

Seismic Design of Adjustable Pallet Racking at Indian Sites: Combining IS 1893 Site Seismicity with EN 16681 Rack Provisions, Test-Derived Joint Stiffness and a Unified Analysis Framework

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Abstract

Adjustable pallet racks (APR) are thin-walled, cold-formed steel structures whose seismic mass is dominated by the stored goods and whose down-aisle lateral stiffness depends almost entirely on semi-rigid beam-end connectors and base plates. India has no dedicated code for storage racking, so designers either adapt the general Indian Standards (IS 1893, IS 800, IS 801) with rack factors borrowed from EN 16681, or design to the European rack standards (EN 15512, EN 16681, EN 15620) using Indian site seismicity. This paper sets out both routes, identifies the source of every design equation, and clarifies the role of the three damping values that appear in rack design: 5% for the design spectrum, 3% for the unloaded steel rack, and about 10% for the loaded rack, which EN 16681 represents through a fixed spectrum modification factor. A worked example for a five-level, single-bay rack in IS 1893 Zone IV gives a design base shear of 7.47 kN, 19% lower than when the payload is treated as rigidly attached mass. The IS 1893 parabolic force distribution places 45% of the base shear at the top level, compared with 33% under the linear EN 1998-1 distribution, and gives a 12% higher overturning moment. Checked with the elastic 3%-damped spectrum as EN 16681 requires, the top-level acceleration (0.25–0.35 g) exceeds the lower-bound friction capacity of both plastic and wooden pallets, so sliding must be addressed explicitly. A closed-form sensitivity study shows that overestimating sway stiffness by a factor of two underestimates drift by about 40% once second-order effects are included, and can lead the designer to neglect second-order effects altogether. The results support a framework in which joint and base stiffness come from the manufacturer's own component tests and second-order effects are classified by the inter-storey drift sensitivity coefficient.

Keywords: pallet racking; seismic design; semi-rigid connections; cold-formed steel; EN 16681; IS 1893; pallet sliding; second-order effects

1. Introduction

Steel storage racks are now a primary structural system in warehouses and distribution centres. In many facilities the rack, not the building, carries most of the stored value and most of the seismic mass. Post-earthquake reconnaissance and shake-table studies have repeatedly shown that racks are vulnerable to down-aisle sway collapse, connector failure, base-plate damage and the shedding of unit loads [1–4]. Their behaviour differs from that of a conventional steel building in four respects: (i) the payload typically forms 90% or more of the seismic mass and can slide on the beams; (ii) members are perforated, open, cold-formed sections in which local, distortional and flexural-torsional buckling interact; (iii) beam-to-upright and upright-to-floor joints are semi-rigid, with a non-linear moment-rotation response that includes initial looseness; and (iv) the down-aisle direction is usually an unbraced sway frame whose stiffness is sensitive to gravity load.

Several international documents address these features. In Europe, EN 15512 [5] covers static design, EN 16681 [6] covers seismic design and EN 15620 [7] covers tolerances and clearances, building on the FEM 10.2.08 recommendation [8]. In the United States, ANSI MH16.1 [9] is referenced by ASCE/SEI 7 [10] for steel storage racks. Australia has AS 4084 [11]. India has no equivalent. IS 1893 (Part 1) [12] provides the seismic hazard and spectrum, and IS 800 [13] and IS 801 [14] cover steel and cold-formed member design, but none of these addresses rack-specific phenomena such as payload sliding, connector looseness or product mass participation.

Indian designs are therefore produced either by adapting general Indian Standards with rack factors borrowed from EN 16681, or by designing wholly to the European standards with Indian site seismicity. Practice varies between designers and suppliers, and three recurring problems are observed. First, damping values are mixed: the 2% value for steel in the 2002 edition of IS 1893, which the 2016 edition withdrew, is still used, sometimes alongside the 3% of EN 16681, which applies only to specific checks. Second, the period is estimated from building formulae instead of a model of the rack. Third, connector stiffness is taken from catalogues or generic literature values rather than from tests on the actual hardware.

The contributions of this paper are:

1. A side-by-side statement of the Indian Standard (IS) route and the European Standard (EN) route for APR seismic design at Indian sites, with the source of every equation identified (Table 2).
2. A clarification of the damping values used in rack design and where each one applies, with the error caused by the withdrawn 2% multiplier quantified.
3. A worked example with explicit conclusions on base shear, force distribution, overturning, pallet sliding and beam behaviour.
4. A closed-form sensitivity analysis showing that errors in connector and base stiffness are unconservative for drift and second-order effects. This supports the requirement that stiffness be derived from the manufacturer's own component tests.
5. A unified analysis framework (Fig. 7) that integrates component testing, a spring model, modal analysis, stability classification and a stiffness-sensitivity loop.

The paper is intended for structural engineers, specifiers, code committees and researchers. It does not promote any proprietary product or test result.

2. Structural behaviour of adjustable pallet racks

Members. Uprights are open, perforated, cold-formed sections, typically 1.5–3.0 mm thick. Their effective cross-section properties (A_{eff} , I_{eff}) are derived from stub-column and distortional buckling tests (EN 15512, Annex A) rather than from gross-section properties, because the perforation pattern governs local and distortional resistance [5].

