Available online at www.eccomasproceedia.org
Eccomas Proceedia COMPDYN (2025) 3011-3022

ECCOMAS
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COMPDYN 2025
10th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Rhodes Island, Greece, 15-18 June 2025

PERFORMANCE ASSESMENT OF STEEL STORAGE PALLET RACKS USING DIFFERENT SEISMIC ANALYSIS APPROACHES

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Abstract

During past earthquakes, such as the 2012 Emilia Earthquake in Italy, many steel storage pallet racks were damaged or collapsed. Low costs and short time are generally required to retrofit or re-built steel storage racks; however, the loss related to the stored goods, the logistic issues, and the downtime after severe damage or collapse can lead to huge economic consequences. Typically, the steel storage racks are composed of cold-formed open cross-section members and semi-rigid connections; moreover, they are characterized by long periods of vibration and are often sensitive to P-delta effects. To take into account the peculiarities of storage racks and to ensure seismic safety, the International racks' standards specify different approaches for seismic analysis. This paper compares the seismic performance of steel storage racks designed according to force-based approaches suggested by the International codes. A typical steel storage rack configuration has been selected as case study. The seismic analyses were performed in the down-aisle direction. Detailed numerical models that consider both geometric and material nonlinearity were developed. The most suitable strategy to be adopted for seismic performance assessment.

Keywords: Thin-Walled Structures; Storage Systems; Nonlinear Analysis; Non-structural Elements; Seismic Assessment.

ISSN:2623-3347 © 2025 The Authors. Published by Eccomas Proceedia.
Peer-review under responsibility of the organizing committee of COMPDYN 2025.
doi: 10.7712/120125.12629.25331

1 INTRODUCTION

In recent seismic events, a considerable portion of the economic losses following an earthquake has been attributed not only to damage or collapse of primary structures but also to damage to secondary structures, downtime of buildings, and the loss of their contents. In particular, within industrial facilities used for material storage, the behaviour of metallic racking systems is often neglected. In recent years, several studies have investigated the seismic performance of steel storage pallet racks [1-8]. These studies have led to the development of various methodologies aimed at analysing the behaviour of these lightweight structures, which are often overlooked. Such structures are composed of cold-formed steel profiles, making them both strong and versatile. However, steel racks are often very tall, and this, combined with the characteristics of open sections, makes them particularly susceptible to P-Delta effects, which significantly reduce their capacity. Steel storage racks are usually very flexible structures, typically characterised by a large fundamental period of vibration in the longitudinal direction (loading direction). The various calculation methodologies available enable the estimation of the response, capacity, and demand of industrial racking systems subjected to seismic actions. This paper seeks to compare these methodologies to identify their differences and assess the level of accuracy that can be considered to achieve the best cost-benefit ratio, both in terms of computational effort and reliability of the results.

2 METHOD OF ANALYSIS

In accordance with the European rack standard [9], [10], several approaches are available to analyse racks. In particular, depending on the classification based on the elastic critical load ratio for global buckling, i.e., the critical buckling load α_{cr}, a specific analysis method can be selected, as reported in Table 1. Note that the methods herein proposed are defined as force-based design approaches.

Table 1: Method of analysis based on the critical buckling load classification.

Classification	Method of analysis		Material behaviour	Geometric behaviour
$\alpha_{cr} \geq 10$	0a	Approximate	Linear	1st order
$3.333 \geq \alpha_{cr} < 10$	0b			1st order with amplification
$\alpha_{cr} \geq 10$	1a	Finite Element Analysis	Linear	1st order
$3.333 \geq \alpha_{cr} < 10$	1b			1st order with amplification
$\alpha_{cr} \leq 3.333$	2a		Linear or nonlinear	2nd order
	2b			

The difference between "a" and "b" of the different methods relies on the consideration (b), or not (a), the warping effect.

