AS 4084—2012

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Australian Standard®

Steel storage racking

STANDARDS Australia

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- Australian Industry Group
- Australian Steel Institute
- Consult Australia
- Engineers Australia
- Griffith University
- The University of Sydney
- WorkSafe Victoria

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AS 4084—2012

Australian Standard®

Steel storage racking

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PREFACE

This Standard was prepared by the Standards Australia Committee BD-062, Steel Storage Racking, to supersede AS 4084—1993, *Steel storage racking*.

The objective of this Standard is to provide designers of steel storage racking with specifications for hot-rolled and cold-formed steel structural members used for action carrying purposes.

The design provisions of the Standard are based on the limit states method and are intended to supplement AS 4100 and AS/NZS 4600.

This edition incorporates the following major changes to the previous edition:

(a) The Standard is in limit states format.

(b) The Standard provides for internal actions to be determined by linear, geometric nonlinear, and material and geometric nonlinear analyses.

(c) The Standard contains a comprehensive range of tests for determining the stiffness and strength of rack components and subassemblies.

Reference has been made to the European Racking Code (EN 15512) and the American Rack Manufacturers Institute Specification (RMI).

The term 'normative' is used in this Standard to define the application of the appendix to which it applies. A normative appendix is an integral part of a Standard.

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SECTION 1 SCOPE AND GENERAL

1.1 SCOPE

This Standard sets out minimum requirements for the design, fabrication and erection tolerances, test methods, operation and maintenance of steel storage racking in the limit states method.

This Standard applies to adjustable static pallet racking made of cold-formed or hot-rolled steel structural members. It covers racking installed within a building, outside a building, and racking that forms part of the frame of the building.

The Standard does not cover drive-in and drive-through racking, cantilever racking, mobile racking or racking made of materials other than steel.

NOTE: Guidance for the design of drive-in and drive-through racking is available in FEM 10.2.07, and in FEM 10.2.09 for cantilever racking.

1.2 NORMATIVE REFERENCES

The following are the normative documents referenced in this Standard:

NOTE: Documents for informative purposes are listed in the Bibliography.

AS	
1170	Structural design actions
1170.4	Part 4: Earthquake actions in Australia
1391	Metallic materials—Tensile testing at ambient temperature
1657	Fixed platforms, walkways, stairways and ladders—Design, construction and installation
4100	Steel structures
AS/NZS	
1170	Structural design actions
1170.0	Part 0: General principles
1170.1	Part 1: Permanent, imposed and other actions
1170.2	Part 2: Wind actions
4600	Cold-formed steel structures
FEM	The European Federation of Materials Handling
9.831	Calculation Principles for Storage and Retrieval Machines. Tolerances, Deformations and Clearances in the High-bay Warehouse
9.832	Basis of Calculations for Storage and Retrieval Machines, Tolerances, Deformations and Clearances in Automatic Small Parts Warehouses (not Silo Design)

1.3 DEFINITIONS

For the purpose of this Standard, the definitions below apply.

1.3.1 Adjustable pallet racking

Storage system comprising upright frames perpendicular to the aisles and independently adjustable, positive locking pallet beams parallel to the aisles, spanning between the upright frames, and designed to support unit load actions (see Figures 1.3.1(a) to (c)).

1.3.2 Aisle width

Space along which the unit load handling equipment operates (see Figure 1.3.2(a)).

1.3.3 Base plate

Bearing plate bolted or welded to the underside of the upright to transmit vertical and horizontal forces into the floor, and provide structural fastening of the upright frame to the floor.

1.3.4 Bay height

Maximum vertical distance from the ground to the highest point of the unit loads in a racking structure (see Figure 1.3.2(b)).

1.3.5 Bay width

See definition of pallet beam length (see Clause 1.3.31 and Figure 1.3.2(b)).

1.3.6 Ceiling clearance

Minimum vertical distance between the highest part of the upright frame or the highest part of the unit load on the top pallet beam level and the underside of the ceiling or the support steelwork for the ceiling (see Figure 1.3.2(b)).

1.3.7 Closed-face racking

Adjustable pallet racking where the unit loads are supported by pallet beams (see Figure 1.3.1(b)).

1.3.8 Cross-aisle

The direction perpendicular to an operating aisle or parallel to an upright frame.

1.3.9 Diagonal frame brace

Diagonal member in the vertical plane that supplements horizontal braces to join uprights together and form a trussed upright frame that is stiff and stable, and designed to withstand applied design loads (see Figure 1.3.1(b)).

1.3.10 Down-aisle

The direction parallel to an operating aisle or perpendicular to an upright frame.

1.3.11 End frame extension

The protrusion of the uprights of an end upright frame beyond the top surface of the uppermost pallet beam (Figure 1.3.1 (a) and (b)). Where it is required, an end frame extension should be at least 50% of the unit load height.

1.3.12 Finished tolerances

Tolerance of the unloaded racking after fabrication and erection prior to initial loading.

1.3.13 Frame bracing

A system of bracing or batten members used to link a pair of uprights in the vertical plane and in the cross-aisle direction to provide frame stability in the cross-aisle direction.

1.3.14 Frame spacer (back tie)

Horizontal member, usually bolted to upright frames to maintain distance between upright frames in a double entry (double-sided) racking layout and designed to resist applied design loads (see Figures 1.3.1(a) and 1.3.2(a)).

1.3.15 Fully automatic operation

Operation, without manual interference, of fully remote controlled robots for storing and retrieving pallets in a racking system.

1.3.16 Geometric non-linear analysis (GNA)

Elastic analysis where the equilibrium is obtained in the deformed frame configuration.

1.3.17 Geometric and material non-linear analysis with imperfections—Type c (GMNIAc)

Non-linear analysis where the equilibrium is obtained in the deformed frame configuration, where

(a) the cross-section is compact in accordance with AS 4100 and non-perforated except at connection points;

(b) the analysis may be based on prismatic beam-elements; and

(c) the analysis considers torsional (twist) rotations and torsional internal actions, including warping torsion, unless fully laterally restrained in accordance with AS 4100.

1.3.18 Geometric and material non-linear analysis with imperfections—Type s (GMNIAs)

Non-linear analysis where the equilibrium is obtained in the deformed frame configuration, where

(a) the cross-section is non-compact in accordance with AS 4100 or cold-formed;

(b) the analysis is based on discretization of pallet beams and uprights into shell finite elements or finite strips modelling perforations; and

(c) the analysis incorporates local and distortional geometric imperfections.

1.3.19 Horizontal frame brace

Horizontal member that joins two uprights together in a frame by bolting or welding (see Figures 1.3.1(a) and (b)).

1.3.20 Linear analysis (LA)

Elastic analysis where the equilibrium is obtained in the original (undeformed) frame configuration (i.e, the displacements are assumed infinitesimal).

1.3.21 Linear buckling analysis (LBA)

Elastic analysis where the pre-buckling internal actions are obtained from a linear analysis.

1.3.22 Load

The value of a force appropriate for an action.

1.3.23 Load (stub) arm

Short load support arm attached to the upright to support the pallet rail (see Figure 1.3.1(c)).

1.3.24 Manual operation

Operation of machines and positioning of equipment controlled by an operator.

1.3.25 Open-face racking

Adjustable pallet racking where the unit loads are supported by load arms attached to the uprights and rails spanning between the support arms, (see Figure 1.3.1(c)).

1.3.26 Operating clearance

Nominal clearance dimension between static and moving parts to ensure safe operation.

1.3.27 Pallet beam

Horizontal member securely locked into the upright frame by means of pallet beam connectors and designed to support vertical loads and resist horizontal loads (see Figures 1.3.1 (a) and (b)).

1.3.28 Pallet beam connector

Device at the pallet beam ends, secured to upright frames by means of boltless or bolted connections and designed to transmit forces into the upright frames and provide stability within the racking structure (see Figure 1.3.1(b)).

1.3.29 Pallet beam deflection

Maximum vertical distance measured from the beam ends to the lowest point of the pallet beam in a loaded condition.

1.3.30 Pallet beam depth

Vertical distance from the top to the underside of the pallet beam (see Figure 1.3.2(b)).

1.3.31 Pallet beam length

Horizontal distance between the inner faces of columns in adjacent upright frames (see Figure 1.3.1(a)).

NOTE: This dimension is needed to conveniently manoeuvre pallets into a bay taking into account unit load width and minimum clearances required.

1.3.32 Pallet beam safety lock (device)

A positive locking device secured to the pallet beam connector to prevent dislodgement of the pallet beam from the upright frame when subjected to upward forces.

1.3.33 Pallet buffer backstop

A buffering backstop which is specified as an aid for use by forklift truck drivers to deposit a unit load in the correct position in the rack. The impact forces arising from the backstop's use as a buffer must be quantified in the specification if this type of aid is required.

1.3.34 Pallet rail (runner)

A horizontal load-bearing member used to support pallets, which normally runs parallel to the pallet placement direction. The rail may be fitted directly to the upright, a load arm or a beam (see Figure 1.3.1(c)).

1.3.35 Pallet safety backstop

A safety backstop used to prevent accidental damage. The two possible types of safety backstops are as follows:

(a) A safety device that protects against unintentional load movement within the racking and prevents unit loads from protruding or falling into an aisle or into an area accessible to people.

(b) A safety device, usually placed at the back of a storage compartment, to prevent accidental collision of a pallet or its load with other equipment, such as sprinklers.

1.3.36 Plan (horizontal) bracing

A horizontal bracing system which, when used with spine bracing, provides the stabilizing effect to the unbraced upright in front (see Figures 1.3.1(b) and (c)).

1.3.37 Rack (row) length

Maximum horizontal length of continuously connected bays in a racking structure, equal to the sum of upright widths plus bay widths (see Figure 1.3.2 (b)).

1.3.38 Spine (vertical or back) bracing

A bracing system providing the down-aisle stability for racking. The bracing is normally provided in the vertical plane at the rear of the racking (see Figures 1.3.1(b) and (c)).

1.3.39 Tolerance

Permissible positive or negative variation from nominal dimension or position, resulting from either manufacture or erection, or both.

1.3.40 Unit load

Individual stored item that can be placed or retrieved in one operation.

1.3.41 Unit load action

Laden individual pallets or equivalent load action modules.

1.3.42 Unit load upright clearance

Minimum horizontal distance from the inside face of the upright to the nearest part of the unit load in the down-aisle direction (see Figure 1.3.2(b)).

1.3.43 Unit load depth

Horizontal dimension of the unit load in the cross-aisle direction (see Figure 1.3.2(a)).

1.3.44 Unit load depth clearance

Minimum horizontal distance between adjacent unit loads in the cross-aisle direction of a double entry (double-sided) racking situation (see Figure 1.3.2(a)).

1.3.45 Unit load height

Maximum height measured from the underside of the pallet to the highest point of the unit load (see Figure 1.3.2(a)).

1.3.46 Unit load height clearance

Minimum vertical distance between the highest point of the unit load and the underside of the pallet beam immediately above (see Figure 1.3.2(b)).

1.3.47 Unit load overhang

Maximum horizontal distance the unit load protrudes beyond the outer edge of the pallet (see Figure 1.3.2(a)).

1.3.48 Unit load width

Horizontal dimension of the unit load measured parallel to the operating aisle (see Figure 1.3.2(b)).

1.3.49 Unit load width clearance

Minimum horizontal distance between adjacent unit loads on a common pallet beam in the down-aisle direction (see Figure 1.3.2(b)).

1.3.50 Upright

Vertical members that comprise the upright frame and are subject to predominantly compressive forces parallel to their longitudinal axes and have provision for systematic attachment of pallet beams.

1.3.51 Upright frame

Vertical frame assembly composed of uprights and bracings to support design loads transmitted through pallet beams and operating equipment.

1.3.52 Upright frame height

Maximum vertical height of an upright frame assembly including base plates and packing plates (when required) (see Figure 1.3.2(a)).

1.3.53 Upright protector

Component in front of an upright frame that is secured either to the floor or an upright, or both, and designed to resist minor impact loads as specified in Section 2 (see Figure 1.3.1(a)).

1.3.54 Upright splice

Vertical member used to splice two uprights together to form a composite column and designed to support vertical loads and resist horizontal loads (see Figure 1.3.1(a)).

1.3.55 Upright width

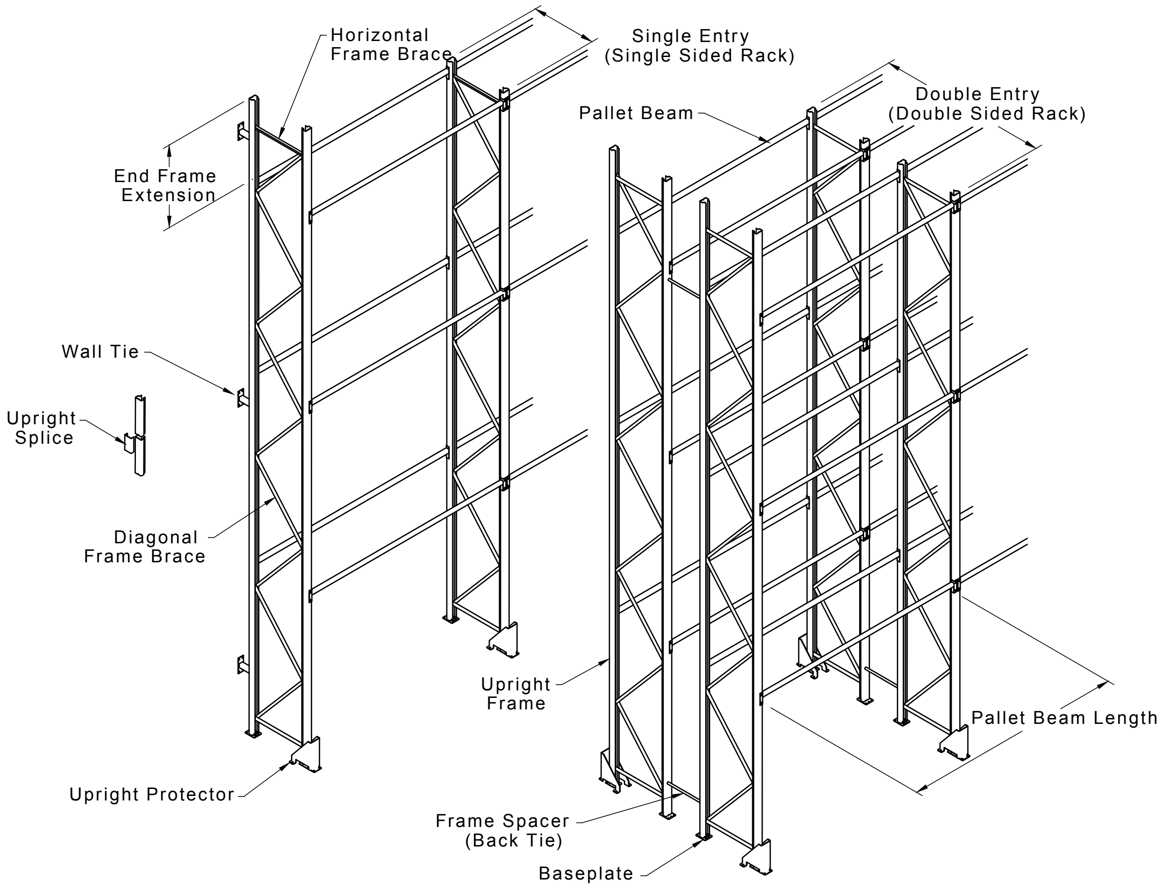
Maximum horizontal distance of an upright measured from flange to flange (outside).

1.3.56 Vertical clearance

Minimum vertical distance between the floor and the underside of the lowest pallet beam; or minimum vertical distance between the top of a lower pallet beam and the underside of an upper pallet beam (see Figure 1.3.2(b)).

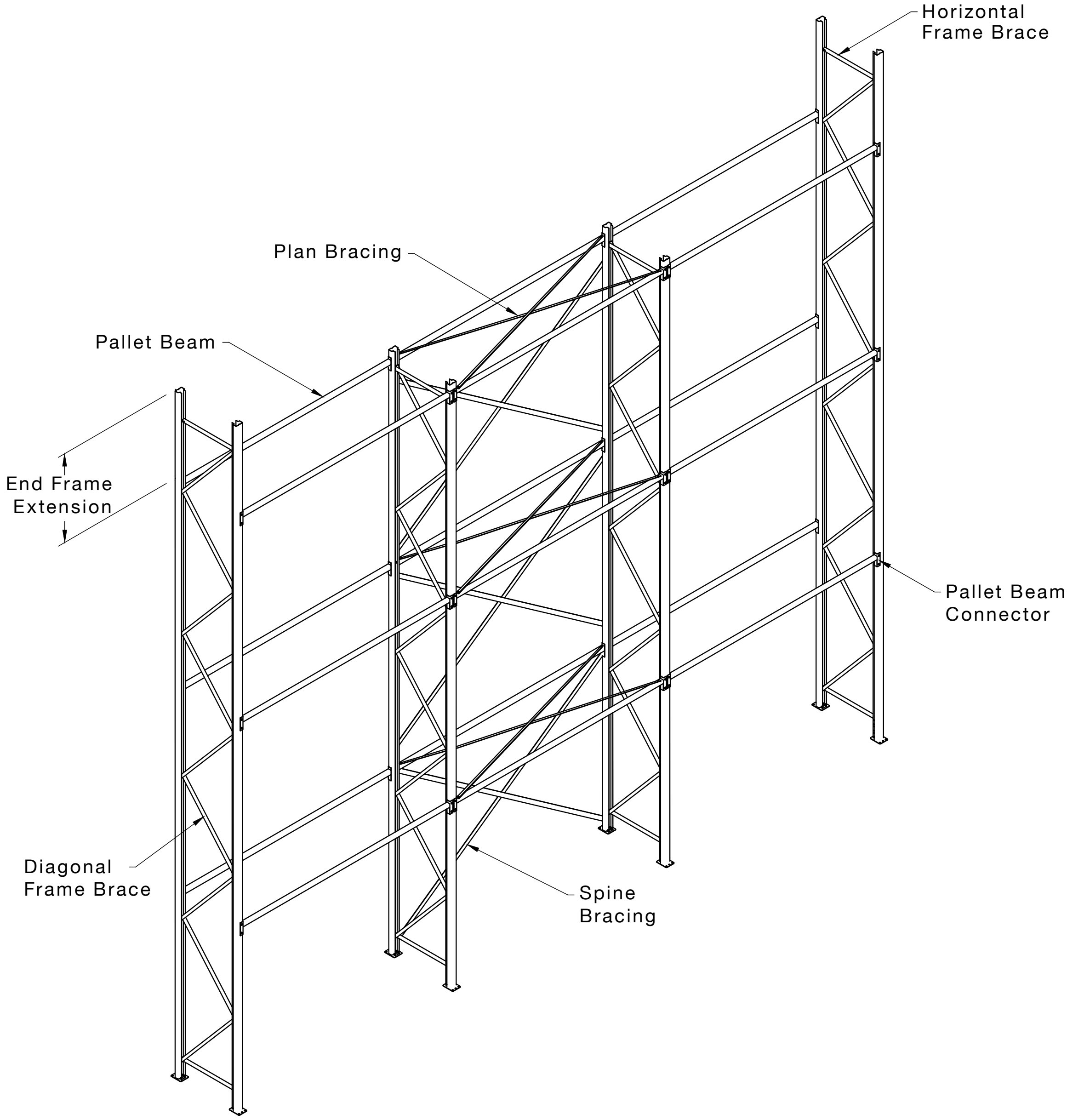
1.3.57 Wall tie (ceiling tie)

Horizontal or vertical member that connects the upright to a wall (or ceiling) to provide stabilizing forces and reduce overturning moments (see Figure 1.3.1(a)).



