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SEISMIC BEHAVIOR AND DESIGN OF STEEL STORAGE RACKS

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SUMMARY

An experimental and analytical study currently in progress at the Department of Structural Engineering of the Politecnico di Milano is devoted to the development of simplified rules for the design of pallet storage steel racks in seismic zones. The work presented here analyzes the sensitivity of different rack configurations both to static equivalent lateral loads and seismic excitation in the longitudinal direction. The analytical models include a semi-rigid formulation representing the partial continuity of the beam-to-column joints, calibrated according to the experimental results. First indications for improving the quality of the seismic response are discussed.

INTRODUCTION

Rack systems for pallet storage are important industrial structures by number and commercial value, yet they have been considered only recently in studies aiming at defining practical design rules for their safe use. These structures are always composed with metal elements: the fabrication of rack structures constitutes, indeed, an important application of cold formed steel products.

The layout of these structures is similar to the more traditional framed systems employed in the civil and industrial construction industry, yet the two differ totally in the geometry of the components and in the typology of the connections between vertical and horizontal elements. In general, for rack structures the beams have boxed cross section and the columns have thin walled open section. Columns present regularly distributed holes along the profile, in order to accommodate the hooks present at the beam ends, realizing connections without bolts or welds, as may be seen in figure 1.

The need for continuously loading and unloading the shelves in service has induced designers to avoid bracing elements in the longitudinal direction of the racks. Therefore, in many cases lateral stability in this direction is ensured only by the connections between beam and column and by the constraint offered by the base of the columns.

In the transversal direction, instead, columns are linked by diagonals and horizontal elements in a truss-like mode. This configuration supplies much higher stiffness to lateral action in this direction. Additionally, most often rack structures are coupled by positioning two racks back-to-back, with links

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between them. The resulting structure is accessible on both sides from the two aisles. The internal columns of the two racks in the assemblage are connected.

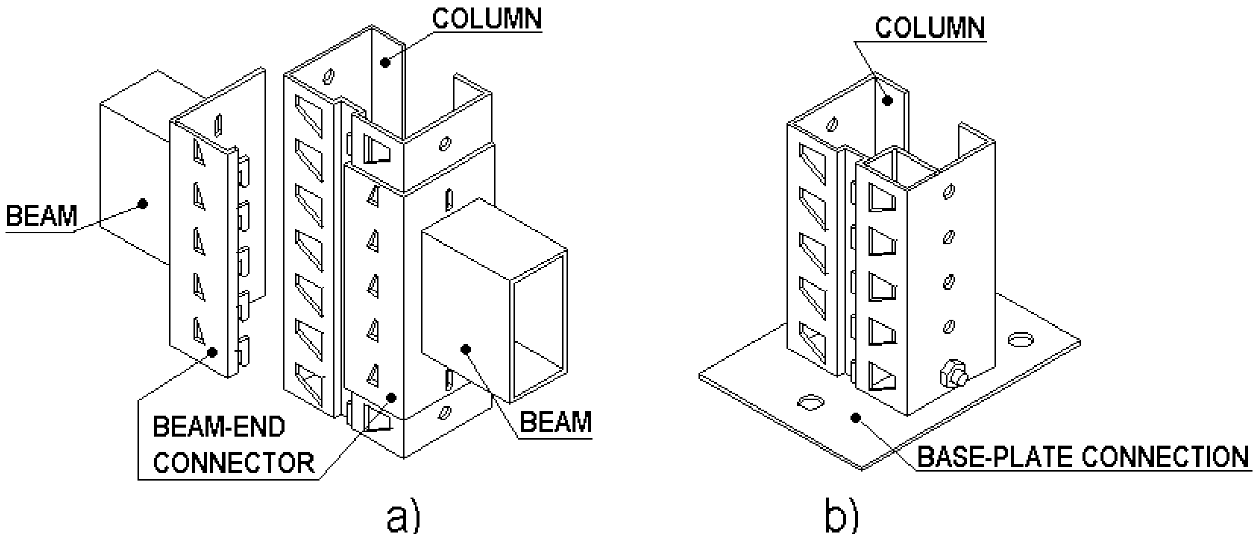

Figure 1. Typical components of rack systems for pallet storage:
a) beam-end connector and b) base-plate connection

Generally, for static loads these structures are analyzed as semi-continuous sway frames, that is, frames with semi-rigid joints, even though the level of continuity supplied by the connections is modest, as discussed in Baldassino [1]. The great variety of elements available on the market and the different typologies of connection have hindered the development of a theoretical approach for the evaluation of their capacity and performance. As a consequence, the most recent design approaches, as in FEM [2] and RMI [3], are based on the use of experimental data relative to the behavior of the components only, without reference to substructures or to the whole structural system, and on the theoretical knowledge currently applied for sizing cold formed elements.