Beam-end connectors. A beam-end connector is a bracket welded to the beam end, carrying hooks (also called lugs or tabs) that engage the perforations of the upright. Moment is transferred by two mechanisms: bearing of the hooks on the edges of the upright slots, and contact between the connector plate and the upright face in the compression zone. Because the hooks seat progressively and the slots deform locally, the moment–rotation ($M-\phi$) response is non-linear, with an initial looseness ϕ_l before full engagement [15, 16]. Neither a pinned nor a rigid idealisation is valid.

Frame action. In the cross-aisle direction, upright frames are braced trusses and are comparatively stiff. In the down-aisle direction the rack is usually unbraced, and lateral stiffness comes only from connector and base-plate rotation. The down-aisle frame is therefore flexible, has a long period and is sensitive to P– Δ effects (Fig. 1).

Mass. The unit loads typically form 90% or more of the seismic mass (96% in the example of Section 7). Pallets are not fixed to the beams and can slide when the level acceleration exceeds the friction capacity. Sliding dissipates energy but creates a load-retention hazard [17].

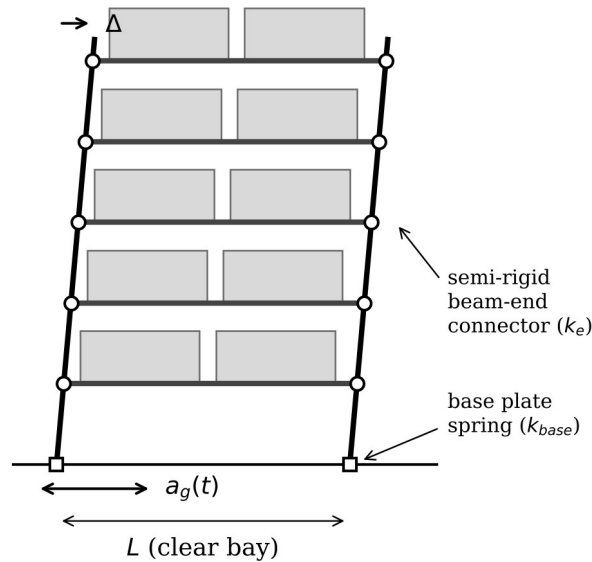


Fig. 1. Down-aisle elevation of a five-level, single-bay rack: payload-dominated mass on semi-rigid beam levels, base-plate springs and the sway profile Δ under ground acceleration a_g . Source: schematic drawn by the authors.

3. Design routes and code background

3.1 The two routes

A design should follow one code family consistently. Mixing member resistances from one family with load factors, damping or behaviour factors from another produces an inconsistent reliability level. In both routes considered here, IS 1893 provides the site seismicity. Table 1 summarises the routes. The ANSI MH16.1/ASCE 7 route used in North America is included for international comparison only.

Table 1. Code routes for APR seismic design at Indian sites.

Aspect	IS route (India)	EN route (Europe)	US route (for comparison)
Site seismic input	IS 1893-1:2016: Z, I, soil type, S_a/g	IS 1893-1:2016 spectrum, with Z/2 as the design-basis peak ground acceleration	ASCE 7: S_s, S_1 , site class
Damping of design spectrum	5%, irrespective of material (IS 1893-1:2016, cl. 7.2.4)	5% reference spectrum reduced by q (EN 1998-1, cl. 3.2.2.5)	5%
Other damping values	None specified for racks	3% for the unloaded rack, used for elastic checks (EN 16681, cl. 6.2); loaded-rack damping (about 10%) through $E_{D3} = 0.8$ (cl. 7.5.2)	None
Member and connection design	IS 800, IS 801	EN 15512 (basis EN 1993)	ANSI MH16.1 (basis AISI S100)
Rack-specific seismic factors	$E_{D1}, E_{D2}, E_{D3}, K_D$ adopted from EN 16681	EN 16681, cl. 7.5	ANSI MH16.1 product-load provisions
Response / behaviour factor	No rack value in IS 1893; $q = 1.5$ adopted from EN 16681	$q = 1.5$ low-dissipative, $q = 2$ with additional requirements (EN 16681, cl. 8.3.2.1)	R per ASCE 7-16 Table 15.4-1 and ANSI MH16.1
Vertical force distribution	Parabolic, h^2 (IS 1893-1:2016, cl. 7.6.3)	Linear in height (EN 1998-1, cl. 4.3.3.2.3)	ASCE 7 / MH16.1
Second-order effects	IS 800	θ -based (EN 16681, cl. 7.2–7.4)	ANSI MH16.1
Tolerances and clearances	EN 15620 (no IS equivalent)	EN 15620	ANSI MH16.1

3.2 Why the IS route borrows rack factors

IS 1893 gives no guidance on (a) the reduced participation of product mass that is free to slide, (b) energy dissipation by pallet–beam friction and by the unit loads, or (c) the participation of different kinds of unit load. EN 16681 addresses these through a unit-load weight modification factor E_{D2} (cl. 7.5.5, Table 5), design spectrum modification factors E_{D1} (sliding) and E_{D3} (damping of the loaded rack) (cl. 7.5.2), and a combined spectrum modification K_D (cl. 7.5.1) [6]. Adopting these factors in the IS route is an engineering judgement and should be stated explicitly in the design basis. In the IS route the payload is also taken at its full storage weight, modified only by E_{D2} . This departs from the imposed-load percentages of IS 1893-1 cl. 7.3.1, which were written for building floors, and is justified because stored goods are quasi-permanent mass.