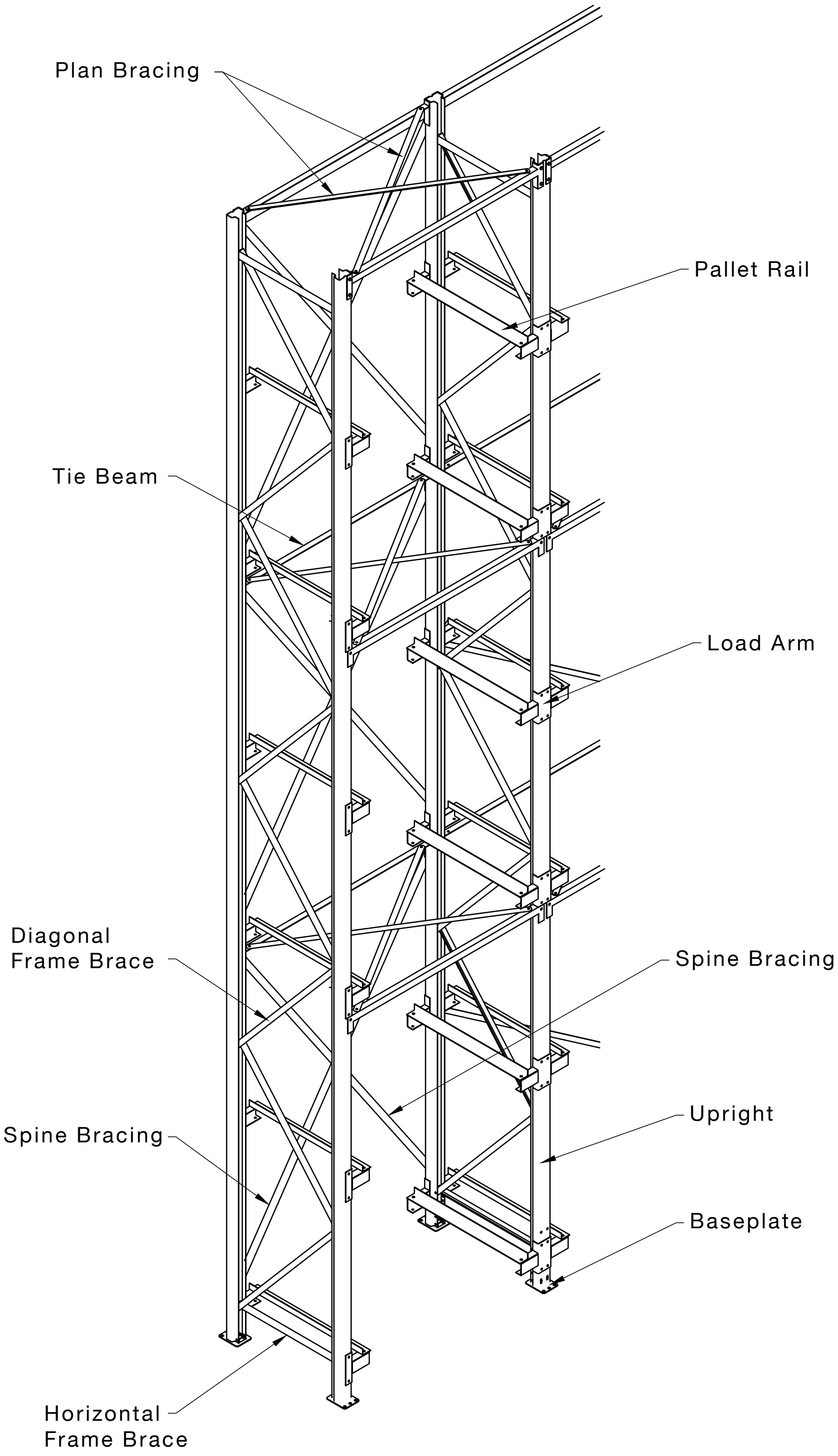
(a) Unbraced racking

FIGURE 1.3.1 (in part) ADJUSTABLE PALLET RACKING



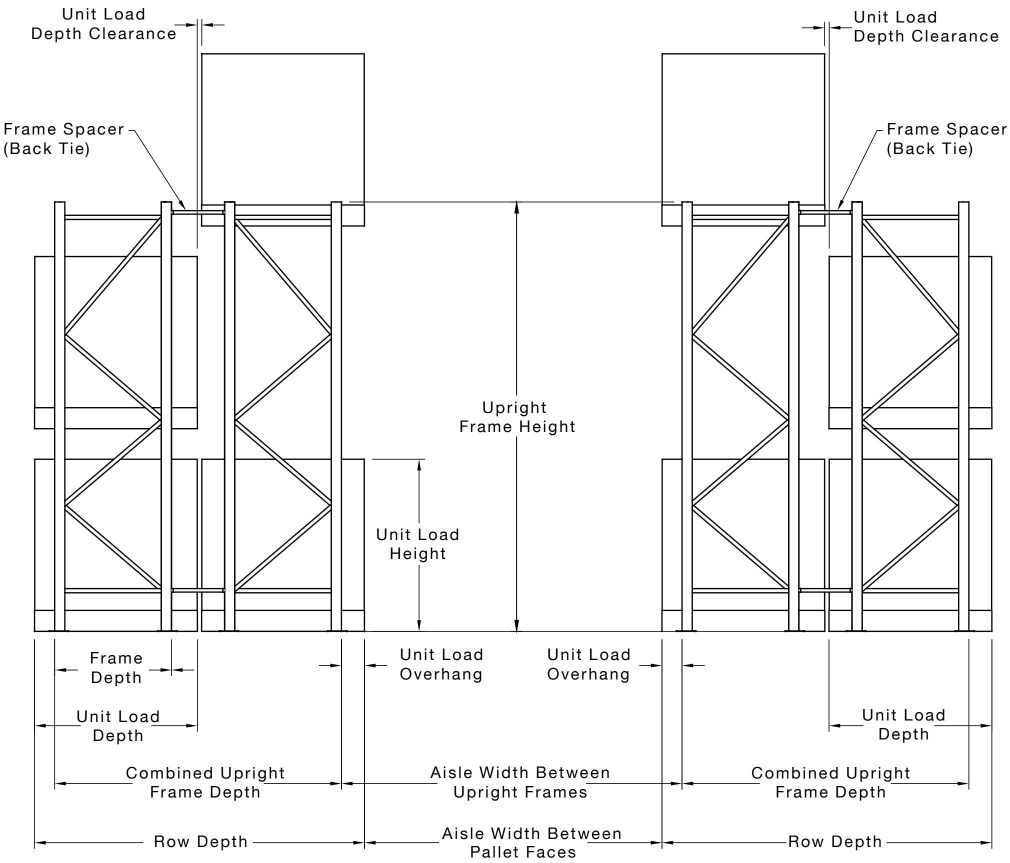
(b) Braced closed-face racking

FIGURE 1.3.1 (in part) ADJUSTABLE PALLET RACKING



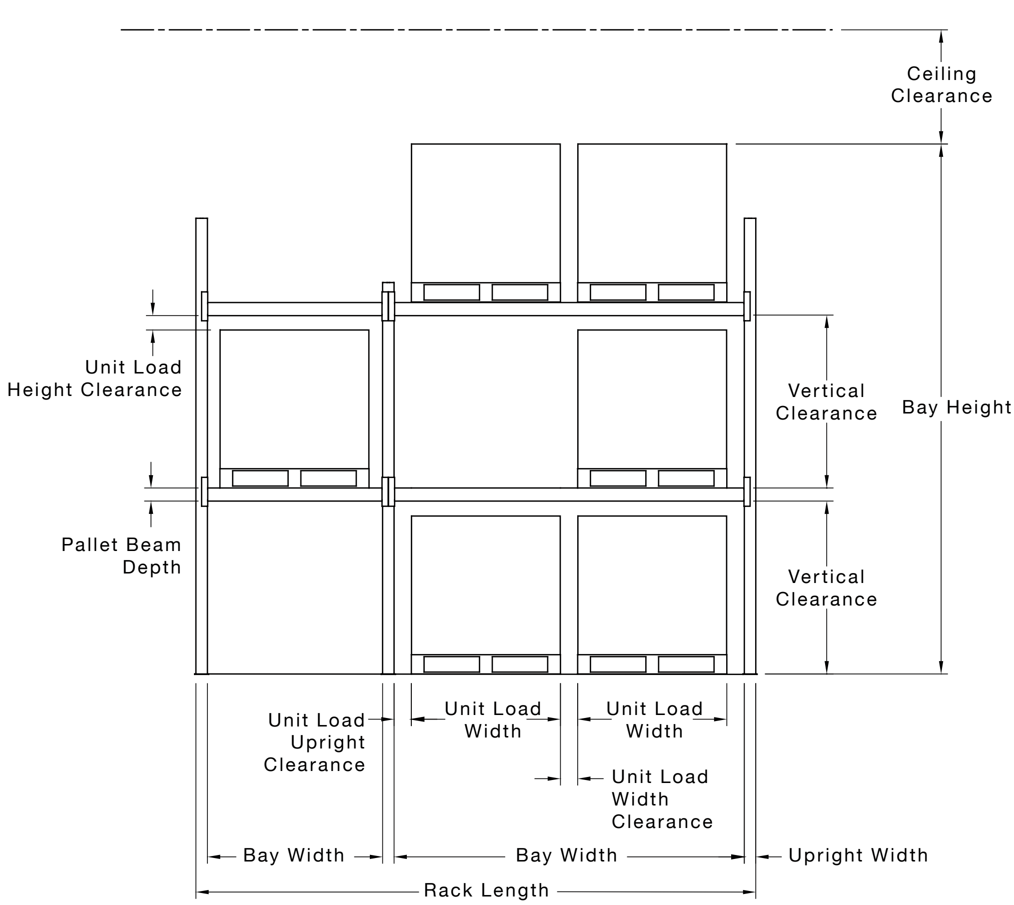
(c) Braced open-face racking

FIGURE 1.3.1 (in part) ADJUSTABLE PALLET RACKING



(a) Aisle width and related dimensions

FIGURE 1.3.2 (in part) DEFINITIONS



(b) Clearance width and related dimensions

FIGURE 1.3.2 (in part) DEFINITIONS

1.4 NOTATION

Symbols used in the text of this Standard are listed below.

The dimensional units for length and stress in all expressions or equations are to be taken as millimetres (mm) and megapascals (MPa), respectively, unless specified otherwise.

a = lever arm (pallet beam to column connection cantilever test)

or

distance between measuring transducers 2 and 3 and transducers 3 and 4 (upright splice test)

or

distance from the face of the upright to the applied action F (shear test on pallet beam to column connection)

A = value of accidental actions

A_e = effective area of the perforated cross-section at the stress f_n

A_{eff} = effective area of the base plate

$A_{net\ min}$ = minimum cross-sectional area obtained by passing a plane through the upright normal to the axis of the upright

A_{ph} = accidental horizontal action

A_{pv}	=	accidental vertical action
A_s	=	tensile stress area of the fixing of the base plate
A_{sp}	=	net cross-sectional area of splice
b	=	upright width
b_p	=	notional plane width of plate element
B_s	=	bi-moment section capacity
C	=	coefficient equal to f_y/f_t
		or
		numerical value between 2.5 and 3 (pallet beam test)
C_m	=	correction factor (pallet beam to upright connection cantilever test)
C_{mx}, C_{my}	=	coefficient for unequal end moment in accordance with AS/NZS 4600
d	=	distance between the centroidal axes of uprights (upright frame shear test)
d_1	=	distance from the anchor bolt in tension to the upright flange in compression (base plate design)
d_2	=	distance to the anchor bolt from the face of the flange of the upright of the base plate
d_3	=	flange width of upright
d_{ij}	=	distance between measuring transducers i and j (floor connection test)
d_u	=	upright depth
D	=	spacing of uprights in a frame (Figure C2)
e	=	eccentricity (Clause 3.3.2.6);
		or
		effective bearing width of the base plate (Clause 6.3.2.1)
E	=	modulus of elasticity (200×10^3 MPa)
EI^*	=	reduced flexural rigidity
f_c	=	stress value in the extreme compression fibre determined in accordance with AS/NZS 4600
f'_c	=	characteristic compressive cylinder strength of concrete at 28 days
f_{cb}	=	design bearing strength of the concrete slab
f_n	=	stress value determined in accordance with AS/NZS 4600 based on the full unreduced cross-section
f_{oc}	=	elastic buckling stress in an axially-loaded compression member
f_{od}	=	elastic distortional buckling stress
f_{ol}	=	elastic local buckling stress
f_{ox}, f_{oy}, f_{oz}	=	values of f_{oc} for flexural buckling about the x and y axes, respectively, and torsional buckling about the z axis
f_{ub}	=	specified ultimate tensile stress of the fixing material of the base plate
f_t	=	measured yield stress of a specimen
f_y	=	nominal yield stress

f_{ya} = weighted average yield stress

F = applied action

F_1, F_2 = applied axial and transverse actions, respectively

F_p = design value of the pull-out resistance of the floor fixing in the floor material used in the installation of the base plate

F_{sd} = maximum expected design action of the upright (upright splice test)

F_t^*, F_v^* = design tension and shear forces in the anchor bolt of the base plate

F_{ti} = maximum measured action for test number i (shear test on pallet beam to column connection)

g, g_1, g_2 = eccentricities (Clause 3.3.2.6)

G = value of permanent action (dead load)

h = total height of upright frame

or

distance to the topmost beam level (Figure 1.7.1)

or

height to the first beam level (Clause 3.3.4.3)

or

storey height (Figure 4.2.2.4)

or

minimum clear distance between connecting points (pallet beam to column connection cantilever test)

or

distance from the floor to the top of the pallet beam (portal frame test, Clause 7.5.2)

or

length of the frame (upright frame shear test *and* upright frame bending and shear test)

h_c = distance from the top of the half round to the top of the pallet beam (portal frame test, Claus 7.5.2)

h_i = distance to beam level i (Figure 1.7.1)

h_p = length of longest plane web element (Figure 4.2.2.4)

H = horizontal action per pallet beam (portal frame test)

H_i* = initial geometric imperfection horizontal action summed over all connected uprights (Clause 3.3.2.4)

I = second moment of area of members for the relevant axis of buckling

I* = reduced second moment of area of members when considering effects of geometric member imperfections

I_b = second moment of area of the pallet beam about the axis perpendicular to the plane of bending

I_c = second moment of area of the upright about the axis perpendicular to the plane of bending

I_w	=	cross-sectional torsion warping constant computed for the gross section
J	=	St. Venant torsion constant of the cross-section computed for the gross section
k	=	coefficient for stiffened and unstiffened elements
k_b	=	pallet beam stiffness (EI_b/L) (portal frame test)
k_c	=	upright stiffness (EI_c/h_c) (portal frame test)
kc_{ti}	=	slope of test curve number i (upright frame shear and bending test)
k_d	=	design value of connection stiffness
k_m	=	average value of connection stiffness
k_{ni}	=	slope of test curve number i as defined in Clause 7.2.2.2
k_o	=	*initial* slope of moment-rotation curve (pallet beam to upright connection cantilever test)
k_s	=	coefficient based on 95% fractile at a confidence level of 75%
k_{ti}	=	slope of test curve number i (upright frame shear test)
l	=	length between panel points along uprights
l_1	=	length of plateau at constant contact pressure of the base plate
l_2	=	distance shown in Figure 6.3.2.4 of the base plate
l_e	=	effective length for flexural buckling in the down-aisle direction; *or*
		effective length for flexural buckling in the cross-aisle direction
l_{ez}	=	torsional effective length
l_i	=	length between panel points of upright number i (Clause 3.3.2.4)
L	=	unsupported length of member
		or
		distance between the centroids of the two upright frames (portal frame test, Clause 7.5.2)
		or
		distance between pins (upright splice test)
		or
		twice the length of an upright section (floor connection test)
M^*	=	moment design action
		or
		amplified internal bending moment of the splice
M_b	=	average moment applied to the base plate (floor connection test)
M_c	=	design moment (pallet beam to upright connection cantilever test)
M^*_{ecc}	=	moment due to eccentricities of the splice
M_n	=	corrected moment (pallet beam to upright connection cantilever test)
M_o	=	elastic buckling moment
M_{sp}	=	moment of resistance of the splice

M_{sx}, M_{sy} = moment section capacity in accordance with AS 4100

M_t = measured moment (pallet beam to upright connection cantilever test)

M_x*, M_y* = moment design action about the x-axis and y-axis, respectively

n = number of tests results in a group

n_f = number of upright frames in one row of bays in the down-aisle direction;

or

number of upright frames connected together and acting together in the cross-aisle direction

N^* = design value of the vertical action on the frame; *or*

axial design action

N_c = compression capacity

N_{cr} = elastic critical (buckling) value of the vertical action

N_i^* = design axial action in member number *i*

N_s = axial section capacity according with AS 4100

Q = action on the beam where the pallet is placed in the perfect position (variable action);

or

form factor of a compression member including effect of perforations

Q' = design action on the beam where the pallet is placed with the maximum misalignment.