For what concerns seismic design, there is only a limited number of specific studies. Codes developed for the buildings are not adequate, because of the specificity of these structures**.** The features that highly condition the seismic response are the great deformability, due to the geometry of components and the level of masses associated with working loads. Additionally, also the different responses in the longitudinal and transversal direction, associated respectively to the structural scheme of semi-continuous frame and of truss, complicate the definition of specific approaches suitable for inclusion in norms and codes.

A research program devoted to developing specific recommendations for the seismic design of steel racks for pallet storage is currently in progress at the Department of Structural Engineering of the Politecnico di Milano. In this program, an experimental phase is intended to highlight the characteristics of the cyclic response of the nodes; a second, analytical phase, to which the work presented here belongs, aims at characterizing the seismic behavior of these structures and possibly at deriving indications useful for the development of rules for their seismic design.

EXPERIMENTAL CHARACTERIZATION OF THE CONNECTIONS

In Italy, the experimental research on components of the steel racks for pallet storage started in 1992 at the University of Trento. Several hundreds of samples, including stub columns, beam-to-column joints, and joints at the column base, were tested under monotonically increasing loads in order to acquire knowledge of the behavior parameters related to design.
More recently, at the Politecnico di Milano two different types of beam-to-column connections have been tested under cyclic load. The two types are shown in figure 2.

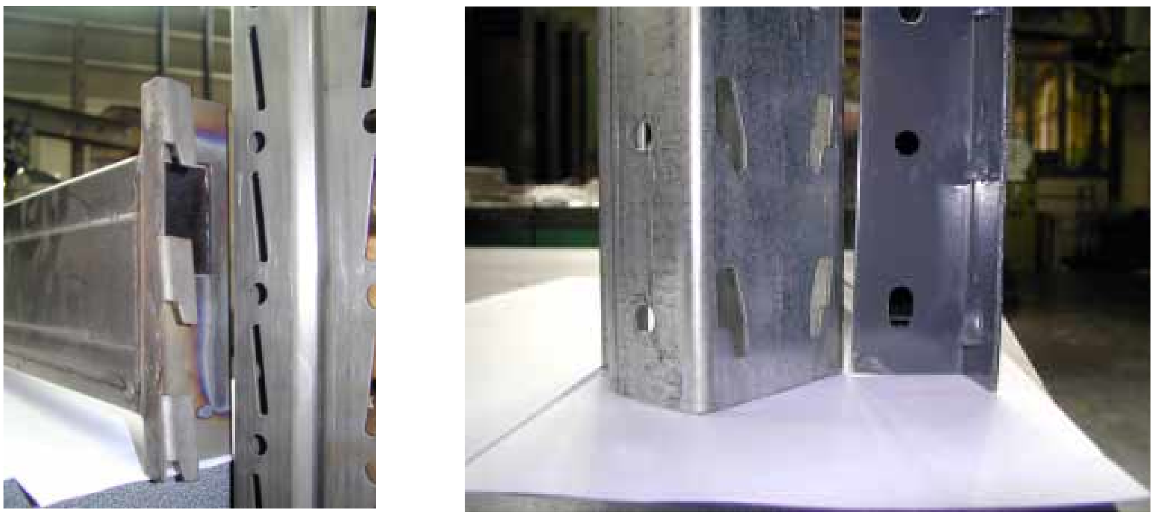

Figure 2. Details of the connection systems studied: type A (left) and B (right)

The details of the research program, as well as the main results, are reported in Bernuzzi [4]. A total of 22 tests were performed. For each type of connection the following types of tests have been carried out:

- tests under monotonically increasing loads, in order to determine the performance of the connection both in normal working conditions and in the presence of live loads that tend to unlock the beam from the column;
- cyclic tests with constant range, carried out imposing different displacement values, in order to evaluate the decay of the main behavior parameters as a function of the number of cycles. Besides tests with symmetric load history, also nonsymmetric load paths have been employed, in order to simulate the effect associated to the presence of pallets on the beams;
- cyclic tests with variable range, according to different load histories, that aimed at evaluating the decay of the behavior parameters as a function of the load paths considered.

Figures 3 and 4 show an experimental moment versus rotation diagram from a monotonic test and typical diagrams from cyclic testing of the two types of connections that were examined, respectively. In these figures, as well as in the following ones, the moment is expressed in non-dimensional form, as a ratio to the plastic moment of the adjacent beam.