3.3 Source of each equation

Table 2 lists every equation used in this paper and its source. Equations labelled “mechanics” are standard structural results. Equation (8b) and the relations in Section 8 are derived here.

Table 2. Source of each equation.

Eq.	Quantity	Source
(1)	IS 1893 spectral shape, medium soil (type II), equivalent static method	IS 1893-1:2016, cl. 6.4.2 [12]
(2)	Damping correction factor η	EN 1998-1:2004, Eq. (3.6) [18]; used with $\xi = 3\%$ per EN 16681, cl. 6.2 [6]
(3)	Sliding factor E_{D1}	EN 16681:2016, cl. 7.5.2 [6]
(4)	Design horizontal acceleration coefficient A_h	IS 1893-1:2016, cl. 6.4.2 [12], with K_D added
(5)	Combined spectrum modification K_D	EN 16681:2016, cl. 7.5.1 [6]
(6)	Seismic weight with unit-load modification E_{D2}	EN 16681:2016, cl. 7.5.5 [6]
(7)	Base shear $V_b = A_h W$	IS 1893-1:2016, cl. 7.6.1 [12]
(8a)	Vertical distribution, IS route	IS 1893-1:2016, cl. 7.6.3 [12]
(8b)	Vertical distribution, EN route	EN 1998-1:2004, cl. 4.3.3.2.3 [18]
(9)	Single-degree-of-freedom period check	Mechanics (structural dynamics)
(10)	Pallet sliding check	EN 16681:2016, cl. 9.2.2.1, 7.5.3 and Table 4 [6]
(11)–(13)	Beam end and mid-span moments, deflection with end springs	Mechanics (slope-deflection); limit $L/200$ from EN 15512 [5]
(14)	Global sway imperfection	EN 15512 [5]
(15)	Inter-storey drift sensitivity θ and amplification	EN 16681:2016, cl. 7.2–7.4 [6]; EN 1998-1, cl. 4.4.2.2 [18]
(16)	Upright interaction check	EN 15512 / EN 1993-1-1, cl. 6.3.3 form

Eq.	Quantity	Source
		[5, 19]

4. Seismic demand

4.1 Design spectrum

For a medium soil site (type II), IS 1893-1:2016 gives the 5%-damped spectral acceleration coefficient for the equivalent static method as

$$\frac{S_a}{g} = \begin{cases} 2.50, & T \leq 0.55 \text{ s} \\ 1.36/T, & 0.55 < T \leq 4.0 \text{ s} \\ 0.34, & T > 4.0 \text{ s} \end{cases} \quad (1)$$

Cross-aisle braced frames typically lie on the plateau. Down-aisle sway frames typically lie on the $1/T$ branch (Fig. 2). The period must be obtained from a modal analysis of the spring model described in Section 6, not from empirical building formulae, which were calibrated for buildings with very different mass and stiffness distributions.

4.2 Damping: three values, three purposes

Three damping values appear in rack design, and much of the inconsistency in practice comes from applying one where another belongs.

- **5%: the design spectrum.** IS 1893-1:2016 (cl. 7.2.4) requires 5% damping for calculating A_h , irrespective of the material of construction. EN 16681 uses the EN 1998-1 design spectrum (cl. 7.5.1), whose behaviour factor q is defined relative to the 5%-damped elastic response (EN 1998-1, cl. 3.2.2.5). No damping correction is therefore applied to the design spectrum in either route.
- **3%: the unloaded steel rack.** EN 16681 (cl. 6.2) specifies $\xi = 3\%$ for the steel structure of the unloaded rack. It is used with the elastic spectrum ($q = 1$) to evaluate E_{D1} (Eq. 3) and the inertia forces for the sliding check (Eq. 10), through the correction factor

$$\eta = \sqrt{\frac{10}{5 + \xi}} \geq 0.55, \eta(3\%) = 1.12 \quad (2)$$

- **About 10%: the loaded rack.** The damping of the loaded rack in operation is higher. EN 16681 does not apply it as a viscous damping ratio but through the fixed factor $E_{D3} = 0.80$, which corresponds to a conventional damping ratio of 10% (cl. 7.5.2, Note 2).

The 2% value for steel and its spectral multiplier of 1.40 belong to IS 1893 (Part 1):2002 and were removed in the 2016 edition [12]. Applying that multiplier to the design spectrum would overstate the base shear of the example in Section 7 by 40%. Applying $\eta(3\%)$ to the EN design spectrum in addition to E_{D3} would count damping twice.

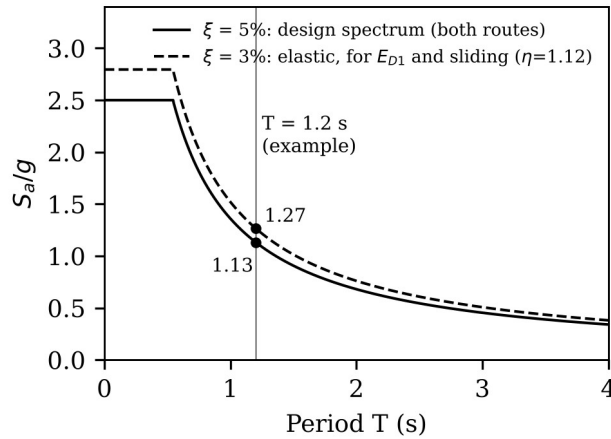


Fig. 2. IS 1893-1:2016 spectrum for medium soil (type II), equivalent static method: 5%-damped design shape (both routes) and the 3%-damped elastic shape used for E_{D1} and the sliding check (5% shape $\times \eta$, a simplification of EN 1998-1 Eqs. 3.2–3.5). Source: plotted by the authors from Eqs. (1) and (2); not reproduced from any code figure.