It should be noted that the Approximate methods (0a and 0b), which are based on the equivalent lateral force approach, have a limitation on the first fundamental period of the rack. The aforementioned period must be no longer than 2s and no longer than $4T_c$, where T_c is the corner period of the spectrum. Therefore, as often happens when dealing with racks, the period is longer, and the approximate methods shall not be adopted. Regarding the analysis with the Finite Element Method, several approaches are available, i.e. the Modal Response Spectrum Analysis (MRSA), Nonlinear Static Pushover Analysis (NSPA), and Nonlinear Dynamic Time-History Analysis (NDTHA). In this study, all these methodologies were performed and compared.

2.1 Modal response spectrum analysis (MRSA)

Modal Response Spectrum Analysis is a technique used to analyse the seismic response of a structure based on its modal properties. By considering the vibration modes of the structure and combining them with the response spectrum characteristic of the site's geographic location, it is possible to estimate the global response of the structure.

2.2 Nonlinear static pushover analysis (NSPA)

The nonlinear static analysis (pushover analysis) allows the evaluation of the structural behaviour beyond the elastic limit, thus in the plastic range, providing valuable insights into its performance under extreme loads. In this case, the structural model must also account for material nonlinearities. The structure will then be subjected to incrementally increasing lateral loads until the target force or displacement is reached, obtaining the capacity curve of the structure.

2.3 Nonlinear dynamic time-history analysis (NDTHA)

Nonlinear Dynamic Time-History Analysis is the most advanced technique available for simulating the response of a structure subjected to seismic action. The structural model will be subjected to a specific accelerogram in the time domain, taking into account both nonlinearities and the inertial contribution of the structural elements. This method can be defined as the most comprehensive and realistic approach for evaluating the dynamic behaviour of a structure under a seismic event.

2.4 Structural members' standard verification

Concerning the resistance checks of structural members, the European standard [9] requires verifying beams, uprights, and connections. As regards the beams, in the majority of the cases, the verification is limited to assessing that the bending demand does not exceed the bending capacity; in some circumstances, the beam could be subjected to torsion. Therefore, the bending capacity is computed by adopting a suitable reduction factor that takes into account the lateral-torsional buckling. Referring to uprights, the verification is not straightforward. As a matter of fact, the flexural and torsional buckling lengths should be computed for each singular member. Then, the flexural, torsional and flexural-torsional buckling loads are required to fulfil equations that combine the action of the different forces. Although complex, commonly, such verifications optimise the rack as much as possible, manufacturers and practitioners esteem and adopt this approach.

Considering the scope of this study, and in order to avoid unnecessary computational onus, the comparison between the different seismic design approaches was carried out by adopting a simpler, thus conservative, methodology. As also suggested by the European rack standard [9], [10], the members' verifications can be performed in accordance with the EC3-1-1 [11]. In this standard, the general method for flexural-torsional and lateral-torsional buckling is presented. Following this approach, the structural members' verification can be conducted, ensuring that:

$$\frac{\chi_{op} \cdot \alpha_{ult,k}}{\gamma_{M1}} \geq 1.0 \tag{1}$$

Where $\alpha_{ult,k}$ is the minimum load amplified of the design loads to reach the characteristic resistance of the most critical cross-section of the structural component considering its in-plane behaviour without taking flexural-torsional or lateral-torsional buckling into account; χ_{op} is the reduction factor for the non-dimensional slenderness $\bar{\lambda}_{op}$ which takes into account flexural-torsional and lateral torsional buckling.

The global non-dimensional slenderness $\bar{\lambda}_{op}$ for the structural component should be determined from:

$$\bar{\lambda}_{op} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr,op}}} \tag{2}$$

Where $\alpha_{cr,op}$ is the minimum amplifier for the in-plane design loads to reach the elastic critical resistance of the structural component.

For further details regarding the general method, interested readers can refer to EC3-1-1, §6.3.4 [11].