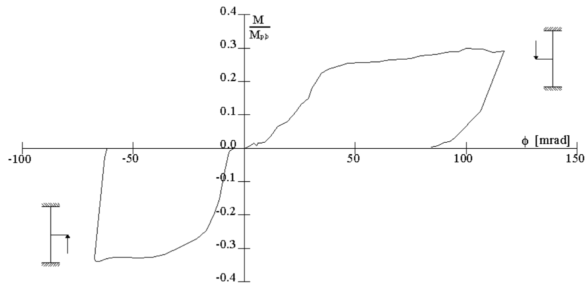

Figure 3. Typical monotonic moment-rotation curve from a beam-end connection test

Considering the experimental results, it is worth pointing out that:

- the first cycle is always stable and regular. It is similar to the cycle typical of the nodes of the most traditional steel structures, characterized by a progressive and regular decrease of stiffness. Both branches of loading and unloading of the first cycle in the plastic range are close to the monotonic response that has been detected experimentally;
- the shape of the subsequent cycles changes significantly under the influence of the residual deformations present in the details of the connection system and in the node area. The moment versus rotation diagram is characterized by a gradual decrease of stiffness in the reloading branch when the number of cycles increases. The unloading stiffness is practically constant and similar to that of the first cycle and to the elastic stiffness of the monotonic test.

Additionally, the curve envelopes are rather well correlated with the monotonic curve, in spite of the initial sliding that was detected in the monotonic tests and that is due to locking of the details of the connection system. During the cyclic tests with variable range, the node response has been found to depend strictly from the imposed load history and, in particular, from the excursions with significant extension in the plastic field.

In the numerical simulations reported in this work, the rack configurations considered were designed with profiles connecting according to the first kind of node tested, modeled as in Figure 5.

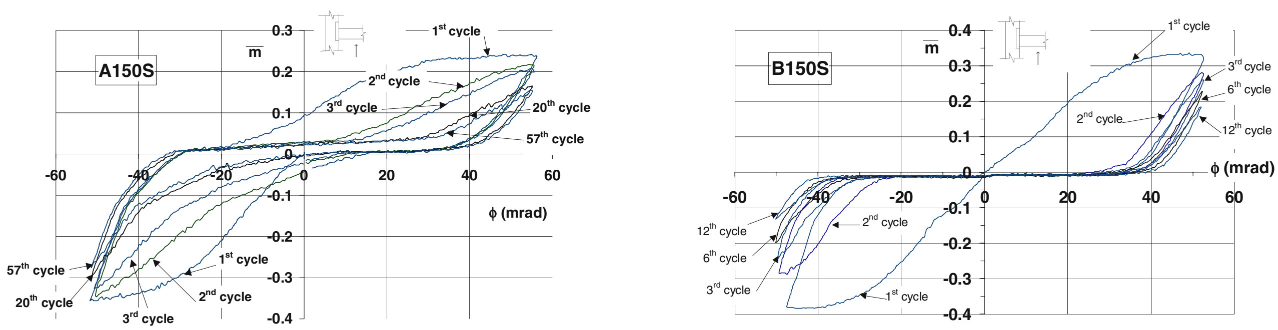

Figure 4. Cyclic behavior of the joints of type A and B

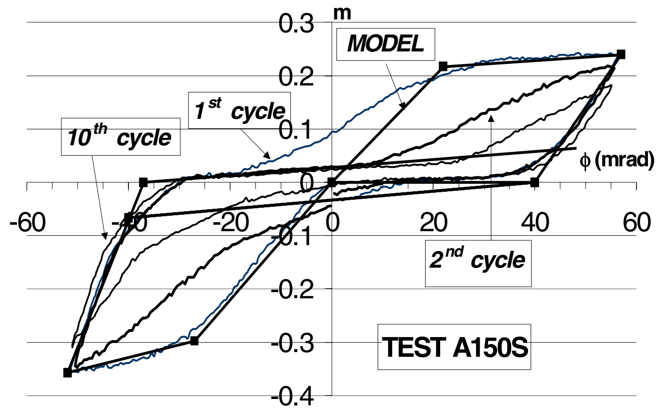

Figure 5. The cyclic model adopted in the analyses

NUMERICAL STUDIES OF SEISMIC BEHAVIOR

The literature treating the behavior of steel rack structures under horizontal load is quite limited. Consequently, the construction of these structures in seismic areas is not specifically regulated in the codes for rack systems design, that usually make reference to the general criteria developed for building frames. This numerical study is intended as a contribution for characterizing the structural behavior of storage racks subject to seismic action and to formulating corresponding design rules.

The cyclic behavior of nodes previously characterized in the experimental part of the research program has been utilized in the numerical analyses discussed in the following. Stiffness of the beam-to-column nodes has been modeled through the relationship of moment versus rotation by means of a finite element that was implemented for describing the cyclic behavior of semi rigid joints, as in Parisi [5].

Various structural configurations have been considered in the analysis, as described in the following. The structures have been analyzed with different combinations of vertical load and seismic action and by means of different methods of analysis. The structural response has been evaluated in terms of those parameters that are commonly adopted to characterize the seismic behavior: the behavior factor, the maximum displacements, and the dissipated energy.