4.3 Rack factors, design acceleration and base shear

The sliding factor depends on the pallet–beam friction coefficient μ_s and on the elastic spectral acceleration $S_e(T_1)$ of the unloaded rack (3% damping, $q = 1$):

$$E_{D1} = \max \left[0.4; \frac{\mu_s g}{S_e(T_1)} + 0.2 \right] \leq 1.0 \quad (3)$$

The design horizontal acceleration coefficient follows IS 1893-1 with the EN 16681 spectrum modification:

$$A_h = \frac{Z}{2} \cdot \frac{I}{R} \cdot \frac{S_a}{g} \cdot K_D \quad (4)$$

$$K_D = 1 - \frac{P_{E,prod}}{P_E} (1 - E_{D1} E_{D3}), E_{D1} E_{D3} \geq 0.4 \quad (5)$$

where $P_{E,prod}/P_E$ is the fraction of seismic weight contributed by the unit loads. EN 16681 permits $K_D = E_{D1} E_{D3}$ when this fraction is at least 90%. The seismic weight and base shear are

$$W = G_{rack} + E_{D2} W_{prod} \quad (6)$$

$$V_b = A_h W \quad (7)$$

The base shear is distributed over the beam levels as

$$\text{IS route: } Q_i = V_b \frac{W_i h_i^2}{\sum_j W_j h_j^2} \quad (8a) \quad \text{EN route: } F_i = V_b \frac{W_i h_i}{\sum_j W_j h_j} \quad (8b)$$

The fundamental period can be checked against a single-degree-of-freedom estimate

$$T = 2\pi \sqrt{\frac{W}{g K_{sway}}} \quad (9)$$

where K_{sway} is the lateral stiffness of the spring model at the effective height and W/g is used as an upper bound on the modal mass. Equation (9) is a check only; the design period comes from the modal analysis.

4.4 Pallet sliding

EN 16681 (cl. 9.2.2.1) treats sliding as a consequence to be assessed rather than as damage. The consequences must be assessed wherever the inertia force on a unit load, calculated from the elastic spectrum with 3% damping, exceeds the lower-bound friction resistance:

$$\frac{a_{i,el}}{g} \leq C_{\mu L} \mu_S \quad (10)$$

Here $a_{i,el}$ is the level acceleration from the elastic spectrum ($q = 1$, $K_D = 1$), and $C_{\mu L} = 0.67$ is the lower-bound correction where no tests are performed (cl. 7.5.3). The reference friction coefficient μ_S for steel beams in normal warehouse conditions is 0.37 for wooden pallets and 0.15 for plastic and steel pallets (Table 4). The reduced design forces must not be used for this check, because K_D already assumes the dissipation that sliding provides. Conversely, the friction limits the horizontal action that a unit load can transfer to the beams to $Q_{p,max} C_{\mu H} \mu_S$, with $C_{\mu H} = 1.50$ (cl. 7.5.3 and 9.3).

5. Semi-rigid beam-end connectors

5.1 Derivation of connector stiffness from tests

EN 15512, Annex A, specifies a cantilever test in which a short beam stub with its connector is fixed to an upright and loaded until failure [5]. The design moment resistance M_{Rd} is obtained statistically from a series of tests. The rotational stiffness k_e is the slope of a line through the origin that isolates equal areas between itself and the experimental $M-\phi$ curve, up to the design moment. ANSI MH16.1 instead uses a secant stiffness at 0.85 of the ultimate moment [9, 16]. The test captures the specific combination of upright perforation geometry, upright thickness, hook geometry, weld detail and steel grade, so a value obtained for one combination does not transfer to another.

5.2 Effect of joint fixity on beam design

For a beam of span L and flexural rigidity EI under uniform load w , with a rotational spring k_e at each end, slope-deflection gives

$$M_{end} = \frac{wL^2}{12} \cdot \frac{1}{1 + 2EI/(k_e L)} \quad (11)$$

$$M_{mid} = \frac{wL^2}{8} - M_{end} \quad (12)$$

$$\delta = \frac{5wL^4}{384EI} - \frac{M_{end}L^2}{8EI} \leq \frac{L}{200} \quad (13)$$

The non-dimensional fixity $k_e L/EI$ controls how moment is shared between connector and mid-span (Fig. 3). A pinned idealisation overestimates beam moment and deflection. A rigid idealisation overestimates connector moment and frame stiffness, and underestimates drift.