3 NUMERICAL MODELS AND SEISMIC DEMAND

3.1 Geometry

The structure of steel storage racks consists of a four-bay frame, with four load levels. The frames are unbraced in the longitudinal direction (Figure 1a-b). The bay's length is 3 m in the longitudinal direction and 1 m in the transversal direction. All the load levels have a height of 2.0 m.

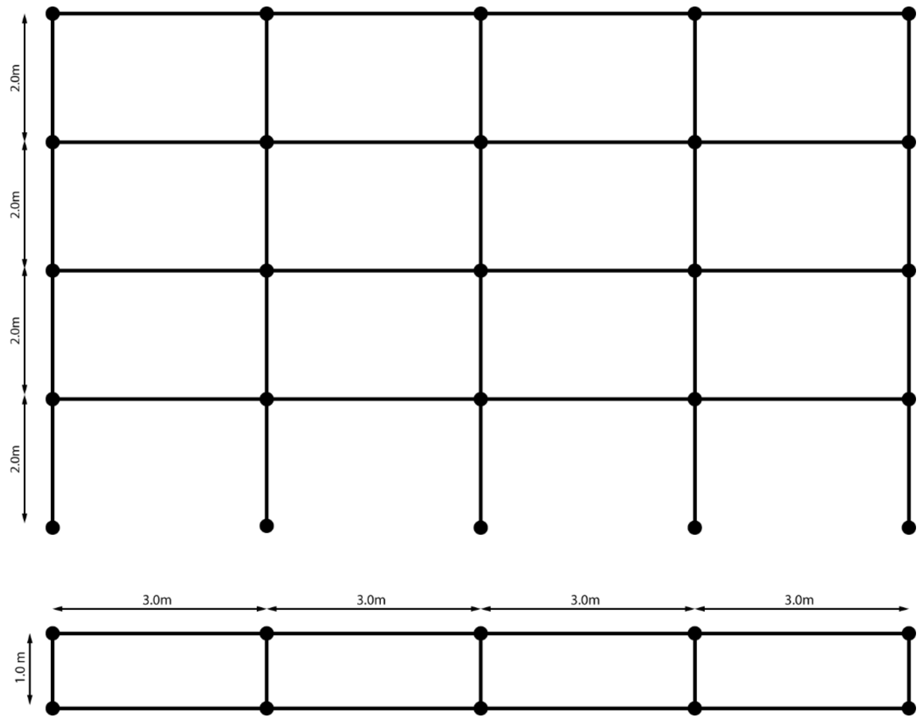


Figure 1: Geometrical configuration of steel rack frame.

Figure 2a illustrates the cross-sections of the uprights, which feature an open omega-shaped profile. In contrast, Figure 2b presents the cross-section of the beams. This hollow rectangular section is formed by the overlapping union of two C-shaped profiles along the shorter side. Table 2 presents the dimensions of the beam and upright sections.

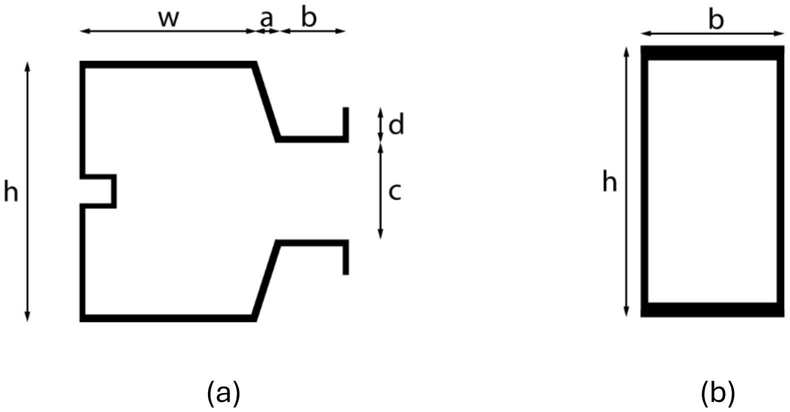


Figure 2: Upright omega and beam rectangular sections.

Table 2: Dimensions of sections of the upright and beam.