The structural layout of the industrial rack structures in the two main directions is substantially different. Because of the above-mentioned requirement of easily accessing the stored goods, structures are not braced in the longitudinal direction. Resistance to the lateral loads in this direction relies completely on a frame behavior, with the semi-rigid joints having only moderate stiffness. Such a configuration makes the longitudinal direction particularly critical from the point of view of the seismic behavior. Accordingly, the characterization of the behavior in this direction has been the target of the analyses carried out in this work. It must be recalled that the common layout of rack structures is by pairs disposed back-to-back and usually linked, resulting in a bi-symmetrical geometry which should rule out mode coupling, as long as loads are regularly distributed, a situation that mostly occurs or is closely approximated most of the time. It may be expected, indeed, that the collapse mechanism be triggered longitudinally, although out-of-plane effects may then take over.

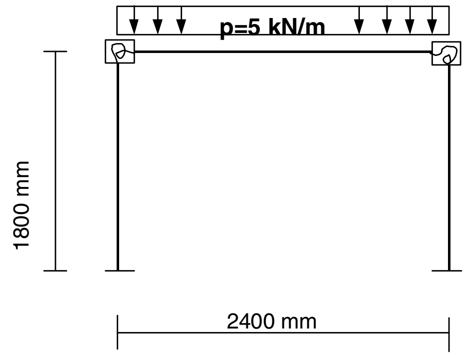

Figure 6. The portal frame

Different structural layouts have been adopted in the analysis. The first case considered is a simple portal frame, as in figure 6. It corresponds to a single basic block of a complete rack system. By studying this portal frame, two main objectives have been reached: the numerical model including the cyclic behavior of the semi rigid joints has been prepared and calibrated, and the fundamental characteristics of the response have been identified, pointing out the role of constraints, the system stiffness, and the type of resistance to seismic action that the structure may supply.

Analyses on a one-bay two-story frame have been carried out together with those of the portal frame. Dimensions were the same as the portal for each story. These analyses were intended to highlight the effect of the aspect ratio of the rack layout on the response level.

On these bases, a complete rack system has been considered, as in figure 7. The structure that has been analyzed has four bays and four stories, that appeared as a very reasonable layout in terms of recurrence. As such, it has been often adopted in literature studies and may be considered somewhat a standard reference structure. Dimensions of each block were 2800×1400 mm. Sections were those tested in the experimental part of the study and are typical of this kind of racks. The yield stress of steel was 280 MPa.

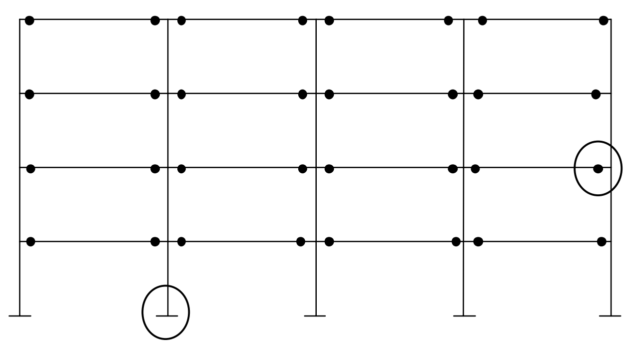

Figure 7. The rack structure (locations where results are discussed are circled)

Finally, a complete rack system with the addition of longitudinal bracings, as in figure 8, has been examined. Even though service requirements of these structures have excluded the use of bracings in this direction, the need of comparing results between braced and unbraced structures and the somewhat unsatisfactory response of the common unbraced layout have suggested this set of analyses. The very frequent disposition joining two racks back-to-back suggests the possibility of introducing suitable kinds of bracing systems without significant reduction of accessibility, as will be discussed in the following.

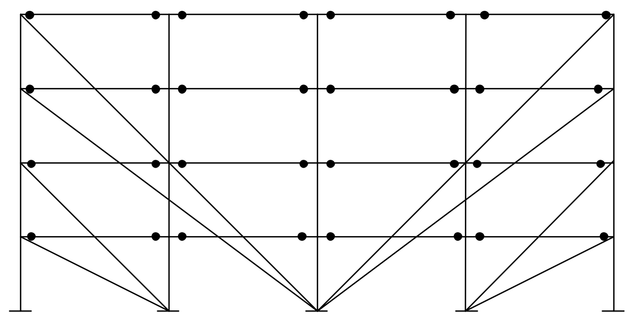

Figure 8. The braced system

In order to avoid reference to commercial products, results have been reported here in non-dimensional form. For moments, a non-dimensional value is given as a ratio of the moment and the plastic moment of the corresponding beam, as in equation 1

$$m = \frac{M}{M_{p,b}} \quad (1)$$

.

Non-dimensional rotations are defined according to Eurocode 3 [6], as in equation 2.

$$\bar{\phi} = \phi \cdot \frac{E \cdot I_b}{L_b \cdot M_{p,b}} \quad (2)$$

In the equations, E is the Young modulus, $M_{p,b}$ is the plastic moment of the beam, L_b is its length, I_b its moment of inertia.