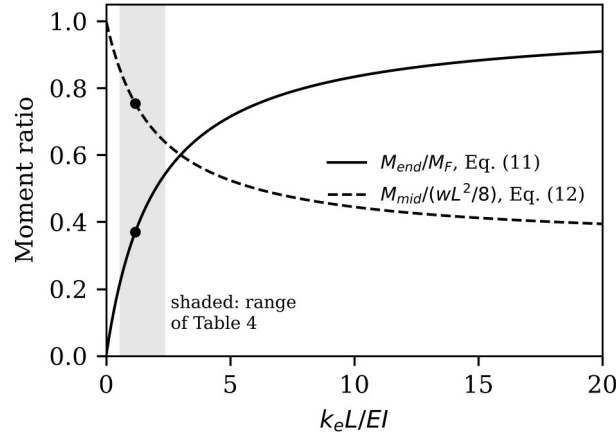


Fig. 3. Effect of joint fixity on beam moments. $M_F = wL^2/12$ is the fixed-end moment; the shaded band is the range covered by Table 4. For $k_e L/EI = 1.17$ (the example in Section 7), $M_{end} = 0.37 M_F$ and $M_{mid} = 0.75 wL^2/8$. Source: plotted by the authors from Eqs. (11) and (12); not reproduced from any code.

6. Global stability and second-order effects

The axial load from the payload substantially reduces the sway stiffness of the down-aisle frame. The global sway imperfection combines out-of-plumb and connector looseness:

$$\phi = \sqrt{\frac{1}{2} + \frac{1}{n_c}} (2\phi_s + \phi_l), \phi_s \geq \frac{1}{500} \quad (14)$$

where n_c is the number of upright columns per beam level, ϕ_s the specified out-of-plumb and ϕ_l the connector looseness from tests [5].

In static design, EN 1993-1-1 (cl. 5.2.1) classifies frames by the elastic critical load factor α_{cr} [19]. In seismic design the relevant measure is the inter-storey drift sensitivity coefficient θ of EN 16681 (cl. 7.3), which follows EN 1998-1 (cl. 4.4.2.2):

$$\theta = \frac{P_E d_r}{V_E h} \approx \frac{q_d}{\alpha_{cr,E}}, M_{II} \approx \frac{M_I}{1 - \theta} \quad (15)$$

Here P_E and V_E are the gravity load and storey shear in the seismic combination, d_r the design inter-storey drift, h the storey height, q_d the displacement behaviour factor and $\alpha_{cr,E}$ the critical load factor under the seismic gravity load. The approximation follows because $d_r = q_d d_{r,el}$ and, for a sway mode, $P_E d_{r,el}/(V_E h) \approx 1/\alpha_{cr,E}$. Second-order effects may be neglected for $\theta \leq 0.1$. Above this, they are included by direct second-order analysis or by the amplification in Eq. (15), which EN 16681 does not recommend for $\theta > 0.3$ (cl. 7.4.2). EN 16681 also limits $P_E/P_{cr,E}$ to 0.5 where $\gamma_I a_{gR} S \geq 0.1 g$ (cl. 7.2). With $q_d = 1.5$, neglecting second-order effects therefore requires $\alpha_{cr,E} \geq 15$ (Fig. 4), a stricter limit than the static value of 10. Because down-aisle stiffness comes only from connector and base rotation while the gravity load is large, unbraced down-aisle frames frequently fall below this limit [3, 20]. Uprights are then verified with test-based effective properties, including distortional buckling, using an interaction of the form

$$\frac{N_{Ed}}{N_{b,Rd}} + \frac{k_y M_{y,Ed}}{M_{b,y,Rd}} \leq 1.0 \quad (16)$$

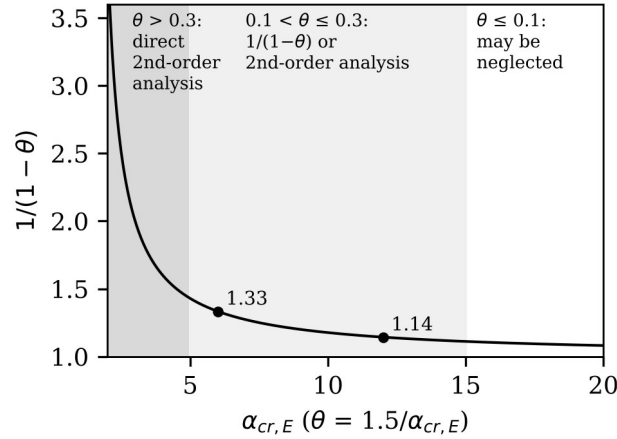


Fig. 4. Second-order amplification $1/(1 - \theta)$ with $\theta = q_d/\alpha_{cr,E}$ and $q_d = 1.5$, showing the ranges of EN 16681 cl. 7.2–7.4. Source: plotted by the authors from Eq. (15); not reproduced from any code figure.

7. Worked example: single bay, down-aisle direction

7.1 Data

A single-bay, five-level rack is considered in the down-aisle direction (Fig. 1). Beam levels are at 1.5, 3.0, 4.5, 6.0 and 7.5 m, and the clear bay is 2.7 m. Each level carries 2 t of unit loads on two beams, so each beam carries 1 t, treated as a uniformly distributed load. The total unit load is 10 t and the rack self-weight 0.4 t, so $P_{E,prod}/P_E = 0.96$. The site is in IS 1893 Zone IV ($Z = 0.24$) on medium soil, with $I = 1.0$. Both routes use $q = R = 1.5$ (EN 16681, cl. 8.3.2.1). The fundamental down-aisle period from a modal analysis of the spring model is $T = 1.2$ s. The pallets are wooden ($\mu_s = 0.37$). $E_{D2} = 1.0$ is taken, the upper bound of EN 16681 Table 5, which is conservative for any unit-load class. Both routes use the same IS 1893 spectrum, so they differ only in the vertical force distribution and in the member design code. Beam values are characteristic (unfactored) and $EI = 210$ kNm² ($I = 100$ cm⁴).