Q_1 = value of one of the variable actions

Q_e = theoretical action on the beam where the pallet is placed with the maximum misalignment

Q_h = maximum specified lateral support action per crane

Q_{ht} = total horizontal action at guide rail level

Q_i = value of typical variable action

Q_{max} = form factor Q for stub upright of thickness t_{max}

Q_{min} = form factor Q for stub upright of thickness t_{min}

Q_{ph} = horizontal placement action

Q_{pv} = vertical placement action

Q_u = unit load action

r_{o1} = polar radius of gyration of the gross cross-section about the shear centre

r_x, r_y = radius of gyration of the gross cross-section about the *x*-axis and y-axis, respectively

R_c = design load or moment based on test results

R_k = characteristic value of the parameter being measured

R_m = mean value of the adjusted test results

or

mean compressive strength of stub column test

R_{ni} = corrected failure load or moment for test number i

R_{ti} = measured failure load or moment for test number i

s = standard deviation of the adjusted test results of the parameter being measured

s_n = standard deviation of the normalized test results of the parameter being measured

S = span of pallet beams (pallet beam test)

Sc_{ti} = transverse bending and shear stiffness of the frame (upright frame bending and shear test)

S_{od} = imperfection multiplier for distortional buckling

S_{ol} = imperfection multiplier for local buckling

S_{ti} = transverse shear stiffness of the frame (upright frame shear test)

t = nominal steel thickness of any element or section exclusive of coatings

t_b = thickness of base plate

t_{max} = maximum thickness of stub upright in a series of tests with identical cross-section and hole dimensions and configurations

t_{min} = minimum thickness of stub upright in a series of tests with identical cross-section and hole dimensions and configurations

t_t = measured thickness for a specimen

Z_e = elastic section modulus of the net section for the extreme compression fibre times [0.5+(Q/2)], where Q is the form factor of a compression member, determined in accordance with Clause 7.3.1

Z_{eff} = elastic section modulus of the effective area about the axis of symmetry in the plane of the base plate

Z_f = elastic section modulus of the full unreduced cross-section for the extreme compression fibre

Z_c = elastic section modulus of the net section for the extreme compression fibre times $\left[1-\left(\frac{1-Q}{2}\right)\left(\frac{f_c}{f_y}\right)^Q\right]$, where Q is the form factor of a compression member, determined in accordance with Clause 7.3.1

$Z_{net\,min}$ = elastic section modulus of minimum cross-sectional area obtained by passing a plane through the upright normal to the axis of the upright

α = test correction coefficient

α_{nx}, α_{ny} = moment amplification factors about the x-axis and y-axis, respectively

β = test correction coefficient

β_x, β_y = monosymmetry section constant about the x-axis and y-axis, respectively

χ = buckling stress reduction factor

χ_{cu} = locus of mean values of test results χ_{ni}

χ_{ni} = buckling stress reduction factor for test number i

Δ = measured side-sway displacement of the frame at the top of the pallet beam (pallet beam test)

or

lateral displacement of the concrete block (floor connection test)

δ = horizontal deflection of the frame (upright frame shear test *and* upright frame bending and shear test)

δ_{ci} = deformation at the design load or moment (R_c) for test number i

δ_i = displacement of transducer number i

δ_o = scaled amplitude of the rack buckling mode

γ_A = partial factor for accidental actions

γ_G = partial factor for permanent actions

γ_{GA} = partial factor for permanent actions under accidental ultimate limit states

γ_Q = partial factor for variable actions

γ_{QA} = partial factor for variable actions under accidental ultimate limit states

η = design load amplification factor

λ = non-dimensional slenderness ratio

λ_{ni} = non-dimensional slenderness ratio for test number i

θ_b = average rotation of the base plate (floor connection test)

θ_{ni} = corrected value of the central rotation for test number i (pallet beam test)

θ_t = measured rotation of the pallet beam to column connection (pallet beam to upright connection cantilever test)

θ_{ti} = measured value of the central rotation in the serviceability limit state for test number i (pallet beam test)

θ_n = corrected rotation of the pallet beam to column connection (pallet beam to upright connection cantilever test)

θ_p = plastic rotation of the pallet beam to column connection (pallet beam to upright connection cantilever test)

ϕ = frame imperfection (out-of-plumb)

or

capacity factor

ϕ_o, ϕ_i, ϕ_{i-1} = initial imperfection angles of upright (Clause 3.3.2.4)

ϕ_ℓ = looseness of pallet beam to column connection

ϕ_s = maximum specified out-of-plumb divided by the height, which depends on the tolerance grade of the racking given in Table 1.7.1(a)

ϕ_t = capacity factor for tension

ϕ_v = capacity factor for shear

ρ = true load amplification factor, equal to the ratio of the action on the beam arising from pallets placed with maximum misalignment, to the corresponding action on the beam arising from pallets placed in the perfect position

ω_{max} = maximum value of sectorial coordinate

1.5 USE OF ALTERNATIVE MATERIALS OR METHODS

This Standard assumes the use of steel of structural quality as specified in AS/NZS 4600 for the design of cold-formed steel structural members and AS 4100 for the design of hot-rolled steel structural members. It shall not be interpreted so as to prevent the use of materials or methods of design or construction not specifically referred to in this Standard, provided that the requirements of Section 4 are complied with.

1.6 GENERAL REQUIREMENTS FOR RACKING INSTALLATIONS

A racking installation shall comply with all of the following:

(a) The racking installations shall have, in one or more conspicuous locations, a permanent, corrosion-resistant plaque not less than 125 mm long and 250 mm high with maximum load action figures, written in a large font of at least 25 mm high, mechanically secured to the racking structure at 2 m above the floor level, which shall display the following:

  (i) Permissible working unit load limit.

  (ii) Permissible total working unit load limit for each pallet beam level.

  (iii) Permissible total working unit load limit for each bay.

  (iv) Designer's name.

  (v) Racking manufacturer's name, supplier's name and trademark, and the installation date.

  (vi) The maximum distance from base plate level to the first beam level, and the maximum distance between first and second beam levels.

  NOTE: Permissible working action is to be determined as the unfactored working action (i.e. actions less action factors), inclusive of pallet weight.

(b) Load application, racking configuration drawings and specifications shall be furnished with each racking installation. A notice shall be included on the drawings that deviations from the drawings may impair the safety of the racking installation.

(c) If the racking is required to be used in more than one configuration, structural drawings shall include each required configuration.

(d) If the maximum extent of damage assumed in the design differs from that shown in Figure 8.3.2 then values for maximum damage shall be specified in the drawings and specification.

1.7 TOLERANCES AND CLEARANCES

1.7.1 Finished tolerances in unloaded condition

The finished tolerances of pallet racking in the unloaded condition shall be as given in Table 1.7.1(a) for the tolerance grades given in Table 1.7.1(b).

NOTE: Tolerance grades are governed by the type of unit load handling equipment used for various applications, ranging from common manually operated racking to an Automated Storage and Retrieval System (ASRS). Reference should be made to EN 15620 or similar for tolerances and clearances required for specific storage and retrieval equipment (e.g. manual control, semi-automatic or fully automatic with coordinate positioning, and with or without compartment precision positioning).

While this standard is not centric to Australian Standard pallets, additional guidance on clearances and tolerances applicable to non-standard or 'international' pallets can be obtained from EN 15620.

TABLE 1.7.1(a)

FINISHED TOLERANCES

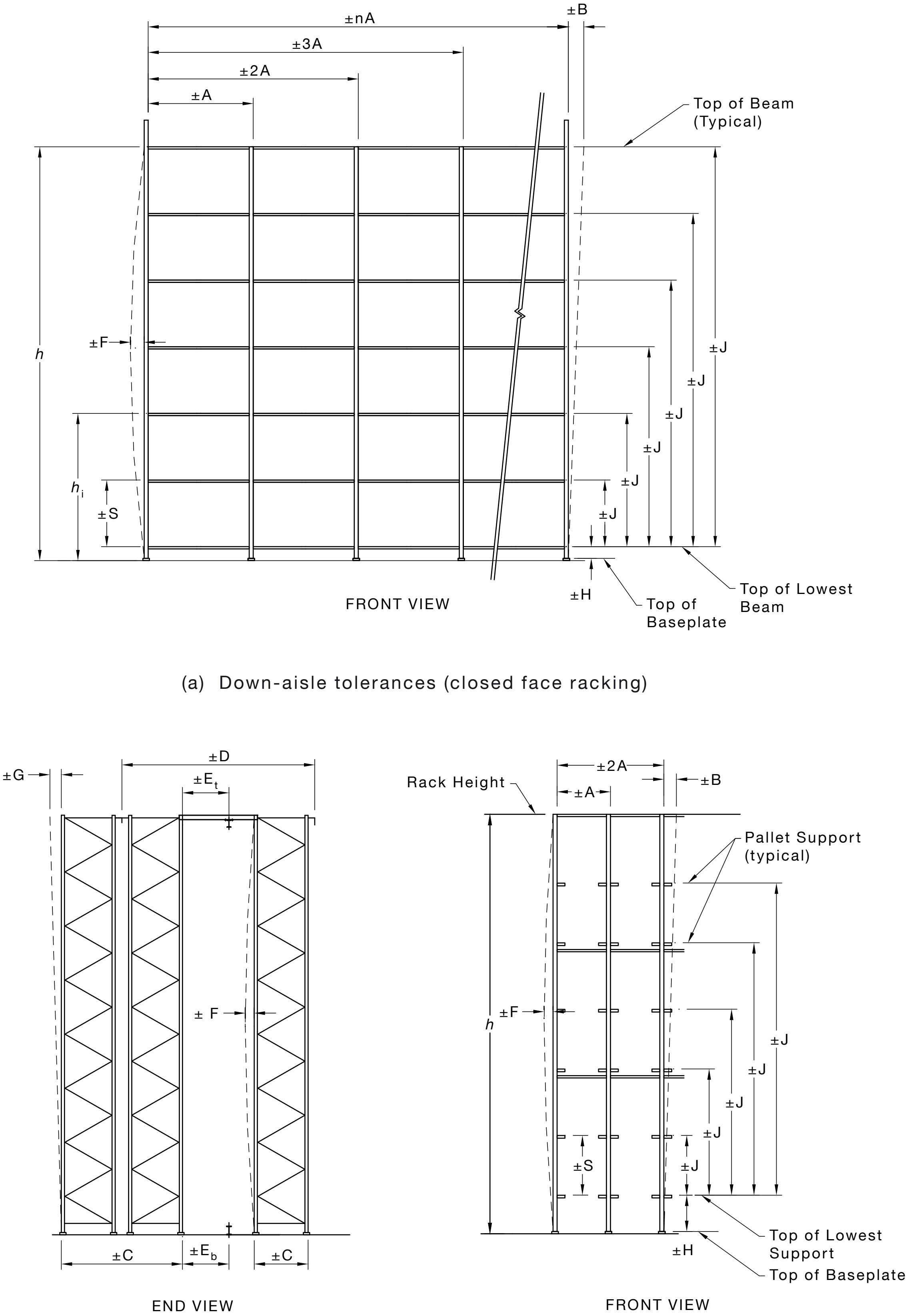
Type of tolerance (see Figure 1.7.1)	Description	Tolerance, mm		
		Tolerance, grade		
		I	II	III
A	Maximum variation in individual bays	±3	±3	±3
nA	Total cumulative deviation in racking length (nA = total deviation, where n is the number of bays)	±30	±20	±10
B	Maximum installation out-of-plumb of upright perpendicular to the plane of the upright frames (down-aisle)	$h/500$	$h/750$	$h/1000$
C	Racking depth (single or multiple frames)	±5	±5	±5
D	String depth	±5	±5	±5
E_b	Rail positioning with regard to the pallet racking measured as the difference between the values at the bottom of the upright frames or clear aisle for Grades I and II	±5	±5	see FEM 9.831 and FEM 9.832
E_t	Rail positioning with regard to the pallet racking measured as the difference between the values at the top of the upright frames or clear aisle for Grades I and II	±5	±5	see FEM 9.831 and 9.832
F	Maximum imperfection of upright with regard to the theoretical longitudinal upright x or y axis	$h/1000$	$h/1000$	$h/1000$
G	Maximum installation out-of-plumb of upright in the plane of the upright frame (cross-aisle)	$h/500$	$h/750$	$h/1000$
H	Distance between top of base plate and top of lowest beam level	±10	±7	±5
J	Maximum deviation of beam level or portal level with regard to the lowest beam level	$\pm h_i/500$ or ±5	$\pm h_i/750$ or ±4	$\pm h_i/1000$
S	Maximum deviation of adjacent vertical beam spacing	±5	±3	±2

NOTE: In Table 1.7.1(a), h is the distance to the topmost beam level and h_i is the distance to beam level i, see Fig. 1.7.1.

TABLE 1.7.1(b)

TOLERANCE GRADES

Tolerance grade	Type of unit load handling equipment
I	Manually operated equipment guided by operator (e.g. wide and narrow aisle racking)
II	Manually operated equipment guided by electrical or mechanical devices (e.g. very narrow aisle racking)
III	Fully automatically operated equipment guided by electrical or mechanical devices (e.g. ASRS racking)



(a) Down-aisle tolerances (closed face racking)

(b) Cross-aisle tolerances

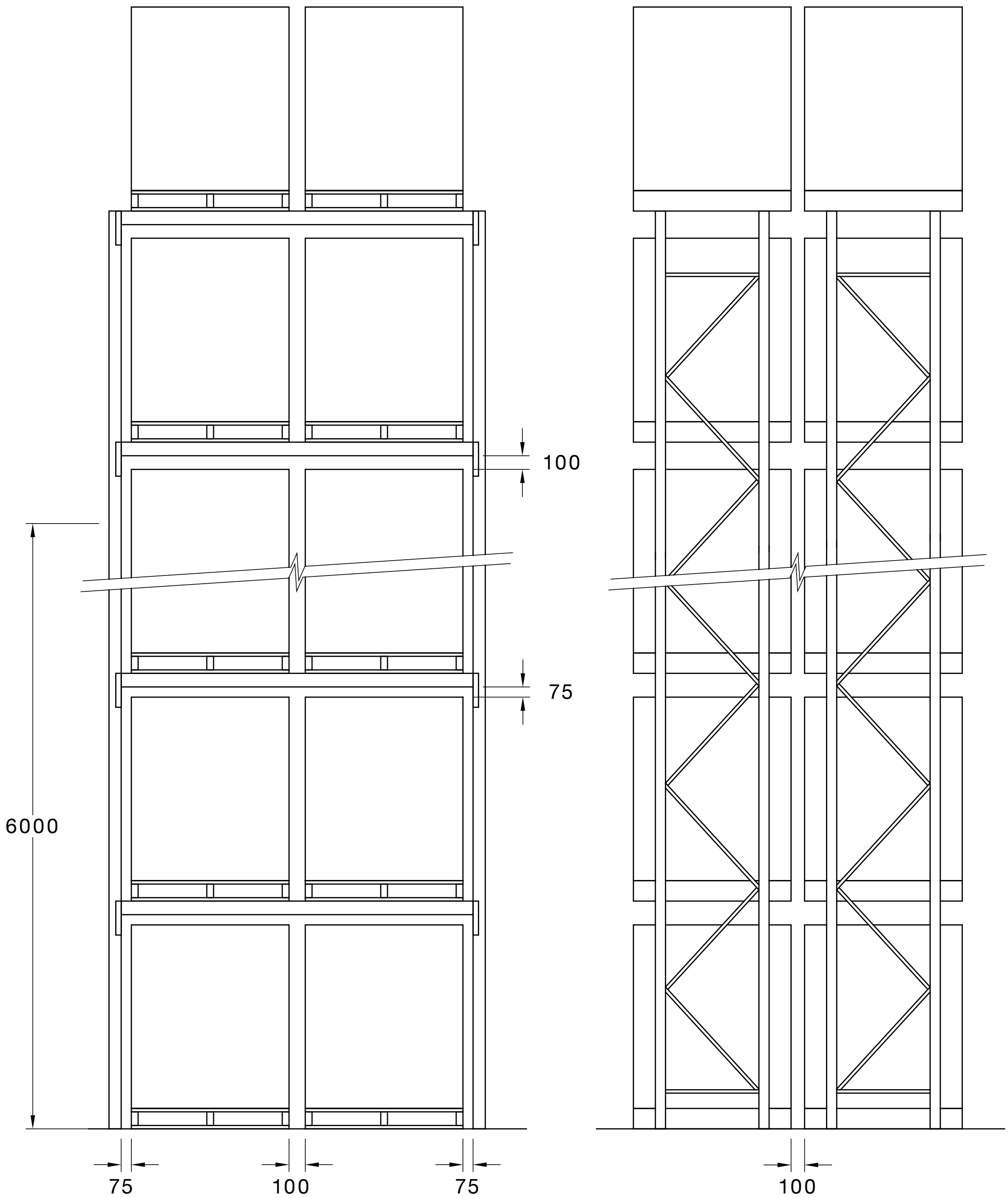
(c) Down-aisle tolerances (open face racking)

FIGURE 1.7.1 TYPES OF TOLERANCES

1.7.2 Unit load operating clearances

Unit load operating clearances shall be as shown in Figure 1.7.2.

NOTE: Clearance (vertically and horizontally) may be reduced in installations utilising automatic unit load handling equipment or fitted with visibility aid devices. The unit load height clearance should be increased to a minimum of 100 mm for beam heights of 6000 mm or greater, and where the equipment operator position is at ground level. For clearances of various types of material handling equipment, see EN 15620 and EN 15629.



DIMENSIONS IN MILLIMETRES

FIGURE 1.7.2 UNIT LOAD OPERATING CLEARANCES

SECTION 2 ACTIONS

2.1 GENERAL

Racking shall be designed for the combinations of permanent, variable and accidental actions in accordance with this Standard and AS/NZS 1170.1.

The influence of the various types of geometric imperfections on the design actions shall be considered.

The term action in this Standard shall be as specified in AS/NZS 1170.0.

NOTE: The actual means by which imperfection effects are incorporated depends on the method of structural analysis adopted.

2.2 PERMANENT ACTIONS

The permanent actions (dead loads) shall comprise the weight of all the construction including walls, floors, ceilings, stairways and fixed service equipment.

Permanent actions shall be determined in accordance with AS/NZS 1170.1.

When estimating permanent actions for the purposes of design, the actual weights of materials and constructions shall be used. The weight of fixed service equipment, such as sprinklers, electrical feeders, and heating, ventilation and air conditioning systems, shall be determined and taken into account whenever such equipment is supported by the structural members of the rack.

2.3 VARIABLE ACTIONS

2.3.1 General

Where applicable, the design shall take into account actions from the following variable actions:

(a) Unit load actions (stored materials).

(b) Vertical and horizontal placement actions.

(c) Actions from rack guided equipment.

(d) Floor and walkway actions.

(e) Thrusts on handrails.

(f) Actions from thermal effects.

(g) Impact and accidental actions.

(h) Actions from geometrical and action imperfections.

(i) Actions from foundation settlement.

Variable actions arising from other equipment connected to the structure shall be determined and taken into account in the design.

2.3.2 Unit loads to be stored

The global analysis and design shall be carried out on the basis that the racking is loaded with unit load actions of maximum weight in each storage location. In this case, the racking shall be designed for a bay action which is equal to the sum of the maximum weight unit load actions stored in that bay.

If specifically agreed to by the end user and documented, the racking may be designed on the basis of a specified bay action which is less than the sum of maximum weight unit loads stored in that bay. In this case, the following conditions shall apply:

(a) The racking shall be assumed to be uniformly loaded in every bay.

(b) The management system in the warehouse must be able to reliably identify unit loads in excess of the specified value and control their distribution within the racking.

(c) Individual storage compartments are designed to carry the maximum weight of unit load.

(d) In upright design, the worst case load distribution shall be considered where maximum weight of unit load action shall be applied to the upper storage positions up to the specified bay action.

(e) The bay action used in the global analysis and design is never exceeded.

The increase in stress and deformation associated with random imperfect placement of unit load actions in the storage compartment may be ignored if it is less than or equal to 12% compared to a storage compartment loaded with perfectly placed unit loads. When the increase is more than 12%, the effect on the beam design shall be taken into account as follows:

$$Q' = \eta Q \quad \text{. . . 2.3.1}$$

where

Q' = design action on the beam where the pallet is placed with the maximum misalignment

η = design load amplification factor

= 1.0 if $\rho \leq 1.12$

= $2\rho - 1.24$ if $1.12 \leq \rho \leq 1.24$

= ρ if $\rho \geq 1.24$

ρ = true load amplification factor, equal to the ratio of the action on the beam arising from pallets placed with maximum misalignment, to the corresponding action on the beam arising from pallets placed in the perfect position

= Q_e/Q

Q_e = theoretical action on the beam where the pallet is placed with the maximum misalignment

Q = action on the beam where the pallet is placed in the perfect position.