For what concerns constraints at the column bases, initially both cases of hinges and of fixed ends had been taken into account. In subsequent analyses, only fixed ends have been considered as explained in the following.

In the finite element model, the beams have been simulated as elastic and the columns with elastoplastic behavior. The beam-to-column nodes have been modeled with the rotational spring elements that include the cyclic behavior derived experimentally, according to the formulation developed in Parisi [5] and implemented within the computer code ANSR, by Maison [7].

The loads

Pallet storage racks in normal working conditions are subject to comparatively high vertical loads. Stored goods are removed and immediately replaced on the racks that are practically never left empty, giving rise to load distributions that are quite uniform in time and in space, along the shelves. Although no reference load values and load distributions are prescribed or suggested in design codes, a uniformly distributed load with a value of 5 kN/m has been considered here as reference value after surveying several instances of industrial storage racks.

Various situations of vertical load have been considered on the standard rack:

- a vertical load equal to 5 kN/m uniformly distributed on all bays;
- uniformly distributed loads of lower intensity, corresponding to 80 percent, 67 percent, 62.5 percent, and 50 percent of the reference load;
- different cases where a full load was uniformly distributed at a single story, while a reduced load of 50 percent of the reference one was applied at the other levels have been also examined.

A uniform load equal to 62.5 percent of the maximum corresponds to the total load of these last cases, uniformly spread.

The vertical loads have been combined either with horizontal loads in a monotonically increasing load, pushover analysis, or with seismic action, expressed as base acceleration, for increasing values of the peak ground acceleration. When horizontal loads have been applied, their profile corresponded to the applicable equivalent static distribution.

Single-bay systems

The analyses carried out on simple single-bay structures were aimed at qualifying the behavior that these typologies may supply and at obtaining first indications on the possible range of response for design, as in Bernuzzi [8]. Analyses with monotonically increasing lateral loads, dynamic analysis, as well as seismic analysis in the time domain with increasing peak ground acceleration have been carried out for these structures.

A first group of analyses has been carried out for vertical loads and for monotonically increasing horizontal loads. Initially the basis of the columns had been considered alternatively as hinged or built-in, making reference to the hinged situation as lower limit of the range of the constraint stiffness. Indeed, the actual constraint is believed to realize intermediate situations.

For the portal frame, the horizontal load at first yield has been initially examined. If the horizontal load was applied to the beam-to-column joint on the left-hand-side, the first yield has been reached at the corresponding node on the right, when the bases were hinged. It occurred at the base on the right-hand-side when the bases where built-in. Table 1 shows the horizontal load as percentage of the vertical loads for which first yield was reached. Results on the right-hand-side column of the table show the effect of geometric nonlinearity.

Table 1. Horizontal load at yielding for the portal

Constraints at column base	Horizontal load (as % of the vertical)	Horizontal load (as % of the vertical, with geometric nonlinearity)
Hinges	14	11
Built-in ends	50	48.5

This last phenomenon slightly reduces the load when the bases are built-in, with a decrease amounting to approximately 3 percent. When the bases are hinged, a significant capacity reduction occurs, decreasing values that are already very limited.

Similar analyses were carried out for the single bay two-story structure with built-in columns. The effect of geometric nonlinearity can be seen in figure 9, showing a comparison of horizontal displacement of the two stories. This effect increases with height, but remains limited. In any case, all the subsequent analyses have been carried out considering geometric nonlinearity.

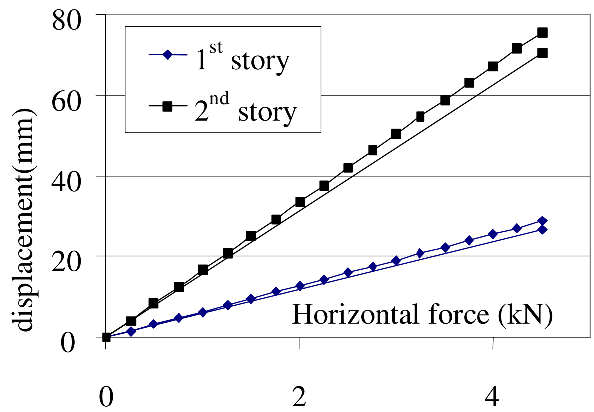

Figure 9. Effect of geometric nonlinearity

For the single bay two-story frame with vertical loads and static equivalent horizontal load of increasing value, the effect of aspect ratio given by the higher number of stories has reduced significantly the capacity of seismic response, bringing the structure to the elastic limit for a total horizontal load equal to 21.5 percent of the vertical for the case of built-in bases and to 7.9 percent in the case of hinges. Columns had been considered elastic.