	Upright	Beam	
h	110	110	[mm]
w	70	-	[mm]
a	10	-	[mm]
b	15	60	[mm]
c	30	-	[mm]
d	10	-	[mm]
t	-	1.5	[mm]

3.2 Beam-to-upright connection

The beam-to-upright connection is modelled adopting the uniaxial material pinch_asm, provided by the SeismoStruct library [12]. As indicated in the EC3-1-8 [13], the rotation stiffness $S_{j,btc}$, which declares if the connection is a semi-rigid joint or a flexible joint is:

$$S_{j,btu} = 0.5 * \frac{E * I_b}{L_b} \tag{3}$$

where E is the Young modulus, I_b is the moment of inertia, L_b is the span of the beam, and the subscript b indicates that they are referring to the beam.
To define the connection in SeismoStruct [12], three points are needed for the definition of the backbone. For a general parametrisation, the backbones are deduced from the $S_{j,btu}$ stiffness and the moment resistance of the beam section $M_{b,Rd}$. The moment resistance of the beam-to-upright connection ($M_{btu,Rd}$) has been estimated equal to 60% of $M_{b,Rd}$, meanwhile the initial stiffness of the connection (k_{btu}) has been estimated equal to two times $S_{j,btc}$.

Table 3: Backbone values for the beam-to-column connections

	Backbone in negative direction		Backbone in positive direction	
	K_i	F_i	K_i	F_i
Point 1	K_{btu}	$M_{btu,Rd}$	$0.9 \cdot K_{btu}$	$0.9 \cdot M_{btu,Rd}$
Point 2	$-0.15 \cdot K_{btu}$	$0.6 \cdot M_{btu,Rd}$	$-0.135 \cdot K_{btu}$	$0.54 \cdot M_{btu,Rd}$
Point 3	$-0.3 \cdot K_{btu}$	$0.3 \cdot M_{btu,Rd}$	$-0.27 \cdot K_{btu}$	$0.27 \cdot M_{btu,Rd}$

From Table 3, it is possible to observe that the values in the positive direction have been derived from the negative ones, with a reduction of 10%.

An example of the backbone curve obtained is herein shown, for the case study beam-to-upright connection (Figure 3).

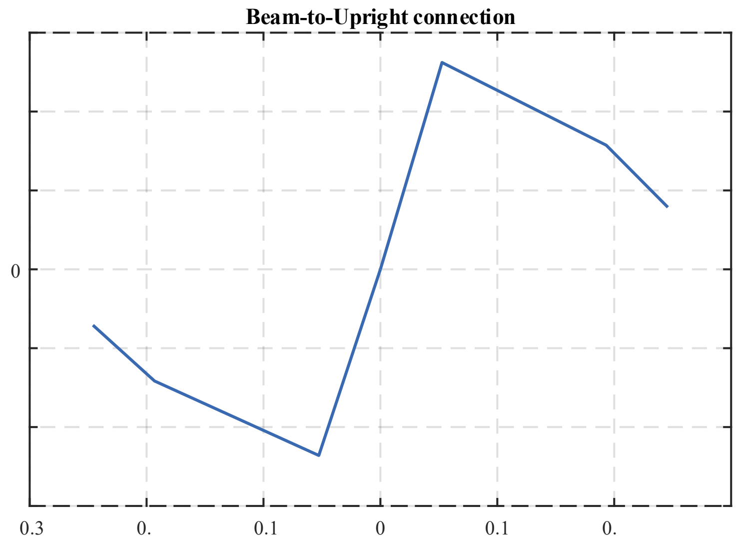


Figure 3: Backbone curve obtained for the behaviour of the beam-to-upright connection.

