BS EN 16681:2016

BSI Standards Publication

Steel static storage systems — Adjustable pallet racking systems — Principles for seismic design

bsi.

National foreword

This British Standard is the UK implementation of EN 16681:2016.

The UK committee abstained from voting on this standard as they are of the opinion that there are generally no requirements in the UK to consider seismic loading, and that the whole of the UK may be considered an area of very low seismicity in which the provisions of EN 16681 need not apply.

The role of racking structures is such that they reside in consequence classes CC1 or CC2. Exceptionally, however, certain racking structures may warrant an explicit consideration of seismic actions. Background information on application in the UK has been published in PD 6698:2009.

The UK participation in its preparation was entrusted to Technical Committee MHE/8, Steel shelving, bins and lockers.

A list of organizations represented on this committee can be obtained on request to its secretary.

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

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Steel static storage systems - Adjustable pallet racking systems - Principles for seismic design

Systèmes de stockage statique en acier - Systèmes de rayonnages à tablettes ajustables - Principes pour le calcul parasismique

Ortsfeste Regalsysteme aus Stahl - Verstellbare Palettenregale - Leitsätze für die erdbebensichere Gestaltung

This European Standard was approved by CEN on 7 April 2016.

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European foreword

This document (EN 16681:2016) has been prepared by Technical Committee CEN/TC 344 "Steel static storage systems", the secretariat of which is held by UNI.

This European Standard shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by December 2016 and conflicting national standards shall be withdrawn at the latest by December 2016.

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0 Introduction

0.1 Effects of seismic actions on racking systems

Racking systems are load bearing structures for the storage and retrieval of goods in warehouses. The goods are generally stored on pallets or in box containers.

Racking systems are constructed from steel components; although components are standardized, they are only standard to each manufacturer. These components differ from traditional steel components in the following regard:

a) continuous perforated uprights;

b) hook-in connections;

c) structural components for racking, which generally consist of cold formed thin gauge members.

In respect of the loads, the self-weight of a rack structure is typically very small or negligible with respect to the total mass, whereas in a typical building the percentage of dead and permanent loads will be much greater.

The nature and the distribution of the goods stored on racking systems strongly affect the response and the safety of the structure under seismic actions. In fact:

— unit loads are in general simply supported vertically by the racking structure and kept in their position when loaded by inertial actions only by friction;

— unit loads are in general sub-structures with distinct dynamic characteristics in terms of frequency and damping, and their behaviour affect the response of the system.

During real earthquakes or earthquake simulated on shaking tables, movements of pallets on pallet beams were observed; these were either very small ones, contributing to the dissipation of energy by means of friction, or very large, with movements of the pallets that produced their falling between beams or outside the rack in the aisle. For this reason, friction between pallet and pallet beam and internal damping in the unit load has a relevant influence in the dynamic response of the rack and affects the entity of the inertial actions.

Also, the safety of the installation related to the movement and eventual falling of the pallets requires a proper assessment.

This European Standard deals with all the relevant and specific seismic design issues for racking systems, based on the criteria of EN 1998-1:2004, Eurocode 8.

0.2 Requirements for EN Standards for racking and shelving in addition to Eurocodes

While the basic technical description of an earthquake is the same for all structures, the general principles and technical requirements applicable for conventional steel structures have to be adapted for racking systems, in order to take into account the peculiarities of racking to achieve the requested safety level.

Also, the methods of analysis and the design requirements need to be addressed to the peculiarity of racking structures.

The scope of CEN/TC 344 is to establish European Standards providing guidance for the specification, design methods, accuracy of build and guidance for the user on the safe use of steel static storage systems.

This, together with the need of harmonized design rules was the reason that European Racking Federation ERF/FEM Racking and Shelving has taken the initiative for CEN/TC 344. CEN/TC 344 is in the course of preparation of a number of European Standards for specific types of racking and shelving and particular applications, which exist in the European Standards (EN) and working group activities (WG).

0.3 Liaison

CEN/TC 344 “Steel Static Storage Systems” liaise with CEN/TC 250 “Structural Eurocodes”, CEN/TC 135 “Execution of steel structures and aluminium structures” and CEN/TC 149 “Power operated warehouse equipment”.

0.4 Additional information specific to EN 16681

This European Standard is intended to be used with EN 1998-1, EN 15512 and related standards.

EN 1998-1 is the first of 6 parts; it gives design rules intended to be used for structures fabricated with conventional materials, including steel.

EN 15512 is the reference standard for the design of racking structures and components; it addresses the principles of the EN 1990, Eurocode, and EN 1993 series, Eurocode 3, to the adjustable pallet racking systems and it needs to be applied also when actions are produced by an earthquake.

1 Scope

This European Standard specifies the structural design requirements applicable to all types of adjustable pallet racking systems fabricated from steel members, intended for storage of unit loads and subject to seismic actions.

This European Standard gives also guidelines for the design of clad rack buildings in seismic zones, where requirements are not covered in the EN 1998 series.

This European Standard does not cover other generic types of storage structures. Specifically, this European Standard does not apply to mobile storage systems, drive-in, drive-through and cantilever racks or static steel shelving systems.

This European Standard does not apply to the design of seismic isolated racking structures.

2 Normative references

The following documents, in whole or in part, are normatively referenced in this document and are indispensable for its application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.[1)]

EN 1090-2, *Execution of steel structures and aluminium structures - Part 2: Technical requirements for steel structures*

EN 1990 (all parts), *Eurocode - Basis of structural design*

EN 1993 (all parts), *Eurocode 3 - Design of steel structures*

EN 1998-1:2004[2)], *Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings*

EN 15512:2009, *Steel static storage systems - Adjustable pallet racking systems - Principles for structural design*

EN 15620, *Steel static storage systems - Adjustable pallet racking - Tolerances, deformations and clearances*

EN 15629:2008, *Steel static storage systems - Specification of storage equipment*

EN 15635:2008, *Steel static storage systems - Application and maintenance of storage equipment*

EN 15878:2010, *Steel static storage systems - Terms and definitions*

ETAG 001 series, *Guideline for European technical approval of metal anchors for use in concrete*

1) Complementary rules to existing Norms specific for seismic applications are included in the following annexes:

— Annex I "*Data to be exchanged between the Specifier/End User and the rack's Supplier*" as complement to EN 15629:2008

— Annex J "*Complementary rules to* EN 15635" as complement to EN 15635:2008

— Annex K "*Complementary rules to* EN 15629" as complement to EN 15629:2008

2) This document is impacted by the amendment EN 1998-1:2004/A1:2013.

3 Terms and definitions

For the purposes of this document, the terms and definitions given in EN 15878:2010 and EN 1998-1:2004 and the following apply.

3.1
associated mass
portion of the total mass of the structure affecting the seismic behaviour of the structural element or of the substructure analysed

3.2
mass regularity
in elevation: situation in which the mass of the individual load level remains constant or reduces gradually without abrupt changes from the base to the top of the rack

in plan: situation in which the mass is distributed without significant horizontal eccentricity with respect to the lateral resisting system

3.3
principal directions
down aisle direction and cross aisle direction in a rack

[SOURCE: EN 15878:2010, definitions 3.2.14 and 3.2.15, modified — the content of the defined term stems from these two original definitions]

3.4
PRSES
person responsible for storage equipment safety

[SOURCE: EN 15635:2008, 3.18]

3.5
rack filling grade reduction factor
$\mathbf{R_F}$
statistical reduction factor intended to take into account the probability that not all of the pallets will be present and at their maximum weight at the time of the design earthquake

3.6
seismic weight
value of weight of a mass allowed in seismic design for the calculation of the seismic action

3.7
specifier
person or company that provides the supplier with a specification based on user's requirements

[SOURCE: EN 15629:2008, 3.23]

4 Symbols and abbreviations

4.1 Symbols

For the purposes of this document, a number of the following symbols may be used together with standard subscripts, which are given later. Additional symbols and subscripts are defined where they first occur.

$A_{E,d}$	design value of the seismic action for the reference return period
a_g	design ground acceleration on type A ground
a_{gR}	reference ground acceleration (PGA) for the reference return period of 475 years
b	distance between the uprights axes
β	lower bound factor for the design spectrum
$C_{\mu L}$; $C_{\mu H}$	correction factors for unit load-beam friction coefficient (lower and upper bound values)
$d_{r,i}$	design inter-storey drift above storey i
ξ	viscous damping ratio expressed as percentage of critical damping
e_i	height of the centre of gravity of the unit load at level i
E	Young modulus of the material
E_d	value of the effect due to the design action
E_{D1}	design spectrum modification factor
E_{D2}	pallet weight modification factor
E_{D3}	design spectrum modification factor
E_{Edx}	effect due to the application of the seismic action along the horizontal axis x
E_{Edy}	effect due to the application of the seismic action along the horizontal axis y
E_{Edz}	effect due to the application of the seismic action along the vertical axis z
θ	inter-storey drift sensitivity coefficient
θ_i	inter-storey drift sensitivity coefficient between levels i and i+1
θ_p	rotation capacity of the beam-end connector
$F_{b,Rd}$	bearing strength of bolted connection
f_k	characteristic strength of the material
$F_{E,i}$	horizontal force at level i of the rack
$F_{v,Rd}$	bolt’s shear strength
Φ_k	rotation at Mk, both positive and negative
G_i	permanent load
$G_{k,i}$	characteristic value of the permanent action

g	gravity acceleration
γ_I	importance factor
γ_M	material's factor
γ_f	load factor
η	damping correction factor
H	height of the frame bracing pitch or the distance from floor to first horizontal
H/b	height to minimum width ratio of an unit load
h_i	inter-storey height above the storey i
H_i	horizontal action on the unit load at level i
I	moment of inertia of the upright
k_S	coefficient related to number of tests
K	buckling length factor
K_D	effective spectrum modification factor
L	beam's length
λ	seismic shear force calculation coefficient in LFMA
$\overline{\lambda}$	adimensional slenderness (EN 15512 and EN 1993-1)
M_k	characteristic bending strength obtained from monotonic tests, both positive and negative
μ_m	mean value of the unit load-beam friction coefficient obtained from tests
$\mu_{n,i}$	individual test result of the unit load-beam friction coefficient
N_{Rd}	axial strength of a member
ΔN_{base}	additional vertical force at the base of the uprights
ΔN_i	additional vertical action at beam-upright intersection
μ_S	unit load-beam friction coefficient
P_c	constant downward load in the bending test
$P_{cr,E}$	Euler critical load
P_E	total gravity load in the seismic design situation
$P_{E,i}$	total gravity load at and above the considered storey i, in the seismic design situation
$P_{E,prod}$	total product weight on the rack
q	behaviour factor
q_d	displacement behaviour factor
q_{rack}	behavior factor of the rack

$Q_{k,i}$ characteristic value of a variable action

$Q_{P;max}$ specified weight of the unit load (see EN 15629, 6.7.1)

$Q_{P;rated}$ specified value of the weight of unit loads for the compartment, upright frame or global down aisle design (see EN 15512) as specified by the Specifier (see also EN 15629:2008, 6.7.1).

$Q_{p,rated,i}$ specified weight of the product stored at the level i in the seismic condition

R_d design resistance of the element

R_F rack filling grade reduction factor

s standard deviation of a number of tests results

S soil parameter

S_a seismic coefficient specified in EN 1998-1:2004, 4.3.5.2 for the analysis of non structural components

$S_d(T)$ ordinate of the design spectrum (normalized by g)

$S_{d,mod}(T)$ ordinate of the modified design spectrum for racks (normalized by g)

$S_e(T)$ ordinate of the elastic spectrum (normalized by g)

T period of vibration

T_1 fundamental period of vibration

T_B, T_C limits of the constant spectral acceleration branch

T_D period value defining the beginning of the spectrum constant displacement range

V_E seismic base shear force

$V_{E,i}$ total seismic storey shear at the considered storey i

W_E weight of the seismic mass considered in the analysis

$W_{E,G}$ weight of permanent loads

$W_{E,Q}$ weight of the variable loads

$W_{E,tot}$ total weight of the seismic mass of the rack

$W_{E,UL}$ seismic design weight of the unit load to be considered in the seismic analysis

$\psi_{2,i}$ combination factor for variable actions

Z distance from the ground level to the fixing level of the rack

4.2 Abbreviations

LFMA Lateral Force Method of Analysis

MRSA Modal Response Spectrum Analysis

LDMA Large Displacement Method of Analysis

5 Performance requirements and compliance criteria

5.1 Applicability

Non-seismic design shall comply with EN 15512. The reference to the tests and quality control of components and materials is based on EN 15512.

In case of very low seismicity conditions, the racking structures need not to be designed for earthquake (see also EN 1998-1:2004, 3.2.1, (5)P).

National Regulations shall be followed to define general conditions of applicability of the seismic design.

It is recommended to consider as very low seismicity cases either those in which the design ground acceleration on type A ground, $\gamma_I\ a_{gR}$, is not greater than 0,04 g, or those where the product $\gamma_I\ a_{gR}\ S$ is not greater than 0,05 g.

5.2 Performance requirements

5.2.1 No collapse requirement

The racking structure shall be designed and constructed to withstand the design seismic action without local or general collapse, retaining its structural integrity and a residual load bearing capacity after the seismic event.

Ultimate limit states are those associated with the collapse, or with other forms of structural failure, that may endanger the safety of people.

The structural system shall be verified as having the specified resistance and ductility.

5.2.2 Damage limitation requirement

No specific design requirement is prescribed in this European Standard. The movement of the stored unit loads does not constitute damage.

NOTE Reference is made to Annex J (normative) for integrity controls after a seismic event.

5.2.3 Movement of unit loads

Movement of unit loads shall be considered in the design when appropriate.

NOTE 1 Seismic accelerations can cause sliding of the pallets on the supporting beams, when the inertial horizontal forces on the pallet exceed the static friction force between pallet and beam.

This effect has been demonstrated by full scale tests to occur for small values of ground accelerations (low intensity earthquakes) with wooden or plastic pallets on painted or zinc coated steel beams, because of the structural amplification of the seismic forces at the highest storage levels.

The consequences of these phenomena are the reduction of the seismic action on the rack, due to the energy dissipation and the limitation of the horizontal action that can be transferred from the pallet to the rack structure, and the risk of unit loads falling, that can cause local or global collapse of the rack, or injury to people.

NOTE 2 The modification of the seismic response of the structure is considered in this European Standard by means of three coefficients that estimate the effects of typical phenomena of racking structures, such as energy dissipation due to the pallet-beam friction, damping due to the movement of the stored products, pallet flexibility, and others:

— E_{D1} and E_{D3} is the design spectrum modification factors,

— E_{D2} is the mass modification factor.

The PRSES shall assess the risks related to the sliding of the unit loads and the possibility of their falling from the rack.

The guidelines for this evaluation are given in 9.2.2.

6 Ground conditions and seismic action

6.1 General

The earthquake motion at a given point of the Earth's surface is defined in EN 1998-1 and related National Annexes.

When the soil properties are not known in sufficient detail to determine the site soil conditions, Ground type D is assumed.

6.2 Damping

When not otherwise specified, the viscous damping ratio ξ of the unloaded rack's steel structure, expressed as percentage of the critical damping, shall be assumed equal to:

$$\xi = 3\ \%$$

NOTE The damping ratio of the loaded racking structure in operating conditions is higher and is taken into account in E_{D3} (see 7.5.2).

6.3 Importance factor γ_I

Importance factors not less than the ones defined in Table 1 shall be used. The Specifier is responsible for the selection of the Importance Class and the design life for the rack.

NOTE 1 The minimum 30 year design life is solely in relation to the seismic design and differs from the normal static design life of minimum 10 years as given in EN 15512.

Higher importance factors can be specified. Unless otherwise required, the importance factor for the rack need not to be greater than the importance factor specified for the part of the building in which the racks are located.

Table 1 — Importance factors for racks

Importance Class	Description	Importance factor γ_I	
		30 years design life	50 years design life
I	Warehouses with fully automated storage operations	0,67	0,8
II	Standard warehouse conditions, including picking areas	0,84	1,0
III	Any kind of rack with random public access for private customer	N/A	1,2
IV	Hazardous product storage *(Hazardous products storage is subject to the approval of National Authorities)* Strategic facilities	N/A	1,4

The default value of conventional warehouse racks shall be 30 years for Class I and II, and 50 years for clad rack buildings.

If National regulations require different design life this shall be considered.

NOTE 2 Importance class I: 20 % probability of exceedance of the seismic action in 30 years and 50 years.

Importance class II: 10 % probability of exceedance of the seismic action in 30 years and 50 years.

Importance class III: 5,8 % probability of exceedance of the seismic action in 50 years.

Importance class IV: 3,65 % probability of exceedance of the seismic action in 50 years.

6.4 Horizontal component of the seismic action

The horizontal component of the seismic action shall be evaluated according to EN 1998-1.

6.5 Vertical component of the seismic action

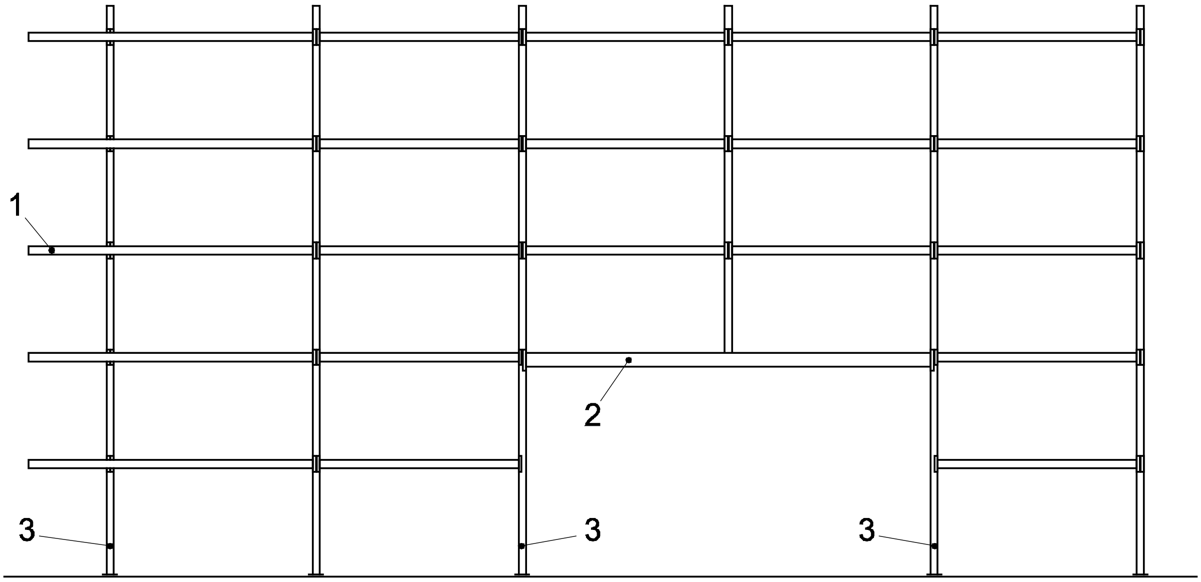
The vertical component of the seismic action shall be evaluated according to EN 1998-1.

The vertical component of the seismic action need only to be taken into account in the following relevant cases as shown in Figure 1:

a) cantilever components;

b) beams supporting columns (for example in order picking tunnels);

c) elements or substructures supporting cantilevers or supported by beams.



Key

1 cantilever components

2 beams supporting columns (for example in order picking tunnels)

3 their directly associated supporting elements or substructures

Figure 1 — Elements to be designed for vertical component of the seismic action

6.6 Design ground displacement

The design ground displacement shall be evaluated according to EN 1998-1.

6.7 Racks supported by suspended floors

The design of racks supported by suspended floors shall be based on realistic model including the structure on which the rack is installed.

Alternatively, response spectra including the effect of the dynamic response of the supporting structure and its interaction with the supported rack can be used; guidance is given in Annex E (informative).

7 Methods of analysis

7.1 General

The reference method for the evaluation of the seismic effects on the racking structures is the modal response spectrum analysis. This shall be performed using a linear elastic model of the structure and the modified design spectrum $S_{d,mod}(T)$ defined in 7.5.1, and, when applicable, the design spectrum for the vertical component defined in 6.5.

7.2 Limitation of the vertical load referred to the critical Euler Load

In all cases when $\gamma_I \; a_{gR} \; S \geq 0{,}1\text{g}$ (or other value prescribed in the National Annex to EN 1998-1 or National Regulations) the following limitation shall be fulfilled:

$$P_E / P_{cr,E} \leq 0{,}5$$

where

P_E is the total gravity load of the rack in the seismic design situation;

$P_{cr,E}$ is the Euler critical load.