From the analyses carried out up to this point, it appeared that constraints without -or with very limited- flexural continuity at the base of the columns were completely inadequate. This fact may be explained considering that the beam-to-column joints are semi-rigid with a very low level of stiffness. The moment versus curvature diagram of these joints follows by close the curve acting as lower bound of the range for which joints may be considered semi-rigid according to the Eurocode 3 [6].

It is worth pointing out again that the joints considered in the experimental part of the study and modeled in the analytical phases are of common use in this kind of structures. Unless these structures have very strong rotational constraints at the base, they result almost unrestrained to lateral mechanisms. As a result, a first design indication highlights the need of a very effective constraint to rotation at the base of the columns, in order for these structures to be capable of withstanding even a very modest lateral load. From these remarks, stems the need for experimental research aimed at evaluating the actual level of rotational stiffness of the constraints at the base of real structures, and for considering the possibility of controlling and possibly improving the effectiveness of these constraints also in the storage racks already in use. Having considered this as a fundamental requirement, subsequent analyses have been carried out only considering built-in bases.

The monotonic analysis was then pushed to the post-elastic field. By keeping the columns indefinitely elastic, the contribution of the semi-rigid joints to nonlinearity may be evidenced. This result may be observed in the upper nonlinear branch of the force versus displacement curve of Figure 10, that corresponds to this situation. The lower nonlinear branch of the diagram allows for plasticity of the columns. With horizontal load applied at the left side, the right-hand-side column base yields first, followed by the semi-rigid joints and, lastly, by the second column base.

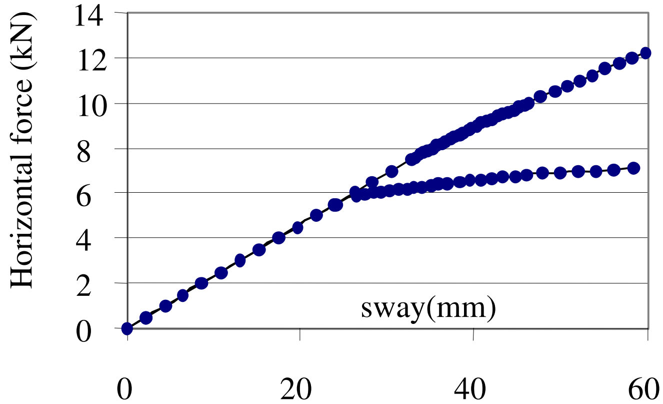

Figure 10. Total horizontal force and generated sway

Dynamic analysis

For the portal structure, built-in at the base, the characteristics of the dynamic response have been evaluated. By examining the first six frequencies and modal shapes, it appeared that the first mode is the only significant one when excitation is in the horizontal direction. The five subsequent modes correspond to oscillations of the beam only, which confirms the limited stiffness of the semi rigid nodes.

Table 2 gives the fundamental period for the possible range of stiffness values of the beam to column joint. Hinges and built-in ends are the extremes of the range, while the current semi rigid situation is just above the half range.

Table 2. Fundamental periods

Beam-to column joints	Fundamental period (seconds)
Hinges	0.30
Semi-rigid joints	0.45
Built-in ends	0.55

Seismic analysis

Seismic analyses in the time domain with an accelerogram compatible with the Eurocode 8 spectrum [9] have been carried out for increasing values of the peak ground acceleration within the range of 0.1-0.6 g.

The frame reaches the elastic limit for a peak base acceleration of approximately 0.27 g. Considering this result and comparing it with values already obtained for monotonically increasing horizontal load that do not contain the horizontal amplification factor, this last may be evaluated as approximately equal to 1.85.

The peak ground acceleration that has been taken for the upper limit of the range is very high. For such value, a corresponding force reduction factor with respect to the elastic case has been found equal to 0.69.

The behavior factor is equal to 0.60/0.27, that is 2.22. The corresponding global ductility request amounts to approximately 1.91, which is a relatively limited value.

The total shear at the base as a function of the peak ground acceleration is reported in figure 11: first yielding occurs for 0.27g, higher accelerations considered in the figure are 0.30, 0.35, 0.40, 0.50, and 0.60g. Figure 12 shows the behavior of the semi rigid springs for the accelerogram with the maximum peak ground acceleration considered. The extension of post-elastic behavior required by the springs is not excessive and is compatible with the results obtained from cyclic tests.