3.3 Base-plate connection

The base-plate connection is modelled adopting, as for the btu connection, with the uniaxial material pinch_asm. Once again, considering the stiffness classification of EC3-1-8 [13], the rotation stiffness $S_{j,base}$ which declares if the connection is a semi-rigid joint or a flexible joint is:

$$S_{j,base} = 30 * \frac{E * I_u}{h_u} \tag{4}$$

where E is the Young modulus, I_u is the moment of inertia, h_u is the height of first load level and the subscript u indicates that they are referring to the upright.
For this case, the three points needed for the backbone are deduced from the $S_{j,base}$ stiffness and the moment resistance of the upright section $M_{u,Rd}$. The moment resistance of the base-plate connection ($M_{base,Rd}$) has been estimated equal to 90% of $M_{u,Rd}$, meanwhile the initial stiffness of the connection (K_{base}) has been estimated equal to 7% of $S_{j,base}$.

Table 4: Backbone values for the base-plate connections.

Backbone in the negative and positive direction		
	K_i	F_i
Point 1	K_{base}	$0.9 \cdot M_{base,Rd}$
Point 2	$0.30 \cdot K_{base}$	$M_{base,Rd}$
Point 3	$-0.40 \cdot K_{base}$	$0.3 \cdot M_{base,Rd}$

From Table 4, it is possible to observe that the values in the positive direction are the same due to the symmetric behaviour of the base-plate joints. An example of the backbone curve obtained is herein shown (Figure 4).

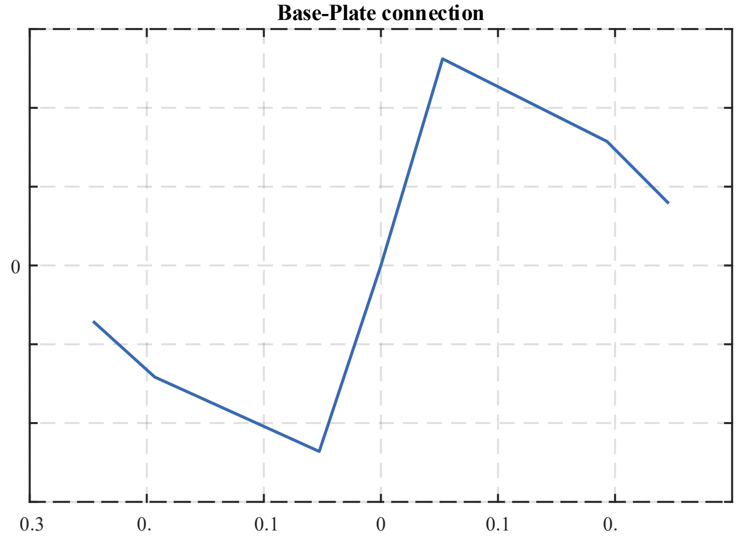


Figure 4: Backbone curve obtained for the behaviour of the base-plate connection.

3.4 Definition of the seismic demand

The case study is located in Modena, Italy, on soil classified as category C with topographic category 1. The PGA for the Life Safety Limit State is equal to 0.24 g, which qualifies the area as a medium-high seismic zone according to the Code.

For the spectra-based analyses, four spectra were calculated using the Italian Code NTC18 [14] approach; the four a_g intensities are: 0.07 g; 0.09 g; 0.24 g; 0.3 g. The four spectra represent the design limit states in accordance with NTC18 [14], which are immediate occupancy (IO), damage limitation (DL), life safety (LS), and collapse prevention (CP).

To perform the Nonlinear Time History Analysis, seven Ground Motions compatible with the spectrum of the area were selected. The selection was carried out using the software Rexel [15] within the period range of 0.15 to 3 seconds, ensuring compatibility with natural accelerograms that could be scaled (Figure 5).