$P_{cr,E}$ shall be obtained either from buckling analysis or approximated according to Annex B, C and G of EN 15512:2009.

7.3 Inter-storey drift sensitivity coefficient

The requirement to account for second order effects is related to the maximum value of the inter-storey drift sensitivity coefficient defined as:

$$\theta_i = \left(P_{E,i}\, d_{r,i}\right) / \left(V_{E,i}\, h_i\right) \tag{1}$$

$$\theta = \max\left[\theta_i\right]$$

where

θ_i is inter-storey drift sensitivity coefficient for the fundamental mode at storey i;

$P_{E,i}$ is total gravity load above the considered storey, in the seismic design situation;

$d_{r,i}$ is design inter-storey drift, evaluated as the difference of the average lateral displacements at the top and bottom of the storey under consideration and calculated according to 7.4.7 by means of linear elastic 1st order analysis;

$V_{E,i}$ is total seismic storey shear at the considered storey i;

h_i is inter-storey height above the considered storey i;

θ is Inter-storey drift sensitivity coefficient for the fundamental mode.

Alternatively, the inter-storey drift sensitivity coefficient θ is obtainable as follows:

$$\theta = q_d \times P_E / P_{cr,E} \qquad (2)$$

where

P_E is total gravity load of the rack in the seismic design situation;

$P_{cr,E}$ is Euler critical load;

q_d is displacements behaviour factor.

NOTE 1 The total gravity load in seismic conditions P_E is determined from the specified value of unit loads $Q_{p,rated}$ and the rack filling factor R_F defined in 7.5.4.

NOTE 2 Refer to the EN 1998 series for the definition of q_d.

7.4 Analysis procedures

7.4.1 General

The procedures described below shall be applied (see Tables 2 and 3).

Other methods of analysis can be used according to EN 1998-1.

7.4.2 Second order effects

7.4.2.1 General

When $\theta \leq 0,1$, second order effects can be neglected.

Second order effects shall be considered according to 7.4.2.2 for low dissipative concept and 7.4.2.3 for dissipative concept.

7.4.2.2 Low dissipative concept (behaviour factor q ≤ 2)

Analysis methods directly considering second order effects shall be used (see Annex A); the applicable methods of analysis are summarized in Table 2.

Alternatively they can be approximated by multiplying seismic action effects obtained from first order analysis by a factor equal to $1/(1-\theta)$.

Amplification of 2nd order effects by factor $1/(1-\theta)$ is in general conservative; it is not recommended if $\theta > 0,3$ as results tend to be unduly conservative.

7.4.2.3 Dissipative concept (behaviour factor q > 2)

The applicable methods of analysis for dissipative design concept are summarized in Table 3.

If $\theta \leq \theta_1$ analysis methods directly considering second order effects should be used (see Annex A).

Alternatively they could be approximated by multiplying the seismic action effects obtained from first order analysis by a factor equal to $1/(1-\theta)$.

If $\theta \leq \theta_2$, pushover analysis according to EN 1998-1 or large displacement analysis presented in 7.4.5 (Large Displacement Method of Analysis - LDMA) shall be used.

If $\theta > \theta_2$, a time history analysis including large displacements and nonlinear behaviour of materials and connections shall be used.

Time-history analysis shall be according to EN 1998-1.

The following values may be assumed:

$\theta_1 = 0{,}3$

$\theta_2 = 0{,}5$

Table 2 — Summary of methods of analysis for low dissipative structural behaviour

θ	**Method of analysis**	**Second order effects**
$\theta \leq 0{,}1$	LFMA (7.4.3) or MRSA (7.4.4)	Negligible
$\theta > 0{,}1$		Shall be either considered directly in the analysis or approximated
NOTE For racks not regular in plan and elevation, refer to EN 1998–1.		

Table 3 — Summary of methods of analysis for dissipative structural behaviour

θ	**Method of analysis**	**Second order effects**
$\theta \leq 0{,}1$	LFMA (7.4.3) or MRSA (7.4.4)	Negligible
$\theta \leq \theta_1$		Shall be either considered directly in the analysis or approximated
$\theta \leq \theta_2$	Pushover analysis according to EN 1998–1 or LDMA according to 7.4.5	
$\theta > \theta_2$	Time history analysis including geometrical and material nonlinearity according to EN 1998–1	
NOTE For racks not regular in plan and elevation, refer to EN 1998–1.		

7.4.3 Lateral Force Method of Analysis (LFMA)

This method of analysis can be applied when response is not significantly affected by contribution of higher modes of vibration in each principal direction.

This requirement is considered fulfilled either for structures:

— that are stiffness and mass regular in elevation with fundamental period of vibration T_1 in the two principal directions less than the following values:

$T_1 \leq 4 \times T_C$

$T_1 \leq 2$ s

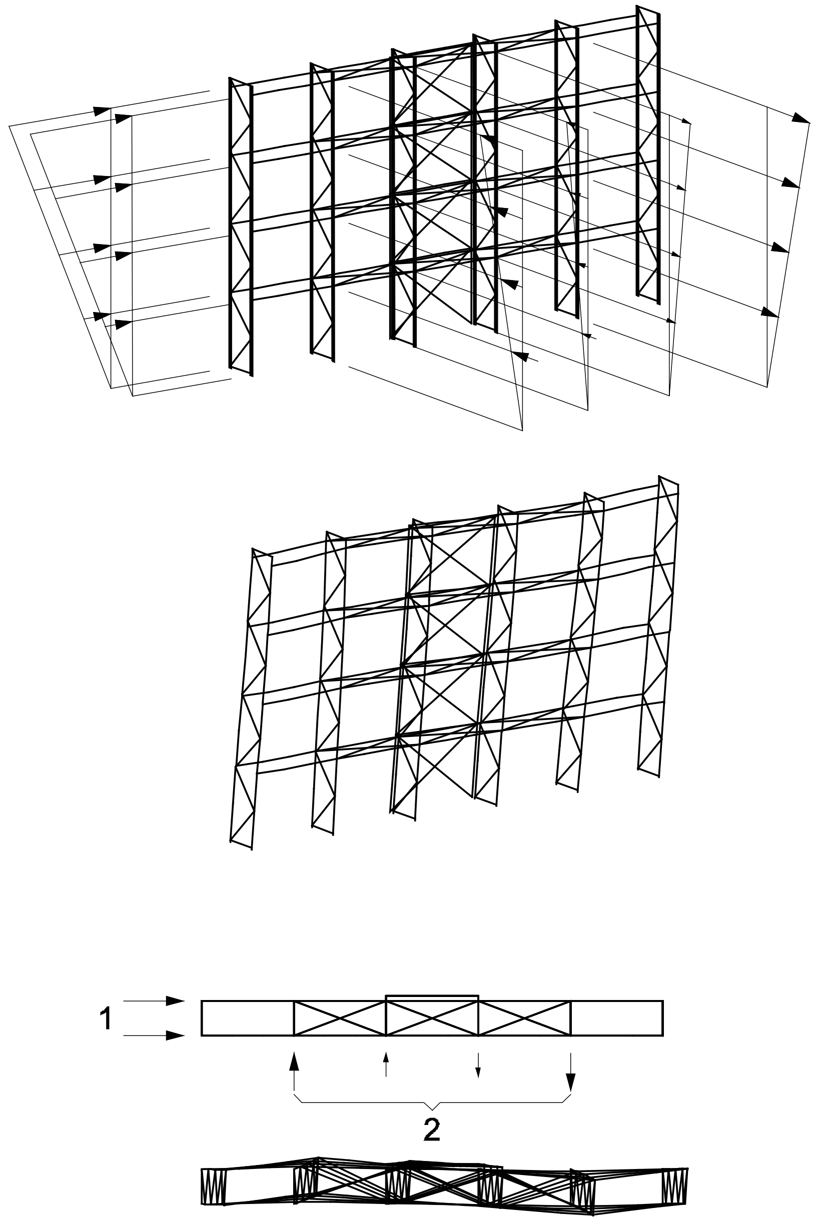
where T_C is the upper limit of the constant spectrum acceleration branch of the design spectrum, given in EN 1998-1 and in the relevant national Standards, or

— in which the modal mass associated to the fundamental period is greater than 90 % of the total mass.

See 8.1.4 for regularity.

For structures not regular in plan, the system of applied horizontal forces shall be homothetic to the three-dimensional fundamental modal shape, including the torsional component. Figure 2 shows an example of system of forces on a non-regular rack in plan.

NOTE The system of forces applied to the structure has a resultant equal to the base shear in the relevant direction, and a null resultant in the other direction.



Key

1 system of longitudinal actions

2 system of actions in transversal direction with null resultant

Figure 2 — Example of system of forces on rack non-regular in plan

Simplified formulas for the evaluation of the fundamental period typical for buildings are not allowed for racks.

The seismic base shear force V_E for each main direction is determined as follows:

$$V_E = S_{d,\mathrm{mod}}(T_1) \times W_{E,tot} \times \lambda \tag{3}$$

where

$S_{d,mod}(T_1)$ is ordinate of the modified design spectrum, in g units, defined in 7.5.1;

T_1 is fundamental period of vibration for translational motion in the direction under consideration;

$W_{E,tot}$ is total weight of the seismic mass of the rack.

λ = 0,85 for racks with 3 or more load levels (excluded floor) and $T_1 \leq 2T_c$; otherwise λ = 1,0.

If the fundamental period is not evaluated, the maximum value of the design spectrum shall be assumed.

The vertical distribution of the seismic base shear shall be according to EN 1998-1.

7.4.4 Modal Response Spectrum Analysis (MRSA)

The stiffness matrix of the model may include the terms that reduce the stiffness of the system due to the vertical loads.

For structures complying with the criteria for regularity in plan (see 8.1.4) the analysis can be performed using two planar models, one for each principal direction.

Structures not complying with these criteria shall be analysed by means of a spatial model.

The method shall be applied according to EN 1998-1.

NOTE In Annex A, MRSA methods including second order effects are presented.

7.4.5 Large Displacement Method of Analysis (LDMA)

When large displacements analysis is performed, the load history shall be defined as follows:

1) the target displacement is first calculated; the target displacement is the maximum horizontal displacement in the direction considered, obtained from MRSA multiplied by the behaviour factor q; the point in which the target displacement is calculated is the target point;

2) the pattern of horizontal forces shall be determined using a distribution matching the modal shape of the fundamental mode in each of the two principal directions;

3) the vertical loads shall be first applied to the structure, then the horizontal forces shall be incremented until the target point reaches a displacement equal to 1,1 times the target displacement; under this condition the structure and all components must remain stable.

The nonlinear behaviour of materials and connections shall be taken into account.

7.4.6 Combination of the effects of the components of the seismic action

The horizontal and vertical components of the seismic action shall be combined according to EN 1998-1.

The general method applicable to racking structures is:

$$0{,}30 \times E_{Edx} \text{ “+” } 0{,}30 \times E_{Edy} \text{ “+” } E_{Edz} \tag{4}$$

$$E_{Edx} \text{"+"}\, 0{,}30 \times E_{Edy} \text{"+"}\, 0{,}30 \times E_{Edz} \quad (5)$$

$$0{,}30 \times E_{Edx} \text{"+"}\, E_{Edy} \text{"+"}\, 0{,}30 \times E_{Edz} \quad (6)$$

where

"+" implies "to be combined with"

E_{Edx} = action effects due to the application of the seismic action along the principal x-axis of the structure;

E_{Edy} = action effects due to the application of the same seismic action along the principal y-axis of the structure;

E_{Edz} = action effects due to the application of the vertical component of the design seismic action (when applicable).

The sign of each component in the above combinations shall be taken as the most unfavourable for the effect under consideration.

7.4.7 Displacements calculation

Displacements induced by the design seismic action shall be calculated according to EN 1998-1, based on the modified spectrum $S_{d,mod}(T)$ defined in 7.5.1.

NOTE For the evaluation of the displacements, the lower bound factor β is ignored in the definition of the design spectrum S_d (T).

7.5 Design parameters for seismic analysis

7.5.1 General

The seismic design of racks shall be performed using the following modified design spectrum:

$$S_{d,\mathrm{mod}} = K_D \times S_d(T) \quad (7)$$

where

$S_d(T)$ is the reference design spectrum defined in EN 1998-1

E_{D1} and E_{D3} are the design spectrum modification factors, with $E_{D1} \cdot E_{D3} \geq 0{,}4$

$$K_D = 1 - P_{E,prod} / P_E \times (1 - E_{D1} \times E_{D3}) \quad (8)$$

where:

P_E is the total weight of the rack in the seismic design situation (including dead weight, permanent weight, live load in the seismic situation and the stored product weight);

$P_{E,prod}$ is the total product weight stored on the rack, in the seismic design situation.

$K_D = E_{D1} \times E_{D3}$ can be assumed when $P_{E,prod}$ is greater or equal to 90 % of P_E.

7.5.2 Design spectrum modification factors

The design spectrum modification factors E_{D1} and E_{D3} take into account the curtailment and modification to the ordinate of the design spectrum.

a) E_{D1} is affected by the following parameters:

1) intensity of the seismic action;

2) number of load levels, total mass and flexibility of the racking structure, expressed by the period of vibration (dominant period in the direction considered);

3) maximum horizontal force that can be transmitted by the unit load to the beams, expressed in terms of the unit load-beam friction coefficient;

$$E_{D1} = \max\left[0{,}4; \mu_S / S_e\left(T_1\right) + 0{,}2\right] \leq 1{,}0 \tag{9}$$

where

μ_S	is the reference value of the unit load-beam friction coefficient given in 7.5.3
T_1	is the fundamental period of vibration of the racking structure in the considered direction (the period with highest modal participating mass in the considered direction)
$S_e(T_1)$	is the ordinate of the elastic spectrum defined in EN 1998-1 (normalized by g) with the viscous damping as defined in 6.2.

When unit loads are restrained on the beams by means of any special system (for example materials increasing the friction between pallet and beam), E_{D1} = 1,0.

b) E_{D3} is a reduction coefficient of the seismic action.

The value of E_{D3} is 0,80

NOTE 1 E_{D3} is introduced to account for the dissipative phenomena typical of the dynamic behaviour of racking structures under seismic actions that are not included in the mathematical formulation presented in this European Standard, but that are observed on racks that have suffered earthquakes, and from tests performed on shaking tables.

NOTE 2 The value of E_{D3} = 0,8 corresponds to a conventional viscous damping ratio ξ of 10 % of the loaded rack describing dissipative mechanisms existing in the whole system.

7.5.3 Unit load-beam friction coefficients

The following parameters shall be used to consider the effects of the friction between unit load and support beams:

μ_S	reference value of the friction coefficient
$C_{\mu L}$	lower bound correction coefficient of the statistical distribution of μ_S
$C_{\mu H}$	upper bound correction coefficient of the statistical distribution of μ_S

Reference values of μ_S and values of $C_{\mu L}$ and $C_{\mu H}$ are obtained from tests in accordance with Annex B.

If tests are not performed, $C_{\mu L}$ = 0,67 and $C_{\mu H}$ = 1,50 shall be assumed and recommended unit load-beam friction coefficients are given in Table 4.

Table 4 — Recommended reference values for the pallet-beam friction coefficient

Materials in contact	Environment	Reference value of pallet-beam friction coefficient μ_S
Steel beams – all coatings Wooden pallet	Warehouse normal conditions	0,37
Steel beams – all coatings Plastic and steel pallet	Warehouse normal conditions	0,15
NOTE Warehouse normal conditions are defined in EN 15629.		

In all other conditions, unit load-beam coefficients shall be defined by tests considering appropriate warehouse conditions.

7.5.4 Design seismic weight of the unit load

The design weight of the unit load $W_{E,UL}$ to be considered in the evaluation of the horizontal seismic action, shall be determined as follows:

$$W_{E,UL} = R_F \times E_{D2} \times Q_{P;rated} \tag{10}$$

where

R_F is rack filling grade reduction factor, related to the occupancy of stored goods in the rack that can be assumed during the seismic event, to be defined by the Specifier based on statistical evaluations: for analysis in cross aisle direction R_F = 1,0 shall be assumed; for analysis in down aisle direction $R_F \geq 0{,}8$ shall be assumed;

E_{D2} is unit load weight modification factor (see 7.5.5);

$Q_{P;rated}$ is specified value of the weight of unit loads for the compartment, upright frame or global down aisle design (see EN 15512) as specified by the Specifier (see also EN 15629:2008, 6.7.1).

NOTE E_{D2} modifies the period and the horizontal action.

Unless otherwise specified, R_F = 1,0 shall be considered.

7.5.5 Unit load weight modification factor

Unless otherwise specified, the values of E_{D2} in Table 5 shall be considered, depending on the type of unit load and stored goods for specified classes of the product.

Table 5 — Unit load weight modification factors

Class	E_{D2}	Stored good classes	Example
A	1,0	COMPACT CONSTRAINED	Frozen goods (cold storage) Steel sheet package Coils and paper rolls
B	0,8	WEAK	Big number of pieces stored on the pallet whose size is small in comparison to the pallet size, including goods stabilized by stretch wrapping
C	0,7	LOOSE AND UNCONSTRAINED	Goods that can easily move around, inside the container (e.g. granulated materials)
D	1,0	LIQUID	Unit load containing liquid that can slosh in the container

NOTE The unit load weight modification factor E_{D2} represents the effects of the interaction between the unit load and the racking structure. This coefficient affects the response to earthquake in terms of participating mass and modification of the period of vibration.

7.5.6 Other seismic weights

All the permanent and live loads other than the unit load weight shall be considered in the seismic analysis.

Refer to EN 15512 for the definition of:

a) Dead loads:

   1) weights of materials and constructions;

   2) weights of fixed service equipment.

b) Minimum floor and walkway loads.

Refer to EN 15512 for the design values.

7.5.7 Weight of the seismic masses

The weight of the seismic mass W_E shall be determined considering all the permanent and variable loads supported by the rack.

$$W_E = W_{E,G} + W_{E,Q} + W_{E,UL} \tag{11}$$

where

$W_{E,G} = \Sigma\ G_{k,i}$	is the permanent load;
$W_{E,Q} = \Sigma\ \psi_{2,i}\ Q_{k,i}$	is the variable load;
$W_{E,UL}$	is the design seismic weight of the unit load defined in 7.5.4;
$G_{k,i}$	is the characteristic value of the permanent load;
$Q_{k,i}$	is the characteristic value of the variable load;
$\psi_{2,i}$	is the combination factor of the variable loads, defined in 9.2.1.1.

7.5.8 Position of the centre of gravity of the unit load

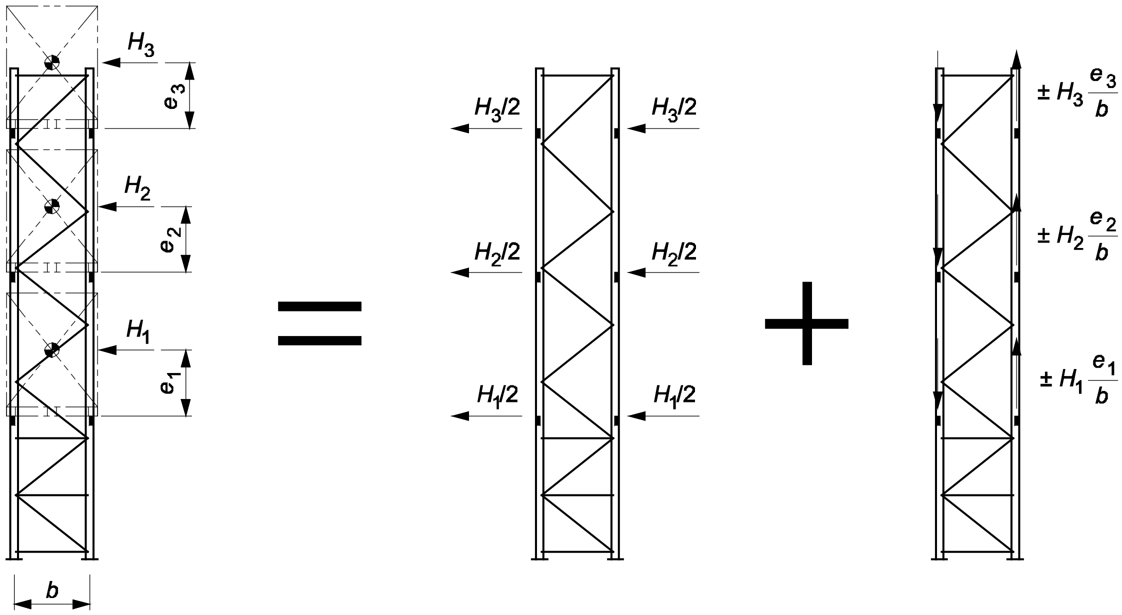
The position of the centre of gravity of the unit load above the beams shall be considered in the analysis as specified in the following of this clause.

a) Cross aisle direction:

   The elevation of the centre of gravity of the unit load with respect to the support beams (vertical eccentricity) shall be considered.