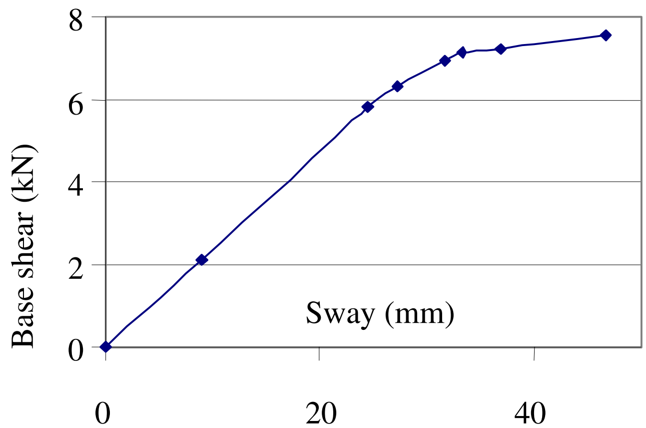

Figure 11. Base shear versus sway for increasing levels of peak ground acceleration

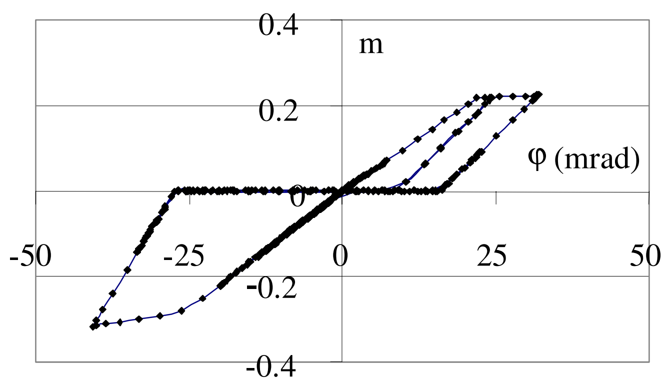

Figure 12. Behavior of the semi-rigid joints

As to the diagram of energy versus time, reported in figure 13, the first significant increment that may be appreciated in the figure corresponds to the beginning of the strong motion phase, while the high step occurring at about 10.5 seconds corresponds to yielding of the column base.

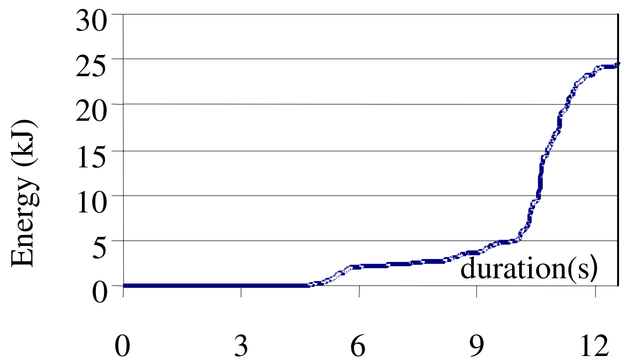

Figure 13. Dissipated energy

Behavior of the standard rack

In order to study the behavior of complete rack structures, the structural configuration shown in figure 7 has been considered. The structure is composed with the same sections used for the portal scheme.

The results obtained for the portal structure had shown that the structure reaches its elastic limit for low values of horizontal force. The objective of the analyses on the standard rack has, then, been to determine the level of horizontal forces that brings it to the elastic limit in the presence of vertical loads typical of working conditions. The load distributions observed for service conditions during numerous surveys to real installations have suggested the vertical load layouts and intensity values adopted here.

The analyses have been carried out with the vertical load corresponding to the case under exam and with the relevant horizontal forces distributed according to a static equivalent profile in a monotonically increasing load history.

On the basis of real case observations, the load configuration with the above mentioned intensity value of 5 kN/m, is not infrequent and may be taken as reference in the analysis. The lateral load multiplier for uniform vertical load has then been determined for a value equal to 5 kN/m and for fractions of it down to 50 percent.

The horizontal load multipliers were found to be extremely small, often at the point of not being compatible with design in seismic zones. In all the cases examined, first yielding was reached at the semi-rigid joint highlighted in figure 7.

The stress values at the base of the column indicated in the same figure were close to the elastic limit in every case. The values of the horizontal force multiplier as a function on the vertical load intensity are shown in figure 14. They are expressed as percentage of the vertical loads. The intensity values considered are 50, 62.5, 67, 80, and 100 percent with respect to the 5 kN/m. Only at the lowest vertical load levels an acceptable level of horizontal forces may be reached.

Further analyses have been carried out with non-uniform distributions of the vertical load and the corresponding static equivalent force distribution in the horizontal direction.

In particular, a load layout with a single story loaded at the maximum level, while the others carried only half of the maximum load, has been considered. The position of the full loaded story has been sequentially modified. At every story, be it fully or partially loaded, the distribution of the load was uniform. The total load on the structure was always 62.5 percent of the maximum. In these load conditions, the maximum values of stress are found in correspondence of the semi-rigid joints of the right-hand-side column or in some case at the base of the indicated internal column, with little differences.