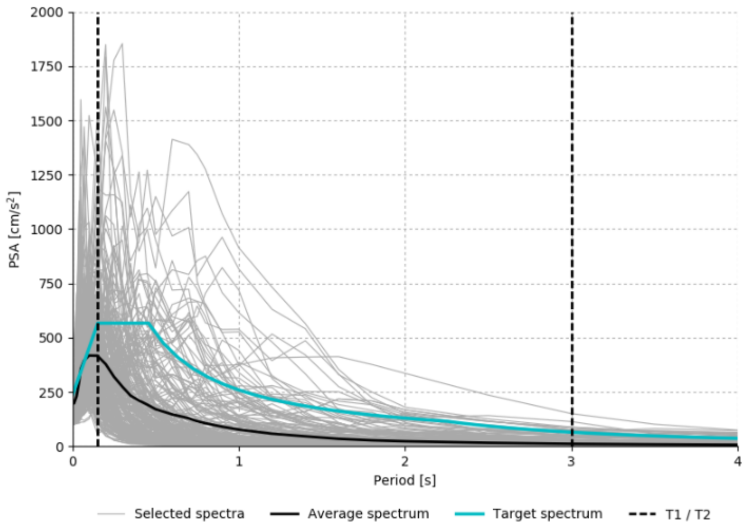


Figure 5: Selection of Ground Motions based on the Code spectra.

4 RESULTS

In the current section, the results of the analyses conducted are presented, comparing the key control parameters that influence the behaviour of pallet storage racking systems. It should be noted that, even if the models developed are three-dimensional, the analyses performed focused on the behaviour in the down-aisle direction. It is well-known, indeed, that rack systems have significant differences in terms of structural response in cross-aisle and down-aisle directions. Thus, in this paper, the response in the cross-aisle direction was neglected, leaving this study to future works. Concerning the results, it is important to mention that, as suggested by the Italian [14] and European [16], standards, the results of NDTHA are summarised in the average response across the seven accelerograms used for the four seismic intensity levels.

4.1 Interstorey drift and top displacement

The first comparison concerns the maximum interstorey drift calculated for each seismic intensity across the three types of analyses performed (Figure 6). From these results, it can be observed that the MRSA exhibits a trend similar to the NSPA. In contrast, the NDTHA generally shows higher interstorey drift values for all seismic intensities. Notably, for the higher seismic intensity (LS and CP), the difference in terms of interstorey drift increases, particularly on the ground floor, where the NDTHA shows a soft-storey mechanism. These results demonstrate the importance of considering the hysteretic behaviour of the rack connections, which commonly exhibit a pinched hysteretic response with low energy dissipation. Clearly, this effect cannot be considered in static (or non-cyclic) analyses.