2.3.3 Vertical placement actions

2.3.3.1 *General*

The minimum vertical placement actions specified in Clauses 2.3.3.2 and 2.3.3.3 shall be applied in applications where unit loads are placed in position by mechanical equipment or by hand, i.e. tolerance grade I and II as per Table 1.7.1 (b).

2.3.3.2 *Goods placed with mechanical equipment*

Beams, supporting arms (if any), and end connections shall be designed for an additional vertical placement action (Q_{pv}), equal to 25% of one unit load action, placed in the most unfavourable position for the particular determination (moment or shear force). When corresponding design capacities are determined by test (see Clause 7.4), due allowance shall be made for the additional placement load as specified in this Section.

No placement actions need be applied when checking beam deflections (see Clause 4.2.5).

2.3.3.3 *Goods placed by hand*

Beams, supporting arms (if any), and end connections shall be designed for an additional vertical placement action (Q_{pv}), equal to 100% of one unit load, placed in the most unfavourable position for the particular determination (moment or shear force).

2.3.4 Horizontal placement actions

2.3.4.1 *General*

In applications where unit loads are placed in position, the following minimum horizontal placement actions (variable actions) shall be applied in both the down-aisle and cross-aisle directions, as specified in Clauses 2.3.4.3 and 2.3.4.4 respectively.

These horizontal forces shall be applied separately, not simultaneously, in each principal direction of the racking.

NOTE: The minimum horizontal placement action is not intended to represent an impact action arising from misuse of the racking.

An accidental action in the horizontal direction (see Clause 2.4.3) shall be taken into consideration but need not be considered at the same time as the horizontal placement action.

2.3.4.2 *Effects of operational methods*

The following operational methods shall be used to determine the horizontal placement action (Q_{ph}):

(a) Where goods are placed with manually operated mechanical equipment (e.g. forklift trucks):

 (i) For racks up to 3 m in height, Q_{ph} shall be an action of 0.5 kN applied at any height up to the top of the rack.

 (ii) For racks over 6 m in height, Q_{ph} shall be an action of 0.25 kN applied at the top of the rack, or an action of 0.5 kN applied at any height up to 3 m.

 (iii) For racks between 3 m and 6 m, Q_{ph} shall be the worst case of an action at the top of the rack whose magnitude is determined by linear interpolation between (i) and (ii), or the action of 0.5 kN applied at any height up to 3 m.

(b) Where goods are placed by automatic storage and retrieval equipment, Q_{ph} and its position shall be specified by the materials handling equipment supplier. However, the minimum value of Q_{ph} shall be 0.25 kN.

(c) Where backstops are used, it shall be clearly defined whether these are safety backstops or buffering backstops, and the Q_{ph} shall be defined by consultation between the user and the racking designer. For manually operated mechanical equipment, Q_{ph} shall be subject to a minimum value of 0.25 Q_u where Q_u is the unit load action.

 Buffer backstops and pick and deposit stations with positioning devices shall be considered to give rise to variable actions whereas safety backstops shall be considered to give rise to accidental actions. The actions arising from both of these shall be used with the relevant action factors.

(d) Q_{ph} as specified in Item (c) shall be taken into account when designing the following racking components in the direct vicinity of the backstop:

 (i) The backstop device itself.

 (ii) The connection of the backstop with the racking component concerned (beam or upright).

(iii) The part of the upright to which the backstop or the beam supporting the backstop is directly connected.

(iv) The upright frame bracing in the direct vicinity of this upright part.

Due to damping and spreading effects, a reduced Q_{ph} may be considered as follows:

(A) $Q_{ph} = 0.1Q_u$ for frame anchoring design based on the upright being unloaded with Q_{ph} acting on one frame in the topmost position.

(B) $Q_{ph} = 0.1Q_u$ for overall frame design (bracing and upright) and Q_{ph} acting on one frame on the topmost pallet position.

(e) If the goods are hand loaded: $Q_{ph} = 0.25$ kN.

2.3.4.3 *Application of the horizontal placement action in the down-aisle direction*

In the down-aisle direction, the horizontal placement action occurs at the beam levels and amplifies the down-aisle sway caused by frame imperfections.

2.3.4.4 *Application of the horizontal placement action in the cross-aisle direction*

In the cross-aisle direction, the most unfavourable location for the placement action shall be one of the following:

(a) The top of the uppermost beam level in order to maximize the forces in the bracing system.

(b) Midway between two bracing nodes of the upright frame lattice in order to maximize the cross-aisle bending moment.

NOTE: In order to determine the design bending moments, a global analysis of the complete upright frame need not be carried out. It is sufficient to add positive and negative bending moments of magnitude $Q_{ph}l/6$, where l is the length between panel points along uprights.

(c) The mid-span of a pallet beam in the horizontal plane in order to provide the maximum minor axis bending moment. This case need not be incorporated in the global cross-aisle analysis and an action of 0.5 Q_{ph} shall be considered to be carried by a single beam in the horizontal plane through the neutral axis. It shall be permissible to ignore the interaction with the vertical action causing Q_{ph}.

If the installation is designed for unit actions to be rolled or slid into position, the placement actions (Q_{ph}) generated by the loading process shall be determined and used in the design.

2.3.5 Actions from rack-guided equipment

In racking operated by rack-guided cranes, the probability of all cranes imposing horizontal actions in the same direction and at the same position in the racking simultaneously, as shown in Figure 2.3.5, decreases as the number of cranes increases. Therefore, if the upright frames are joined over the aisles, the total horizontal force (Q_{ht}) at guide rail level shall be the value given in Table 2.3.5.

TABLE 2.3.5

TOTAL HORIZONTAL ACTIONS AT GUIDE RAIL LEVEL

Number of cranes	Total horizontal force, Q_{ht}
1, 2	$\sum Q_h$
3	$0.85\sum Q_h$
4	$0.70\sum Q_h$
≥5	$3.0Q_h$

where

Q_h = maximum specified lateral support action per crane

Q_{ht} = reduced sum of lateral forces acting at the crane top guide rail, which is connected to a member joining all the upright frames together, as shown in Figure 2.3.5.

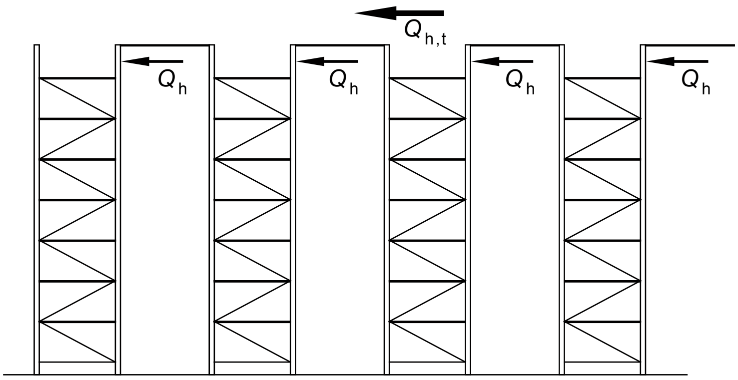


FIGURE 2.3.5 HORIZONTAL FORCES FROM RACK-GUIDED EQUIPMENT

If the horizontal force (Q_h) is specified as a result of an eccentric one-sided force applied to the crane rail, then the values given in Table 2.3.5 shall not be used.

The horizontal force from rack-guided equipment shall be combined with the placement action if this constitutes the worst case.

Consideration shall be given to accidental impact forces in the down-aisle direction, if any, arising from a crane hitting a rack-mounted buffer.

2.3.6 Floor and walkway actions

Uniformly distributed and concentrated live (variable) action requirements shall be determined from AS/NZS 1170.1 and AS 1657, as appropriate. If the loads arising from stored materials or racking systems supported by the floor exceed the values stated in AS/NZS 1170.1 and AS 1657, as appropriate, the actual values shall be used. Particular attention shall be given to the concentrated loads applied by upright frames.

Where moving equipment applies dynamic forces to the structure, these effects shall be taken into account as quasi-static actions. In order to distinguish between the effective vertical wheel actions of the different types of trucks, the relevant static forces shall be multiplied by the dynamic factors given in Table 2.3.6.

TABLE 2.3.6

DYNAMIC FACTOR

Truck type	Dynamic factor
Pedestrian controlled truck with velocity less that 5.0 km/h	1.2
Ride-on truck with a velocity less than 7.5 km/h	1.4
Ride-on truck with a velocity less that 10.0 km/h	2.0

When repeated cycles of rolling loads are involved, due consideration shall be given to fatigue design.

Due consideration shall be given to the effects of horizontal forces applied by the equipment and how these forces are to be accommodated by the structure.

Particular attention shall be given to concentrated actions applied by the bases of upright frames which are supported directly by a flooring system.

2.3.7 Actions arising from installation or repair

When the installation method requires the installers to use safety harnesses, suitable anchorage points capable of arresting an accidental fall shall be provided.

NOTE: Residual deformation of the members may occur in the event of a fall.

2.4 ACCIDENTAL ACTIONS

2.4.1 General

Accidental actions are defined as impact actions which are more severe than those which occur under ordinary 'good practice' operating conditions.

Provision shall be made in the structural design for uses and actions that involve unusual vibration or dynamic forces.

Impact damage caused by fork-lift trucks or other moving equipment against racking uprights shall be avoided by appropriate operator training and safety measures. The minimum requirements for the protection of corner uprights shall be as follows:

(a) An upright protector with a height of not less than 400 mm shall be positioned at the end upright of each run of racking between cross-aisles.

(b) An upright protector shall be positioned at all those uprights positioned at aisle and gangway intersections.

(c) The upright protector shall be designed for an energy absorption of not less than 400 Nm in any direction at any height between 0.1 m and 0.4 m.

(d) The upright protector shall be positioned in such a way that, after its deformation by absorbing an impact, the upright will not be damaged.

NOTES:

1 Uprights other than corner uprights may be protected in a direction normal to the aisle, depending on the preference of the user.

2 The protection of uprights of racking served by mechanically guided handling equipment may not be necessary.

3 As an alternative to the use of upright protectors, the installation may be designed to survive the complete removal of a section at the bottom of an upright.

2.4.2 Accidental vertical actions

Racking components directly above a unit load action shall be able to absorb an accidental vertical force (A_{pv}). In general, this force shall be applied at the end of a beam in order to verify that the connector does not disengage. Upward placement actions are accidental variable actions and shall be considered with an action factor (γ_A) (given in Clause 2.7.4) as follows:

(a) If goods are placed with manually operated equipment (e.g. fork-lift trucks):

$A_{pv} = 5.0$ kN

(b) If goods are placed with automatic mechanical equipment (e.g. storage and retrieval machines):

$A_{pv} = 0.5Q_u$

but $A_{pv} \geq 0.25$ kN and $A_{pv} \leq 5.0$ kN

where

Q_u = weight of unit load action.

2.4.3 Accidental horizontal actions

An accidental overload in the horizontal direction shall be taken into consideration as follows:

(a) For tolerance grade I racking (see Table 1.7.1(b)):

From floor to 1.0 m height on the aisle side upright:

$A_{ph} = 2.5$ kN in the cross-aisle front to back direction

$A_{ph} = 1.25$ kN in the down-aisle direction.

These actions shall be treated as occurring separately, in any position between the floor and a height of 1 m. The accidental actions may be taken by the upright itself or it may require that each upright shall be reinforced or protected.

NOTE: Uprights do not need to be designed for accidental horizontal actions provided upright protectors are installed up to a height of 1 m, or shorter if it can be shown that shorter upright protectors prevent direct upright impact by forklift trucks. Upright protectors shall be designed for the full accidental horizontal actions.

(b) For tolerance grades II and III racking (see Table 1.7.1(b)):

$A_{ph} = 0.5$ kN at any position in either the down-aisle or cross-aisle direction.

NOTES:

1 The specified value of A_{ph} may not be acceptable for certain types of equipment. The machinery to be used should be checked and confirmed (i.e. during a malfunction the action may increase until the clutch on the delivery device slips).

2 If upright protectors are installed, they shall be designed to support the specified value of accidental horizontal A_{ph}.

2.5 WIND ACTIONS

Where relevant, wind actions shall be determined in accordance with AS/NZS 1170.2.

When racking is exposed to wind, no account shall be taken of the shielding effects of fully or partially loaded racking upwind of the run in consideration. Each individual row of racking (between adjacent aisles) shall be designed to resist the full wind pressure, suction and wind friction forces.

For a clad rack installation, the assembled rows need not be capable of resisting more than the wind force calculated for the complete assembly.

The fully or partially loaded racking shall be considered to be impermeable to wind in the loaded areas unless the effect of permeability can be quantified.

NOTE: When determining the limiting deflections for the proper functioning of automatic equipment, a lower value of wind load may be used when specified by the supplier of the equipment.

2.6 SEISMIC ACTIONS

Provision shall be made for earthquake effects and associated lateral forces determined in accordance with AS 1170.4. For each such installation, the storage racking shall be designed, manufactured, and installed in accordance with such provisions.

NOTE: Guidance on the appropriate seismic mass, limit states load combinations, analysis procedures, detailing provisions and installation requirements for racking structures (as distinct from building structures which is the prime focus of AS 1170.4) can be obtained from the RMI Specification or FEM 10.2.08.

2.7 ACTION COMBINATIONS

2.7.1 General

The weight of unit loads and global frame imperfections shall together constitute a single action. Placement actions shall constitute a separate action.

Global imperfections and placement actions shall be combined in one direction at a time. The combination of imperfections or placement actions in one direction with imperfections or placement actions in the other orthogonal direction need not be considered.

2.7.2 Combination of actions for ultimate limit states

The design values of actions shall be combined using whichever of the following gives the larger value:

(a) Considering only the most unfavourable variable action:

$$\sum \gamma_G G + \gamma_Q Q_1$$

(b) Considering all unfavourable variable actions that may occur simultaneously:

$$\sum \gamma_G G + 0.9 \sum_i \gamma_Q Q_i$$

(c) Design for accidental actions:

$$\sum \gamma_{GA} G + \sum_i \gamma_{QA} Q_i + \gamma_A A$$

where

γ_G = partial factor for permanent actions (see Clause 2.7.4)

G = value of permanent action (dead load), (see Clause 2.2)

γ_Q = partial factor for variable actions (see Clause 2.7.4)

Q_1 = value of one of the variable actions, (see Clause 2.3)

Q_i = value of typical variable action, (see Clause 2.3)

γ_{QA} = partial factor for variable actions under accidental ultimate limit states (see Clause 2.7.4)

γ_{GA} = partial factor for permanent actions under accidental ultimate limit states (see Clause 2.7.4)

γ_A = partial factor for accidental actions (see Clause 2.7.4)

A = value of accidental actions, (see Clause 2.4)

Action combinations for wind and seismic actions shall be determined in accordance with AS/NZS 1170.0.

2.7.2.1 *Analysis in the down-aisle direction*

In considering imposed actions from stored materials, the worst load (action) pattern shall be considered for each of the following criteria:

(a) Overall stability in the down-aisle direction.

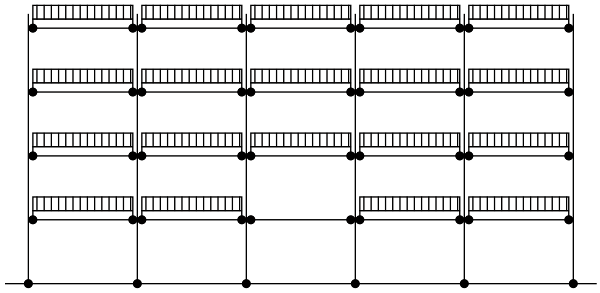
(b) Bending and buckling of the uprights.

(c) Beam deflections and mid-span bending moments.

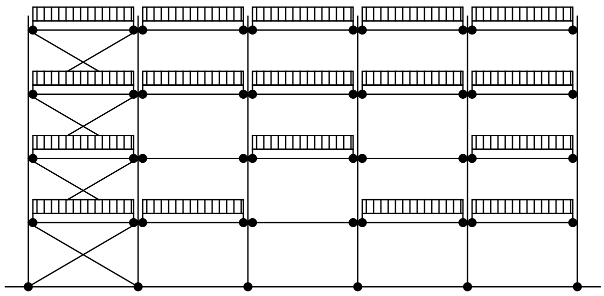
(d) Moments in beam to upright connectors.

For overall stability in the down-aisle direction, the fully loaded structure with the actions arising from imperfections as specified in Clause 3.3.2 shall be considered.

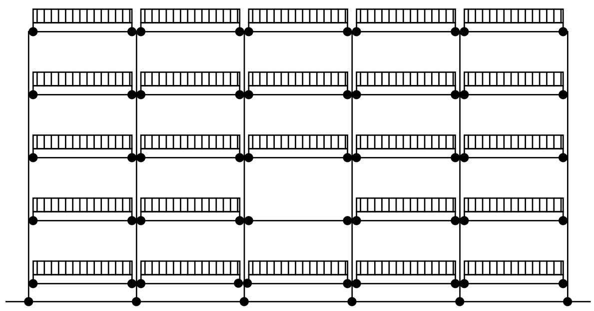
For the design of uprights, the structure to be fully loaded shall be considered with the exception of a single unloaded beam near the middle of the structure at the lowest level, as shown in Figure 2.7.2.1(a). In braced racking, an alternative action pattern giving rise to single curvature in the uprights shall also be considered, as shown in Figure 2.7.2.1(b).



a) Typical loading pattern for an unbraced rack



b) Typical loading pattern for a braced rack



c) Additional loading pattern when the lowest beam is near the ground

FIGURE 2.7.2.1 TYPICAL PATTERN LOAD FOR DOWN-AISLE DIRECTION

Pattern load may induce a reversal of bending moment in the beam to upright connections. When this occurs, consideration shall be given to the stiffness and strength of the connector in the reverse direction. Connectors shall have a minimum reverse capacity of 50% of the capacity in the usual direction of action. Unless a larger capacity is justified by tests, the reverse capacity assumed in the design shall therefore be restricted to 50% of the capacity in the usual direction of the action.

NOTES:

1 If the lowest beam is near the ground, e.g. the centre-line of the lowest beam is 300 mm or less above the ground, it may be more critical to omit the action from a single beam at the second level and the case shown in Figure 2.7.2.1(c) should also be considered.

2 This down-aisle analysis gives rise to primary axial actions and down-aisle bending moments in the uprights.

2.7.2.2 *Analysis in the cross-aisle direction*

Analysis of cross-aisle action combinations shall incorporate cross-aisle frame imperfections.

Actions due to handling equipment normally arise horizontally in the cross-aisle direction.

2.7.3 Combination of actions for serviceability limit states

The design values of actions shall be combined using whichever of the following gives the larger value:

(a) Considering only the most unfavourable variable action:

$$\sum \gamma_G G + \gamma_Q Q_1$$

(b) Considering all unfavourable variable actions which may occur simultaneously:

$$\sum \gamma_G G + 0.9 \sum_i \gamma_Q Q_i$$

Placement and accidental actions need not be considered at the serviceability limit state.