7.2 Results

Table 3. Worked example, down-aisle direction.

Step	Input / expression	IS route	EN route
S_a/g (5%)	$1.36/T$, $T = 1.2$ s, Eq. (1)	1.133	1.133
$S_a(T_i)/g$ (3%, $q = 1$)	$(Z/2) \times 1.133 \times \eta$, $\eta = 1.118$, Eq. (2)	0.152	0.152
E_{D1}	$\max[0.4; 0.37/0.152 + 0.2] \leq 1.0$, Eq. (3)	1.00	1.00
K_D	$E_{D3} = 0.80$, $P_{E,prod}/P_E = 0.96$, Eq. (5)	0.808	0.808
Seismic weight W	$(0.4 + 1.0 \times 10)$ t $\times g$, Eq. (6)	102.0 kN	102.0 kN
A_h	$(0.24/2)(1.0/1.5) \times 1.133 \times 0.808$, Eq. (4)	0.0732	0.0732
Base shear V_b	$A_h W$, Eq. (7)	7.47 kN	7.47 kN
Top-level force	Eq. (8a) / (8b), equal W_i	3.40 kN (45%)	2.49 kN (33%)
Overturning moment at base	$\sum Q_i h_i$	45.8 kNm	41.1 kNm
Elastic top-level acceleration $a_{5,el}/g$	3% elastic, $q = 1$, $K_D = 1$	0.346	0.253
Sliding capacity $C_{\mu I, \mu_s}$	0.67×0.37 (wood) / 0.67×0.15 (plastic)	0.248 / 0.10	0.248 / 0.10
Beam moments	$w = 3.63$ kN/m, $L = 2.7$ m, $k_e = 91$ kNm/rad	M_{end} 0.81, M_{mid} 2.49 kNm	same
Beam deflection	Eq. (13): 11.96 – 3.53 mm	8.43 mm $\leq$ 13.5 mm	same

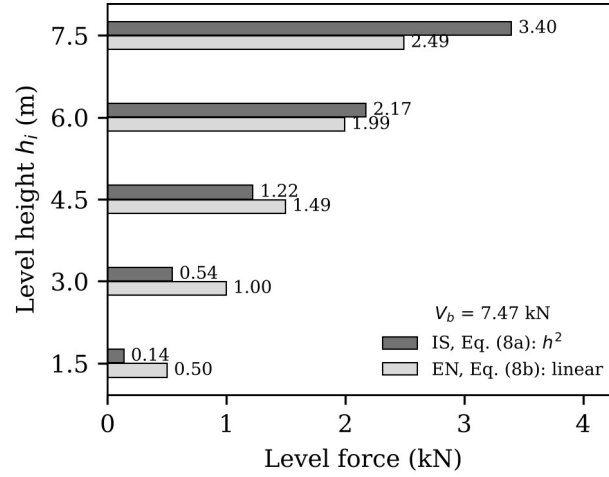


Fig. 5. Level forces for the IS (parabolic, Eq. 8a) and EN (linear, Eq. 8b) distributions of the same base shear $V_b = 7.47$ kN. Source: computed by the authors.

Table 4. Beam response for different joint idealisations (same beam as Table 3).

Joint model	$k_e L/EI$	M_{end} (kNm)	M_{mid} (kNm)	δ (mm)	$\delta/(L/200)$
Pinned	0	0.00	3.31	11.96	0.89
0.5 k_e	0.59	0.50	2.81	9.80	0.73
Test value k_e	1.17	0.81	2.49	8.43	0.62
2 k_e	2.34	1.19	2.12	6.80	0.50
Rigid	∞	2.21	1.10	2.39	0.18

7.3 Conclusions from the example

- Rack factors reduce the base shear by 19%.** With $K_D = 0.808$ the base shear is 7.47 kN, against 9.25 kN if the payload were treated as rigidly attached mass. With wooden pallets at this site $E_{D1} = 1.0$, so the whole reduction comes from the loaded-rack damping factor E_{D3} . The reduction would be larger only at sites with stronger shaking, where E_{D1} falls below 1.0.
- The force distribution, not the base shear, distinguishes the routes.** For the same base shear, the IS parabolic distribution puts 45% of V_b at the top level against 33% for the EN linear distribution. Top-level force is 37% higher and base overturning moment 12% higher. Connector and upright design at the upper levels, and base uplift, are therefore route-dependent.
- Pallet sliding must be addressed explicitly.** The elastic top-level acceleration is 0.25 g (EN) to 0.35 g (IS). This exceeds the lower-bound capacity of plastic pallets (0.10) at the upper three levels with the IS distribution and the upper four with the EN distribution, and that of wooden pallets (0.248) at the top level in both. The consequences of sliding, or load retention, must be assessed for these levels. Checking sliding with the reduced design force (0.17 g at the top level in the IS route) would have suggested, wrongly, that wooden pallets stay in place.
- Joint fixity governs beam design.** Compared with a pinned idealisation, the test-based connector stiffness reduces mid-span moment by 25% (3.31 to 2.49 kNm) and deflection by 30% (11.96 to 8.43 mm). The beam meets the L/200 limit with a ratio of 0.62. A rigid idealisation would understate mid-span moment by 56% and overstate connector moment by a factor of 2.7.
- Damping must be applied once.** Applying the withdrawn IS 1893:2002 multiplier of 1.40 to the design spectrum would raise the base shear to 10.46 kN, an overestimate of 40%.

