspondence with the joint, and the subsequent formation of a soft story mechanism. Not all uprights were subject to this phenomenon. Those, where the upright bracings converge in the beam-to-column connection, did not present substantial deformation. Indeed the presence of the upright bracing in the node prevented inwards rotation of the upright flange.

5.2 Braced specimens

Among all four specimens, only IPA2 showed a ductile performance, without a sudden local failure in its elements. IPB2 and IPD2 both have had local failures, after which they are unloaded. IPC2 was a very resistant rack, its testing had to stop because the hydraulic jack reached its maximum limit.

IPA2 showed a ductile performance thanks to the exploitation of plastic resources of its vertical bracings that are efficiently connected to the rack joints. IPA2 was able to sustain a base shear nearly 15% greater than its yielding value, and its ultimate deformation is almost 3,5 times greater than its yielding value. When this specimen was unloaded, its residual permanent plastic deformation was equal to 2 times the elastic one.

Specimen IPB2 suddenly failed due to the shear failure of one of its bolts that connected the vertical tension bracing to the bracing base-plate at ground level. When this bolt failed, all the rack members were still in their elastic phase. The global stiffness of the rack had a sudden decrease, when the compression diagonals were buckled. After this point, no plastic behavior occurred until the shear failure of the bolt, which did not permit the rack to take any advantage of plastic resources of the diagonals.

For specimen IPC2, test was stopped due to the capacity of hydraulic jack. Nevertheless, design elastic limit of the structure was reached at the end of the test. Pushover curve has almost an elastic branch until the end of test, which means the deformations of rack members mainly remained elastic. Slight plastic global deformation was observed from the descendent branch of the curve showing small residual deformation. In the substructure tests performed within WP2, this bracing element had a brittle rupture at its connection at higher loads.

Specimen IPD2 had a sudden local collapse caused by a connection failure of its tension bracing at the first level. This failure was caused by the bending of the bolt that was connecting the first level bracing to the bracing joint at the first floor. Progressive plastic deformation until failure, concentrated in the bolt and the connection plates caused a slight global plastic performance showing that, if the connection were more efficient, the specimen could have a more ductile performance.

In all the cases, a moderate global torsion was observed due to asymmetry in plan. Though it was limited thanks to the horizontal bracings used at each level.

6. WP5 – NUMERICAL ANALYSES

6.1 Classical analysis at design stage

Numerical models of the 8 case-studies were prepared according to the daily practice using conventional analysis methods (push-over and time history) and recommendations of FEM 10.2.08.

6.2 Local Analysis of the members and connections

The connections show a highly nonlinear behaviour due to various phenomena; these were simulated as nonlinear elements. Inelastic structural properties were treated by using nonlinear link elements in the model. The input data of the link elements were obtained from experimental data provided by each IP. There are two main connection types in the structures; beam to upright connections and base plate to the upright connections. Base plates provide stiffness in the down aisle direction due to the rotational stiffness of the specific assembly, while in the cross aisle direction they are considered simple hinges. The average stiffness and the strength of the connection changes under different axial loads.

6.3 Push over analysis

An incremental static analysis was used to determine the force-displacement relationship, and the capacity curve, for the racking systems provided by the 4 different industrial partners. In the numerical model, horizontal loads coexist with vertical loads, in order to achieve a real loading situation. The analysis is nonlinear in both the geometry and material terms, while it can provide solutions after the bifurcation points or a descendant curve as well.

The structures under investigation were specific design examples of storage racks provided by 4 industrial partners (IP). The structures were separated into two main categories, the braced and unbraced racks. The braced racks are commonly designed for high seismic zones, while the unbraced racks are designed for both medium and low seismic zones. Although this is the principle, there are exceptions to the rule, where unbraced racks are strong enough to deal with a strong earthquake. The cross aisle direction is always braced in a varied way which also depends on the seismicity of the region where the rack is placed to. Two characteristic braced systems for the cross aisle direction are the X bracing and the D bracing. The major difference between the two models is not only the number of the used diagonals but also the symmetry of the configuration. The distribution of the horizontal loading is a crucial point for the push-over analysis. The most common distribution has the shape of the dominant dynamic mode. For the down and the cross aisle directions the horizontal loads are obviously different. For the unbraced systems the dynamic modes on both two directions are dominant, as the behaviour of the system is clear, uncoupled and pure translational. On the other hand, the braced racks which have the bracing system placed eccentrically in a rear plane have a severe torsional behaviour, a situation that creates a coupled global behaviour. Thus, the principal modes of the down aisle direction of a braced rack have a maximum participating mass ratio of 60%. As a result, in a pushover analysis for such systems, a multimodal distribution of the horizontal loads should be used for the down aisle direction. However, the demanded number of modes to collect 90% participating mass from the active modes is in case of a braced model are more than 40. This makes the multimodal analysis complicated and controversial. Since the full scale tests performed by POLIMI had a triangular horizontal loading, the appropriate loads for the pushover analyses was an acceleration, applying a constant force in the height. It is not exactly the real situation but it is numerically stable enough.

6.4 Time history analysis

The seismic behavior of racking systems in down aisle and cross aisle direction was investigated by means of incremental dynamic analysis (IDA) with the statistic evaluation procedures of FEMA P695. Two unbraced down-aisle frames, and two cross-aisle frames were considered. Numerical analyses (6 bays-4 levels) of racks were based on the calibrated numerical models (2 bays, 4 levels). The reason of examining only these models was because the other two unbraced racks of the project did not present experimentally any ductility, and hence it was not useful to investigate statistically their seismic behavior. On the other hand, the braced models present the need of 3D models, which are computationally problematic, as the IDA includes the run of thousands of 3D dynamic analyses and a huge amount of results to be edited. In the cross aisle direction the 2 main types of bracing systems are examined; the X and the D bracing type. For IDA, 44 real ground motions were used. Behaviour factors of each system have been estimated.

The general conclusion from these numerical analyses was that pushover analyses seems to underestimate the actual ductility of the system, while IDA leads to higher q values. However the IDA procedure needs a lot of different archetypes in order to derive an overall quantitative conclusion. IDA lead to higher ductility values for down aisle unbraced frames, than the ones proposed in the design codes. Though, it would not be safe to generalize this result to all systems, remembering that other two unbraced racks tested in research had almost zero ductility. The general discrepancy of the results shows that, to estimate behaviour factor of rack structures, detailed analysis and tests should be performed, because every manufacturer has his own particular approach to design racks, which have a lot of particularities that can hardly be estimated in simple design methods.

7. WP 7: SOFTWARE TOOL DEVELOPMENT

Current analysis method for racks under seismic loading (the multimodal spectral analysis) does not allow to consider geometrical nonlinear effects due to the linear superposition of the modal responses. To overcome this limitation and allow for a multimodal spectral analysis with due consideration of 2^{nd} order effects, a new procedure was developed and implemented in a specific software package: a multimodal spectral stepwise non-linear analysis. It offers the following advantages against other approaches:

- It uses seismic data that is well defined in every national design code (parameterized spectra, importance categories, regional acceleration limits etc.).
- It uses fast, reliable and reasonably accurate algorithms and procedures (modified Newton-Raphson, CQC).
- It can be combined with existing software and it is easy to use and understand even for users without specialized knowledge (simple presentation of variables and data, user is not required to know the implementation details).

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