NOTES:

1 The weight of the unit load action varies up to a maximum, which is used in design. The specifier should define a design value for the weight of the unit load action (or different design values of the weight of the unit load action for the upright frame and/or down-aisle global design), which will not be exceeded. This may result in the specification of design weights which are conservative, thus justifying the 0.9 combination factor for action combinations.

2 Unless unusual conditions prevail, it is usual that the goods to be stored plus the global imperfections constitute the action with the largest effect.

2.7.4 Action factors

The action factors (γ) shall be as given in Table 2.7.4.

NOTES:

1 In racking, the statistical uncertainty regarding the magnitude of weight of unit load actions is considerably less than that for the conventional variable actions in building construction (wind, snow, floor action and the like). Furthermore, users exert a high level of control in the operation of the system. Consequently, unit load actions have an action factor (partial safety factor) between that for other live actions and permanent actions.

2 The main uncertainty in the action related performance of pallet racking is in the interaction with the mechanical handling equipment. It is considered that these effects are incorporated in the accidental actions and placement actions which reflect the likely result of good practice.

TABLE 2.7.4

ACTION FACTORS (γ)

Actions	Ultimate limit state	Serviceability limit state
(a) Permanent actions (γ_G)		
(i) With unfavourable effect	1.3	1.0
(ii) With favourable effect	1.0	1.0
(b) Variable actions (γ_Q)		
(i) Unit loads	1.4	1.0
(ii) Unit loads in crane-operated systems	1.4 or 1.3 (see Note)	1.0
(iii) Placement actions	1.4	1.0
(iv) Other live actions	1.5	
(c) Accidental actions		
(i) γ_A	1.0	
(ii) γ_{GA}	1.0	
(iii) γ_{QA}	1.0	

NOTE: The lower value of action factor of 1.3 may be applicable for a crane-operated warehousing system where all unit loads are weighed prior to entering the racking, and unit loads weighing more than the specified design load are rejected.

2.7.5 Stability against overturning

Using an action factor corresponding to the ultimate limit state, it shall be verified that the empty racking is stable under the action of a single horizontal placement action in the most unfavourable position.

The horizontal placement action shall be resisted by the self-weight of the racking and the floor anchorages.

In every upright frame, the base plates shall be fixed squarely to the uprights and secured to the floor through any shimming material or grouting necessary to ensure that the uprights are solidly supported under the whole area of the base plate. The shimming material shall be steel and shall be prevented from shifting relative to the base plate.

On racking serviced by mechanical equipment, a minimum of two anchors per base plate shall be used.

SECTION 3 STRUCTURAL ANALYSIS

3.1 GENERAL

A racking system shall be designed as follows:

(a) Firstly, a global analysis of the structure shall be made in order to determine the distribution of design actions (such as internal forces, moments or stresses) and displacements, as specified in Clause 3.3.

(b) Secondly, individual elements of the structure and connections shall be checked to ensure that the elements have adequate resistance in the ultimate limit state, and that unacceptable deformations do not develop in the serviceability limit state, as specified in Section 4.

For the purpose of global analysis based on beam elements, system lines coinciding with the centroidal axes of the gross cross-section of the members should typically be used.

The influence of imperfections shall be considered in the analysis and design by taking due account of the following:

(i) Frame (sway) imperfections in accordance with Clauses 3.3.2.1, 3.3.2.2 and 3.3.2.3.

(ii) Member imperfections in accordance with Clause 3.3.2.4.

(iii) Local and distortional imperfections in accordance with Clause 3.3.2.5.

Member, local and distortional imperfections may implicitly be accounted for in the design provisions depending on the type of structural analysis.

NOTES:

1 Pallet racking systems are usually fabricated from cold-formed sections and, therefore, elastic methods of global analysis are usually the most appropriate. However, non-linear connection behaviour may be incorporated in the analysis provided that the non-linear characteristic used is based on test results which demonstrate adequate rotation capacity.

2 Although the pallet racking is a spatial structure, it is assumed that for the purposes of global analysis, it can be treated as comprising a set of plane frames lying in the vertical planes parallel and perpendicular to the aisles, and in the horizontal plane, each of which is initially taken to operate independently. Alternatively, the internal actions and displacements may be determined from a three-dimensional analysis of the racking structure.

3 Imperfections in one plane may be neglected in the global analysis of the other plane.

4 Where axial tension is predicted to develop in uprights, uplift of uprights should be considered in the structural analysis, unless the anchorage of the base plate is designed to prevent such uplift.

3.2 DESIGN CRITERIA

3.2.1 Actions and action combinations

Racking design actions shall be determined for the actions specified in Section 2.

Both the ultimate and serviceability limit states shall be considered using the action combinations specified in Clause 2.7. Provisions for determining the serviceability limit state performance and the ultimate limit state resistance shall be carried out in accordance with Section 4.

3.2.2 Design procedure

The analysis of a racking system shall be undertaken by considering the down-aisle and cross-aisle directions. In the case of elastic analysis, in order to design the uprights, the actions arising from the down-aisle and cross-aisle analyses shall be combined using interaction equations. Other elements shall be designed on the basis of one or other plane frame analysis, as appropriate.

The global analysis provides the internal design actions. The design provisions of Sections 4 and 5 are dependent upon the method of global analysis used to determine the design actions.

NOTE: The design of uprights involves combining the axial force arising from the stored materials and the like, enhanced by any additional axial forces arising from placement action and the like, with bending moments about both axes of the section. However, in accordance with Clause 2.7.1, the frame imperfections and placement actions need only be considered in one direction at a time.

3.2.3 Action combinations for analysis

The action combinations for analysis shall be as specified in Clauses 2.7.2 and 2.7.3. Pattern load shall be considered for the down-aisle and cross-aisle directions as specified in Clause 2.7.2.1 and Clause 2.7.2.2, respectively.

3.3 GLOBAL STRUCTURAL ANALYSIS

3.3.1 Methods of structural analysis

A comprehensive analysis of a complete frame, or in long racking a representative number of bays, in either the down-aisle or cross-aisle direction, shall be carried out using one or a combination of the following methods of analysis, within the limitations of Clause 3.3.8:

(a) Linear analysis (LA).

(b) Linear buckling analysis (LBA).

(c) Geometric non-linear analysis (GNA).

(d) Geometric and material non-linear analysis with geometric imperfections (GMNIA), types GMNIAc and GMNIAs for frames composed of compact and non-compact members, respectively.

The requirements for each method of analysis in regard to geometric and material modelling are set out in Clause 3.3.3. Geometric imperfections and eccentricities between members shall be considered in the structural model, as specified in Clause 3.3.2, or indirectly in the design of members. The specified geometric imperfections depend on the method of analysis.

Structural analysis programs used to perform LA, LBA, GNA and GMNIA analyses shall be validated against benchmark analytical solutions, well-documented experimental tests, or similar benchmark results.

3.3.2 Design imperfections and eccentricities

3.3.2.1 *Frame (sway) imperfections in unbraced frames*

The effects of frame imperfections shall be considered in global analysis either by means of an initial sway geometric imperfection or by a closed system of equivalent horizontal forces.

NOTES:

1 More sophisticated modelling of the global imperfection than an initial frame imperfection or a closed system of equivalent horizontal forces may be carried out, however, care needs to be taken in the generation of the model to reflect the practical application.

2 As upright frames are usually braced in the cross-aisle direction, this Clause may only apply to down-aisle imperfections. For cross-aisle imperfections see Clause 3.3.2.2.

The effect of looseness of the beam to upright connector shall be included in the calculation of the frame imperfection or by direct modelling in the moment-rotation characteristics of the upright-to-pallet beam connection.

The frame imperfection (ϕ) shall be determined from the following:

$$\phi = \phi_s + \phi_\ell$$

where

ϕ_s = maximum specified out-of-plumb divided by the height, which depends on the tolerance grade of the racking given in Table 1.7.1(a)

ϕ_ℓ = looseness of beam-upright connector

ϕ ≥ 1/250 for GNA analysis

≥ 1/500 for GMNIA analysis.

NOTE: If the effect of the looseness of the beam to upright connector is included in the modelling of the connection used in the global analysis, ϕ_ℓ may be set equal to zero in the above equations.

In the absence of tests of connector looseness, ϕ_ℓ shall be taken as 0.01 when calculating the initial out-of-plumb in the plane of the beams.

These initial frame imperfections shall apply in all horizontal directions, but may be considered in one direction at a time.

The initial frame imperfections may be replaced by a closed system of equivalent horizontal forces. These equivalent horizontal forces shall be applied at each level and shall be proportional to the factored vertical loads applied to the structure at that level, as shown in Figure 3.3.2.1.

For the design of the base plate and floor fixings, the horizontal reactions at each base support shall be determined using the sway imperfection ϕ and not the equivalent horizontal forces. In the absence of actual horizontal loads, the net horizontal reaction is zero.

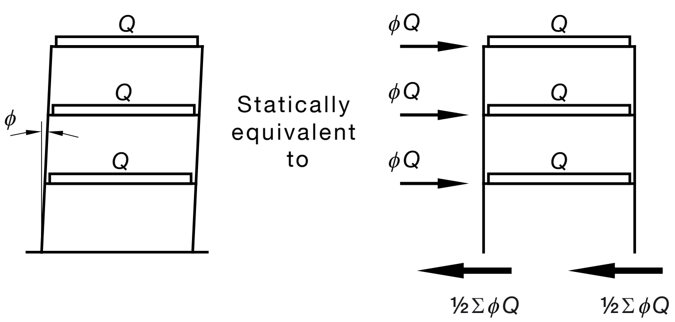


FIGURE 3.3.2.1 EQUIVALENT HORIZONTAL FORCES

3.3.2.2 *Frame (sway) imperfections in braced frames*

The imperfections described in this Clause shall be included in the global analysis.

The initial frame imperfection (see Figure 3.3.2.2) shall be determined from the following:

$$\phi = \sqrt{\left(\frac{1}{2} + \frac{1}{n_f}\right)}\ 2\phi_s \qquad \ldots 3.3.2.2$$

where

n_f in the down-aisle direction = the number of upright frames connected by pallet beams in one row of bays

n_f in the cross-aisle direction = the number of upright frames connected together (e.g. by top ties, run spacers or by intermediate floors) and acting together.

NOTE: Rational analysis may allow the use of more than one row of upright frames in the cross-aisle direction (e.g. due to the top bracing or the diaphragm action of a floor).

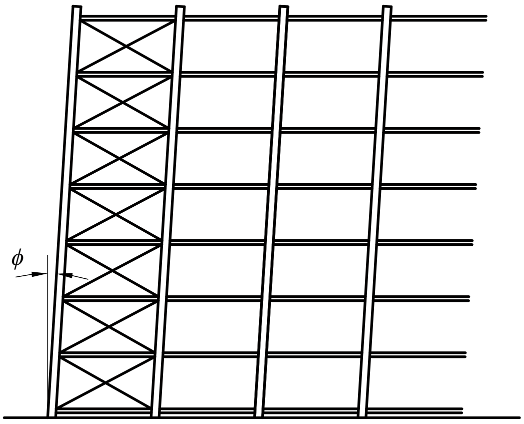


FIGURE 3.3.2.2 GLOBAL BRACING IMPERFECTIONS

3.3.2.3 *Imperfections in racking partially braced in the down-aisle direction*

This Clause applies to frames where the cross-bracing shown in Figure 3.3.2.2 extends over the lower part of the height and the frame is unbraced above this part. The initial frame imperfection ϕ in Clause 3.3.2.2 shall be applied over the braced portion of the height. The initial frame imperfection ϕ in Clause 3.3.2.1 shall be applied over the unbraced portion of the height.

3.3.2.4 *Member imperfections*

3.3.2.4.1 *Uprights*

When GMNIA structural analysis is used for determining internal actions in accordance with Clauses 3.3.1 and 3.3.3, geometric imperfections of uprights shall be included in the structural model.

NOTES:

1 Member imperfections may be implemented in the structural model by performing an LBA analysis with restraints to prevent sway, scaling the buckling mode to an amplitude of δ_o, and superimposing the scaled buckling mode onto the perfect straight-member geometry.

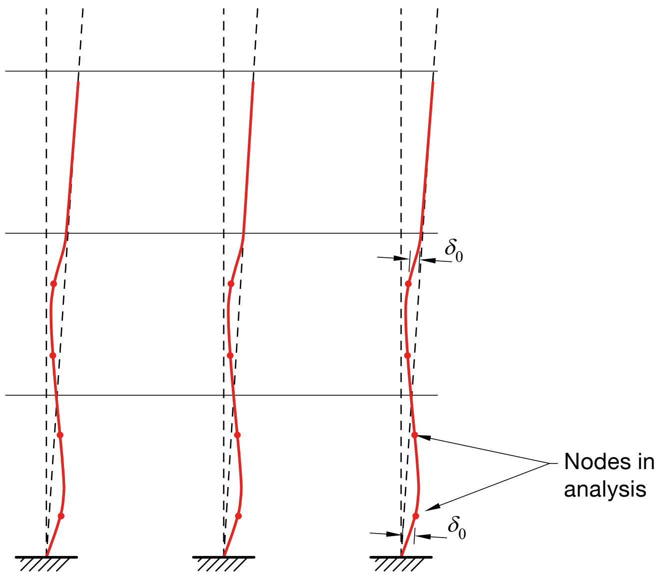
2 Alternatively member imperfections may be implemented in the structural model by off setting nodes relative to the straight member geometry by δ_o. In this case, it is sufficient to include geometric imperfections in the uprights of the lower two storeys only. It is also usually sufficient to introduce two additional nodes per member as shown in Figure 3.3.2.4(a).

3 The magnitude of the member geometric imperfection should be in accordance with the damage of uprights tolerated during service. For the damage levels specified in Clause 8.3.2, values of $\delta_o = L/200$ and $\delta_o = L/333$ apply to the down-aisle and cross-aisle directions, respectively. If the damage levels exceed the member imperfection allowed for in the structural design standard, e.g. $L/1000$ in AS 4100, then member imperfections shall also be included in LA and GNA analyses.

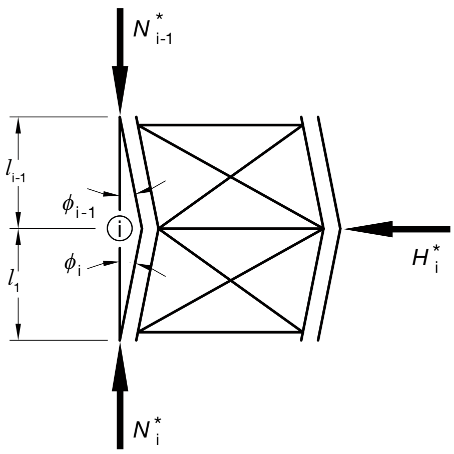
Alternatively, the effects of geometric member imperfections may be implicitly accounted for by using a reduced flexural rigidity ($EI^* = 0.8EI$) for all members and connections.

3.3.2.4.2 *Bracing member*

Local bracing imperfections give rise to self equilibrating systems of forces (see Figure 3.3.2.4(b)) which shall be used in the design of the bracing members and their connections only. A first order analysis of a subassembly, as shown in Figure 3.3.2.4(b) may be used.



(a) Upright imperfections



(b) Local bracing imperfections

FIGURE 3.3.2.4 IMPERFECTIONS

For uprights without splices: $\phi_o = 1/400$.

For uprights which incorporate splices: $\phi_o = 1/200$.

For $l_i \geq l_{i-1}$: $\phi_{i-1} = \phi_o \sqrt{\frac{1}{2}\left(1 + \frac{1}{n_u}\right)}$ but $\phi_{i-1} \leq \phi_o$

and $$\phi_i = \phi_{i-1}\frac{l_{i-1}}{l_i}$$

For $l_i \le l_{i-1}$: $\phi_i = \phi_o \sqrt{\frac{1}{2}\left(1+\frac{1}{n_u}\right)}$ but $\phi_i \le \phi_o$

and $$\phi_{i-1} = \phi_i \frac{l_i}{l_{i-1}}$$

where n_u is the number of uprights per bracing system.

The initial geometric imperfection may be applied as a horizontal force (H_i^*) calculated as follows:

$$H_i^* = N_{i-1}^*\,\phi_{i-1} + N_i^*\phi_i \qquad \ldots 3.3.2.4$$

where

H_i^* = summed over all connected uprights

N^* = design axial load in upright

If $l_i = l_{i-1}$, $N_i^* = N_{i-1}^*$ and $\phi_i = \phi_{i-1}$ then $H_i^* = 2\,N_i^*\phi_i$

3.3.2.5 *Local and distortional buckling imperfections (GMNIAs only)*

When GMNIAs structural analysis is used for determining internal actions in accordance with Clauses 3.3.1 and 3.3.3, geometric imperfections in the shapes of the local and distortional buckling modes shall be included in the structural model.

NOTES:

1 Local buckling is defined as a buckling mode where the junctions between component plates remain essentially un-displaced, as shown in Figure 3.3.2.5(a). Distortional buckling is defined as a buckling mode where one or several component plates undergo in-plane displacements, as shown in Figure 3.3.2.5(b).

2 The local and distortional buckling modes and buckling actions may be determined from an LBA structural analysis based on shell finite element or finite strip discretization of the uprights.

3 Imperfections in the shapes of the local and distortional buckling modes may be included in the structural model by multiplying the local and distortional buckling modes by imperfection multipliers and superimposing these scaled imperfections onto the perfect geometry.

4 The imperfection multipliers (s_{ol}) for local buckling may be determined as follows:

$$s_{ol} = 0.3t\sqrt{f_y / f_{ol}} \qquad \ldots 3.3.2.5(1)$$

where

t = plate thickness

f_{ol} = elastic local buckling stress.

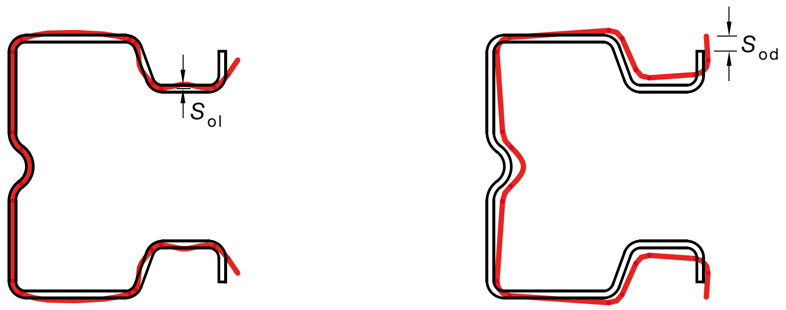
5 The imperfection multipliers (s_{od}) for distortional buckling may be determined as follows:

$$s_{od} = 0.3t\sqrt{f_y / f_{od}} \qquad \ldots 3.3.2.5(2)$$

where

t = plate thickness

f_{od} = elastic distortional buckling stress.



(a) Local buckling (b) Distortional buckling

FIGURE 3.3.2.5 BUCKLING IMPERFECTIONS

3.3.2.6 *Eccentricities*

3.3.2.6.1 *Bracing eccentricities*

If the eccentricities between system lines are greater than the limits specified in this Clause, the eccentricities shall be included in the global analysis and the resulting secondary moments shall be included in the member design.