8. Sensitivity to joint and base-plate stiffness

The down-aisle sway stiffness K is dominated by k_e and k_{base} . Consider a model whose stiffness K_{model} differs from the true stiffness K_{true} by the ratio $r = K_{model}/K_{true}$. On the 1/T branch of the spectrum, Eqs. (1), (7) and (9) give

$$\frac{V_{b,model}}{V_{b,true}} = \sqrt{r}, \quad \frac{\Delta_{model}}{\Delta_{true}} = \frac{1}{\sqrt{r}}, \quad \frac{\alpha_{cr,model}}{\alpha_{cr,true}} = r$$

because $T \propto 1/\sqrt{K}$, $S_a \propto 1/T$ and the spectral displacement $S_d = S_a (T/2\pi)^2 \propto T$. On the plateau of the spectrum the base shear is insensitive to K , but the drift error is larger ($\Delta \propto 1/r$). These relations are derived here. They are exact only when the sway stiffness scales uniformly with k_e and k_{base} , and they indicate the direction and order of the error.

Suppose the stiffness is overestimated by a factor of two ($r = 2$), for example by using a generic connector value or a value from different hardware. The computed base shear is then 41% too high, which is conservative, but the first-order drift is 29% too low. For a frame with a true $\alpha_{cr,E} = 6$ ($\theta = 0.25$), the model gives $\alpha_{cr,E} = 12$ ($\theta = 0.125$). The amplification is then taken as 1.14 instead of 1.33, and the total drift including P- Δ is underestimated by about 40% (Fig. 6). For a frame with a true $\alpha_{cr,E} = 10$ ($\theta = 0.15$), the model gives $\theta = 0.075 \leq 0.1$, and the designer would neglect second-order effects altogether. An overestimated stiffness therefore looks conservative for forces but is unconservative for drift, second-order moments in uprights, rack-to-building clearances and the choice of analysis method.

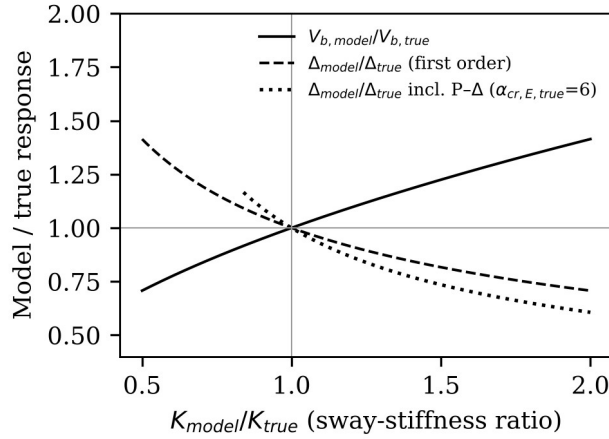


Fig. 6. Ratio of model to true response as a function of the sway-stiffness error r (1/T branch). The dotted curve assumes a true $\alpha_{cr,E} = 6$ and $q_d = 1.5$, and is shown only where the model $\theta \leq 0.3$. Source: computed by the authors from the closed-form relations in Section 8.

These results support a clear recommendation: **rack manufacturers should maintain in-house (or accredited third-party) testing capability to derive the moment-rotation characteristics of their own beam-end connectors and base plates**, in accordance with EN 15512 Annex A or ANSI MH16.1. Tests are needed for each combination of upright section, thickness, connector and beam profile supplied. The design should use the stiffness from those tests and should check sensitivity at lower and upper values. Generic literature values are not a substitute, because the stiffness depends on details (slot geometry, hook seating, weld position and steel grade) that vary between products.

9. Proposed analysis framework

Fig. 7 shows the framework recommended by the authors. It can be implemented in any frame-analysis program with rotational springs, modal analysis and linear buckling, such as RSTAB or SAP2000. Its distinctive features relative to common practice are:

1. **Test-to-model traceability.** Effective section properties, connector and base stiffness, and looseness are taken from component tests on the actual hardware (blocks A and B), not from catalogues or generic values.
2. **Damping applied once and in the right place.** The 5% design spectrum is used with q and K_D . The 3% elastic spectrum is used only for E_{D1} and sliding (block C).
3. **Period from the model.** The design period comes from modal analysis of the spring model (block E), not from building formulae.
4. **Stability-driven choice of analysis.** $\alpha_{cr,E}$ from a linear buckling analysis gives θ , which selects first-order, amplified or direct second-order analysis (blocks F and G).
5. **Rack-specific verifications.** Beyond member checks, the framework verifies drift, beam deflection, pallet sliding from the elastic spectrum, base uplift and shear, and EN 15620 clearances (block H).
6. **Stiffness-sensitivity loop.** Blocks D–H are repeated with k_e and k_{base} at lower and upper values (block I), so that the design is not tuned to a single stiffness assumption.