   The masses of the unit loads shall be positioned above support beam level (at the elevation of their centre of gravity) for the evaluation of the period of vibration and of the seismic action and for the design of the pallet beams and their connections.

   Figure 3 shows an example of the effect of the vertical eccentricity of the centre of gravity of the unit load.



Key

b	width of the upright frame
H_1, H_2, H_3	seismic action acting on unit loads
e_1, e_2, e_3	vertical eccentricity of the centre of gravity of the unit loads above pallet beams

Figure 3 — Effect of the vertical eccentricity of the centre of gravity of the unit load

NOTE In Annex C is presented a method to consider the effect of the vertical eccentricity of the centre of gravity of the of the unit load in the numerical model of the upright frame.

In some circumstances, the computational effort involved with this requirement is considerable. A practical alternative approach, that will reduce computational effort, is to place the mass at the beam level and to apply additional actions to the members as explained in Annex D (informative).

b) Down aisle direction:

In general, the vertical eccentricity of the unit load can be neglected in the analysis in down aisle direction, except for single bay racks.

In Figure 4 an example of the effect of the vertical eccentricity of the centre of gravity of the unit load in the model in down aisle direction is reported; it is shown that the overturning forces of adjacent unit loads are in opposite direction and their effect is null, apart in the side uprights where their effect can be neglected (except in the single bay runs).

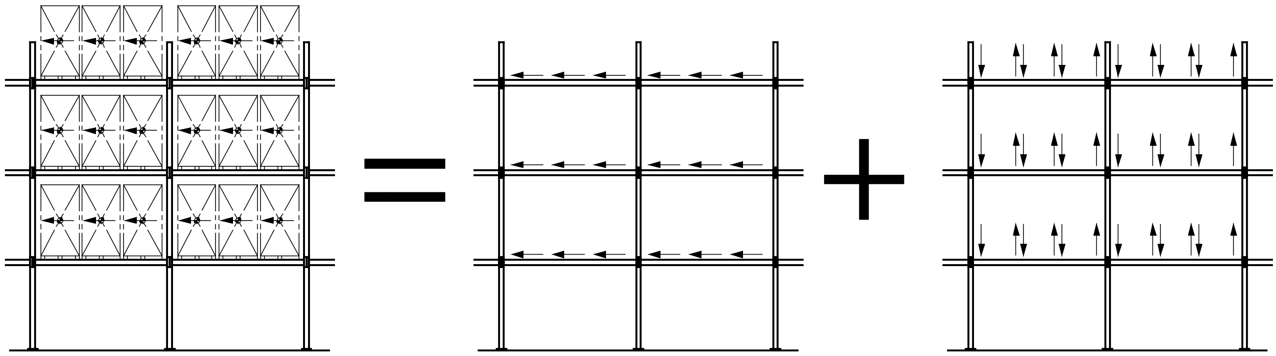


Figure 4 — Effect of the vertical eccentricity of the centre of gravity of the unit load in the model in down aisle direction

7.5.9 Positioning tolerances

The eccentricity due to the placement tolerances of the unit loads may be neglected in the seismic design.

The systematic eccentricities due to the rack configuration shall be taken into account.

7.5.10 Structural regularity criteria

Structures shall be classified as "regular" or "non-regular".

NOTE The concept of regularity is related to both mass and stiffness distribution, in plan and in elevation. The criteria for pallet racks are given in 8.1.4.

This distinction has implications in seismic design, as described below:

— the structural analysis that can be carried out using either a simplified planar or a spatial numerical model;

— the method of analysis that can be either lateral force procedure (base shear method) or modal response spectrum analysis;

— the value of the behaviour factor q, which shall be decreased for non-regularity in elevation when dissipative behaviour is assumed; the reduction of the behaviour factors is provided in 8.1.3.

The analysis method chosen shall be according to Table 6.

Table 6 — Consequences of structural regularity on seismic analysis and design

Regularity		Allowed simplification[a]		Behaviour factor q
Plan	Elevation	Model	Method of analysis	
Yes	Yes	Planar	LFMA or LDMA for dissipative concept	Reference value
Yes	No	Planar		Decreased for dissipative structural behaviour (see also 8.1.3)
No	Yes	Spatial	LFMA or LDMA for dissipative concept	Reference value
No	No	Spatial		Decreased for dissipative structural behaviour (see also 8.1.3)

[a] The reference method is the modal response spectrum analysis. See also requirements in 7.4.

7.6 Modelling assumptions for structural analysis

7.6.1 Sub-modelling

It is allowed to consider the rack as a number of separate sub-structures, provided that the following conditions are met:

a) each lateral resisting system shall be analysed individually, subject to the pertinent seismic action, by means of sub-models (i.e. upright frames, vertical bracing);

b) each subsystem shall be analysed considering all the seismic resistant elements with the masses that affect the behaviour of the sub-structure (i.e. rear bracing, horizontal bracing and connected upright frames);

Torsional effects shall be taken into account when relevant.

7.6.2 Distribution of the masses

The most unfavourable loading configuration for the seismic analysis shall be considered.

At least the following shall be considered:

a) Seismic action in cross aisle direction:

   The analysis for earthquake in the cross aisle direction can be performed considering one isolated upright frame, which is regarded as a subsystem.

   The determinative loading configuration for the design shall be evaluated for each of the structural elements of the upright frame; it is necessary to consider at least the following loading configurations:

1) all levels fully loaded with their specified loads;

2) all levels partially loaded at 2/3 of their specified loads;

3) only top level loaded with its specified loads.

NOTE 1 In general, the first loading configuration is determinative for the design of components (uprights and upright frame bracings), the second and the third may affect the design of the anchoring (baseplates and anchor bolts).

NOTE 2 In principle, all the possible loading configurations that lead to the most severe load case conditions in the frame are considered; the probability that an intermediate condition exists at the same time as the design earthquake occurs is small and therefore it is not necessary to consider further conditions.

The upright frame may always be considered as mass regular in elevation.

b) Earthquake acting in down aisle direction:

The analysis in the down aisle direction is a global analysis and the most relevant action is obtained when the rack is fully loaded. Nevertheless, the mass distributions that maximize the internal forces in each element should also be considered for checking the uplift at the base of the upright in braced racks.

When the vertical bracing subsystem is not located at the end bays of a row, when checking the uplift it is permissible to consider that the uprights involved in the vertical bracing system are subjected to maximum 30 % of the gravity load. However, the seismic action shall be determined with the rack being fully loaded.

7.6.3 Specific modelling requirements for the analysis

a) Rules for the global analysis of the racks are given in EN 15512:2009, Clause 10;

b) where not otherwise specified, the effects of the eccentricities of the connections of the diagonal elements on the uprights or of the vertical bracing columns can be considered according to EN 15512;

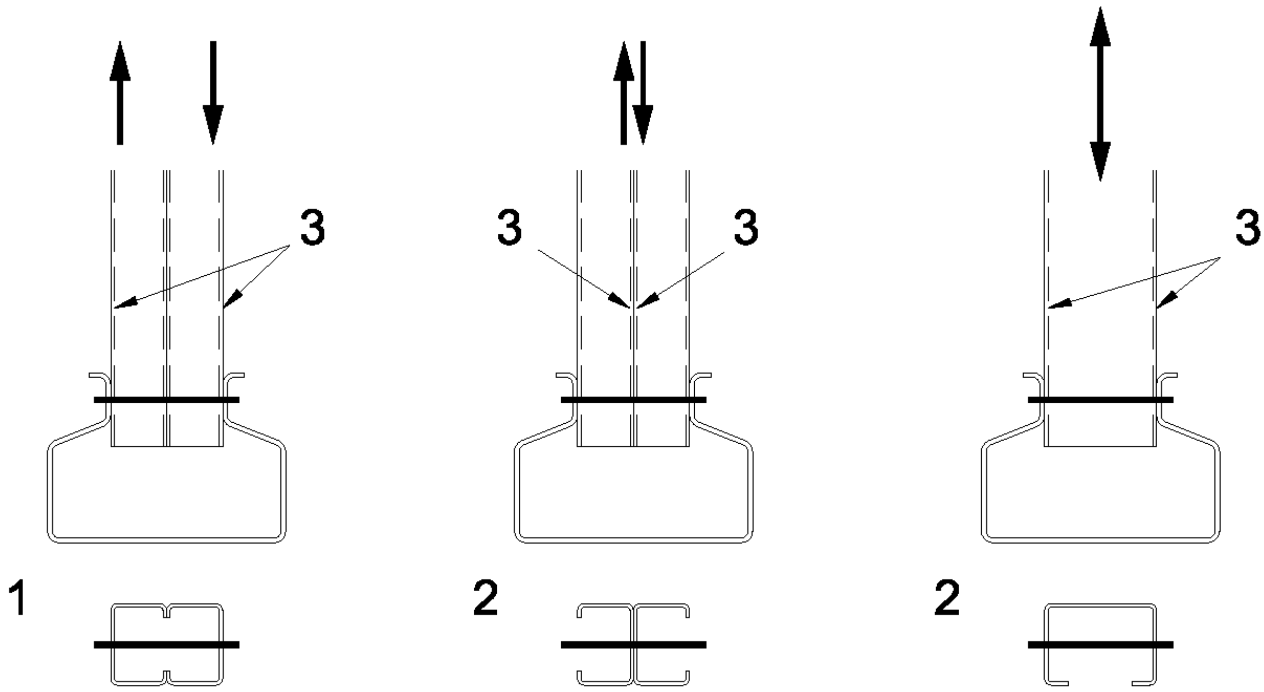
c) in the down aisle direction, the stiffness of the beam-to-upright connection and of the floor connections (baseplates) shall be the value of the stiffness obtained from static load tests according to EN 15512:2009, A.2.4 and A.2.7 respectively;

NOTE 1 In dissipative design concept, the beam-end connector test coefficient is $\eta = 1$ (η defined in EN 15512:2009, A.2.4.5.1).

d) in the cross aisle direction, the shear stiffness of the upright frame shall be equal to the design value obtained according to EN 15512, A.2.8. In case of unsymmetrical connections of the frame bracings diagonals, the effects of the torsion in the upright introduced by the diagonals shall be explicitly taken into account in the design of the upright;

NOTE 2 Ideally the connection between uprights and diagonals transfer the action to the upright symmetrically in the plane of the frame without producing torsion in the upright.

Figure 5 shows typical configurations of the upright frame bracing members and their effects on the action transferred to the upright.



Key

1 upright with unsymmetrical connection of diagonals
(torsion induced in the upright by the diagonals)

2 upright with symmetrical connection of the diagonals
(action transferred symmetrically in the plane of the frame without torsion in the upright)

3 webs

Figure 5 — Action transferred by the upright frame bracing members to the upright

e) when bracings with tension only diagonals are used, only the active elements in tension shall be considered in the model;

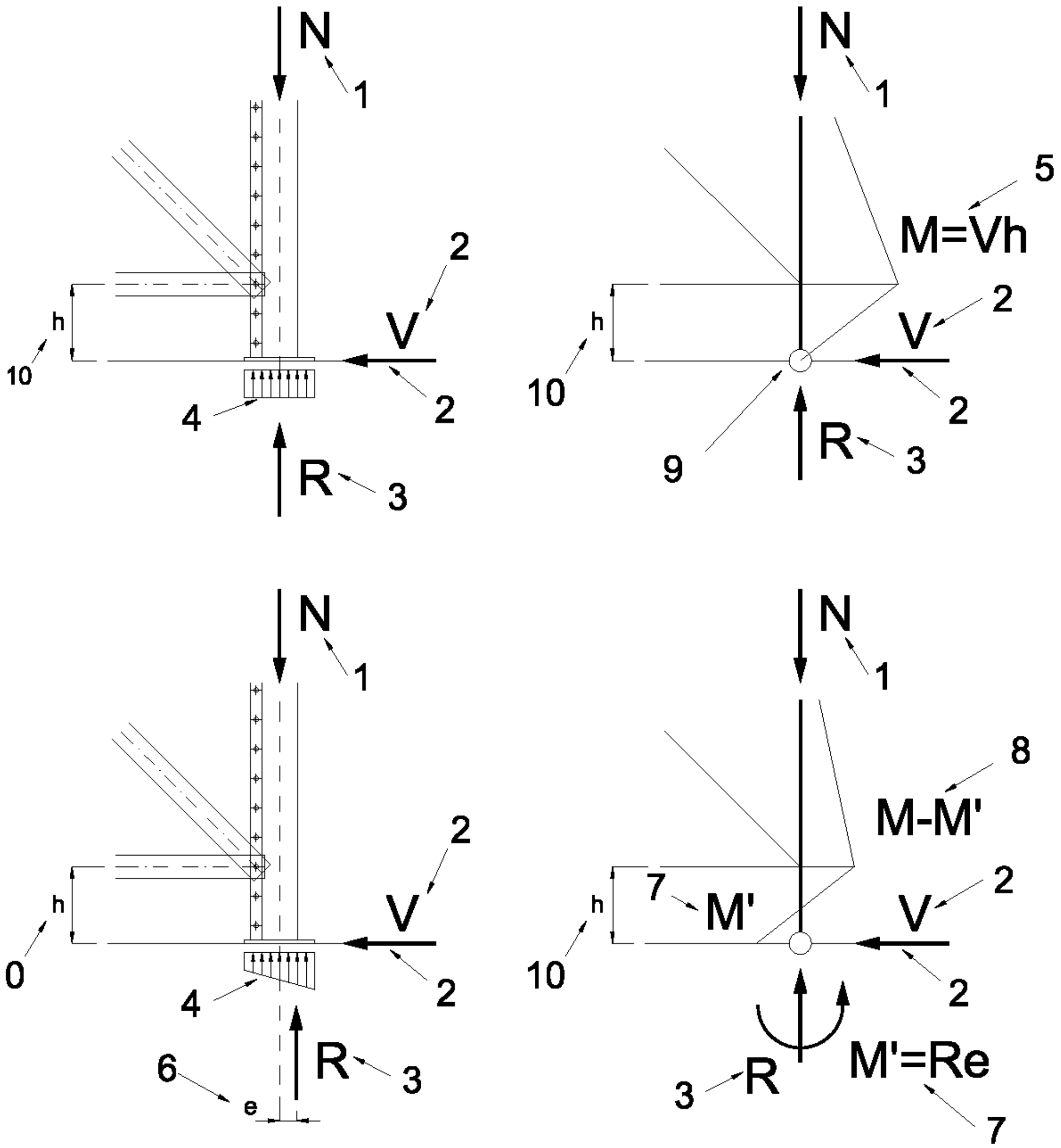
f) the elements in the model shall be consistent with the load path of the horizontal forces;

g) vertical bracing shall be modelled with the appropriate eccentricity to the rack elements and shall include the elements connecting the vertical bracing to the rack;

h) horizontal bracings shall be modelled including the stiffness of their connections.

7.6.4 Moment redistribution near the upright's base due to the floor reaction

The effects of the eccentricity of the bracing's diagonals at the base of the uprights shall be properly modelled and the bending moment considered in the design of the upright; it is permitted to take into account the positive effect of the bending strength generated by the axial force in the upright at the base (see Figure 6), provided that the structural scheme adopted for the design is statically determined.



Key

1 axial load in the upright

2 shear reaction at the upright's base

3 vertical reaction at the upright's base, equal to the axial load N

4 pressure on concrete slab with resultant N

5 peak value of the bending moment generated by V

6 eccentricity of the vertical floor reaction

7 bending moment generated by the eccentricity of the axial reaction

8 reduced peak value of the bending moment

9 hinge

10 vertical eccentricity of the first horizontal

Figure 6 — Effect of the bending moment generated by the axial force in the upright

NOTE For a flat ended upright, the bending moment at the base in cross aisle direction is determined by modelling the stiffness of the upright to floor connection in cross aisle direction equal to EI/H where: EI is the flexural rigidity of the upright in the cross aisle direction and H is the height of the frame bracing pitch or the distance from floor to first horizontal, if any.

8 Specific rules

8.1 Design concepts

8.1.1 General

Earthquake resistant racks shall be designed according to one of the following concepts:

a) low dissipative structural behaviour. The effects of seismic action are calculated by means of elastic global analysis without taking into account significant nonlinear material behaviour;

b) dissipative structural behaviour. Zones of the structure can undergo plastic deformation (dissipative zones).

The value of the behaviour factor q depends on the structural type and on the classification of the member's cross section (refer to EN 1993-1 for classification scheme).

Table 7 shows the range of the reference values of the behaviour factors in relation to the design concept adopted.

Table 7 — Design Concepts and reference values of the behaviour factor

Design Concept	Range of reference values of the behaviour factor q
Concept A Low dissipative structural behaviour	$q \leq 2$
Concept B Dissipative structural behaviour	$q > 2$

8.1.2 Materials

8.1.2.1 General

Structural steel shall comply with EN 15512 and the EN 1993 series.

8.1.2.2 Provisions for low dissipative design concept

In design concept A, material's properties shall comply with EN 15512 without any additional requirement.

8.1.2.3 Provisions for dissipative design concept

In design concept B, the distribution of material properties, such as yield strength and toughness, shall be such that dissipative zones form where they are intended to in the design; yielding is expected to develop in dissipative zones before other zones leave the elastic range during an earthquake.

Such requirements are considered to be fulfilled if the yield strength of the steel in dissipative zones and the design of the structure conform to the conditions given in EN 1998-1.

In the project specification, the designer shall specify the required fracture toughness of steel and welds and the lowest service temperature adopted in combination with the earthquake action. Refer to EN 1998-1 and to National Regulations.

In those areas where the designer requires plasticity, the components shall exhibit the necessary strength and ductility.

In bolted connections of earthquake resisting structure minimum bolt grade 8.8 shall be used.

8.1.3 Structural systems

The reference values of the behaviour factors q for racks are given in 8.3.

For non-regular in elevation assemblies designed in concept B (dissipative) the reference value of q shall be reduced by 20 % (see 8.1.4 for regularity criteria), but the value of q need not be less than 1,5.

NOTE For the purpose of this European Standard, the following structural systems are considered, according to their seismic behaviour:

a) moment resisting frames: horizontal seismic forces are resisted by the flexural behaviour of members and connections. In design concept B, dissipative zones are mainly located in plastic hinges near, or in, the beam-upright joints, and energy is dissipated by means of cyclic bending;

b) frames with concentric bracings, in which members subject to axial forces withstand the horizontal seismic action. In design concept B, dissipative zones are mainly located in the tension diagonals.

Other mechanisms for energy dissipation are allowed to be considered as described in EN 1998-1.

The behaviour factor q accounts for the energy dissipation capacity of the structure.

Seismic resisting structures connected to the rack (such as independent bracings, frames or shear walls) shall be designed according to EN 1998-1 or National Regulations.

8.1.4 Regularity criteria

8.1.4.1 General

Regularity criteria for racks relate to both stiffness and mass distribution, in plan and in elevation. For a regular configuration, all the following criteria shall be met.

When the beams are in a regular pattern it is permissible to assume the mass regularity condition in plan and in elevation for the relevant conditions for seismic design.

NOTE In storage racks, the seismic weight is mostly due to the stored unit loads and this means that it is impossible to control the mass regularity for all the possible pallet configurations.

In order to consider the structure regular, both the structural configuration and layout shall be regular (see 8.1.4.4).

8.1.4.2 Cross aisle direction

a) Regularity in plan:

   Racks in cross aisle direction can be considered as stiffness and mass regular in plan when upright frames and bay loads are the same for the whole run.

   The planar analysis of the isolated upright frame considering its associated mass is in any case allowed.

   When upright frames are structurally connected in cross aisle direction, they shall be analysed together with their connections;

b) Regularity in elevation:

The upright frame can be regarded as regular if:

1) diagonal bracings have the same type of pattern. The variation of the scheme is limited to a ratio of 2 between the maximum and the minimum pitches (excluding the lower one), and they are without interruptions from the floor to the top load level (stiffness regularity), and

2) the ratio of the maximum and the minimum distance in elevation between the pallet beams, and between floor and 1st level pallet beam, is less than 2. When the first beam level is less than 1,2 m from the floor it may be excluded from this criterion.