The effects of bracing eccentricities may be neglected if the following conditions are met:

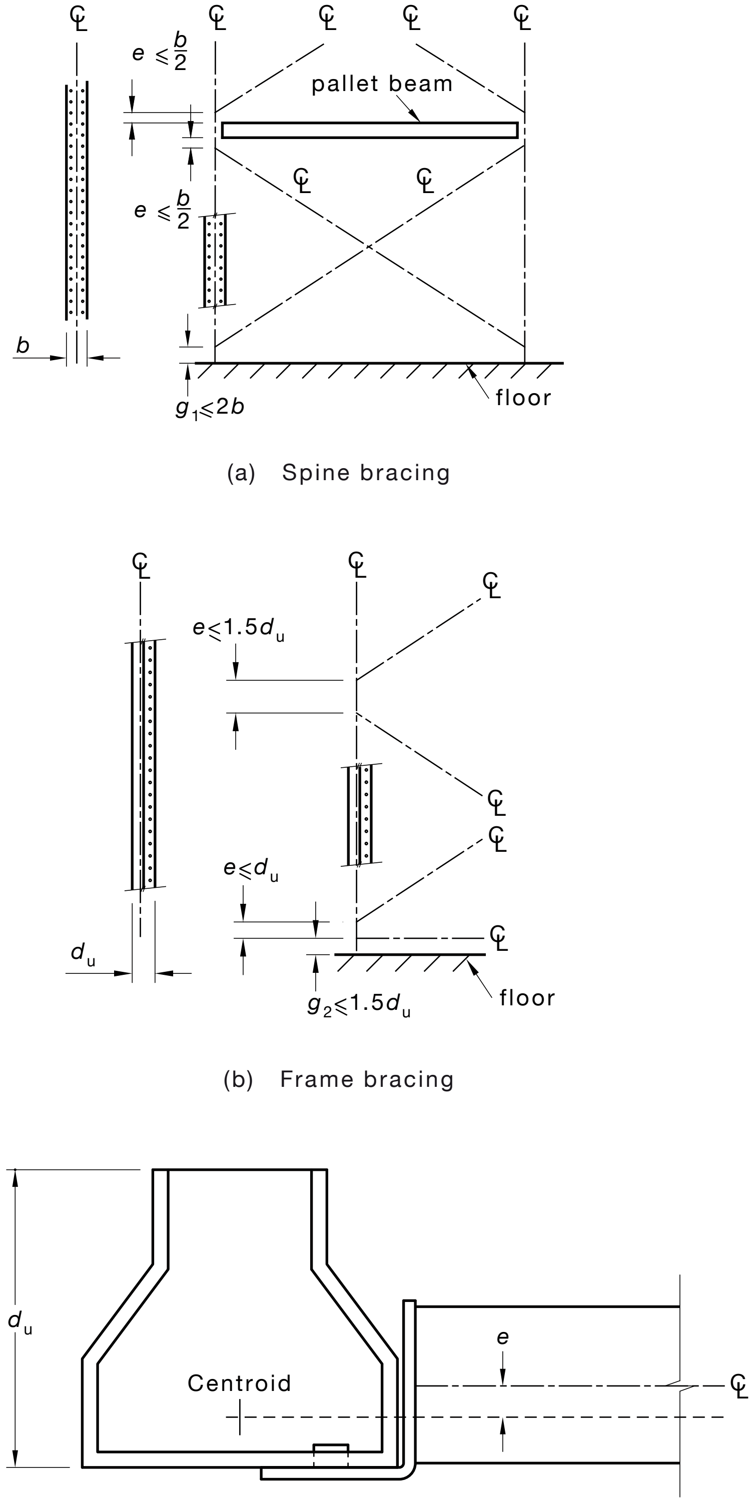
(a) For the spine bracing, the intersection point of the centre-lines of a horizontal member and a diagonal falls within a vertical dimension (e) equal to one half of the upright width (b) (see Figure 3.3.2.6(a)). Where beams are used as horizontal members, the intersection point shall be taken as the intersection of the centre-lines of a diagonal and the top or bottom flange line (see Figure 3.3.2.6(a)).

(b) For the upright frame bracing, the intersection point of the centre-lines of the horizontal bracing member and the consecutive diagonal bracing falls within a vertical dimension (e) equal to the upright depth (d_u) (see Figure 3.3.2.6(b)), and the intersection point of the centre-lines of two consecutive diagonal bracing members falls within a vertical dimension (e) equal to 1.5 times the upright depth (d_u) (see Figure 3.3.2.6(b)).

(c) The eccentricity (g_1), i.e. the vertical distance from the floor to the lower spine bracing node point, is not greater than two times the upright width (b) (see Figure 3.3.2.6(a)) and eccentricity (g_2), i.e. the vertical distance from the floor to the lower frame bracing node point is not greater than 1.5 d_u, as shown in Figure 3.3.2.6(b).

If a global analysis including eccentricities in the cross-aisle direction is required, the bases of the structure shall be considered to be pinned unless the base stiffness is determined by testing in accordance with Clause 7.9.

NOTE: It is good practice for the angle of inclination of the diagonals of upright frames from the horizontal to be between 20° and 70°.



(a) Spine bracing

(b) Frame bracing

(c) Between beams and uprights

FIGURE 3.3.2.6 ECCENTRICITIES

3.3.2.6.2 *Eccentricities between beams and uprights*

The centroidal axis of the beam may not coincide with the centroidal axis of the upright. This results in an eccentricity (e) in the cross-aisle direction, as shown in Figure 3.3.2.6(c).

The eccentricity (e) in Figure 3.3.2.6 (c) may be neglected when e is less than $0.25d_u$, where d_u is the depth of the upright.

3.3.3 Basis of structural analysis

For the purpose of global structural analysis, the system lines shall typically coincide with the centroids of the gross cross-sections.

In a global analysis, the actual number of bays may be considered. Alternatively, if all bays have the same configuration of beam levels, it is sufficient to consider a representative number of bays. In this case, the minimum number of bays is five or the actual number of bays, whichever is the lesser.

The applicable methods of analysis are set out in Clause 3.3.1. The requirements for each method of analysis in regard to geometrical and material modelling shall be as given in Table 3.3.3.

TABLE 3.3.3

METHODS OF STRUCTURAL ANALYSIS

Analysis	Material	Member imperfection	Frame imperfection	Basis of structural analysis
LA	Elastic	No	Out-of-plumb (see Note 2) or equivalent horizontal actions (see Note 3)	Equilibrium is obtained in the original (undeformed) frame configuration, (i.e. the displacements are assumed infinitesimal). The analysis may be based on prismatic beam-elements. Gross cross-section properties may be used ignoring perforations. The analysis shall account for semi-rigidity of connections and base plates, and reduced shear stiffness of upright frames.
LBA	Elastic	No	Out-of-plumb (see Note 2) or equivalent horizontal actions (see Note 3)	The pre-buckling internal actions may be obtained from an LA analysis. The analysis may be based on prismatic beam-elements. The analysis may, or may not, consider torsional (twist) rotations and torsional internal actions. Gross cross-section properties may be used ignoring perforations. The analysis shall account for semi-rigidity of connections and base plates, and reduced shear stiffness of upright frames.
GNA	Elastic	No	Out-of-plumb (see Note 2) or equivalent horizontal actions (see Note 3)	Equilibrium is obtained in the deformed frame configuration. The analysis may be based on prismatic beam-elements. Gross cross-section properties may be used ignoring perforations. The analysis may, or may not, consider torsional (twist) rotations and torsional internal actions. The analysis shall account for semi-rigidity of connections and base plates, and reduced shear stiffness of upright frames.

(*continued*)

TABLE 3.3.3 (*continued*)

Analysis	Material	Member imperfection	Frame imperfection	Basis of structural analysis
GMNIAc and GMNIAs	Inelastic	Yes (see Note 1)	Out-of-plumb (see Note 2) or equivalent horizontal actions (see Note 3)	Equilibrium is obtained in the deformed frame configuration. The plasticity modelling shall be based on flow theory. The modelling of the non-linear stress-strain relationship shall be based on recognized models for hot-rolled or cold-formed steel, as appropriate. Residual stresses shall be modelled directly or indirectly (e.g. through the stress-strain curve). The analysis shall account for semi-rigidity of connections and base plates, and reduced shear stiffness of upright frames. *For type c-analysis—* the cross-section is compact in accordance with AS 4100 and non-perforated except at connection points; the analysis may be based on prismatic beam-elements; and the analysis shall consider torsional (twist) rotations and torsional internal actions, including warping torsion, unless fully laterally restrained in accordance with Clauses 5.3 and 5.4 of AS 4100. *For type s-analysis—* the cross-section is non-compact in accordance with AS 4100 or cold-formed; the analysis is based on discretization of pallet beams and uprights into shell finite elements or finite strips modelling perforations; and the analysis incorporates local and distortional geometric imperfections (see Note 4).

NOTES:

1 Member geometric imperfections shall be modelled in accordance with Clause 3.3.2.4.

2 Out-of-plumb shall be modelled in accordance with Clauses 3.3.2.1 to 3.3.2.3.

3 Equivalent horizontal action shall be determined in accordance with Clause 3.3.2.1.

4 Local and distortional geometric imperfections shall be determined in accordance with Clause 3.3.2.5.

3.3.4 Analysis of braced and unbraced racking in the down-aisle direction

3.3.4.1 *General*

The stability in the down-aisle direction shall be demonstrated by a rational analysis which takes account of the following:

(a) The destabilizing effect of axial compressive loads in the uprights (second-order effect).

(b) The moment-rotation characteristics of the beam to upright connections.

(c) The moment-rotation characteristics of the upright to floor connections.

(d) The shear stiffness of the bracing system and its connections.

(e) The moment-rotation characteristics of splices in the uprights.

(f) The actions arising from down-aisle frame imperfections as specified in Clauses 3.3.2.1 and 3.3.2.2, as appropriate.

(g) The actions originating from handling equipment.

NOTES:

1 If it is not possible to calculate the shear stiffness of the bracing system and its connections, the shear stiffness should be determined by a test of a suitable component or subassembly as specified in Clause 7.7.

2 Bracing members introduce additional axial forces into the adjacent uprights which should be considered in the design of these members.

3 Plan bracing will also transfer horizontal actions in the down-aisle direction back to the plane of the vertical spine bracing and thereby introduce additional axial force into the adjacent uprights.

4 The eventualities (described in Notes 2 and 3) are often taken into account by providing an additional braced frame adjacent to the plan bracing.

5 In 2D analysis of spine-braced racking, plan bracing should be installed at each level of vertical bracing connection points to ensure the front row of uprights is braced in the down-aisle direction in an equal manner to the rear row of uprights. The connections between plan bracing and spine bracing should be detailed to preferably avoid warping and torsional actions in the upright.

6 Racking may be propped in the down-aisle direction against the building or other substantial structure. If advantage is taken of propping the racking against the building or other substantial structure in the design, the force in the prop or props should be calculated and the prop and supporting structure designed accordingly.

3.3.4.2 *Moment-rotation characteristics of beam end connectors*

The moment-rotation characteristics of the beam to upright connections shall be determined as the design values of stiffness and moment resistance obtained by testing in accordance with Clause 7.5. The treatment of the flexibility of the beam-end connectors shall be considered by either modelling the beam-upright connectors—

(a) as rotational springs of constant stiffness. In this case, the looseness of the beam-upright connectors shall be incorporated in the frame imperfections in accordance with Clause 3.3.2.1. The joint stiffness shall be determined from tests in accordance with Clause 7.5; or

(b) as non-linear rotational springs. In this case, the looseness of the beam-upright connectors shall either be incorporated in the global analysis by including an appropriate initial looseness in the non-linear spring characteristic or the beam-upright connector looseness shall be incorporated in the frame imperfections in accordance with Clause 3.3.2.1. The joint looseness shall be determined from tests in accordance with Clause 7.5.3.

3.3.4.3 *Moment-rotation characteristics of the connection to the floor*

For a flat-ended upright, the stiffness of the upright to floor connection shall be assumed to be EI/h, where—

EI = flexural rigidity of the upright in the down-aisle direction

h = height to the first beam level with a minimum value of 1 m.

If a higher value of the stiffness of the floor to upright connection is to be included in the analysis, it shall be determined by test in accordance with Clause 7.9 with an axial force appropriate to the installation being designed.

Alternatively, a more detailed variation of the stiffness of the floor to upright connection with axial force may be included in the analysis.

NOTE: For non-seismic design, the connection of the upright to the floor may conservatively be modelled as a pinned connection.

3.3.5 Analysis of braced and unbraced racking in the cross-aisle direction

The stability in the cross-aisle direction shall be demonstrated by a rational analysis that takes into account all of the following:

(a) The shear flexibility of the bracing system, including the flexibility of the connections between the uprights and the bracing members. The shear stiffness shall be determined in accordance with Clause 7.7.

(b) The moment-rotation characteristics of splices in the uprights.

(c) The actions originating from handling equipment.

(d) The moment-rotation characteristics of the upright to floor connections (see Note 1).

(e) The overall stability of the braced frame.

(f) The actions arising from frame imperfections in the cross-aisle direction as specified in Clause 3.3.2.2.

NOTES:

1 It is usually safe to assume a pinned connection between the uprights and the floor. A non-zero moment-rotation characteristic may only be used if it is assured that full contact with the floor will be maintained.

2 Racking may be propped in the cross-aisle direction against the building or other substantial structure. If advantage is taken of propping against the supporting structure, the force in the prop or props should be calculated and the prop and the supporting structure designed accordingly.

3.3.6 Methods of analysis for stability in the down-aisle direction

For determining the elastic critical load for buckling in the down-aisle direction, a rational LBA method shall be used, or the approximate LBA method described in Appendix A may be used in lieu.

The approximate LBA method described in Appendix A may only be used for standard pallet racking arrangements that conform to the following:

(a) Constant beam length.

(b) Approximately constant height between beam levels except for the height from the floor to the first beam level.

(c) The same upright section throughout the racking.

(d) The same beam section throughout the racking.

(e) No change of beam levels within the length of the racking.

(f) The same beam end connector type throughout the racking.

If using the simplified method of Appendix B for determining the elastic critical load, splices of adequate stiffness in the upper part of the structure shall be ignored. This simplified method shall not be used if there are splices below the third beam level of the structure.

If there is a splice below the third beam level, either it shall be demonstrated that the splice does not introduce any loss of stiffness or a GNA analysis shall be carried out taking into account the flexibility of the splice.

3.3.7 Simplified methods of analysis for stability in the cross-aisle direction

For determining the elastic critical load for buckling in the cross-aisle direction, a rational LBA method shall be used, or the simplified LBA method described in Appendix C may be used in lieu.

3.3.8 Frame classification

Frame classification shall be based on the elastic critical action ratio (N^*/N_{cr}), where—

N^* = design value of the vertical action on the frame

N_{cr} = elastic critical value of the vertical action for buckling in a sway mode.

In classifying a frame, the following shall be considered:

(a) If $N^*/N_{cr} \leq 0.1$, a frame shall be classified as stiff, i.e. the frame response to in-plane horizontal forces is sufficiently stiff for it to be acceptably accurate to neglect any additional internal forces or moments arising from horizontal displacements of the nodes. In such a case, an LA analysis without amplification shall be sufficient.

Any other frame shall be classified as a flexible frame and the effects of the horizontal displacement of its nodes shall be taken into account in its design.

NOTES:

1 The critical action may be determined from a rational LBA analysis.

2 Approximate methods of estimating N_{cr} with sufficient accuracy for frame classification are given in Appendices A and C in reference to Clauses 3.3.6 and 3.3.7.

3 Unbraced racking is invariably classed as flexible frames in the down-aisle direction and therefore second order effects should be considered.

(b) If $0.1 < N^*/N_{cr} \leq 0.3$, an LA analysis may be used in which second order effects are incorporated using moment amplification factors in accordance with Clause 3.3.9.

(c) If $N^*/N_{cr} > 0.3$, a GNA or GMNIA analysis shall be required in which second order effects are treated directly.

3.3.9 Design actions

When the design actions have been obtained from a GNA or GMNIA analysis, the analysis accounts for second order effects and moment amplification shall not be required.

When the design actions have been obtained from an LA analysis, second order effects shall be treated indirectly by one of the following methods:

(a) Using the amplified sway moment method whereby the sway moments obtained by an LA elastic analysis shall be increased by multiplying by the ratio—

$$\frac{N_{cr}}{N_{cr} - N^*} \qquad \ldots 3.3.9$$

where the elastic critical buckling action (N_{cr}) shall be determined in accordance with Clause 3.3.8.

(b) The simplified method described in Appendix B. This method may be regarded as a version of the amplified sway moment method. It is also subject to the requirements stated in Clause 3.3.6 for the approximate LBA method described in Appendix A.

SECTION 4 DESIGN PROCEDURES

4.1 GENERAL

All design computations for limit states actions, deflections and the like shall be made in accordance with conventional methods of structural design and provisions as specified in AS/NZS 4600 for cold-formed steel components and structural systems and AS 4100 for hot-rolled steel components and structural systems, as applicable, except as modified or supplemented by this Standard. In cases where adequate methods of design calculations are not available, the following tests shall be conducted:

(a) Tests on uprights (see Clause 7.3).

(b) Pallet beam tests (see Clause 7.4).

(c) Pallet beam to upright connection tests (see Clause 7.5).

(d) Upright frame tests (see Clause 7.6).

(e) Tests for shear stiffness of upright frames (see Clause 7.7).

(f) Tests on upright splices (see Clause 7.8).

(g) Tests on floor connections (see Clause 7.9).

(h) Charpy type impact tests (see Clause 7.10).

The effect of perforations (slots) on the load carrying capacity of compression members is accounted for by the modification of the design equations in AS/NZS 4600 and the use of the minimum net area instead of the gross area in the relevant calculations described in Section 5.

It may be necessary to consider the fatigue limit state if the rack is subjected to repeated loading. Design for fatigue shall be carried out to Section 6 of AS/NZS 4600 or Section 11 of AS 4100, as appropriate.

4.2 DESIGN CRITERIA

4.2.1 General

The design procedure depends on the type of global structural analysis used to determine the design actions, as per Clause 3.3.1. Clauses 4.2.2, 4.2.3 and 4.2.4 specify the applicable design procedure for each type of global structural analysis.

4.2.2 Design based on LA or GNA structural analysis

The design of uprights requires the use of interaction equations (see Clause 3.5 of AS/NZS 4600 and Clauses 8.3 and 8.4 of AS 4100) considering the combined effects of compression and bending. Effective flexural buckling lengths for determining the compression capacity (N_c) shall be in accordance with Clauses 4.2.2.1 and 4.2.2.2 for the down-aisle direction and Clauses 4.2.2.3 and 4.2.2.4 for the cross-aisle direction. The effective buckling length for torsion shall be in accordance with Clause 4.2.2.5.

4.2.2.1 *Design based on LA structural analysis considering the down-aisle direction*

Bending moments determined from LA analysis shall be amplified in accordance with Clause 3.3.9. The effective length (l_e) for flexural buckling in the down-aisle direction shall be taken as the height between beam levels.

4.2.2.2 *Design based on GNA structural analysis considering the down-aisle direction*

The effective length (l_e) for flexural buckling in the down-aisle direction shall be determined from the elastic buckling action (N_{cr}) as follows:

$$l_e = \pi \sqrt{\frac{EI}{N_{cr}}} \qquad \ldots 4.2.2.2$$

where

I = second moment of area for the relevant axis of buckling

N_{cr} = elastic buckling action under consideration, determined in accordance with Clause 3.3.8.