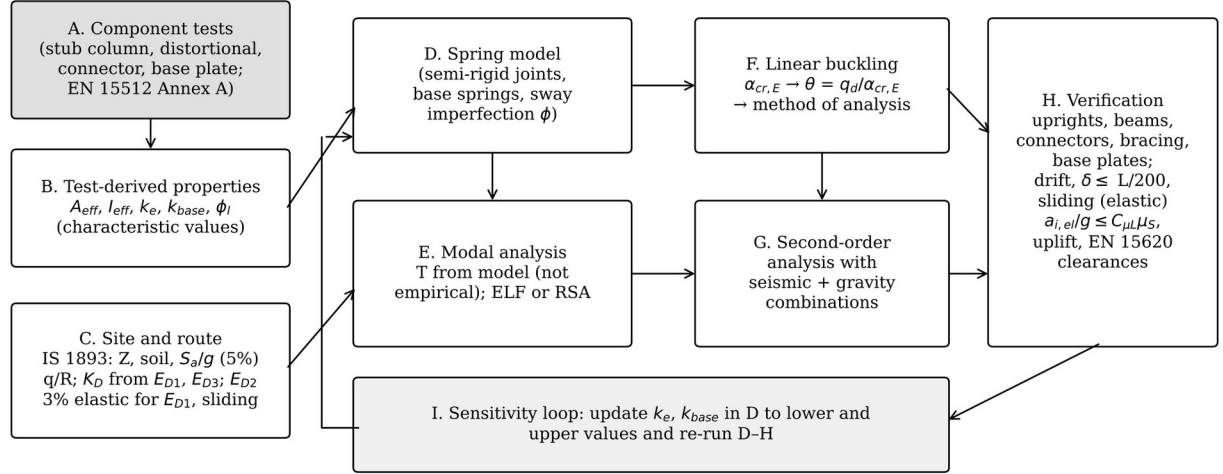


Fig. 7. Proposed analysis framework for APR seismic design. Source: developed by the authors.

10. Discussion

Choice of route. For Indian projects, the EN route is the more complete option. It is written for racks and covers sliding, product mass participation, connector testing, second-order effects and clearances within one family. The IS route is acceptable where a client or authority requires Indian Standards for member design. Its reliance on EN 16681 rack factors should then be declared, and the combination of IS 800/IS 801 resistances with EN 16681 factors has not been calibrated as a system. Section 7 shows that, with a common spectrum, the routes differ mainly in how forces are distributed up the height. A full comparison would also include load combinations, partial factors and member resistance models.

International context. The same principles appear in ANSI MH16.1 and ASCE 7 in the United States, where product-load provisions and connector tests serve the purposes of E_{D2} and EN 15512 Annex A, and in AS 4084 in Australia. The framework in Section 9 is not tied to any one code family and can be used with any of them, provided the factors are taken consistently from one family. A dedicated Indian standard for storage racking, drawing on these documents and on Indian test data, would remove the present need to borrow factors across code families.

Limitations. The worked example is a single bay with equal level masses, uses the equivalent lateral force method and maps the IS 1893 spectrum directly onto the EN route. The sensitivity relations of Section 8 assume uniform stiffness scaling and elastic behaviour. They do not capture connector non-linearity, looseness or cyclic degradation. Experimental validation on full-scale racks is recommended as future work, including shake-table tests and cyclic connector tests on Indian-manufactured hardware.

11. Conclusions

1. APR seismic design should follow one code family consistently. When designing to European rack standards, IS 1893 should be used only for site seismicity.
2. Three damping values have distinct roles. The design spectrum is 5%-damped in both routes. The 3% unloaded-rack damping of EN 16681 is used only for the elastic spectrum in the sliding factor and sliding check. The higher damping of the loaded rack enters only through $E_{D3} = 0.8$. The 2% value for steel from IS 1893:2002 no longer applies; using its multiplier would overstate the example's base shear by 40%.
3. EN 16681 rack factors reduced the base shear by 19% in the example. They must be computed for the actual site, pallet type and unit-load class and stated in the design basis.
4. For the same base shear, the IS parabolic distribution gives a 37% higher top-level force and a 12% higher overturning moment than the EN linear distribution.
5. Sliding must be checked with the elastic 3%-damped spectrum, not with reduced design forces. In the example, plastic pallets slide at the upper three or four levels and wooden pallets at the top level.
6. Semi-rigid connector stiffness changes beam mid-span moment by 25% and deflection by 30% relative to a pinned idealisation. Neither pinned nor rigid idealisations are acceptable.
7. Overestimating connector or base stiffness by a factor of two underestimates drift by about 40% once second-order effects are included, even though base shear appears conservative. It can also lead to second-order effects being neglected wrongly. Rack manufacturers should therefore have in-house or accredited testing capability to derive the stiffness of their own semi-rigid joints.
8. A second-order spring model, with the analysis method selected by $\theta = q_d/\alpha_{cr,E}$ and with a stiffness-sensitivity loop, is proposed as the minimum analytical standard for APR seismic design.

Acknowledgements

[To be completed.]

Declaration of competing interest

[To be completed by the authors, including any affiliation with rack manufacturers or suppliers.]

Data availability

All values used in the worked example and figures are given in the paper.

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