8.1.4.3 Down aisle direction

a) Regularity in plan:

1) racks not braced in the down aisle direction, or with symmetric bracings in the front and rear lines of uprights, are stiffness regular in plan;

2) single run racks braced with spine bracing are not stiffness regular in plan; the design rules are specified in 8.3.3;

3) double run racks with spine bracing only shall be regarded as non-regular in plan.

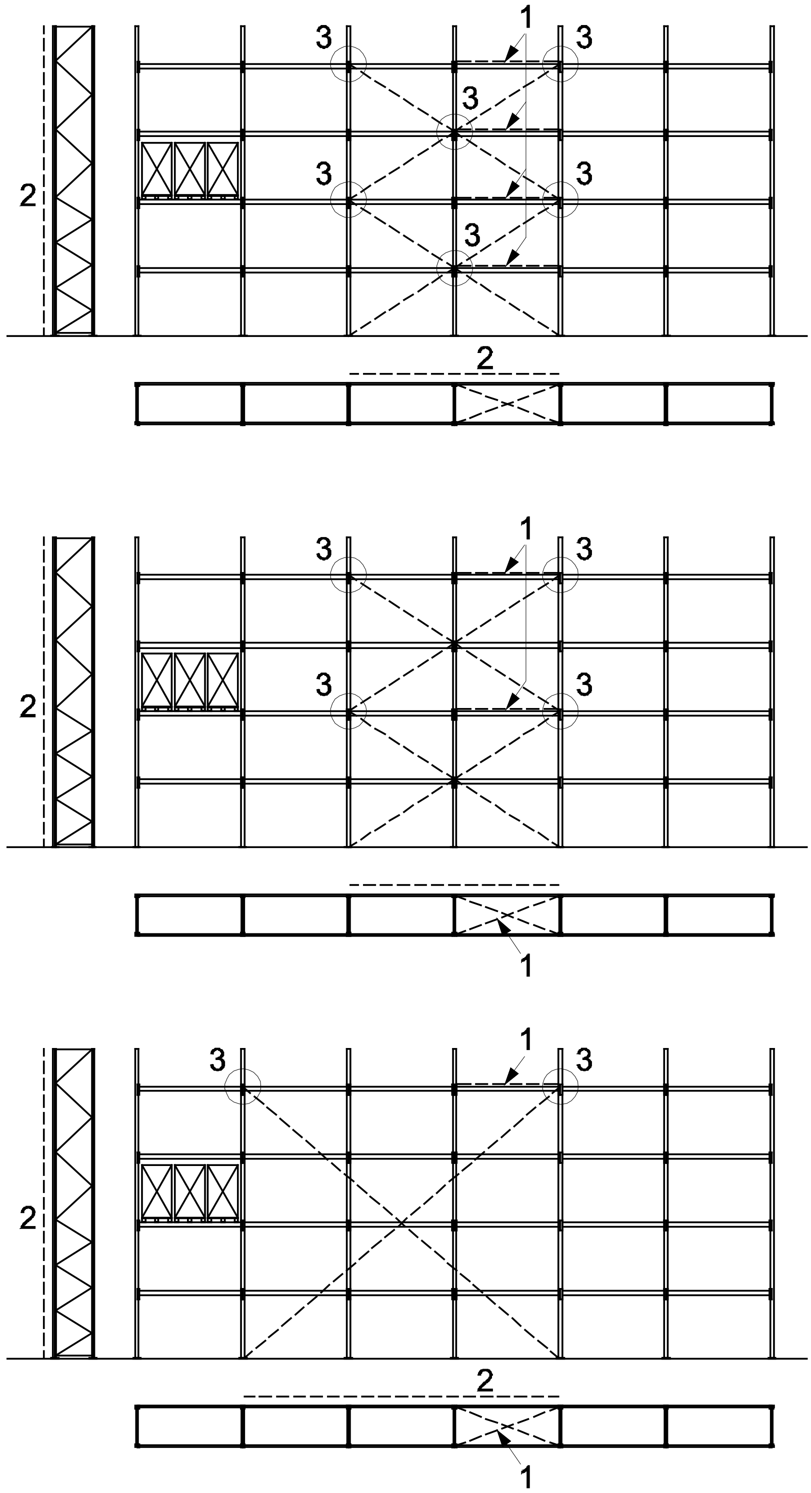
NOTE This is because of the possible eccentricity of the mass in plan.

b) Regularity in elevation:

1) position of pallet beams. The rack can be regarded as regular if the pallet beams are at the same level for the whole length of the run, and the ratio between the maximum and the minimum height between the pallet beams, and between the floor and 1st beam level, is less than 2. When the first beam level is less than 1,2 m from the floor it may be excluded from this criterion;

2) type of vertical bracing. Racks with continuous vertical bracing connected at regular spacing from the floor to the top load level are stiffness regular in elevation. Partially braced racks are not stiffness regular in elevation.

Figures 7 to 11 show examples of regular and non-regular racks in plan and elevation.



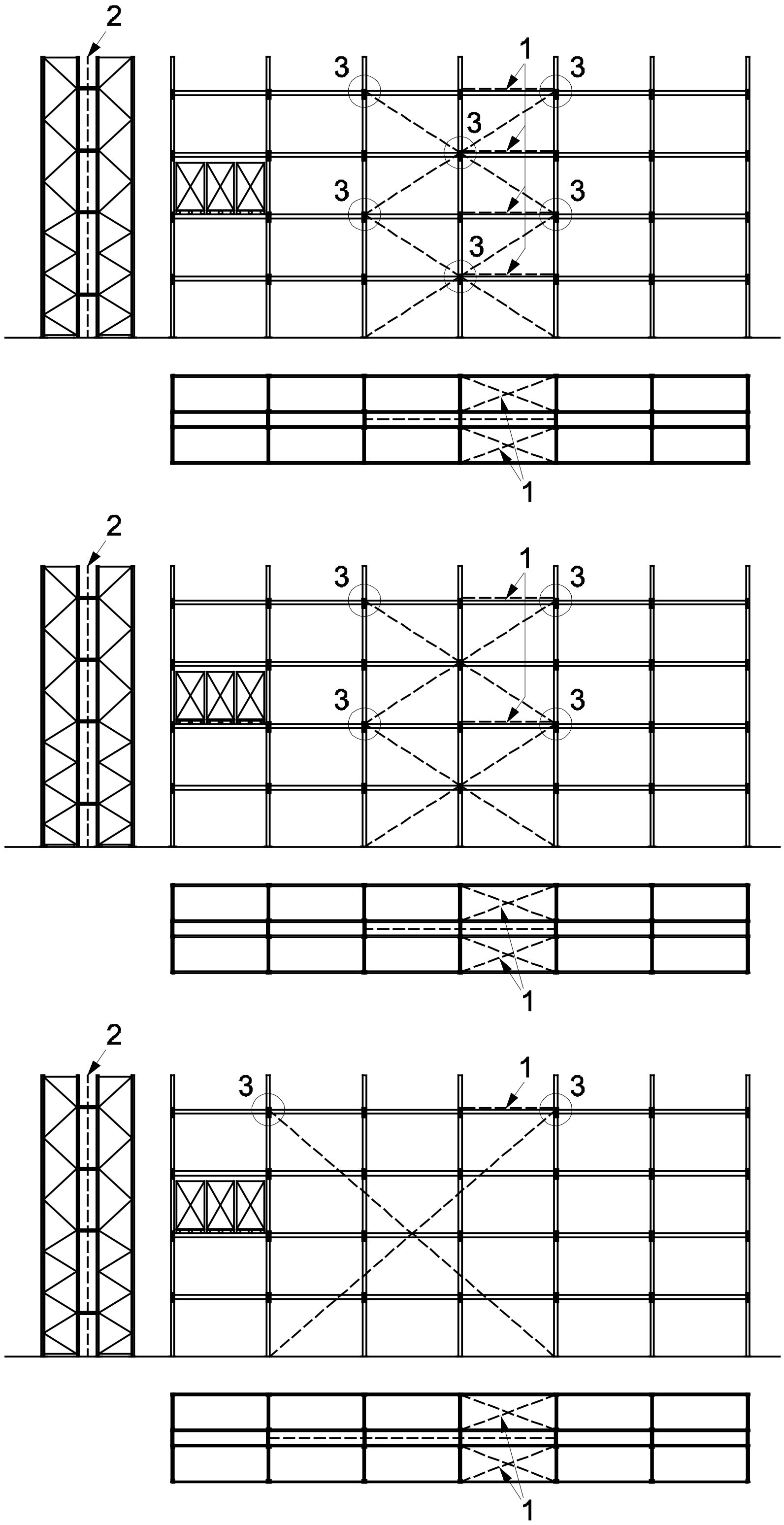
Key

1 horizontal bracings

2 vertical bracing

3 levels connected to the vertical bracing

Figure 7 — Rack not stiffness regular in plan and regular in elevation in down aisle direction



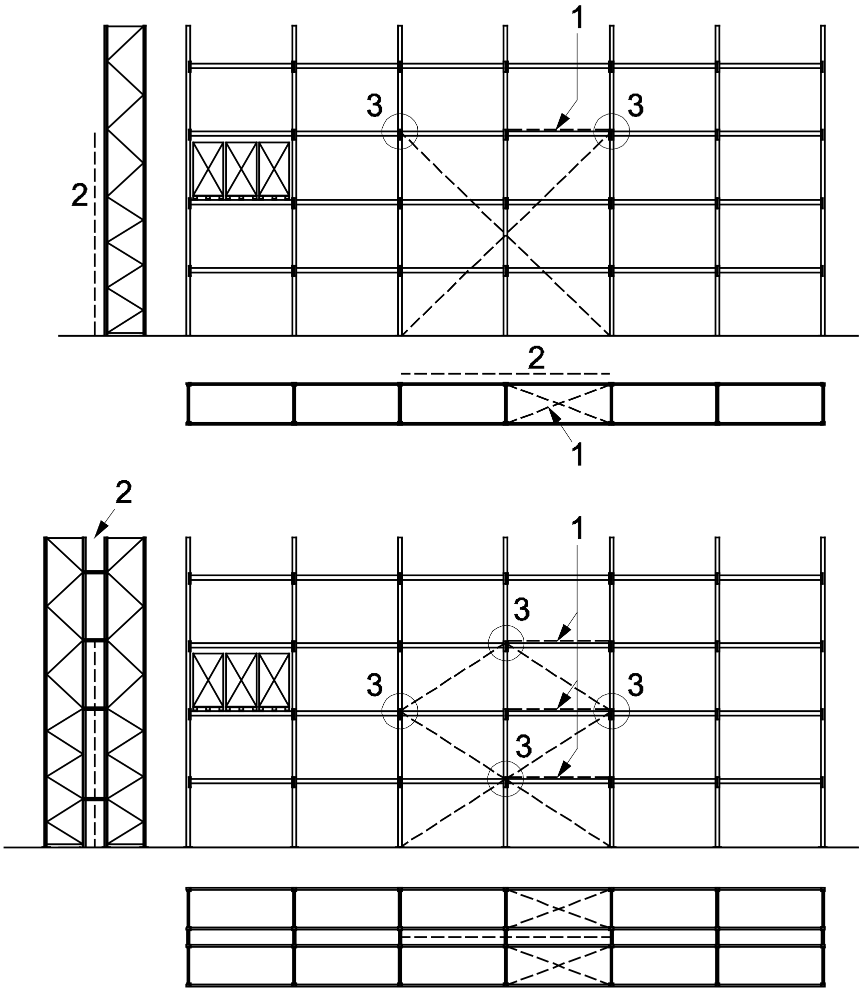
Key

1 horizontal bracings

2 vertical bracing

3 levels connected to the vertical bracing

Figure 8 — Racks not mass regular in plan and regular in elevation in the down aisle direction



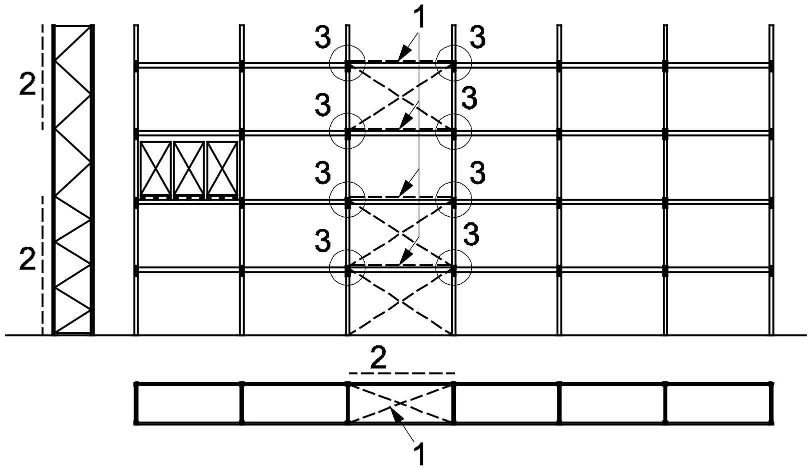
Key

1 horizontal bracings

2 vertical bracing

3 levels connected to the vertical bracing

Figure 9 — Partially braced rack not regular in elevation in down aisle direction



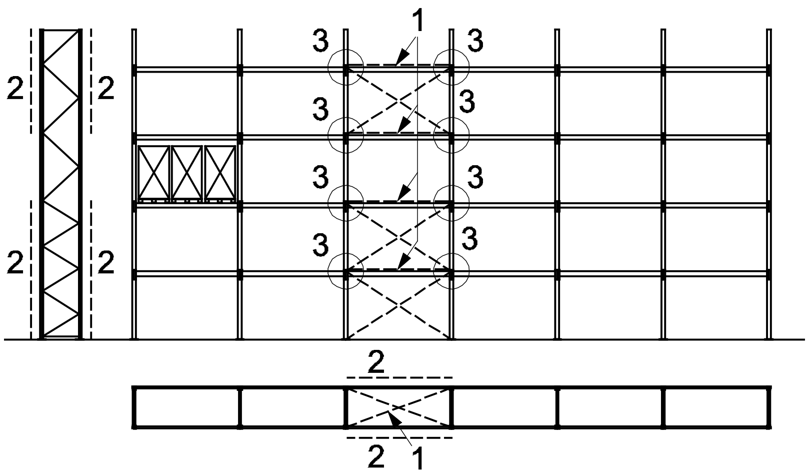
Key

1 horizontal bracings

2 vertical bracing

3 levels connected to the vertical bracing

Figure 10 — Braced rack not stiffness regular in plan and not regular in elevation in down aisle direction (vertical bracing interrupted)



Key

1 horizontal bracings

2 vertical bracing

3 levels connected to the vertical bracing

Figure 11 — Braced rack regular in plan and not regular in elevation in down aisle direction (vertical bracing interrupted)

8.1.4.4 Layout regularity

Racks are normally provided in long runs. Parallel runs that are structurally independent or that are continuously connected to each other along the length and at the top of the uprights can be considered as regular. All other cases are not regular.

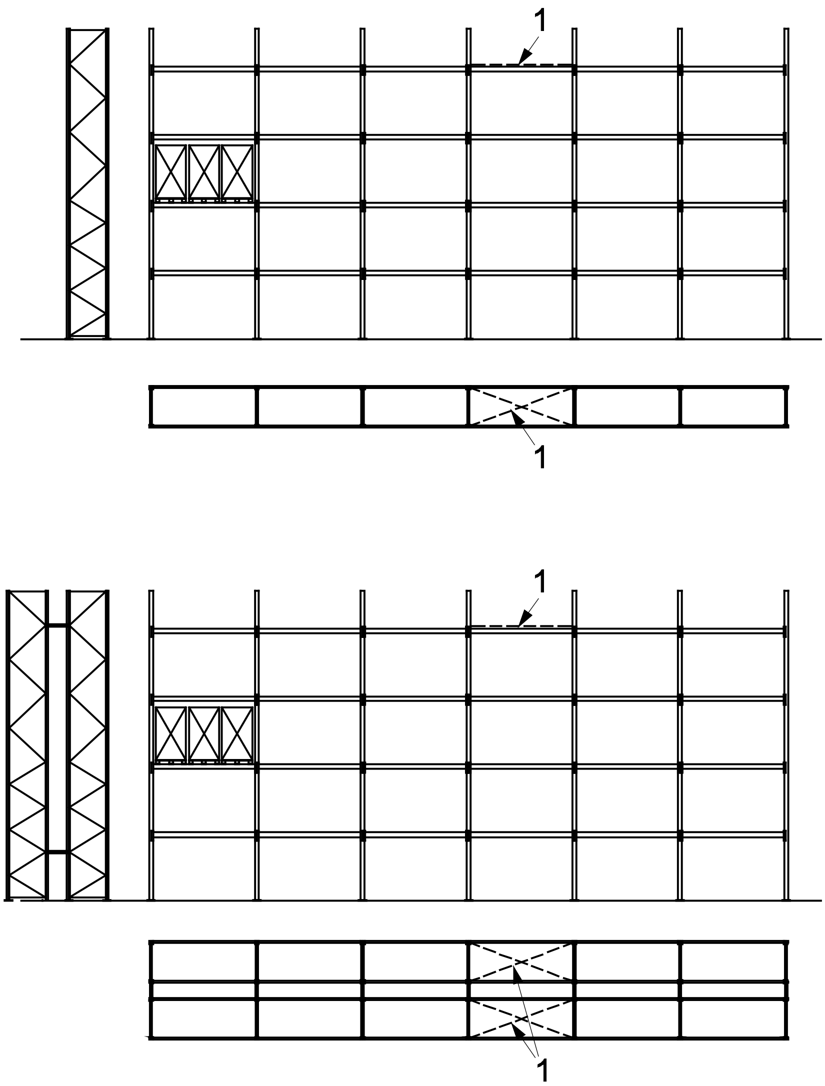
8.1.5 Unbraced racks

When a unit load-beam friction coefficient μ_S is less than 0,1, unbraced racks shall have plan bracing in seismic design (see also Figure 12).

For racks with five beam levels or less it is sufficient to provide one set of horizontal bracing at the top-most beam level at least every 10 bays.

For racks with more than five beam levels, additional horizontal bracing is required at a frequency of one set per 5 levels at least every 10 bays. The plan bracing shall be evenly distributed in the height of the rack with one set at the top-most level.

The horizontal bracings shall be designed as specified in 8.3.3.3.



Key

1 plan bracing

Figure 12 — Unbraced rack showing plan bracing

8.1.6 Rules for the design of low dissipative structures

a) For members that are part of the earthquake resisting structure, the rules on materials given in 8.1.2.2 apply;

b) the strength of members and connections shall be evaluated according to the rules for elastic or plastic resistance specified in EN 15512;

c) bolts shall be at least snug tight and nuts shall incorporate some form of locking device;

NOTE 1 A tooth flanged nut is an example of a suitable device.

d) unless otherwise specified, the behaviour factor $q > 1,5$ shall only be used if members that contribute to the seismic resistance of the structure in compression or bending have a section classification 1, 2 or 3;

NOTE 2 q > 1,5 is allowed to be used when the gross section of perforated uprights fulfil the requirements of EN 1993–1-1 (maximum width-to-thickness ratio) for class 3 in compression.

e) Bracing schemes shall be according to EN 1998-1. For racking, it is also permissible to use K, D, Z bracings, and X bracings without horizontal members, in which the resistance to the horizontal actions is provided by diagonals in compression. The behaviour factor q = 1,5 may be assumed, provided that the design effect due to seismic action in all bracing members and their connections is increased of a factor 1,5. Higher values of q may be used if demonstrated by test;

f) for bolted shear connections the shear strength of bolts $F_{v,Rd}$ shall be 1,20 times higher the bearing resistance $F_{b,Rd}$ of the connected profiles:

$$F_{v,Rd} / F_{b,Rd} > 1,20 \tag{12}$$

This requirement needs not to be applied when the strength of the connection is q times greater than the calculated bolt shear due to seismic action.

8.1.7 Rules for the design of dissipative structures

Dissipative structures shall comply with design rules for steel structures stated in EN 1998-1.

Specific detailing rules are given in Annex F (normative); testing procedures for beam-upright connections and floor connections are given in Annex G (normative).

8.1.8 Anchoring conditions

Post installed anchor bolts shall be designed according to ETAG 001, Annex C.

For low dissipative design concept, post installed anchor bolts shall be certified by ETAG for application as specified in Table 8.

In very low seismic zone, the designer of the slab shall specify the cracked or uncracked conditions for post installed anchor bolts in the concrete.

Performance category of post-installed anchor bolts for racks designed according to dissipative concept shall be as defined in Annex E of ETAG 001.