If the frame is unbraced in the down-aisle direction, it is permitted to determine N_{cr} for the corresponding braced frame by applying appropriate restraint against down-aisle displacement in the LBA analysis.

Alternative methods are as follows:

(a) For unbraced frames and frames braced in the down-aisle direction (spine braced frames), $l_e = L$, where L is the height between beam levels.

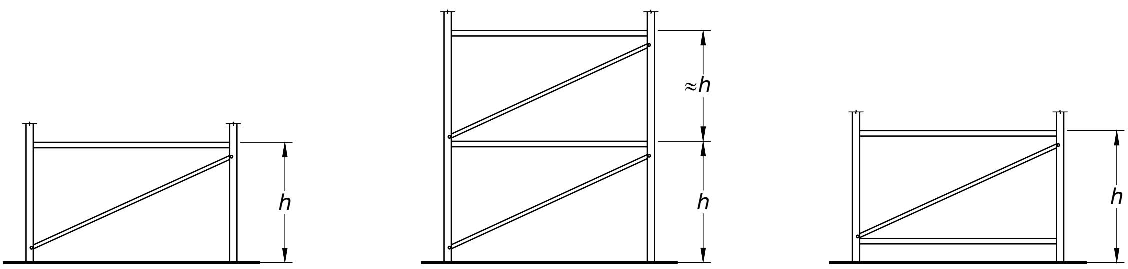
(b) For the bottom length of frames braced in the down-aisle direction, provided that a base plate is fitted to the upright and the floor is concrete.

$l_e = 0.9L$, where L is determined from one of the following cases:

(i) *Case 1* The bracing nodes do not coincide with the beam nodes (i.e. the conditions stated in Clause 3.3.2.6 are not satisfied): L is the height to the first floor beam level (h in Figure 4.2.2.2(a)).

(ii) *Case 2* The bottom beam is at a height above the floor similar to the height between adjacent beams: L is the height from the floor to the first beam level (h in Figure 4.2.2.2(b)).

(iii) *Case 3* The bottom beam or bracing node is close (see Clause 3.3.2.6) to the floor: L is the height from the floor to the second beam level or the beam level above the bracing node (h in Figure 4.2.2.2(c)).



(a) Case 1 (a) Case 2 (a) Case 3

FIGURE 4.2.2.2 FRAMES BRACED IN THE DOWN-AISLE DIRECTION

4.2.2.3 *Design based on LA structural analysis considering the cross-aisle direction*

Bending moments determined from LA analysis shall be amplified in accordance with Clause 3.3.9. The effective length (l_e) for flexural buckling in the cross-aisle direction shall be taken as the height between node points.

4.2.2.4 *Design based on GNA structural analysis considering the cross-aisle direction*

The effective length (l_e) for flexural buckling in the cross-aisle direction shall be determined from an LBA analysis following the procedure specified in Clause 4.2.2.2.

Alternative methods are as follows:

(a) For the bottom length of an upright in a braced upright frame l_e = 0.9L, where L is the height from the floor to the second node point (h in Figures 4.2.2.4(a) and (b)), provided that—

  (i) the bracing members of the upright frame are connected to both flanges of the upright;

  (ii) the bracing eccentricities satisfy the requirements of Clause 3.3.2.6;

  (iii) a base plate is fitted to the upright; and

  (iv) the floor is concrete.

  otherwise $l_e = L$.

  NOTE: In a braced frame, if the bottom node is not near the floor as per Clause 3.3.2.6, the length between the floor and the first node should be considered as being free to sway.

(b) For all other parts of the upright in a braced upright frame, $l_e = L$, where L is the height between node points (h_p in Figures 4.2.2.4(a) and (b)).

  NOTE: The situation shown in Figure 4.2.2.4(c) arises frequently and special care should be taken with the stability of the unbraced upper portion of the uprights.

(c) For horizontal and diagonal bracing members in an upright frame, provided the bracing member is welded with a minimum fillet weld length of 20 mm to both flanges of the uprights, for in plane buckling, $l_e = 0.9L$. For all other cases, $l_e = L$.

  If the connections at the ends of a bracing member do not coincide with its system lines, i.e., the eccentricities do not comply with Clause 3.3.2.6, the member shall be designed for combined axial action and bending.

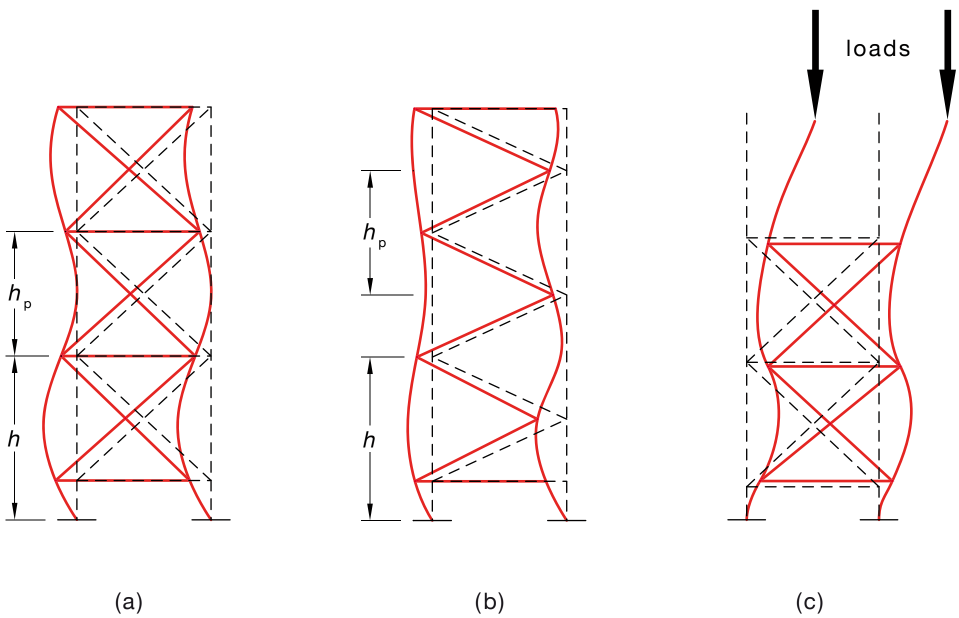


FIGURE 4.2.2.4 IN-PLANE BUCKLING MODES FOR BRACED UPRIGHT FRAMES

4.2.2.5 *Effective length for torsional buckling of uprights*

The elastic torsional buckling stress (f_{oz}) specified in AS/NZS 4600 may be calculated using a torsional effective length (l_{ez}) as determined in this Clause.

Unless determined by a rational buckling analysis, the following torsional buckling length shall be used:

(a) l_{ez} = 1.0 times the distance between bracing points when the connections provide full torsional restraint.

(b) l_{ez} = 0.5 times the distance between bracing points when the connections provide full torsional restraint and full warping restraint.

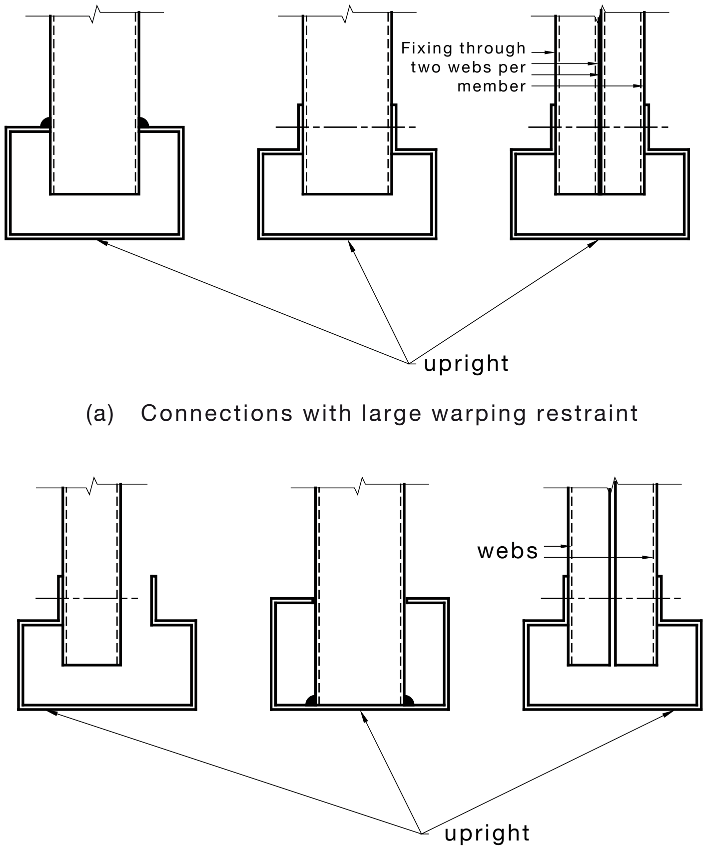
NOTES:

1 In practice, it is difficult to obtain full torsional and full warping restraint. Figures 4.2.2.5(a) and 4.2.2.5(b) offer guidance on the torsional effective length for typical brace to upright connections.

2 For end connections similar to Figure 4.2.2.5(a), which may be regarded as providing large torsional restraint and large warping restraint, l_{ez} = 0.7 times the distance between the bracing points.

3 For end connections similar to Figure 4.2.2.5(b), which may be regarded as providing large torsional restraint and partial warping restraint, l_{ez} = 1.0 times the distance between the bracing points.

4 Lower values of l_{ez}, for instance where l_{ez} is greater than 0.5 times the distance between bracing points, may also be obtained by comparison between theoretical torsional and flexural-torsional buckling expressions with results of compression tests on uprights as defined in Clause 7.3.2.



(a) Connections with large warping restraint

(b) Connections with partial warping restraint

FIGURE 4.2.2.5 BRACING MEMBER CONNECTIONS

4.2.3 Design based on GMNIAc structural analysis

For the strength limit state, the following shall be satisfied:

(a) The member capacity shall support the factored limit states actions multiplied by $1/\phi$, where $\phi = 0.9$.

Alternatively, the section capacity requirement—

$$\frac{N^*}{\phi N_s} + \frac{M_x^*}{\phi M_{sx}} + \frac{M_y^*}{\phi M_{sy}} \leq 1 \qquad \text{. . . 4.2.3(1)}$$

shall be satisfied, where the section capacities (N_s, M_{sx}, M_{sy}) are determined in accordance with AS 4100.

If uprights are subjected to primary torsional loads, the section capacity requirement, if imposed, shall be as follows:

$$\frac{N^*}{\phi N_s} + \frac{M_x^*}{\phi M_{sx}} + \frac{M_y^*}{\phi M_{sy}} + \frac{B^*}{\phi B_s} \leq 1 \qquad \text{. . . 4.2.3(2)}$$

where

B_s = bi-moment section capacity

$$= \frac{I_w}{\omega_{max}} f_y \qquad \text{. . . 4.2.3(3)}$$

I_w = warping constant

ω_{max} = maximum value of sectorial coordinate

ϕ = 0.9

(b) The connections shall satisfy the requirements of Section 9 of AS 4100, or the design actions shall be limited to—

(i) the moment and shear capacities of pallet beam to upright connections as determined by testing in accordance with Clause 7.5.1 and Clause 7.5.4, respectively, and Clause 7.9 for base plate connections; and

(ii) the combined moment and shear capacity determined from the following:

$$\frac{M^*}{M_c} + \frac{V^* - (M_c/a)}{V_c} \leq 1 \qquad \text{. . . 4.2.3(4)}$$

where

M^* = moment design action

M_c = moment design capacity of the connection (see Clause 7.5.1)

V^* = shear design action

a = lever arm of the applied action in the cantilever test (see Clause 7.5.1)

V_c = shear design capacity of the connection (see Clause 7.5.4).

4.2.4 Design based on GMNIAs structural analysis

For the strength limit state, the following shall be satisfied:

(a) For the ultimate limit state, the members shall be required to support the factored limit states actions multiplied by $1/\phi$, where $\phi = 0.9$.

(b) The connections shall satisfy the requirements of Section 5 of AS/NZS 4600, or the design actions shall be limited to—

(i) the moment and shear capacities of the connections determined by testing as per Clauses 7.5.1 and 7.5.4, respectively, and Clause 7.9 for base plate connections; and

(ii) the combined moment and shear capacity per Clause 4.2.3 (b), as applicable.

4.2.5 Serviceability criteria

The fully loaded racking with the actions arising from the imperfections as specified in Clauses 3.3.2.1 and 3.3.2.2, but not placement or accidental actions, shall be considered. It shall be verified that under these actions, the sway of the racking shall be less than $h/200$, where h is the total height of the racking.

Where required by frame classification in accordance with Clause 3.3.8, this analysis shall take second-order effects into account.

NOTE: Sway is the horizontal movement in addition to any initial out-of-plumb.

In the serviceability limit state, pallet beam deflections shall not exceed 1/180 of the span measured relative to the ends of the beam.

SECTION 5 DESIGN OF COLD-FORMED STEEL UPRIGHTS AND PALLET BEAMS

5.1 GENERAL

If the structural design is based on an LA or GNA global analysis, the design procedure involves the calculation of axial and bending design capacities of the uprights and pallet beams, and the use of interaction equations to account for combined action in the uprights. For cold-formed steel members, calculation shall be in accordance with AS/NZS 4600, except as modified in Clauses 5.2 and 5.3 (of AS/NZS 4600) to account for the influence of perforations.

5.2 UPRIGHTS

5.2.1 Section properties

Exceptions to the provisions of AS/NZS 4600 for computing the section properties are specified in Clauses 5.2.2 and 5.2.3. Except as noted, all cross-sectional properties shall be based on full unreduced and unperforated sections considering rounded corners.

5.2.2 Flexural capacity

The nominal section and member bending capacities shall be determined in accordance with AS/NZS 4600, with Z_e and Z_c determined as follows:

Z_e = elastic section modulus of the net section for the extreme compression fibre times $\left[0.5+\left(Q/2\right)\right]$

Z_f = elastic section modulus of the full unreduced cross-section for the extreme compression fibre

Z_c = elastic section modulus of the net section for the extreme compression fibre times $\left[1-\left(\frac{1-Q}{2}\right)\left(\frac{f_c}{f_y}\right)^{Q}\right]$

The value of Q shall be determined in accordance with Clause 7.3.1.

In the calculation of the elastic buckling moment M_o, involving the determination of f_{ox}, f_{oy} and f_{oz}, the section properties shall be based on the full unreduced cross-section considering rounded corners. Furthermore, J, β_y, r_{o1} and I_w may be computed assuming sharp corners.

Inelastic reserve capacity (see AS/NZS 4600) shall not be considered for perforated members.

The distortional buckling flexural capacity shall be determined as per Clause 3.3.3.3 (a) of AS/NZS 4600 with M_y replaced by $Z_{net\ min} f_y$, where $Z_{net\ min}$ is the elastic section modulus of the minimum cross-sectional area obtained by passing a plane through the upright normal to the axis of the upright, except that in the definition of the slenderness (λ_d) then the full value of $M_y = Z_f f_y$ shall be used.

5.2.3 Axial capacity

5.2.3.1 *Effective area*

The nominal compression capacity shall be determined in accordance with AS/NZS 4600, with A_e determined as follows:

A_e = effective area of the perforated cross-section at the stress f_n, where f_n shall be determined in accordance with AS/NZS 4600 based on the full unreduced cross-section

$$= \left[1-(1-Q)\left(\frac{f_n}{f_y}\right)^Q\right] A_{net\ min} \qquad \ldots 5.2.3.1$$

5.2.3.2 *Sections not subject to flexural-torsional buckling*

Radii of gyration shall be based on gross section properties computed considering rounded corners.

5.2.3.3 *Doubly-symmetric or singly-symmetric sections subject to torsional or flexural-torsional buckling*

Radii of gyration shall be based on gross section properties computed considering rounded corners.

f_{ox}, f_{oy} and f_{oz} shall be the stresses calculated in accordance with Clause 5.2.2.

5.2.3.4 *Singly-symmetric sections subject to distortional buckling*

The axial load capacity shall be determined as per Clause 3.4.6 of AS/NZS 4600 with A replaced by $A_{net\ min}$.

5.2.4 Combined compression and bending

Combined compression and bending shall be designed for by the use of interaction equations. The use of moment amplification and effective buckling lengths shall be as specified in Clause 4.2.2. The terms C_{mx}, C_{my}, α_{nx}, and α_{ny} of Clause 3.5 AS/NZS 4600 shall be taken as 1.0.

5.3 PALLET BEAM

Pallet beams shall be analysed as simply supported, or by rational analysis when partial end-fixity is considered.

At service actions, the deflections measured with respect to the ends of the beam shall not exceed the limit given in Clause 4.2.5.

SECTION 6 CONNECTIONS AND BASE PLATES

6.1 GENERAL

Adequate strength of connections to withstand the calculated resultant forces and moments, and adequate rigidity where required, shall be established by test or, where possible, by calculation. Test procedures for various connections are specified in Section 7.

6.2 PALLET BEAM CONNECTIONS

Pallet beams shall have connection locking devices (or bolts) capable of withstanding an accidental upward force of 5 kN per connection without failure or disengagement. This requirement may be deemed to be met if a Grade 4.6 M6 bolt is installed.

6.3 BASE PLATES AND ANCHORAGE

6.3.1 General

The design of base plates and anchors specified in Clause 6.3.2 applies to concrete floor slab only. For other types of floors, see relevant references, e.g. FEM 10.2.02.

The floor slab and its supporting foundation (e.g. soil and subsoil or structural framework) shall be designed to safely support the imposed loads from racking uprights with acceptable settlement or deflection. The supplier shall provide the racking layout (footprint) indicating the racking dimensions, the upright loads and the base plate areas for the purpose of the structural assessment of the floor.

6.3.2 Base plates

The bottom of all uprights shall be furnished with base plates to transfer upright actions and moments (as required) into the floor. These forces and moments shall be consistent in magnitude and direction with the racking analysis.

In every upright frame, the base plates shall be fixed squarely to the uprights and secured to the floor through using packing material or grouting necessary to ensure that the uprights are solidly supported under the whole area of the base plate. The packing material shall be steel and prevented from shifting relative to the base plate.

Alternatively, appropriate tests may be used to determine the minimum thickness of the base plate.

6.3.2.1 *Effective area*

The base plate shall be designed in accordance with Clauses 6.3.2.2, 6.3.2.3 and 6.3.2.4 as appropriate, assuming the bearing pressure is uniformly distributed over the effective bearing width of the base plate, shown as shaded in Figure 6.3.2.1:

$$e = t_b \sqrt{\frac{f_y}{2 f_{cb}}} \quad \ldots 6.3.2.1$$

where

e = effective bearing width (see Figure 6.3.2.1)

t_b = thickness of the base plate

f_y = yield stress of the base plate

f_{cb} = design bearing strength of the concrete slab

$= 0.85\phi f_c'$

ϕ = capacity reduction factor

= 0.6

f_c' = compressive strength of the concrete floor at 28 days, which shall be assumed to be 25 MPa unless otherwise determined.

When the effective bearing width (e) is greater than the actual distance available to the base plate edge, then the actual distance shall be used.