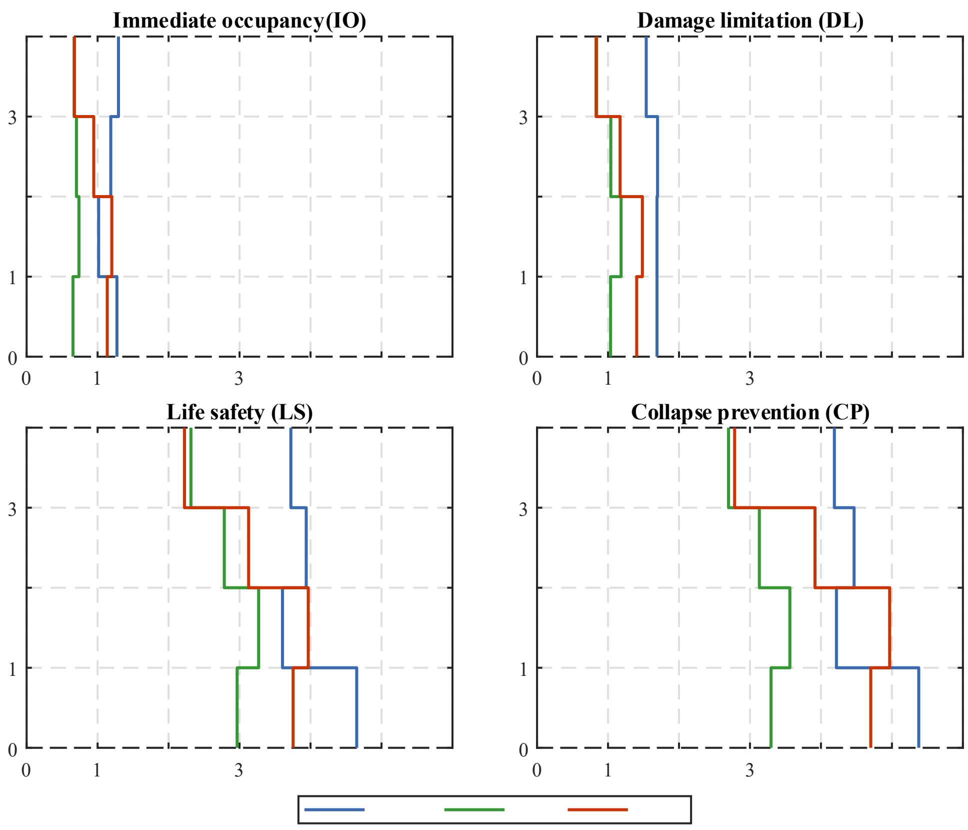


Figure 6: Comparison of results from the three different analyses in terms of interstorey drift.

Figure 7 compares the maximum displacements at the top load level for the three types of analyses. It can be observed that MRSA exhibit the maximum displacements for all the considered return periods, reaching over 30 centimetres for the highest seismic intensity.
Conversely, NDTHA shows lower displacements if the highest return period is considered. Anyway, the results of the two nonlinear analyses (NDTHA and NSPA) are in agreement, demonstrating the potential benefit of performing nonlinear analyses for a better estimate of the seismic rack performance.

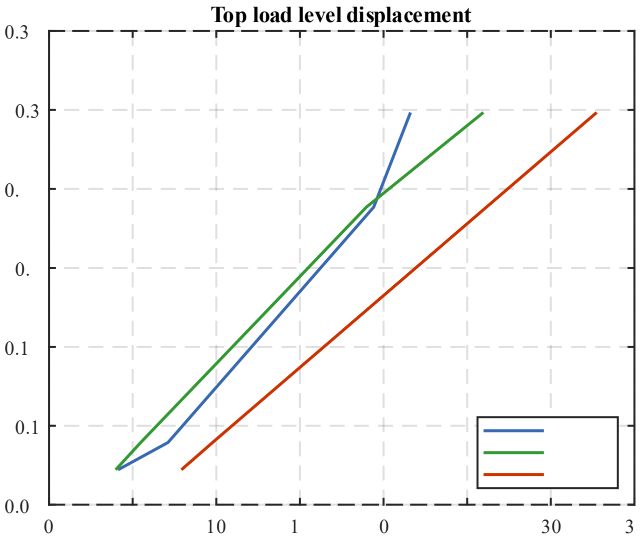


Figure 7: Comparison of results from the three analyses regarding top load level displacement.

4.2 Connections performance

The performance of the connections is a critical aspect in the seismic behaviour of pallet racking systems, particularly due to their semi-rigid nature and tendency to exhibit pinched hysteresis under cyclic loading. Two main types of joints were examined in this study: the base-plate connection and the beam-to-upright connection.