Table 8 — Minimum performance category for post installed anchor bolts for racks designed in Low dissipative Concept

Seismicity		Importance Class			
	$a_g \cdot S$	I	II	III	IV
Very Low[a]	$a_g \cdot S \leq 0{,}05g$	ETAG 001 Part 1 to Part 5			
Low[a]	$0{,}05\ g < a_g \cdot S \leq 0{,}1g$	ETAG 001 Part 1 to Part 5 for cracked concrete		C2[b]	C2[b]
	$a_g \cdot S > 0{,}1g$	C1[b]	C1[b]	C2[b]	C2[b]

$a_g = \gamma_I\, a_{gR}$ design ground acceleration on type A ground.

[a] Seismicity according to EN 1998–1:2004, 3.2.1.

[b] Performance Category according to ETAG 001, Annex E.

Performance category of post-installed anchor bolts for racks designed in dissipative design concept shall be as defined in Annex E of ETAG 001.

8.2 Structural systems withstanding the seismic action

In the case of a rack that is regular in plan it may be assumed that the lateral force resisting systems can also be considered in separate cross-aisle and down-aisle analyses.

If a structural system other than the rack is provided to withstand the seismic action, it should be designed with the criteria of EN 1998-1 or National Regulations.

NOTE Typical rack's structural systems withstanding seismic actions are:

a) upright frames, in the cross aisle direction;

b) one of the following systems, in the down aisle direction;

1) unbraced frames: the stability is provided by beam – to – upright joints, without vertical bracing;

2) a single line of vertical bracings: the bracing system consists of the following elements:

i) a spine bracing placed behind the rear frame, which can be an independent structure connected to the rack, or bracing elements connected directly to the rack,

ii) horizontal bracings connecting the front unbraced line of uprights to the rear braced uprights.

The vertical bracing withstands the horizontal seismic action.

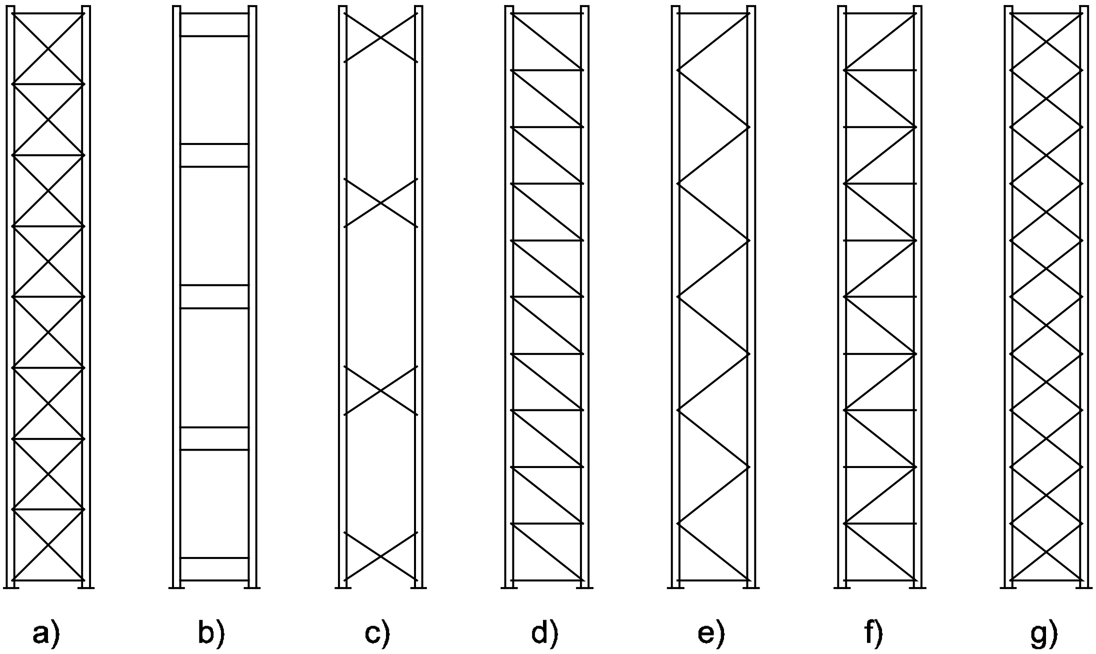
The horizontal bracings and the upright frames connected to the horizontal bracings withstand the induced torsional effects due to the eccentricity of the seismic action with respect to the vertical bracing;

3) symmetrical vertical bracing (each alignment of uprights is braced): vertical bracings withstanding the seismic action are present in a limited number of bays, in each alignment of uprights. Horizontal bracings connecting the front and the rear uprights are also provided according to the design rule for unbraced racks (see 8.1.5).

8.3 Structural types and behaviour factor

8.3.1 Upright frames

Typical frame bracing configurations are shown in Figure 13; in Table 9 , the seismic design procedures are described.



Key

(a) X-braced frame with horizontal elements
(b) battened frame
(c) partially braced frame
(d) Z-braced frame
(e) D-braced frame
(f) K-braced frame
(g) X-braced frame

Figure 13 — Upright frame bracing arrangements

Frames designed in low dissipative concept with pattern type a) and diagonals working in tension and compression (Table 9, (a.3)), shall have the horizontal member designed for 50 % of the horizontal shear force in the frame scheme.

Table 9 — Seismic design procedure for upright frames

Frame type	Structural type	Design concept	Reference behaviour factor	Increase factor for the seismic action in the bracing's elements and connections
a	(a.1) Diagonal bracing with only active tension diagonals	Dissipative	According to EN 1998-1	According to EN 1998-1
	(a.2) Diagonal bracing with only active tension diagonals	Low dissipative	2,0	1,0
	(a.3) Diagonal bracing with tension and compression diagonals	Low dissipative	1,5	1,0 or 1,5 [a]
b	Dissipative battened frame can be used, provided that the requirements of moment resisting frames of EN 1998–1 are met; otherwise q = 1,5 can be used with increase factor for the seismic action 1,5 in battens-to-upright connection			
c			1,0	1,0
d-e-f-g	Low dissipative		1,5	1,5 [b]

[a] The diagonals of scheme a.3 is designed with increase factor of the seismic action 1,5 when their failure is caused by local instability mechanism in the member in compression

[b] An increase factor of the seismic action 1,5 is applied to all the bracing members and their connections (see 8.1.6 e)) in order to reduce the risk of global collapse mechanisms.

8.3.2 Moment resisting frames

8.3.2.1 Low dissipative concept

A minimum value of q = 1,5 can be assumed for the global analysis, provided that the elements of the floor connection, including its connection to the upright, are designed with an increase factor for the seismic action 1,5 applied to the bending moment calculated from the analysis. This increase factor is not required to be applied to the design of the anchor bolts.

The strength of the baseplate and its connection to the upright shall be calculated either theoretically or by tests.

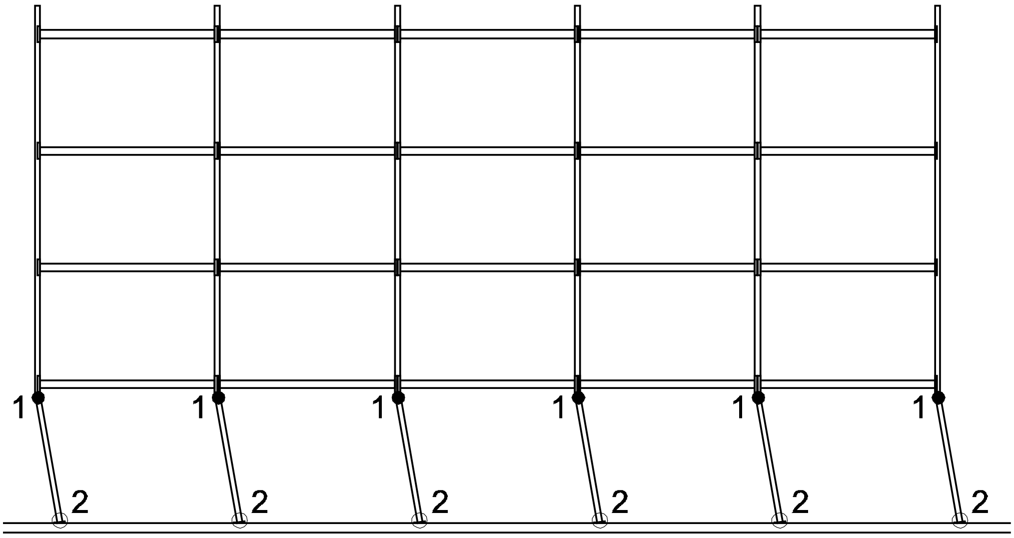
Tests shall be performed according to EN 15512:2009, A.2.7 on floor connections installed with the floor fixings, including compression, tensile and null axial force cases, to evaluate the bending strength and the stiffness.

Tests on floor connection in compression can also be performed without floor fixings installed; in this case, in order to consider the effect of the floor fixings, one additional test with floor fixings installed shall be executed applying pure bending moment (with null axial force), and the bending strength obtained from this test can be added to the bending strength derived from tests executed without floor fixings.

It is allowed to consider the floor connection hinged, provided that the collapse mechanism of the frame is not soft storey type.

NOTE Soft storey mechanism is a failure mechanism at one storey level, typically the ground level, further the development of plastic hinges at column ends accompanied by excessive storey drift located at that storey.

In Figure 14, a soft storey collapse mechanism typical for unbraced racks is represented.



Key

1 plastic hinge in the upright below the first level

2 plastic hinge in the floor connections

Figure 14 — Soft storey collapse mechanism of unbraced frame

Bolted baseplates to upright connections using bolts in slot holes in the direction of the force or oversized holes shall be slip resistant according to the requirements of EN 1993-1-8 and EN 1090-2, or they shall be properly designed to avoid rotations without rely on friction; otherwise the floor connection shall be considered as hinged.

The behaviour factor $q = 2$ can be used when the above requirements are fulfilled and the following conditions a) and either b) or c) or d) are met:

a) the condition of 8.1.6 d) is fulfilled;

b) at least one bolt is provided to secure the beam end connector to the upright. Washers shall be fitted under both the nut and bolt head and the nut shall be at least snug tight. The properties of the beam-end connectors determined without bolt can be considered for the design of the frame and of the beam-end connector; or

c) it can be demonstrated that, removing the upper and lower hook/stud of the beam end-connector, the remaining resisting hocks/studs are able to support the vertical unit loads applied. The locking pin should fulfil the requirement of EN 15512:2009, 6.4.2.a); or

d) the beam-end connector is demonstrated to have a rotation capacity in both positive and reverse direction, equal or greater to 3 times the rotation measured at the value of bending moment corresponding to characteristic bending strength.

Moment redistributions in beams according to EN 15512:2009 is not permitted in seismic design of moment resisting frames designed according to low dissipative concept.

8.3.2.2 Dissipative concept

Dissipative behaviour $q > 2$ may be assumed when the requirements of EN 1998-1 for moment resisting frames are fulfilled.

8.3.3 Racks with vertical bracings in down aisle direction

The vertical bracing considered in this European Standard is the “concentric diagonal bracing” type; Figure 15 shows the typical elements of the bracing system.

Other types of vertical bracing can be used and they shall be analysed with the methods of EN 1998-1.

a) The design of the vertical bracings and of the seismic resistant elements in the down aisle direction shall be carried out using the design rules and the related behaviour factor defined hereunder for vertical bracings, considering the mass distribution on the rack as specified in 7.6.2 b).

b) The upright frames connected by horizontal bracings shall be designed for the torsion, using the design rules for the upright frames defined in cross aisle direction. At least, the following distribution of the loads maximizing the eccentricity in plan shall be considered:

   1) single entry block fully loaded;

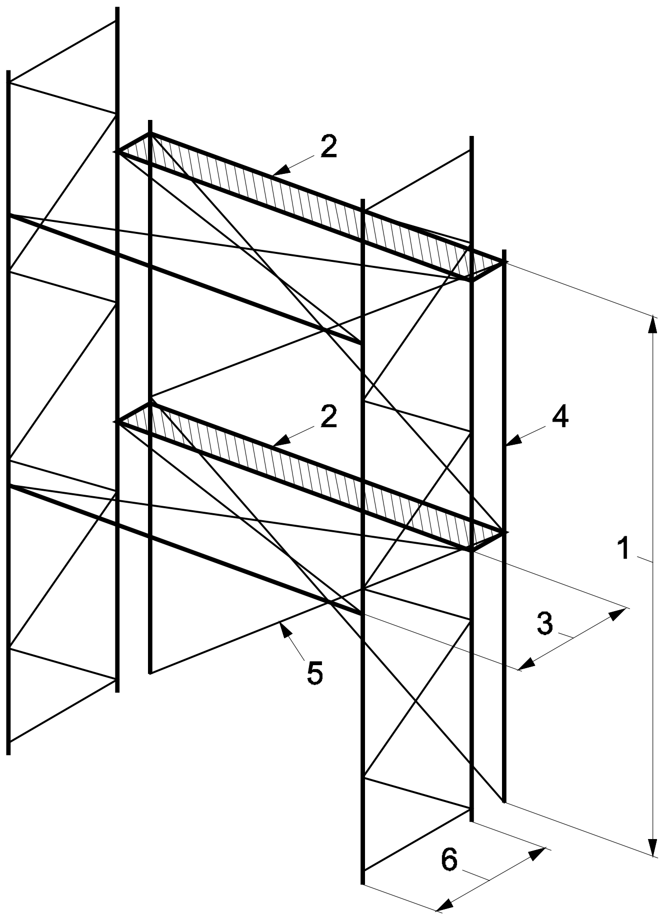
   2) double entry block fully loaded on one side only.

NOTE 1 When an automated loading management system can ensure the respect of the design loading configuration in double entry racks, where one run side is fully loaded and the other side partially loaded, this configuration can be considered as the most severe unbalanced case for the design of the rack.

NOTE 2 The behaviour factor of the structure is defined by the considered mode of vibration and the structural types.

c) The connection eccentricities of the bracings shall be taken into account according to EN 15512:2009, 8.6. The connection elements shall be capable of transferring the forces obtained from the analysis. If this is not achieved then the spine bracing may not be fully effective and additional stresses may be induced in the uprights.

d) When dissipative design concept is applied, the elements connecting the rack to the bracings shall be designed with the requirements specified for the connections of dissipative members according to EN 1998-1.



Key

1 vertical bracing
2 rack to spine bracing connection
3 plan bracing
4 spine bracing upright
5 spine bracing
6 frame bracing

Figure 15 — Elements of the bracing system

8.3.3.1 Low dissipative design concept

In Table 10 are reported the behaviour factors and the design rules to be applied for racks with X bracings.

Table 10 — Seismic design procedure for spine bracing

Typology	Reference behaviour factor q	Detailing rules for bracing elements and connections
Tension only diagonals with horizontal compression element	2	Diagonal adimensional slenderness $\bar{\lambda} > 1{,}3$ [a]
Tension and compression diagonals with horizontal compression element	1,5	[a]
Tension and compression diagonals without horizontal compression element	1,5	Clause 8.1.6 e)
[a] The distance of the bolt to the edge is to be at least 2,5 times its diameter of the bolt when the failure mode is the bolt's bearing.		

If flats, round bars or cables, or other components with $\bar{\lambda} > 2$ are used as diagonal bracing, the construction shall guarantee that those elements shall always be in tension under the gravity loads; in order to fulfil this condition spine bracings with diagonals made of flats, round bars or cables shall have posts not supporting vertical loads forming so an independent bracing tower. The system shall be periodically checked.

NOTE This requirement will reduce the possibility of "shock" loading due to looseness in the spine bracing system.

Pallet beams can be considered as compression elements provided that all the eccentricities are defined in the analysis, with all the components connecting the rack to the vertical bracing.

Unless otherwise demonstrated, the behaviour factor to be used for earthquake action in down aisle direction in case of torsion shall be the smallest one of all resisting subsystems involved in the resistance, including the torsion effects.

8.3.3.2 Dissipative design concept

Vertical bracing designed under design concept B shall be according to EN 1998-1.

8.3.3.3 Horizontal bracings

Horizontal bracings in low dissipative concept shall be designed for the actions obtained from the analysis.

Horizontal bracings in dissipative concept shall be designed according to EN 1998-1.

Horizontal bracings in unbraced racks shall be designed each for an axial strength N_{Rd} greater or equal to 5 kN.

9 Seismic analysis and design

9.1 Actions

9.1.1 Actions to be considered simultaneously with earthquake

The following actions, as defined in EN 15512, shall be considered concurrently with the seismic action:

a) dead loads (permanent action):

   1) weights of materials and constructions;

   2) weight of fixed equipment;

b) unit loads (variable action);

c) snow loads (variable action);

d) floor and walkway loads (variable action). The characteristic vertical static load shall be considered. No dynamic amplification factor should be taken into account;

e) when mechanical handling equipment is supported by the racking it shall be taken into account in the design. The mechanical handling equipment supplier shall provide necessary data to the rack supplier.

   NOTE When stacker cranes are parked at the P&D station the load carriage will be at its lowest position and will be unloaded. This is unlikely to be a critical design case as the centre of gravity is close to the floor.

9.1.2 Actions not to be considered simultaneously with earthquake

The following loads as defined in EN 15512 need not be considered concurrently with seismic actions:

a) wind;

b) vertical placement loads;

c) horizontal placement loads;

d) operational loads caused by rack-guided equipment;

e) thrusts on handrails;

f) thermal loads;

g) actions from global imperfections;

h) impact and other accidental loads;

i) maintenance loads.

9.2 Safety Verifications

9.2.1 Ultimate limit states

9.2.1.1 Combination rules

The design actions shall be combined using the following formula:

$$\sum G_{k,i} \text{ “+” } \sum \psi_{2,i} Q_{k,i} \text{ “+” } A_{E,d} \tag{13}$$

where

“+” implies “to be combined with”;

Σ implies “the combined effects of”;

$G_{k,i}$ is characteristic value of permanent action i;

$Q_{k,i}$ is characteristic value of variable action i;

$A_{E,d}$ is design value of seismic action for the reference return period;

$\psi_{2,i}$ is combination factor for variable actions.

The values of the combination factors shall be assumed according to national regulations, but not less than:

— $\psi_{2,1} = 1,0$ for unit loads;

— $\psi_{2,2} = 0,8$ for floor loads in storage areas;

— $\psi_{2,3} = 0,3$ for floors not for storage or for walkway loads in operating areas;

— $\psi_{2,4} = 0$ for walkway loads for exclusive access for maintenance.

Combination factor for snow load shall be according to EN 1990.

9.2.1.2 Resistance condition

The following condition shall be met for all structural elements, including connections:

$E_d \leq R_d \{f_k/\gamma_M\}$

where

E_d is design effect, due to the design seismic situation, obtained by combining the actions according to 9.2.1.1;

$R_d \{f_k/\gamma_M\}$ is corresponding design resistance of the element, function of the characteristic strength of the material f_k and the material's factor γ_M.

9.2.1.3 Material's factor γ_M

Material's factor for ultimate limit state are according to EN 15512.

9.2.1.4 Ductility condition

In dissipative design concept the structural elements and the structure as a whole shall comply with the EN 1998 series requirements on ductility.

For low dissipative design concept, no specific requirements on ductility are necessary.

9.2.1.5 Equilibrium condition

The rack shall remain stable under the actions derived by the combination rule of 9.2.1.1, including overturning and sliding.

9.2.1.6 Seismic clearances

The horizontal displacement shall be calculated according to 7.4.7.

The layout of the racks shall include clearances sufficient to prevent collisions:

— between unconnected racks;

— between racks and adjacent building structures. This shall be evaluated taking into account the displacement of the building. The owner of the building shall provide the displacements of the building for the analysis.

The distance between structures shall be calculated according to EN 1998-1:2004, 4.4.2.7 (2)(b).

9.2.2 Movements of the unit loads

9.2.2.1 Unit load sliding

Unit load sliding on a rack is not considered as damage.

Consequences of sliding shall be assessed when the inertial force due to the seismic acceleration on the unit load at each loading position exceeds $C_{\mu L} \times \mu_S \times Q_{P,rated}$. These inertial forces shall be calculated using the elastic spectrum with viscous damping specified in 6.2.

9.2.2.2 Unit load falling

9.2.2.2.1 Unit loads not fixed on the rack

If unit loads are free to slide on the pallet beams, the rack shall be designed according to the rules given in this European Standard.

The risk of falling of the unit loads shall be assessed (see also Annex J).

NOTE 1 The combination of the seismicity of the site, configuration of the rack and friction coefficient between the unit load and beam is allowed to lead to falling of the unit loads, either inside or outside the rack. There is a possibility of causing local or global collapse of the structure, injury to persons and damage to the stored goods, especially in case of high racks and narrow aisles.