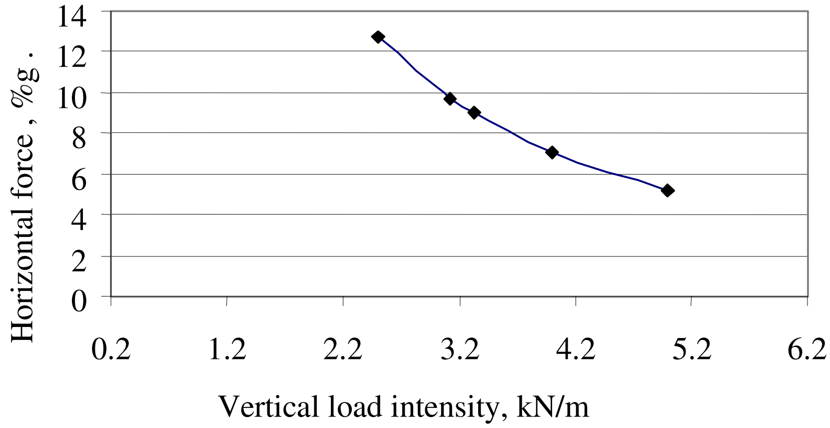

Figure 14. Horizontal force coefficient at yielding versus uniform vertical load intensity

In the first case, the semi rigid spring that reports the maximum stress value and determines the minimum level of the multiplier is the nearest to the story that is carrying the maximum load. The value range of the multiplier at the joints is 9.02-11.3, approximately. Figure 15 presents the behavior as a function of the load centroid height at the indicated column base. In all cases, the multiplier values are within a limited interval and are quite modest.

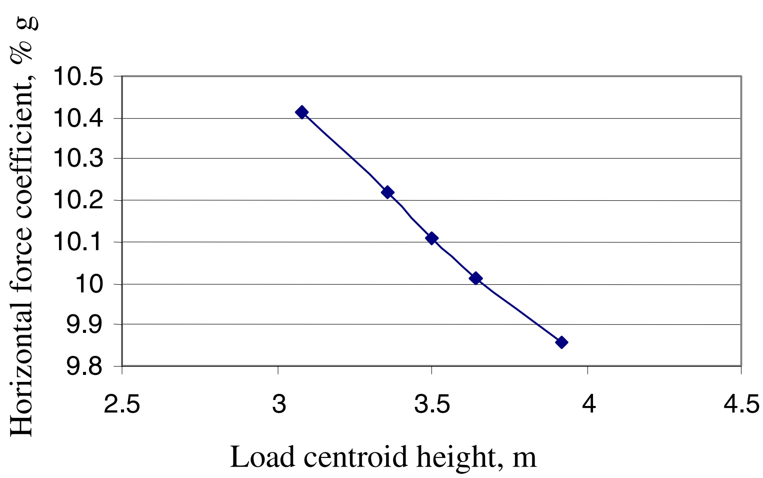

Figure 15. The horizontal force coefficient at yield versus the height of the total load

LIMITATION OF DISPLACEMENTS BY BRACING

According to the previous analyses, the level of horizontal force at the elastic limit for the structures examined is quite limited and needs to be improved for structures to be built even in moderate or low seismicity regions.

The possibility of inserting simple bracing elements that may contain the horizontal deformability has been examined. As mentioned above, constructive practice for storage racks does not contemplate bracing in the longitudinal direction. Yet, racks are almost always disposed in couples positioned back-to-back and tied together in the transversal direction. Bracing elements that do not interfere in service conditions with the need for accessing stored goods in an unlimited manner should be devised and seem to be extremely advisable.

In the last part of this work the effects of a simple bracing system applied to the rack structures have been evaluated. The choice of the bracing type has not been optimized in terms of structural behavior nor of accessibility. The intent of this analysis was to highlight the effects of a simple bracing system that could be possibly applied also to existing structures.

A simple system of steel cables has been applied to the standard rack structure previously studied. The cables section adopted is small, amounting to 78.5 mm^2. The result of this operation is evident in the comparison between the horizontal stiffness of the braced structure and that of the initial unbraced layout. The ratio between the horizontal sways at the top right corner for equal horizontal load is about 20. Consequently, the horizontal force multiplier is about 1.35 g for the braced structure. For this very high level of horizontal forces, the stress level of the cables is extremely low and so are the bending moments in the structure, that tends to change its functioning mode and behave more like a truss system.

CONCLUSIONS

Within a research project on the seismic response of industrial rack systems, this work has analyzed the seismic behavior of these structures in the longitudinal direction, that is particularly sensitive to horizontal action because in common construction practice the lateral response is based only on the semi-rigidity of the beam-to-column joints and on the rotational stiffness of the base constraints. The structures that have been considered in the study are a basic single-bay one story or two-story rack and a multistory multiple-bay structure. The layout adopted for this last is a case frequently cited in literature for storage rack studies and may be considered as reference structure.

The analysis has shown that rack structures may supply a limited-post elastic response, but they exit the elastic field at very low values of horizontal action.

Although the common practice of construction for storage rack systems does not make use of bracings in the longitudinal direction in order to facilitate access to stored goods, it appears that the presence of a suitable bracing system would increment considerably the seismic capacity of these structures which is otherwise unacceptably limited.

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