Base-plate Connection

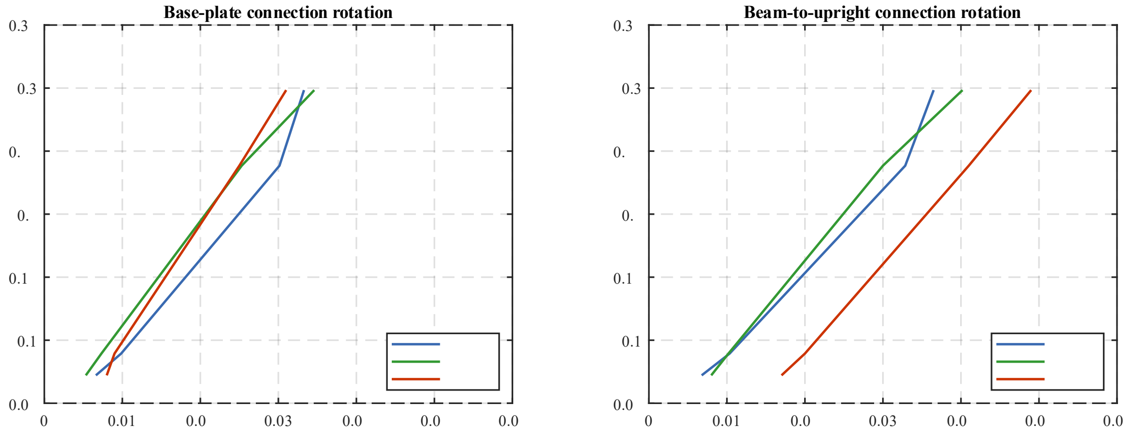
Figure 8a illustrates the rotation response of the base-plate connections across the different analysis methods. All three analyses (MRSA, NSPA, and NDTHA) produced very similar results for base rotation, with only minimal variations observed even at higher seismic intensities. This consistency suggests that base-plate rotations are less sensitive to the modelling approach, possibly due to the relatively higher stiffness of these joints and the reduced moment demand compared to intermediate levels. However, the observed degradation of base stiffness at higher intensities (notably in NDTHA) highlights the importance of accurately capturing this loss of stiffness when using simplified analyses.

Beam-to-Upright Connection

Figure 8b shows the rotation demands at beam-to-upright connections. In contrast with base joints, these connections showed more significant variation between analysis methods. Specifically, MRSA produced the highest rotation values, likely due to the accumulation of modal contributions without considering nonlinear softening or pinching behaviour. The nonlinear methods (NSPA and NDTHA), which more accurately simulate the backbone behaviour of the connection, resulted in lower and more consistent rotations. This confirms that including connection nonlinearity is crucial to avoid overestimating deformation demands.

Furthermore, NDTHA and NSPA captured the progressive degradation of connection stiffness with increasing drift, consistent with the backbone curves previously defined. This behaviour, which cannot be represented in MRSA, leads to a more realistic estimation of rotational demands and overall rack ductility.

Overall, the comparison shows that while simplified methods may offer a first approximation of connection performance, nonlinear analyses are essential to accurately capture the inelastic behaviour, particularly under high seismic demands.



(a) (b)

Figure 8: Comparison of rotation in the (a) base-plate and (b) beam-to-upright connections.

5 CONCLUSIONS

This study has compared three different seismic analysis approaches, Modal Response Spectrum Analysis (MRSA), Nonlinear Static Pushover Analysis (NSPA), and Nonlinear Dynamic Time-History Analysis (NDTHA), applied to a typical steel pallet racking system.

The results confirm that simplified methods like MRSA may significantly overestimate or underestimate key response parameters, especially in the presence of nonlinear and hysteretic connection behaviour. While MRSA provided a faster estimation of global responses, it failed to capture critical phenomena such as stiffness degradation, pinched hysteresis, and soft-storey mechanisms that emerged in nonlinear analyses. The underperformance of MRSA at higher seismic intensities, particularly for base-plate verifications, underlines the limits of relying solely on modal-based procedures.

On the other hand, NSPA and NDTHA yielded consistent and realistic predictions, especially concerning connection rotations and interstorey drifts. NDTHA, although computationally demanding, represents the most robust and comprehensive method, enabling the simulation of the actual time-dependent nonlinear behaviour of the system. NSPA, despite its quasi-static nature, provided results in close agreement with NDTHA and can be seen as a good compromise between accuracy and computational effort.

It is recommended that future studies extend the analysis to include:

- different rack geometries and component variations;
- full 3D modelling with torsional effects using elements with seven degrees of freedom;
- experimental calibration of connection hysteresis curves;
- effects of geometric imperfections and local buckling in thin-walled sections.

Such developments will further improve the understanding of seismic performance and lead to more resilient storage rack designs.

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