In seismic zones, when displacements of the unit loads are likely to occur, additional components shall be installed or appropriate countermeasures shall be taken in order to prevent falling of unit loads inside the rack.

NOTE 2 The real displacement due to sliding under a severe earthquake is quite unpredictable because of the random nature of earthquakes and also for the number of parameters which can affect the behaviour of pallets (friction coefficients, etc.).

9.2.2.2.2 Unit loads fixed on the pallet beams

When the movement of the unit loads on the beams is prevented using any special system (for example materials increasing the friction between pallet and beam) the design spectrum modification coefficient E_{D1}, defined in 7.5.2, shall be assumed equal to 1,0.

9.2.2.3 Unit load rocking and overturning

The designer shall assess the risk related to the stability of unit load rocking.

NOTE See also Annex H for the assessment of the stability of the goods on the load makeup accessory.

9.3 Pallet beam design

9.3.1 Actions on pallet beams

9.3.1.1 Axial force

The axial force in the beams is derived from the global analysis.

9.3.1.2 Horizontal action

The total horizontal action on the pallet beams is equal to the sum of the horizontal reactions at the beam ends obtained from the global analysis, multiplied by:

$$1/E_{D1} \times Q_{P,\max}/Q_{P,rated} \tag{14}$$

where

$Q_{P,max}$ is specified weight of the unit load (see EN 15629, 6.7.1).

It can be considered uniformly distributed over the beam's length.

The pallet beam friction is limiting the horizontal action per unit load to the value $Q_{P,max} \times C_{\mu H} \times \mu_s$ (see also 7.5.3).

The horizontal seismic action acting on the unit load may be obtained by dividing the horizontal seismic force of a compartment by the total number of unit loads in that compartment.

9.3.1.3 Bending moment in the horizontal plane

The horizontal action calculated as specified in 9.3.1.2 shall be applied as uniform distributed load on the pallet beams in the horizontal plane, when the compartment is fully loaded, or on the length of the unit loads when the compartment is analysed as partially loaded.

The horizontal force shall be equally divided between the beams in the compartment.

9.3.1.4 Bending moment in the vertical plane

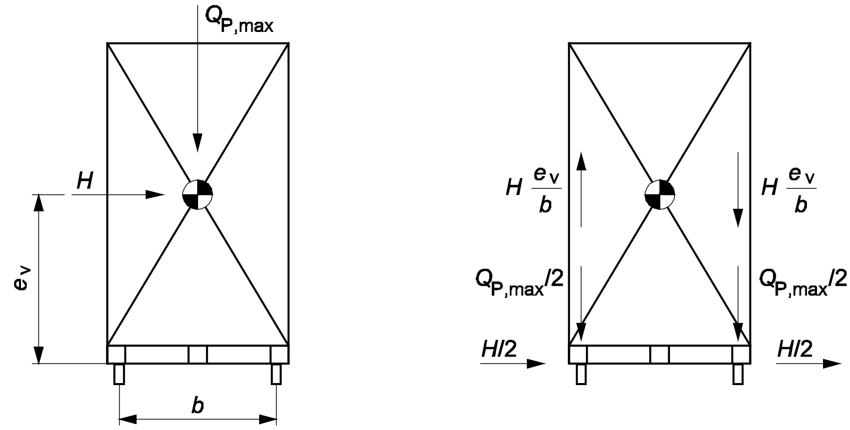
The bending moment in the vertical plane of the pallet beam is due to three different effects:

a) effect of the unit load weight and of the earthquake in the down-aisle direction. The effect of the unit load weight and of the earthquake in the down-aisle direction is derived directly from the global analysis;

b) effect of the vertical eccentricity of the centre of gravity of the unit load. The overturning moment due to the vertical eccentricity of the centre of gravity of the pallet equilibrated by the variation of the vertical load on the support beams shall be taken into account. The limiting effect of sliding on the horizontal action can be taken into account (see 9.3.1.2).

If the vertical eccentricity of the unit load is modelled, then the bending moment in the pallet beams is derived directly from the global analysis. If the vertical eccentricity is not considered in the global analysis, then the effect of rocking is derived from a sub-model.

Figure 16 shows the design actions on pallet beams for earthquake in cross aisle direction.



Key

$Q_{P,max}$	specified weight of the unit load
H	horizontal seismic action applied at the centre of gravity of the pallet
e_V	vertical distance between the centre of gravity of the pallet to the top beam level
b	horizontal distance between the beam axes

Figure 16 — Design actions on pallet beams for earthquake in cross aisle direction

c) eccentricity in cross aisle direction of the position of the unit load. The effect of this eccentricity is the increase of the load to one beam if sliding occurs (see 9.2.2.1); the pallet shall be considered in its extreme position. Depending upon the model adopted, the effect of this eccentricity might be included directly in the global analysis.

The effect of the seismic action in the cross-aisle and down-aisle direction shall be combined according to 7.4.6.

9.3.2 Buckling length in the horizontal plane

The buckling length of the pallet beam in compression shall be obtained by multiplying the system length of the beam L by the buckling length factor K.

The buckling length factor K shall be not less than specified in Table 11.

Table 11 — Recommended buckling length factors of the pallet beams in the horizontal plane

Number of unit loads per compartment	K for single span beams	K for two or more span beams
n out of n	0	0
1 out of 2	0,6	0,5
1 at mid span out of 3	1,0	0,9
2 out of 3	0,6	0,5
2 at mid span out of 4	0,7	0,6
3 out of 4	0,5	0,45

A conservative assumption is to consider the beam’s buckling length equal to the distance between upright system lines (pinned beam ends).

If a horizontal plan bracing is fitted between the beams, the buckling length shall assume pinned end conditions at the node points.

9.3.3 Correction coefficient for horizontal bending

For the design of the pallet beams, the design value of the bending moment in the horizontal plane may be reduced to take into account the bracing effect of the unit loads.

The bending moment obtained from the analysis of the pallet beams may be multiplied by a reduction coefficient defined as follows.

If the horizontal action per unit load exceeds the value $Q_{P,max} \cdot C_{\mu L} \cdot \mu_s$, then the value of the reduction coefficient shall be not less than 0,8, apart the load case of a single unit load at mid span whose value is 1,0, unless otherwise demonstrated.

If the horizontal action per unit load does not exceed the value $Q_{P,max} \cdot C_{\mu L} \cdot \mu_s$, then the value of the reduction factor shall be not less than the one specified in Table 12.

NOTE This is due to the positive effect of pallet friction causing diaphragm action.

Table 12 — Recommended correction coefficient for horizontal bending

Number of unit loads per compartment	Single span beams	Two or more span beams
n out of n	0	0
1 out of 2	0,6	0,5
1 at mid span out of 3	1,0	0,9
2 out of 3	0,6	0,5
2 at mid span out of 4	0,7	0,6
3 out of 4	0,5	0,45

9.3.4 Buckling length factor in the vertical plane

The buckling length of the pallet beam in the vertical plane shall be calculated considering a reduced stiffness of the beam-to-upright connection.

For the purposes of determining the buckling length the beam-to-upright stiffness shall be reduced to 1/3 of the value obtained from tests according to EN 15512 unless it is demonstrated by tests or by rational analysis that the connections maintain their efficiency under the design earthquake.

A conservative assumption is to take the beam's buckling length equal to the distance between upright's axes (pinned beam ends).

9.3.5 Beam design check

9.3.5.1 Checks in seismic conditions

The beams shall be checked according to EN 15512 for axial load, horizontal and vertical bending determined as previously specified.

The following load cases shall be considered:

— a fully loaded compartment;

— the beam loaded by the most unfavourable disposition of the unit loads.

9.3.5.2 Checks in post-earthquake situation

In addition to 9.3.5.1, the pallet beams shall be checked under the vertical load only, with pinned ends and load factor γ_f = 1,0.

NOTE This is required to ensure that after the design earthquake the unit load remains stable on the beams for safe inspection and unloading even if the beam end connector is damaged.

If sliding of the unit loads occurs, the single beam shall be checked for the maximum eccentricity possible for the unit loads in equilibrium on the beams of the compartment.

The effects of continuity shall be taken into account in the design of multi-span beams.

Annex A
(informative)

Analysis methods including second order effects

A.1 General

The aim of this annex is to present the results of seismic structural analysis performed according to the methods of this European Standard specified for low dissipative concept (see 7.4.2.2), where second order effects are relevant. It is essentially a reference for assessing the performance of software packages in terms of second order effects.

Three examples are proposed, with geometries not necessarily realistic for racking, emphasizing in different ways the second order effects:

— Example 1: simple cantilever column

— Example 2: simple beam-column system

— Example 3: plane truss

The examples are treated in a purely elastic context, i.e. considering the EN 1998-1 elastic spectrum with a damping level of ξ = 3 %, and hence with η = 1,118.

$$0 \le T \le T_B: \; S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right] \tag{A.1}$$

$$T_B \le T \le T_C: \; S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \tag{A.2}$$

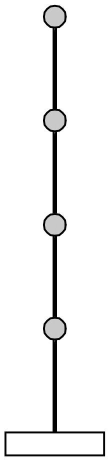
$$T_C \le T \le T_D: \; S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C}{T}\right] \tag{A.3}$$

$$T_D \le T \le 4\text{s}: \; S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C T_D}{T^2}\right] \tag{A.4}$$

The chosen spectrum is a "type 1 – soil B" situation (T_B = 0,15 s; T_C = 0,5 s; T_D = 2,0 s; S = 1,2), with a reference acceleration a_g = 3 m/s^2. Additionally, in order to maximize second order effects, example 1 is also studied for a constant spectrum with intensity equal to 3 m/s^2.

A.2 Example 1

The structure is a simplified model of a 4-level down-aisle frame assuming the masses concentrated at a single position for each level, with a column representative of the global transverse stiffness of the frame.



Properties of the equivalent column:

- Cross-section area: $A = 0{,}01\ m^2$
- Second moment of area: $I = 1{,}5 \times 10^{-5}\ m^4$

Elastic modulus: $E = 210 \times 10^9\ N/m^2$

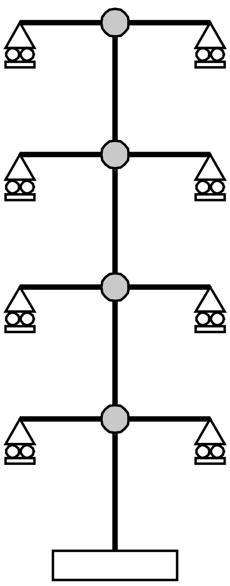
Level height: 2 m (total height = 8 m)

Masses: 3 tonnes per level
(total mass = 12 tons; $g = 9{,}81 m/s^2$)

Support conditions: Column base fixed in horizontal and vertical translation and fixed in rotation

A.3 Example 2

The structure is a single column with beams. Masses are assumed concentrated and located at the beam-columns intersections.



Geometric properties (same for the column and for the beams):

- Cross-section area: $A = 0{,}001\ m^2$
- Second moment of area: $I = 1{,}0 \times 10^{-6}\ m^4$

Elastic modulus: $E = 210 \times 10^9\ N/m^2$

Level height: 2 m (total height = 8 m)

Beam length: 1,5 m on each side of the column
(total beam length of 3 m)

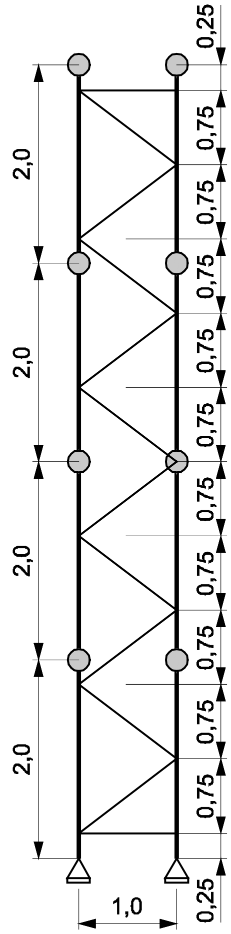
Masses: 3 tonnes per level
(total mass = 12 tons; $g = 9{,}81 m/s^2$)

Support conditions:

- Column base fixed in horizontal and vertical translation and fixed in rotation
- Beam-column connections perfectly rigid
- End of the beams simply supported in the vertical direction

A.4 Example 3

The structure is a typical D-type cross frame. Uprights are continuous all over the height. Diagonal and horizontal bracings are hinged at both ends. Concentrated masses are located every 2 m on each of the two uprights.



Dimensions in metres

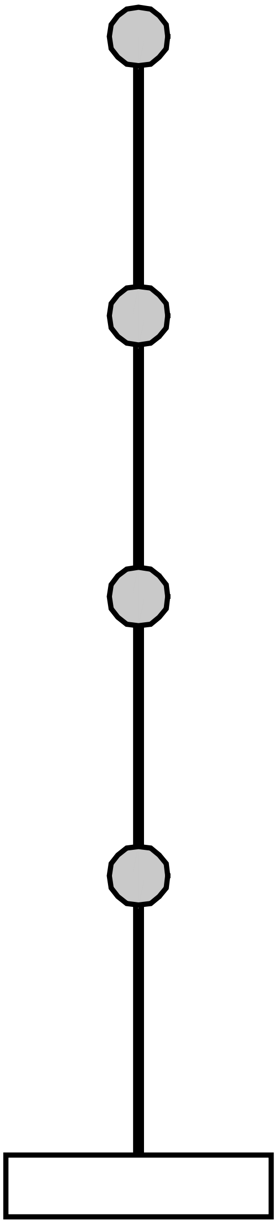
Geometric properties:		
• Uprights	Cross-section area:	$A = 0{,}001\ m^2$
	Second moment of area:	$I = 2{,}0 \times 10^{-7}\ m^4$
• Bracings:	Cross-section area:	$A = 1{,}5 \times 10^{-5}\ m^2$
Elastic modulus:	$E = 210 \times 10^9\ N/m^2$	
Level height	total height = 8 m	
Geometry of the frame	See scheme	
Masses:	3 tonnes per level	
	(8 × 1,5 t; $g = 9{,}81 m/s^2$)	
Support conditions:	Column base fixed in horizontal and vertical translation	

A.5 Reference results

The reference results are summarized in the following Tables A.1 to A.4.

NOTE The results provided here have been obtained using a Modal Response Spectrum Analysis based on a stiffness matrix modified to account for the loss of stiffness due to the compression in the uprights. They are to be considered essentially as indicative and not as absolute reference values, since the application of the methodologies to account for second-order effects is allowed to lead to slightly different outcomes from a numerical tool to another. Some methods are also known to safely provide more conservative results – i.e. larger values of the displacement, drifts and internal forces – than those proposed in the present document (e.g. linear amplified method). Software tools providing outputs significantly lower than those proposed in the present annex are questioned.

Table A.1 — Example 1 — Eurocode 8 spectrum

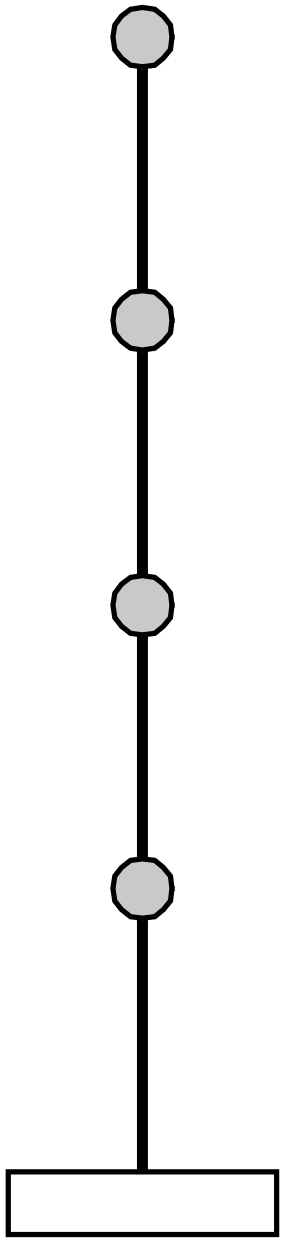


Results of the modal analysis		
Mode	**Period** s	**Mass percentage**
1	4,133	69,33
2	0,500	21,30
3	0,174	6,97
4	0,096	2,40

Displacement under seismic loading			
Total displacement		**Relative displacement**	
Level	**Displacement** m	**Between levels**	**Relative displacement** m
4	0,344 7	3–4	0,132 5
3	0,222 6	2–3	0,111 7
2	0,118 7	1–2	0,083 1
1	0,037 3	0–1	0,037 3

Internal forces under seismic loading *(maximum value between levels)*		
Between levels	**Shear** kN	**Bending moment** kNm
3–4	14,90	32,00
2–3	8,05	43,87
1–2	16,95	43,87
0–1	27,59	73,37

Table A.2 — Example 1 — Constant spectrum

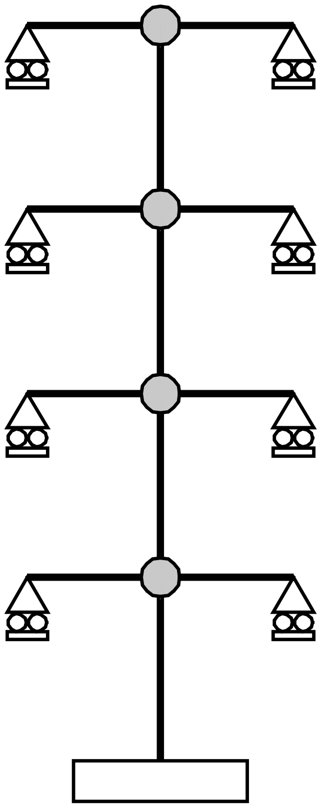


Results of the modal analysis		
Mode	**Period** s	**Mass percentage**
1	4,133	69,33
2	0,500	21,30
3	0,174	6,97
4	0,096	2,40

Displacement under seismic loading			
Total displacement		**Relative displacement**	
Level	**Displacement** m	**Between levels**	**Relative displacement** m
4	1,749 3	3–4	0,620 8
3	1,127 5	2–3	0,560 0
2	0,567 5	1–2	0,410 0
1	0,157 5	0–1	0,157 5

Internal forces under seismic loading *(maximum value between levels)*		
Between levels	**Shear** kN	**Bending moment** kNm
3–4	12,89	43,5
2–3	20,05	115,9
1–2	24,37	199,3
0–1	26,25	268,2

Table A.3 — Example 2 — Eurocode 8 spectrum



Results of the modal analysis		
Mode	**Period** s	**Mass percentage**
1	3,026	86,38
2	0,902	9,48
3	0,509	3,39
4	0,367	0,75

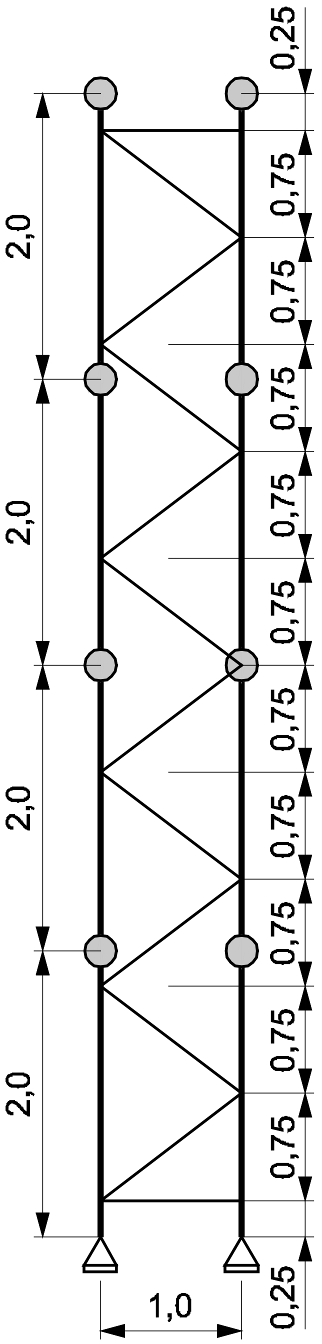
Displacement under seismic loading			
Total displacement		**Relative displacement**	
Level	**Displacement** m	**Between levels**	**Relative displacement** m
4	0,319 3	3–4	0,063 3
3	0,277 8	2–3	0,089 6
2	0,204 1	1–2	0,113 8
1	0,096 6	0–1	0,096 6

Internal forces in the beams under seismic loading *(maximum value between levels)*		
Between levels	**Shear** kN	**Bending moment** kNm
3–4	8,60	10,27
2–3	9,79	12,97
1–2	10,99	16,42
0–1	13,68	21,97

Internal forces in the beams under seismic loading *(maximum value for each levels)*		
Level	**Shear** kN	**Bending moment** kNm
4	3,42	5,14
3	6,32	9,48
2	8,42	12,63
1	9,73	14,60

Table A.4 — Example 3 — Eurocode 8 spectrum

Dimensions in metres



Results of the modal analysis		
Mode	**Period** s	**Mass percentage**
1	1,613	91,14
2	0,565	7,46
3	0,352	1,15
4	0,297	0,15

Displacement under seismic loading			
Total displacement		**Relative displacement**	
Level	**Displacement** mm	**Between levels**	**Relative displacement** mm
4	0,261 2	3–4	0,050 1
3	0,218 0	2–3	0,053 9
2	0,169 1	1–2	0,064 4
1	0,106 5	0–1	0,106 5

Internal forces in the uprights under seismic loading *(maximum value between levels)*			
Between levels	**Normal force** kN	**Shear** kN	**Bending moment** kNm
3–4	43,98	7,71	2,09
2–3	81,54	4,20	2,02
1–2	148,60	5,27	2,02
0–1	217,40	18,27	6,65

Internal forces in the bracing under seismic loading			
Level	**Normal force** kN	**Level**	**Normal force** kN
12	10,40	6	37,94
11	24,66	5	38,68
10	23,39	4	42,14
9	25,37	3	55,21
8	29,89	2	68,62
7	34,47	1	28,40

Support reaction under seismic loading		
Reaction	**Left base** kN	**Right base** kN
Vertical	217,39	217,39
Horizontal	18,27	16,84

Annex B
(normative)

Evaluation of the unit load — beam friction coefficient

B.1 General

The value of the friction coefficient between the unit load and the beam has been demonstrated being very similar in static and dynamic range; for this reason a single value can be considered in the pre and post sliding range.

The value of the friction coefficient is affected by the unit load-beam interface and environmental conditions (temperature, humidity, cleaning of the surfaces at contact); for this reason the friction coefficient shall be determined for each combination of pallet and beam surface treatment (paint type) in use in the specified environmental conditions.

The pigments and the composition of the painting used to produce the colour are allowed to produce variations in the friction coefficient.

B.2 Purpose of the test

The purpose of the test is to determine the unit load-beam friction coefficient in the warehouse conditions.

The actual loading conditions shall be represented, with the pallet placed over the pallet beams or on the real supporting interface.

There are two test arrangements relative to the racks main directions, the down aisle and the cross aisle.

NOTE The unit load-beam friction coefficient μ_S is allowed to depend on the direction.

B.3 Test arrangement

The test set-up consists in a horizontal steel frame of adequate strength and stiffness. Depending on the test method used, the frame is pinned to the ground on one side or positioned horizontally with fixed supports.

The unit load supporting elements to be tested (pallet beams or whichever) are rigidly fixed on the test frame in both directions; the unit load is then placed over the supporting system.

The test frame shall be bigger than the tested elements by at least 200 mm per side.

Adequate stops for the sliding of the unit load during the test should be placed; nevertheless, the unit load shall be free to move for at least 50 mm or for a distance bigger than the beam's width, whichever is larger.

The unit load shall be loaded with homogeneous material which will not deform and slide during the test.

The load mass shall be usually comprised between 500 kg and 900 kg, and it shall not change its configuration with respect to the pallet and shall be rigidly fixed during the test.

NOTE 1 The range 500 kg to 900 kg covers normal applications; different masses and types of unit loads are used if required for specific applications.

The loaded surface on the unit loads shall be centred with 10 mm tolerance in each direction, and similar to the actual working condition of the racking system; thus, it shall be inside the unit load's top surface but not less of the 70 % of the unit load's side length.

NOTE 2 Other configurations are considered if required for specific applications.

The centre of gravity of the unit load shall be at a height from the top of the beam equal or greater than 25 % of the smaller side of the pallet in plan, to take into account the overturning effect, due to the horizontal forces.

Before starting the test, the horizontality of the test frame shall be checked with the tolerance of ± 0,5°.

B.4 Test method

B.4.1 General

Two test methods are defined in this annex. In option 1, the friction coefficient is determined from the slope which shall be applied to the test frame in order to cause sliding of the unit load with the test load. In option 2, the friction coefficient is obtained by pulling or pushing the unit load with the test load applying a gauge measuring the force needed to evoke sliding.

B.4.2 Test method 1

In this test one side of the test frame is lifted (e.g. by using a crane) whereas the opposite side of the frame is pinned to the ground. The slope of the frame is enhanced at sufficiently small increments until the unit load begins to slide.

During the test, the following displacements shall be measured (see Figure B.1):

— vertical displacements of the test frame, taken in two positions along the opposite side with respect to the pinned one, at a distance of no more than 200 mm from the area of the unit load. Both the measured positions shall be at the same distance from the pin, with a tolerance of ± 5 mm;

— relative displacements of the unit load with respect to the frame, in the direction of the slope, of two points along the uplifted side of the unit load, symmetrically placed inside the unit load area of no more than 50 mm from its edges.

In order to avoid dynamic effects, the test frame shall be uplifted with a velocity at maximum equal to 10 mm/sec.

The test is not valid if at each measuring step, the slope of the unit load orthogonal to the sliding direction is larger than 0,5 %.

For each tested couple of loaded beams or supporting elements, the test shall be repeated 50 times with the first 10 test results being discarded. The time interval between one test and the following shall be long enough to avoid the effects of temperature variation caused by the unit load sliding.

Figure B.1 shows the setup for test method 1.

B.4.3 Test method 2

In this test the unit load is pushed or pulled with a horizontal force acting parallel or orthogonally to the tested unit load supporting elements. Care shall be taken that the vector of the applied force acts in the right direction (horizontality as well as its direction in relation to the pallet beams, etc.), that it is applied to the middle of the unit load and that it is enhanced at sufficiently small increments. The horizontal force is measured by a gauge.

During the test, at least the following values shall be continuously recorded (see Figure B.2):

— horizontal pushing or pulling force applied to the unit load measured via a gauge;

— relative displacements of the unit load with respect to the supporting elements, in the direction of the horizontal force, of two points along that side of the unit load where the horizontal pushing/pulling force is applied to, symmetrically placed inside the unit load area of no more than 50 mm from its edges.

The force to be measured to determine the friction coefficient shall be the one at the start of the movement.

For each tested couple of loaded beams or supporting elements, the test shall be repeated 50 times with the first 10 test results being discarded. The time interval between one test and the following shall be long enough to avoid the effects of temperature variation caused by the unit load sliding.

Figure B.2 shows the setup for test method 2.

B.5 Derivation of the results

B.5.1 Test method 1

For each relevant test, the value of the friction coefficient, defined as the trigonometric tangent of the uplifting angle of the frame with respect to the horizontal, is measured for both instruments placed to measure the vertical displacement of the test frame at the instant of sliding of the unit load. The result is the mean value of the two measures.

B.5.2 Test method 2

For each relevant test, the minimum force needed to cause sliding of the unit load shall be recorded. The friction coefficients are calculated by dividing the recorded horizontal forces by the test load.

B.5.3 Evaluation

The results of one test are obtained from a number of repetitions performed on the same sample. The number of repetitions shall not be less than 50.

In order to determine the final value of the friction coefficient, the results of the first 10 tests shall not be used, since the first tests are not really representative of the effective service rack conditions; for this reason, the evaluation of the test result is performed over at least 40 values.

The friction coefficient obtained for the test is the mean value of the friction coefficients obtained from the repetitions.

For each test the different beams and unit loads are used.

The reference value of the friction coefficient μ_S defined in 7.5.3 is the mean value obtained from at least three tests:

$$\mu_S = \mu_m$$

$$\mu_m = \frac{1}{n}\sum_{i=1}^{n} \mu_{ni} \qquad \text{(B.1)}$$

where

μ_{ni}	is individual test result (mean value of ≥ 40 repetitions on the same set-up);
n	is number of tests (≥3).

The coefficients $C_{\mu L}$ and $C_{\mu H}$ defined in 7.5.3 can be obtained from the tests:

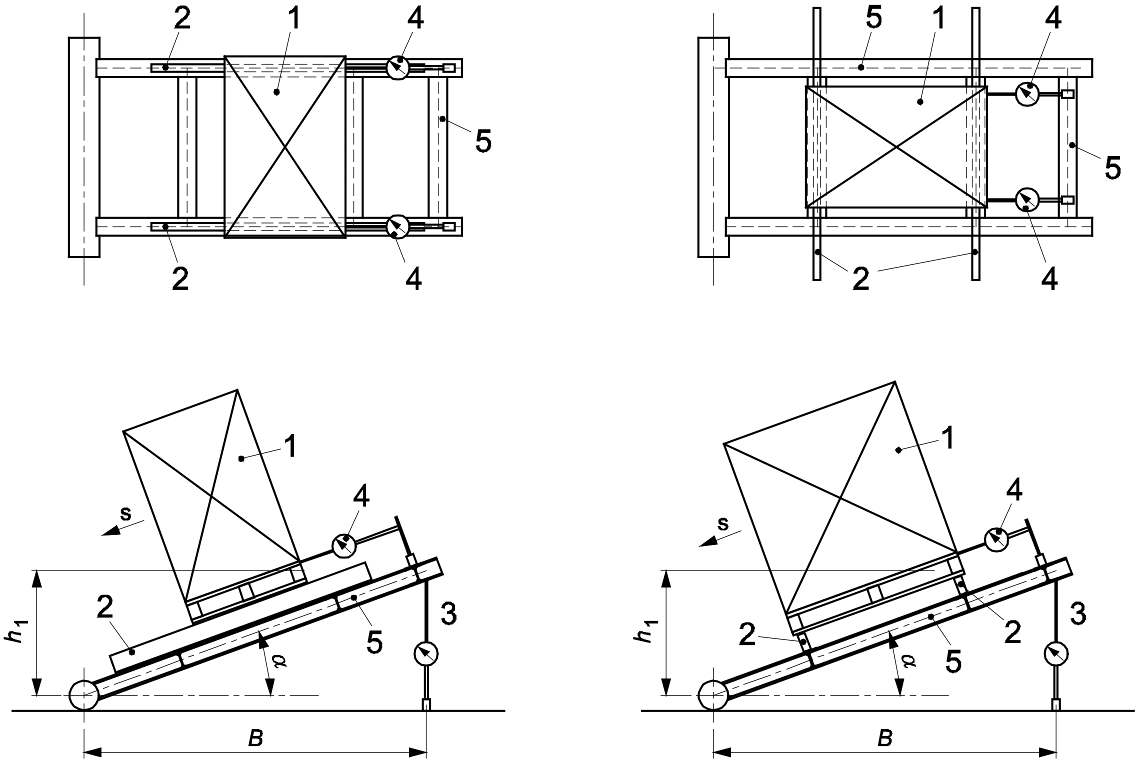
$$C_{\mu L} = 1 - k_S\, s / \mu_m$$

$$C_{\mu H} = 1 + k_S\, s / \mu_m$$

where

$$s = \sqrt{\frac{1}{(n-1)} \sum_{i=1}^{n} (\mu_{ni} - \mu_m)^2} \text{ = standard deviation of the test results} \qquad \text{(B.2)}$$

k_s = coefficient in Table 13 of EN 15512:2009



Key

s sliding direction

1 unit load

2 beams

3 displacement gauge (instrument to measure vertical displacements of the frame)

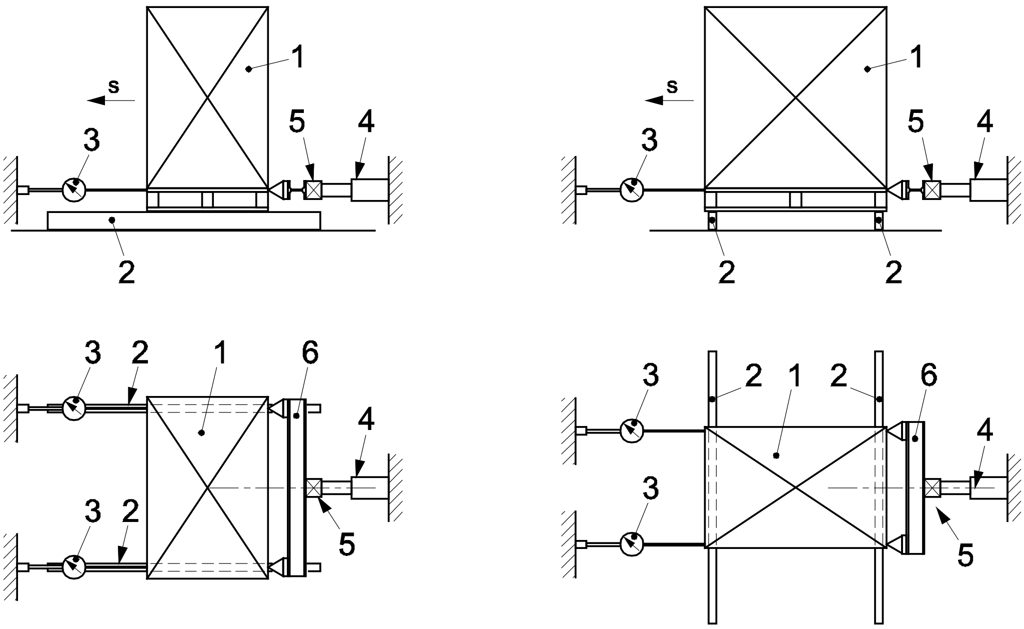
4 displacement gauges (instruments to measure relative displacements between the pallet and its supporting system)

5 frame

B horizontal distance between the axe of instrument 1 and the axe of the frame's pin

α sliding angle = arctan (h_1 /B)

Figure B.1 — Method 1 — frame lift: test arrangement



Key

s sliding direction

1 unit load

2 beams

3 displacement gauges (instruments to measure relative displacements between the pallet and its supporting system)

4 jack

5 gauge measuring the horizontal load applied to the unit load

6 load spreader

Figure B.2 — Method 2 — unit load push-pull: test arrangement

Annex C
(informative)

Principles for modelling the unit load masses

C.1 General

The principles of the modelling of the unit loads supported by the racking structure are reported in 7.5.8. In general, in order to simplify the numerical model, in down aisle direction the masses of the unit loads can be modelled at the level of the pallet beams, and in cross aisle direction they have to be considered at their actual elevation above the support beams.

In the numerical model, the masses of the unit loads are in general modelled as lumped masses, connected to the beam-upright nodes, and not distributed on the pallet beams.

Indeed, the interaction between unit loads and pallet beams is not directly considered in the global analysis, for the following reasons:

— the design actions on the pallet beams are evaluated by means of local analysis, as specified in 9.3, where the positive effects of the friction between pallets and beams is considered;

— the damping effects due to the unit load - beam friction, movement of the beams and the internal movements of the stored goods, are considered numerically in the E_{D1}, E_{D2} and E_{D3} modification factors.

For the purpose of the analysis, it is allowed to model the unit loads by using simplified dummy substructures, in order to consider properly the actual elevation of the centre of gravity.

Dummy substructures should be such that the diaphragm effect of the unit loads is taken into account in cross aisle direction and in down aisle direction when the rack is not vertically braced; in these cases the diaphragm effect has a limited influence on the response of the structure and no influence on its resistance.

In vertically braced racks, all the torsion effects shall be concentrated on the horizontal bracings and on the connected upright frames, and consequently the dummy substructures shall be such that they don't produce diaphragm effect in down aisle direction.

NOTE The actual stiffness and strength of the diaphragm is not predicted and evaluated from the global analysis, while it has significant influence on the behaviour and resistance of the structure.

In general, different set of masses in down aisle and cross aisle direction need to be defined.

C.2 Down aisle direction

In down aisle direction, for the purpose of the global analysis the difference in height between the centre of gravity of the unit loads and their supports on the beams can be neglected for multiple bay racks. The increase of the axial force in the uprights due to the elevation of the centre of gravity to the beams is negligible for the internal uprights (it is null when beams are equally loaded), and it is assumed not affecting the design for the lateral uprights; also the variation of the shear action at the beam's ends can be considered negligible. The masses in down aisle direction can be modelled lumped at the beam-upright intersection.

C.3 Cross aisle direction

In cross aisle direction, the masses of the pallets shall be considered at their actual elevation, because of the height-depth ratio of the rack in this direction.

Effects of vertical eccentricity of the unit load masses are allowed to be neglected for torsion in racks not regular in plan.

When the masses are modelled at the level of the pallet beams, a correction to the actions in the uprights is presented in Annex D.

Alternatively, it is permitted to lump numerically the masses defined in cross aisle direction at nodes positioned in the plane of the upright frames, at the elevation of their centre of gravity, centred between the upright's axes, and rigidly connected to both the uprights at the level of the pallet beams.

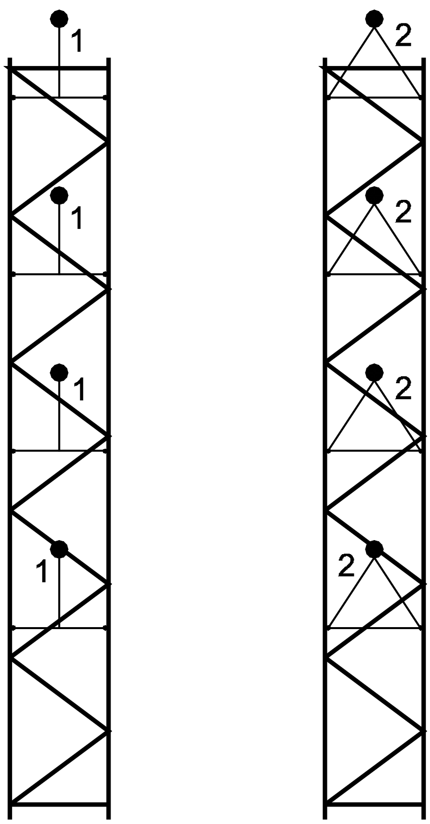
The rigid connection can be realized using either numerical constraints or "dummy" substructures triangular or inverted "T" shapes; only horizontal and vertical inertial actions need to be transmitted to the uprights.

When "dummy" substructures are used:

— the stiffness of the elements shall be properly calibrated, in order to avoid local relevant modes of vibration on the substructures, when they are too flexible, or numerical instability,

— the mass of the dummy substructures shall be null.

Figures C.1 and C.2 show examples of mass modelling in cross aisle direction using substructures.

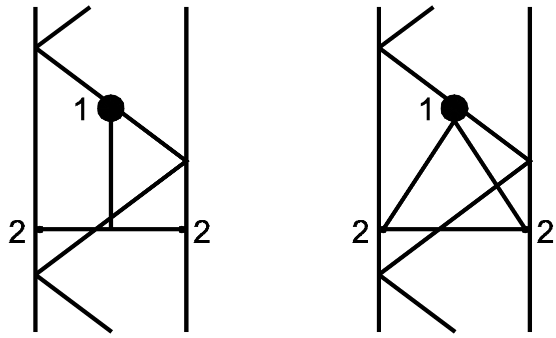


Key

1 dummy substructures - inverted T type

2 dummy substructures - triangular type

Figure C.1 — Masses modelling in cross aisle direction using substructures



Key

1 mass defined in cross aisle direction

2 moment releases (hinged connection)

Figure C.2 — Masses modelling in cross aisle direction using substructures

Annex D
(informative)

Simplified method to evaluate the influence of the centre of gravity of the pallet regarding the beam level

D.1 General

In this Annex, a simplified method to take into consideration the effect of the actual position of the centre of gravity of the unit load, when their masses are modelled at the level of the pallet beams, is described.

The influence of such eccentricity (centre of gravity regarding the beam) is limited to the cross aisle direction.

This approach is also valid for cases where requirements for applying of LFMA are not fulfilled.

D.2 Simplified method for global analysis

An additional vertical force ΔN_i is considered at each beam-upright intersection based on the value of the horizontal action H_i at each level calculated using LFMA method:

$$\Delta N_i = \frac{H_i \times e_i}{b} \tag{D.1}$$

where

H_i is the horizontal action on the unit load at level i;

e_i is the height of the centre of gravity of the unit load at level i;

b is the distance between the uprights axes.

The additional vertical force ΔN_i is added to the axial load in the upright obtained from the analysis.

At the base of the uprights, the additional vertical force is:

$$\Delta N_{base} = \frac{\sum_{i=1}^{n} H_i \times e_i}{b} \tag{D.2}$$

where

n number of load levels

Annex E
(informative)

Principles for the design of racks supported by floors

E.1 General

In this Annex, three methods of design for racks installed on suspended floors are presented (see 6.7), referenced respectively as method 1, 2 and 3.

Methods 1 and 2 are based on the structural analysis of the system made of the racking and the supporting structure.

In method 3, a conventional distribution of force maximizing the seismic action is applied to the racking structure.

E.2 Mass on the rack to be considered with Methods 1 and 2

E.2.1 General

In these methods, situations with different amounts of masses are analysed, in order to investigate the situations maximizing the resonance effects; at least the following cases will be considered:

100 % - 75 % - 50 % - 25 %.

The case of mass from dead weight and permanent loads only 0 % will be considered for light duty pallet racking systems.

The mass is allowed to be considered uniformly distributed on the racking structure (i.e. the rack fully loaded with reduced weight of the unit load will be considered).

The mass of the unit load on the rack is affected by the E_{D2} factor.

Additionally, the upright frame in cross aisle direction is analysed also in the configuration with 100 % pallet weight at the top level.

NOTE This is to maximize the uplift action on the baseplate.

E.2.2 Method 1

Method 1 consists in the MRSA of the system made of the complete supporting structure and the whole row of the rack concerned, using one single numerical model.

The model of the supporting structure can be a simplified one, but it shall provide a reliable representation of its dynamic behaviour.

The MRSA is performed using the elastic spectrum with the damping factor defined for the supporting structure.

The model of the supporting structure includes the mass related to the dead loads and to the variable loads considered in the same way used for its design, apart the tributary mass of the rack that is applied to the rack itself.

For the design of the rack, the effects of the seismic action on the rack are multiplied by $E_{D1}E_{D3}/q_{rack}$, where q_{rack} is the behaviour factor of the rack.

The number of modes considered is chosen in such a way that at least the first two translational modes in each direction of the racking structure are activated.

E.2.3 Method 2

Method 2 is applicable when the total mass of the rack concerned does not exceed 10 % of the total mass of the building (mass related to the dead loads and to the variable loads considered to design the building, as defined for method 1).

In this case a MRSA or LFMA of the rack can be performed, under the conditions defined in Clause 7, without modelling the building.

The used design floor spectrum is the modified spectrum defined in Clause 7, where S_d is defined as $\gamma_I S_a/q$, and S_a is the seismic coefficient specified in EN 1998-1:2004, 4.3.5.2 for the analysis of non-structural components, with Z the distance from the ground level to the fixing level of the rack, q and γ_I are respectively the behaviour factor and the importance factor defined for the rack.

E.2.4 Method 3

Method 3 consists in applying a set of horizontal forces $F_{E,i}$ at all the unit loads, where

$$F_{E,i} = \mu_S \times Q_{p,rated,i} \tag{E.1}$$

and

μ_S is the pallet-beam friction coefficient;

$Q_{p,rated,I}$ is the weight of the product stored at the level i in the seismic condition.

The calculated forces are based on the maximum action that can be transmitted to the racking structure by the friction; the method is conservative and it can always be applied.

E.3 Vertical component of the seismic action

The effects of the structural amplification of the vertical component of the seismic action can be generally neglected when the span of the supporting suspended floor is less than 20 m; otherwise, they shall be considered in all cases.

Annex F
(normative)

Additional detailing rules for dissipative elements (Concept B)

F.1 General

Steel storage racks designed under design Concept B – Dissipative structural behaviour (see 8.1.1) shall fulfil the detailing rules for steel structures of EN 1998-1, with the following additional specific rules.

F.2 Connections participating in the energy dissipation

The strength and ductility under cyclic loading of the connections in dissipative zones of the structure shall be supported by experimental evidence, in order to comply with specific requirements defined for each structural type and structural ductility classes as specified in this Annex.

F.3 Moment resisting frames

The design of moment resisting frames in which the ductility is concentrated in the connections (beam-to-upright connections or floor connections) shall be based on the moment-rotation characteristics determined according to EN 1998-1; for beam-upright connections made with hooked connectors or similar Annex G shall be used.

The rotation capacity θ_p shall be consistent with deformations from the global analysis that shall take into account explicitly the combined effects of geometric nonlinearity, plasticity and degradation of the characteristics of connections.

NOTE Because of the high flexibility of the racking structures, the design approach based on MRSA and application of the q factor lead to uneven amplifications of second order effects. Refer also to Table 3 in 7.4.2.3.

F.4 Requirements for horizontal bracings

Horizontal bracings shall be designed to withstand the seismic action without plasticity.

Annex G
(normative)

Testing procedure for beam-upright and floor connections for dissipative design (concept B)

G.1 Bending test on beam-end connector

G.1.1 Test arrangement

The test setup for monotonic and cyclic test is the same described in EN 15512:2009, A.2.4, with the following modifications:

— the test shall be performed in displacement control;

— the load jack shall apply reversal moments, when cyclic test is performed;

— when required, an additional constant downward load $P_c = 5\ kN$ shall be applied on the beam at a distance not greater than 100 mm from the connector, prior to starting the cyclic or monotonic loading and left constant during the test, simulating the design downward-acting gravity on the unit load that serve to fully engage the beams and their connectors into the column's holes.

Either safety pins or seismic bolts shall be installed if they are planned to be in place in the design.

Refer also to Figure A.5 of EN 15512:2009.

G.1.2 Monotonic test

The purpose of the monotonic test is to evaluate the ultimate moment and the rotational capacity of the beam end connector.

The test procedure, the corrections to the observations and the derivation of the results is the same as described in EN 15512:2009, A.2.4.

Optionally or when required to ensure the engagement of the hooks, the additional downward loads shall be applied.

The test shall be performed both in downward and reverse directions; at least three tests shall be performed in each direction.

G.1.3 Cyclic load test

G.1.3.1 General

In this clause requirements for the loading history and failure criteria are provided; they shall be applied in the cyclic testing of beam end connectors when dissipative design concept is used (Annex F).

The procedure for beam end connectors is designed to take into account the unsymmetrical behaviour in hogging-sagging conditions.

G.1.3.2 Loading history

The loading sequence shall have the following characteristics:

— 1 cycle at 0,25 × M_k

— 1 cycle at 0,50 × M_k

— 1 cycle at 0,75 × M_k

— 1 cycle at M_k

— 3 cycles at 2 × Φ_K

— 3 cycles at 3 × Φ_K

where

M_k is the characteristic bending strength obtained from monotonic tests both positive and negative;

Φ_K is the rotation at M_k, both positive and negative.

More cycles or more intervals may be used if necessary.

NOTE The loading protocol is the same of ECCS-45:1986, except that the load cycles defined in 3.3 are modified.

G.1.3.3 Corrections to the observations

The moment strength shall be corrected as specified in EN 15512:2009, A.2.4.4.

G.1.3.4 Parameters and interpretation of the tests

For the specific parameters for a group of three cycles of equal rotation, refer to ECCS-45:1986, 3.5.

For the parameters of interpretation of the whole test, refer to ECCS-45:1986, 3.6.

G.1.3.5 Acceptance criteria

For dissipative design, the requirements on rotation capacity reported in EN 1998-1:2004, 6.6.4 (2) and (3) for DCH apply.

G.2 Bending test on floor connection

G.2.1 Test arrangement

The test setup for cyclic test is the same as described in EN 15512:2009, A.2.7 with the following modifications:

— the test shall be performed in displacement control;

— the load jack shall apply reversal moments, when the cyclic test is performed.

Anchor bolts shall be installed in the same arrangement as they are planned to be in the design.

Refer also to Figure A.11 of EN 15512:2009.

G.2.2 Cyclic load test

G.2.2.1 General

In this clause requirements for the loading history and failure criteria are provided; they shall be applied in the cyclic testing of the floor connection when dissipative design concept is used (Annex F).

G.2.2.2 Loading history

The loading sequence shall have the following characteristics:

— 1 cycle at 0,25 × M_k

— 1 cycle at 0,50 × M_k

— 1 cycle at 0,75 × M_k

— 1 cycle at M_k

— 3 cycles at 2 × Φ_K

— 3 cycles at 3 × Φ_K

where

M_k is the characteristic bending strength obtained from monotonic tests both positive and negative;

Φ_K is the rotation at M_k, both positive and negative.

More cycles or more intervals may be used if necessary.

NOTE The loading protocol is the same of ECCS-45:1986, except that the load cycles defined in 3.3 are modified.

G.2.2.3 Corrections to the observations

The moment strength shall be corrected as specified in EN 15512:2009, A.2.7.4.

G.2.2.4 Parameters and interpretation of the tests

For the specific parameters for a group of three cycles of equal rotation refer to ECCS-45:1986, 3.5.

For the parameters of interpretation of the whole test refer to ECCS-45:1986, 3.6.

G.2.2.5 Acceptance criteria

For dissipative design, the requirements on rotation capacity for DCH reported in EN 1998-1:2004, 6.6.4 (3) apply.

Annex H
(informative)

Assessment of the stability of the unit load

H.1 General

In this Annex, methods to assess the stability of the unit load for overturning, and the “tilt test” used to verify the strength and stability of the goods stored in the unit load (e.g. stretch wrapping, banding), in order to prevent the risk of their failure and falling, are presented.

H.2 Recommended aspect ratio of the unit load

The following height to minimum width ratio H/b is recommended to reduce the possibility of the unit load overturning.

When $2{,}5 \times \alpha \times S \geq 1{,}1$, $H/b \leq 2{,}0$ for pallets placed at a height equal or greater than 2,50 m above the floor.

When $0{,}7 \leq 2{,}5 \times \alpha \times S \leq 1{,}1\ g$, $H/b \leq 2{,}5$.

The unit load is assumed to have a uniform weight distribution.

H.3 Pallet “tilt test”

The purpose of the test is to ensure that the means used to secure the merchandise in the unit load are sufficiently strong and rigid to keep the merchandise from sliding during earthquake shaking.

The basic test procedure is as follows:

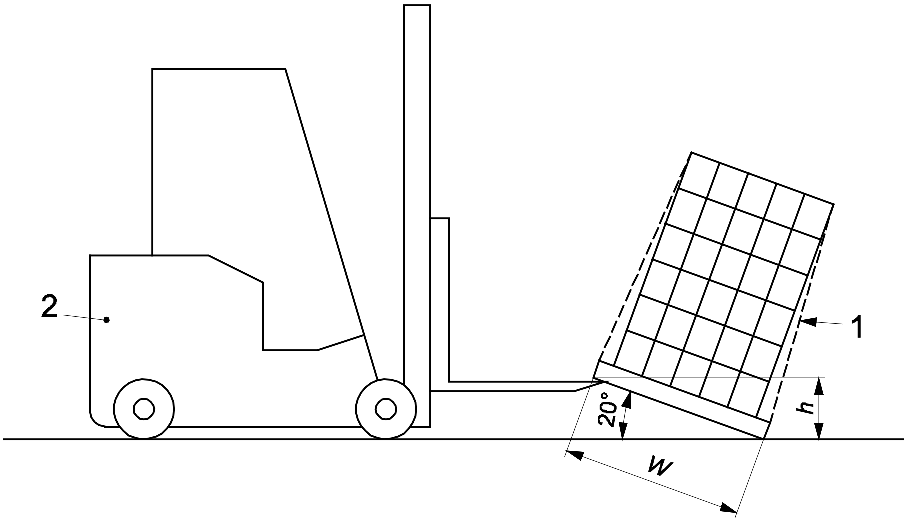
— step 1: the merchandise is bound in the unit load with a securing method;

— step 2: the pallet is lifted on one side to a height that produces an angle of 20° between the ground and the bottom surface of the pallet;

— step 3: if the merchandise remains restrained in place for at least 5 min without appreciable movement, the load secured to the pallet is considered to have adequate confinement and passes tilt test;

— step 4: if the merchandise shifts appreciably or the securing material breaks, the merchandise shall be re-secured using another industry-approved method and retested.

Figure H.1 and Table H.1 show the procedure for the execution of the “tilt test”.



Key

1 product secured in the unit load

2 forklift

W pallet width

Figure H.1 — Pallet tilt test

Table H.1 — Pallet width and required lift

Pallet Width W [mm]	Required Lift h [mm]
1 200	437
1 100	400
1 000	364
900	330
800	291

H.4 End bay Upright frames

It is recommended that the ends of a longitudinal row of racks have upright frames or frame extenders that extend high enough above the topmost shelf to provide sliding and overturning restraint for palletized or individually stored merchandise on the upper-most level.

This is to prevent the merchandise from toppling into the main aisle-ways generally located at the end of a row of storage racks.

If one end of the row abuts a wall, this frame need not have the frame extension.

Annex I
(informative)

Data to be exchanged between the Specifier/End User and the Rack's Supplier

NOTE Complement to EN 15629 for racking installations in seismic areas.

I.1 Information to be provided by the Specifier/End User to the Rack's Supplier

a) Seismic zone or ground peak acceleration;

b) response spectrum type (Type 1, Type 2 or other spectrum specified by National Regulations based on EN 1998-1 approach – in this case name of the Regulation shall be specified e.g. DIN EN 1998-1 etc.);

c) ground type;

d) importance class (refer to Table 1);

e) design life (standard for racking: 30 years / buildings: 50 years; refer to Table 1);

f) warehouse/retail area with public access / Storage of hazardous products;

g) unit load weight specifications:

$Q_{P;max}$ specified weight of the unit load (see EN 15629, 6.7.1);

$Q_{P;rated}$ specified value of the weight of unit loads for the compartment, upright frame or global down aisle design (see EN 15512) as specified by the Specifier (see also EN 15629:2008, 6.7.1);

h) R_F , rack filling reduction factor (only for down aisle direction - refer to 7.5.4);

i) storage environment (e.g. standard, cold store, chill store with wet pallets; refer to 7.5.3);

j) type of load make up accessory (e.g. wooden / plastic pallet; steel box pallet; refer to 7.5.3);

k) class of stored goods ("compact", "weak" etc. refer to 7.5.5, Table 5);

l) seismic sway of the building.

I.2 Information to be provided by the Rack's Supplier to the Specifier/End User

a) Risk related to sliding of unit loads (refer to Annex J);

b) risk related to rocking of stored goods (refer to Annex H);

c) minimum value of ground acceleration defined in 5.2, for which is requested the check of integrity of the racks after a seismic event (refer also to Annex J).

Annex J
(normative)

Complementary rules to EN 15635

J.1 Damage limitation requirement: assessment of the damage after an earthquake

After a seismic event in which the ground acceleration is greater than 0,50 $\gamma_I\ a_{gR}\ S$ or 0,50 $a_{gR}\ S$, whichever is less, the PRSES shall instruct a complete check of the integrity of the racking structure. The assessment of the level of damage to the structural elements is mandatory before returning the rack to use.

NOTE The seismic intensity is obtainable from publicly available information.

The minimum value of 0,50 $\gamma_I\ a_{gR}\ S$ and 0,50 $a_{gR}\ S$ shall be provided by the rack supplier in the User manual.

Damage evaluation criteria and the appropriate actions are given in EN 15635.

J.2 Unit load sliding

After a seismic event, when unit loads are found in a position on the beams outside the range of acceptance according to EN 15620, those pallets shall be repositioned.

J.3 Unit load falling

The Supplier shall inform the User about the possibility of occurrence of movement of the unit loads causing their falling from the rack, based on the criteria of 9.2.2.2.

J.4 Unit load rocking and overturning

The Supplier shall inform the User about the possibility of occurrence of rocking or overturning of the unit loads causing their falling from the rack, see 9.2.2.3 and Annex H.

NOTE Depending on the ratio between the height and width of the unit load and on the seismic acceleration at each load level, rocking phenomena and overturning can occur. The PRSES uses the information provided by the rack supplier to manage the risk taking into account the working conditions of the warehouse.

J.5 Responsibility of the User

The User or the PRSES shall assess the risk, considering the working conditions of the warehouse and the nature of the stored goods, in order to require or prescribe adequate precautionary measures.

Annex K
(informative)

Complementary rules to EN 15629 — Warehouse environmental condition category

(See also EN 15512:2009, 8.9 and EN 15629:2008, 6.10).

a) Normal Condition:

   1) in-house storage equipment;

   2) temperature: not lower than 8 °C;

   3) humidity: service class 1 according to EN 1995-1-1, at a temperature of 20 °C the relative humidity of the air exceeds 65 % only for a few weeks per year;

   4) corrosivity: no progressive corrosion will occur of unprotected steel (only a brown colouring is possible).

In case the humidity does not comply with the above, the mechanical properties of wood based products used in the storage equipment shall be corrected in accordance with EN 1995-1-1.

NOTE 1 Protective coatings used in the industry today will be acceptable for Normal Condition.

Due to loading and unloading operations the coating on the unit load supporting component surface is allowed to disappear locally due to wear and tear.

b) Chilled (“wet cold”) store Condition:

   1) conditions as given for Normal Condition, however

   2) temperature between 8 °C and 0 °C.

c) Cold store Condition:

   1) conditions as given for Normal Condition, however

   2) temperature between 0 °C and – 45 °C.

d) Outdoor Condition:

   1) temperature: site specification;

   2) humidity: site specification;

   3) corrosivity: site specification.

In case a wall of the warehouse building is partly and permanently open, it shall be considered as an Outdoor Condition, otherwise specified differently by the end-user or by the Specifier.

e) Corrosive Condition:

   all the conditions where the environmental situation with regard to corrosivity is not complying with the conditions a) to d) above.

NOTE 2 In such cases the surface treatment of the steel racking or shelving components is agreed between parties. The End User specifies which protection system is required.

Annex L
(informative)

A–deviations

A-deviation: National deviation due to regulations, the alteration of which is for the time being outside the competence of the CEN-CENELEC national member.

This European Standard does not fall under any Directive of the EU.

In the relevant CEN-CENELEC countries these A- deviations are valid instead of the provisions of the European Standard until they have been removed.

Italy

Clause	Deviation
	"*Decreto del Ministero delle Infrastrutture 14/1/2008*" (DM 14/1/2008)
7.5 Design parameters for seismic analysis - **7.5.1 General**	The response spectrum to be applied for the design of pallet racks in the Italian territory is the one defined in the Decree "Decreto del Ministero delle Infrastrutture 14/1/2008" (DM 14/1/2008) in force, until it is updated by a new Code. 3.2.3.2 "SPETTRO DI RISPOSTA ELASTICO IN ACCELERAZIONE"
6.3 Importance factor γ_I	The intensity of the seismic action is determined in relation to the nominal life of the structure, the class of use and the reference period as they are defined in the DM 14/1/2008,2.4.2. The reference period VR shall be not less than 35 years (2.4.3). The classes of use and the coefficient CU for racking systems are defined in the Table L.1 (as well as in Tab 2.4.II Valori del coefficiente d'uso CU):

Table L.1 — Class of use and Coefficient CU for racks

Class of use	Description	Coefficient C_U
I	Warehouses with fully automated storage operations. Warehouses with low occupancy [a]	0,7
II	Normal warehouse conditions, including picking areas [b]	1,0
III	Racks installed in areas with public access	1,5
IV	Hazardous product storage (subject to the approval of National Authorities) Strategic facilities	2,0

[a] In the low occupancy warehouses there is not permanence of people in the storage area but only access for maintenance.

[b] Only authorized and trained workers are permitted to access the storage areas within a warehouse

Clause	Deviation
	"*Decreto del Ministero delle Infrastrutture 14/1/2008*" (DM 14/1/2008)
6.3 Importance factor γ_I	For the only purpose of the evaluation of the seismic action the nominal life of a storage rack is 50 years; it is allowed to consider a reduced life of 35 years when: - racks are not installed on a building suspended floor - racks are not for storage of hazardous product - racks are not installed in areas open to the public access
9.2.1.3 Material's factor γ**M**	Referring to D.M. 14 Gennaio 2008 "Norme Tecniche per le Costruzioni", 4.2.4.1 "VERIFICHE AGLI STATI LIMITE ULTIMI" tab. 4.2.V the safety coefficients for materials are: - Resistance of cross-sections γM 1 = 1,05 - Resistance to buckling γM 1 = 1,05 - Resistance of the connections γM 3 = 1,25

Spain

Clause	Deviation
General	In Spain, the Royal Decree 997/2002 of 27 of September approves the earthquake resistant construction Standard: general part and edification NCSR-02 / NCSE-02. The scope of the standard NCSR-02 / NCSE-02 is extended to all projects and construction works relative to edification, and, as appropriate, to other types of constructions, until specific standards or provisions containing seismic requirements will be approved. This implies that to satisfy the Royal Decree 997/2002 the Standard NCSR-02 / NCSE-02 shall be used where applicable and required by the Spanish Laws.

Bibliography

[1] ECCS-45:1986, *Recommended testing procedure for assessing the behaviour of structural steel elements under cyclic loads*

[2] EN 1995-1-1, *Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings*

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