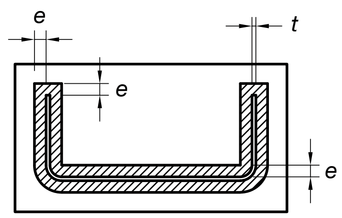


FIGURE 6.3.2.1 EFFECTIVE AREA OF THE BASE PLATE

6.3.2.2 *Base plate supporting axially loaded uprights*

When the upright is centrally loaded, the design axial compressive action (N*) shall satisfy the following:

$$N^* \leq f_{cb} A_{eff} \quad \text{. . . 6.3.2.2}$$

where

A_{eff} = effective area of the base plate.

6.3.2.3 *Combined axial action and bending moment without anchor bolts*

The combined axial action and bending moment without anchors shall be determined as follows:

(a) *Small bending moment* For small bending moment, elastic theory may be used provided the following conditions are met:

(i) $$\frac{N^*}{A_{eff}} + \frac{M^*}{Z_{eff}} \leq f_{cb} \quad \text{. . . 6.3.2.3(1)}$$

(ii) $$\frac{N^*}{A_{eff}} - \frac{M^*}{Z_{eff}} \geq 0 \text{, i.e. } M^* \leq \frac{N^* Z_{eff}}{A_{eff}} \quad \text{. . . 6.3.2.3(2)}$$

where

M^* = design bending moment in Clause 1.4 'moment design action'

Z_{eff} = elastic section modulus of the effective area about the axis of symmetry in the plane of the base plate.

(b) *Large bending moment* For large bending moment, if equation 6.3.2.3(2) is not satisfied, the base plate shall be checked using the stress distribution shown in Figure 6.3.2.3, which is assumed to apply across the effective area of the base plate only.

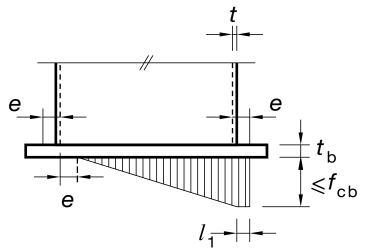


FIGURE 6.3.2.3 EFFECTIVE BEARING LENGTH AND LENGTH OF PLATEAU AT CONSTANT CONTACT PRESSURE

The area under the base plate subjected to the constant stress (f_{cb}) shall not extend beyond the centre-line of the upright flange, so that—

$$l_1 \leq \left(e + \frac{t}{2}\right) \qquad \text{. . . 6.3.2.3(3)}$$

where

l_1 = length of plateau at constant contact pressure

t = thickness of the upright.

6.3.2.4 *Combined axial action and bending moment with anchor bolts*

The effective area of the base plate shall be determined in accordance with Clause 6.3.2.1. However, the base plate thickness (t_b) shall also satisfy the following condition:

$$t_b \geq \sqrt{\frac{6 d_2 F_t^*}{l_2 f_y}} \qquad \text{. . . 6.3.2.4(1)}$$

where

d_2 = distance from the anchor bolt in tension to the nearest face of the upright

= design tension force in anchor bolt

$$= \frac{M^*}{d_1} \qquad \text{. . . 6.3.2.4(2)}$$

l_2 = overall depth of upright

$\leq$ d_3

d_1 = distance from the anchor bolt in tension to the upright flange in compression

d_3 = flange width of upright.

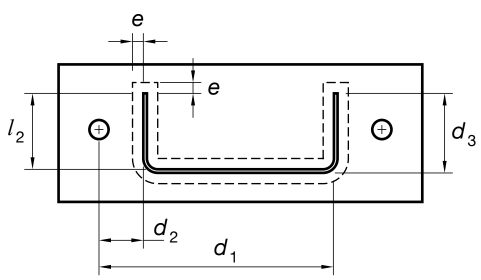


FIGURE 6.3.2.4 EFFECTIVE AREA

The base plate design shall satisfy the following:

$$A_{\mathrm{eff}} f_{\mathrm{cb}} \geq N^* + F_{\mathrm{t}}^* \qquad \ldots 6.3.2.4(3)$$

For base plates supporting uprights with more complex profiles than the simple unlipped channel shown in Figure 6.3.2.4, a similar approach to the design of the base plate may be made, with the distance (d_1) measured from the centre of the effective area of the base plate adjacent to the compression flange. Alternatively, appropriate tests may be used to determine the minimum thickness of the base plate.

6.4 DESIGN OF BASE PLATE ANCHORAGE

6.4.1 General

The design actions in the floor fixings shall be calculated for the most onerous action combination at the ultimate limit state resulting from appropriate actions as specified in Section 2. It shall also be verified that the empty racking is stable when subjected to appropriate horizontal placement actions as specified in Clause 2.3.4.

NOTE: Only the racking self-weight may be considered as resisting action.

In addition, the floor fixings shall be designed to carry the tensile forces necessary to maintain stability against overturning in accordance with Clause 2.7.5.

On racking serviced by mechanical equipment, a minimum of two anchors per base plate shall be used. Each upright-to-floor connection shall be able to transfer design ultimate actions of 5 kN in tension and 8 kN in shear.

6.4.2 Design of anchors

6.4.2.1 *General*

Both the strength of the anchor and its connection to the floor shall satisfy the requirements of Clauses 6.4.2.2, 6.4.2.3 and 6.4.2.4.

6.4.2.2 *Tensile forces*

The tensile force (F_{t}^*) shall satisfy the following:

$$F_{\mathrm{t}}^* \leq 0.9\phi_{\mathrm{t}} A_{\mathrm{s}} f_{\mathrm{ub}}\text{; and} \qquad \ldots 6.4.2.2(1)$$

$$F_{\mathrm{t}}^* \leq \phi_{\mathrm{t}} F_{\mathrm{p}} \qquad \ldots 6.4.2.2(2)$$

6.4.2.3 *Shear forces*

The shear force (F_{v}^*) shall satisfy the following:

$$F_{\mathrm{v}}^* \leq 0.6\phi_{\mathrm{v}} A_{\mathrm{s}} f_{\mathrm{ub}} \qquad \ldots 6.4.2.3$$

6.4.2.4 *Combined tensile and shear forces*

The combined tensile (F_{t}^*) and shear forces (F_{v}^*) shall satisfy the following:

$$\sqrt{\left(F_{\mathrm{t}}^*\right)^2 + 2\left(F_{\mathrm{v}}^*\right)^2} \leq 0.9\phi_{\mathrm{t}} A_{\mathrm{s}} f_{\mathrm{ub}}\text{; and} \qquad \ldots 6.4.2.4(1)$$

$$\sqrt{\left(F_{\mathrm{t}}^*\right)^2 + 2\left(F_{\mathrm{v}}^*\right)^2} \leq \phi_{\mathrm{t}} F_{\mathrm{p}} \qquad \ldots 6.4.2.4(2)$$

where

ϕ_{t} = reduction factor in tension

= 0.7

A_{s} = tensile stress area of the fixing

f_{ub} = specified ultimate tensile stress of the fixing material

F_p = design value of the pull-out resistance of the floor fixing in the floor material used in the installation

ϕ_v = reduction factor in shear

= 0.7

NOTES:

1 Many manufacturers offer proprietary fixing systems for which tests have been conducted in order to determine the ultimate tensile strength of the fixing in concretes of varying strength. In these circumstances, the manufacturer's fixing specifications should be followed and the manufacturer's values may be used for F_p.

2 For anchorages in concrete, the following parameters are significant for the value of F_p:

(a) The thickness of the structural concrete floor (an added screed will not contribute to the strength of the anchorage).

(b) The actual embedment length of the anchor in the solid concrete floor.

(c) The actual hole diameter in the concrete.

(d) The quality of the concrete.

(e) The percentage of reinforcement in the top of the slab.

(f) Whether the anchorage is in the tension or compression zone of the concrete.

(g) The spacing of anchors within an anchor group.

(h) The distance between the anchor and the edge of the concrete slab.

(i) The difference between the size of the hole in the base plate and the diameter of the anchorage.

3 When the concrete slab is placed directly on the soil, the tensile stresses in the upper layers of the concrete are generally small and the top of the slab may be regarded as being in the compression zone.

6.5 DESIGN OF FRAME SPACERS

In double entry racking, at least two frame spacers (see Figures 1.3.1(a) and 1.3.2(a)) are required between each adjacent pair of upright frames. The frame spacers shall be located near the node points of the upright frames and spaced as widely apart as practicable. The lowest spacer shall be positioned at the level of the first bracing node above the floor.

Each frame spacer shall have a capacity greater than or equal to the horizontal placement action and the horizontal accidental action.

6.6 UPRIGHT SPLICES

In general, splices in an upright frame shall be avoided. A splice should not be allowed below the lowest beam level or below 1.5 m, and no more than one splice should be located between any two adjacent beam levels.

Splices shall be designed either by calculation or by testing in accordance with Clause 7.8.

Splices shall have, as a minimum, the strength of the weaker of the connected members or shall be designed for a concentric compressive force (N*) and a bending moment (M*).

Splices and connections shall be designed in such a way that actions (axial force, shear force and bending moment) can be transmitted to the effective portions of the cross-section.

If it can be shown by test (see Clause 7.8) that the stiffness and the moment resistance of the two uprights connected by the splice are greater than or equal to those of the upright in one piece and with the same length, it shall be permissible to ignore the presence of the splice in the global analysis.

If the construction details at the ends of a member are such that there is doubt regarding the line of action of the action, a suitable bending moment due to eccentricity (M^*_{ecc}) shall be assumed in the design.

If a hinged splice is assumed in the global model, then only the axial strength and the shear strength need to be checked; the stiffness need not be checked.

The splice joint shall be designed as follows:

$$\frac{N^*}{\phi A_{sp} f_y} + \frac{M^*_{ecc} + M^*}{M_{sp}} \leq 1 \qquad \ldots 6.6$$

where

N^* = concentric compressive force

A_{sp} = net cross-sectional area of the splice

ϕ = 0.9

M^*_{ecc} = moment due to eccentricities

M^* = amplified internal bending moment of the splice

M_{sp} = design moment of resistance of the splice.

SECTION 7 TEST METHODS

7.1 INTRODUCTION

7.1.1 General

Material properties shall be determined in accordance with applicable test procedures in AS 1391. For this purpose, tensile coupons are taken, after the completion of testing of members, from flat portions of the specimen at regions of low moment and shear.

If the effects of cold-work and finishing cure schedules are to be accurately accounted for by test, the test specimens shall be formed and subjected to the same curing procedure as is used or contemplated in the prototype. This is essential because different manufacturing methods and finishing processes produce different amounts of cold working and thermal exposure (e.g. cold working of a specimen by brake-pressing is less than in a cold-roll formed prototype).

Test specimens shall be fully described prior to testing and any dents or other defects shall be noted and the condition of welds, if any, inspected and described. All cross-sectional dimensions of each specimen shall be measured prior to testing at several points along the length and photographs of specimens shall be taken prior to, during, and after testing.

NOTE: The purpose of these tests is for design and not for purchase acceptance tests.

7.1.2 Testing apparatus and fixtures

The tests shall be carried out in a testing machine, or by means of hydraulic jacks in a test frame, or by application of measured weights. The testing machine or load-measuring apparatus shall comply with the requirements of AS 4100.

The mass of load distribution beams and other fixtures shall be measured and included in evaluating the test data.

7.1.3 Instrumentation

Dial gauges or other deflection measuring devices are required at appropriate points to obtain proper alignment and to measure load-deflection behaviour accurately.

Strain gauges may be used if behaviour characteristics other than ultimate loads and load-deflection relations are desired. Extensometers shall be used for coupon tests.

For members subject to twisting (such as channel and zed sections), the twist angle shall be measured.

7.1.4 Reduction and presentation of test data

For each test, the report shall include the following:

(a) A sketch of the specimen with all dimensions.

(b) A sketch of the test set-up with all dimensions, including locations and kinds of gauges, loading and support arrangements and an identification of the loading apparatus, such as testing machine and jacks, with information on the range used and the smallest increment readable for that range.

(c) The results of the coupon tension tests shall be presented in the form of a table of elongation versus load or, alternatively, strain versus stress. Yield stress and ultimate strength shall be determined by any of the methods prescribed in AS 4100 and AS/NZS 4600.

    NOTE: It is desirable to include stress-strain curves in the data presentation.

For presentation of the results of the test, all load, deflection and other recorded data shall be reduced to actual values by correcting, where appropriate, for initial readings, weights of loading apparatus, e.g. loading beams.

These reduced measurements shall be presented in tables showing load versus the particular measured quantity, e.g. deflection or strain. In these same tables, observations of special events, such as flange buckling or connection failure, shall be noted at the particular load at which they occurred.

Graphic presentation of load-deformation curves is advisable, at least for the mid-span deflections. Depending upon observations made during the test and on inspection of tabulated data, graphic presentation of selected or all other load-deformation data is desirable.

7.2 EVALUATION OF TESTS FOR DETERMINING STRUCTURAL PERFORMANCE

7.2.1 Interpolation of test results

7.2.1.1 *Corrections to test results*

Raw test results shall be adjusted to account for differences between the actual thickness of the material used in the test and the nominal thickness specified by the manufacturer, and to account for the difference between the yield stress of the material in the test sample and the minimum yield stress guaranteed by the manufacturer. The way in which adjustments are to be made depends upon the nature of the test being made, and is described for each test separately in Clauses 7.3 to 7.10.

When samples are prepared for tensile tests to determine the yield stress of the material, they shall be cut from an undamaged region of the test piece, away from heat affected zones, and away from bends in the section and from other areas where cold working effects can influence the result.

NOTE: Alternatively the test pieces may be cut from the original coil, before cold-forming.

7.2.1.2 *Corrections to failure loads or moments*

The following procedure shall be adopted, when specified in Clauses 7.3 to 7.10, to apply a correction to the failure load or failure moment due to variations in the yield stress of the material and the thickness of the test specimen.

$$R_{ni} = R_{ti}\left(\frac{f_y}{f_t}\right)^{\alpha}\left(\frac{t}{t_t}\right)^{\beta} \qquad \text{. . . 7.2.1.2}$$

where for the specimen:

R_{ni} = the corrected failure load or moment for test number i

R_{ti} = the measured failure load or moment for test number i

f_t = the measured yield stress for the specimen

f_y = the nominal yield stress

t_t = the measured thickness for the specimen

t = the nominal thickness

α = 0 when $f_y \geq f_t$

= 1.0 when $f_y < f_t$.

Where not otherwise specified in Clauses 7.3 to 7.10:

β = 0 for $t \geq t_t$

$$\beta = \frac{\frac{b_p}{t}}{k\sqrt{\frac{E}{f_t}}} \text{ but } 1 \leq \beta \leq 2 \text{ for } t < t_t$$

where

k = 0.64 for stiffened elements

= 0.21 for unstiffened elements

b_p = the notional plane width of plate element.

7.2.1.3 *Derivation of characteristic values*

After the individual results of a group of tests have each been corrected for variations in thickness and yield stress, the characteristic value of the parameter being measured, R_k, shall be calculated as follows:

$$R_k = R_m - k_s s \qquad \text{. . . 7.2.1.3 (1)}$$

where

R_m = the mean value of the adjusted test results

$$= \frac{1}{n}\sum_{i=1}^{n} R_{ni} \qquad \text{. . . 7.2.1.3 (2)}$$

R_{ni} = individual test result, corrected for thickness and yield stress (Clause 7.2.1.2)

n = number of tests results in the group ($n \geq 3$)

s = the standard deviation of the adjusted test results

$$= \sqrt{\frac{1}{(n-1)}\sum_{i=1}^{n}(R_{ni} - R_m)^2} \qquad \text{. . . 7.2.1.3 (3)}$$

k_s = coefficient given in Table 7.2.1.3.

TABLE 7.2.1.3

k_s COEFFICIENT BASED ON 95% FRACTILE AT A CONFIDENCE LEVEL OF 75%

Number of tests n	k_s
3	3.37
4	2.63
5	2.33
6	2.18
7	2.08
8	2.00
9	1.95
10	1.92
15	1.82
20	1.76
30	1.73
40	1.71
50	1.69
100	1.68
∞	1.64

7.2.1.4 *Characteristic values for a family of tests*

A family of tests shall consist of a series of tests in which (normally) one design parameter (e.g. span, thickness) is varied. This Clause enables a family of test results to be treated as a single entity.

In order to carry out the evaluation of the characteristic strength, a suitable expression which defines the relationship between the test results and one or more relevant parameters in the test series shall be used. This design expression may be based on the appropriate equations of structural mechanics or on an empirical basis.

The design expression is considered to be the mean value of the test results. The standard deviation s_n is calculated for the whole test series after first normalizing the test results by dividing each result by the corresponding value in the design expression. The characteristic strength R_k for a particular set of parameters within a family is given by:

$$R_k = R_m (1 - k_s s_n) \quad \text{. . . 7.2.1.4}$$

where

R_m = value given by the design expression

k_s = value given in Table 7.2.1.3 with n = total number of tests in the family

s_n = standard deviation of the normalized test results.

NOTE: The more accurately the design expression reflects the mean measured strength, the more favourable the values resulting from the evaluation. The coefficients in the design expression may be adjusted in order to optimize the correlation.

7.2.2 Derivation of design capacity and procedure to define stiffness and curves

7.2.2.1 *Design capacity*

The failure load or failure moment, R_{ni}, shall be taken to be the maximum corrected load or moment, as indicated in Figure 7.2.2.2 (a), or the corrected load or moment corresponding to four times the first yield deformation if the load or moment-deformation curve does not reach a maximum, as shown in Figure 7.2.2.2 (b).

NOTE: The first yield deformation may be calculated as the deformation at the intersection between a line representing the elastic stiffness deformation and a line representing the inelastic stiffness deformation as illustrated in Figure 7.2.2.2 (b).

For a group of tests, the characteristic failure load or moment R_k shall be calculated in accordance with Clause 7.2.1.3 or Clause 7.2.1.4 as appropriate. The design load or moment is then R_c, where:

$$R_c = \phi R_k \qquad \text{. . . 7.2.2.1}$$

where

ϕ = capacity factor equal to 1.0 for members and 0.9 for connections.

NOTE: Any value of the design load or moment may be chosen less than or equal to the allowable maximum in order to optimize the possibly conflicting requirements for stiffness and strength. Thus, by reducing the design strength, a greater design stiffness may be achieved.

7.2.2.2 *Procedure to derive a bi-linear curve and stiffness*

A bi-linear load or moment deformation relationship consists of a linear stiffness together with a design strength chosen as described by Equation 7.2.2.1.

The stiffness of the assembly shall be obtained as the slope k_{ni} of a line through the origin which isolates equal areas (A1, A2) between it and the experimental curve, below the design load or moment corrected for yield and thickness, R_c, as shown in Figure 7.2.2.2, provided that:

$$k_{ni} \leq 1.15\frac{R_c}{\delta_{ci}} \qquad \text{. . . 7.2.2.2 (1)}$$

where

δ_{ci} = deformation at the design load or moment, R_c, for test number *i.*

NOTE: This provision is designed to limit the difference between the deformation at failure assumed in the model and that indicated by the test, to 15% in cases where the assembly behaves non-linearly.

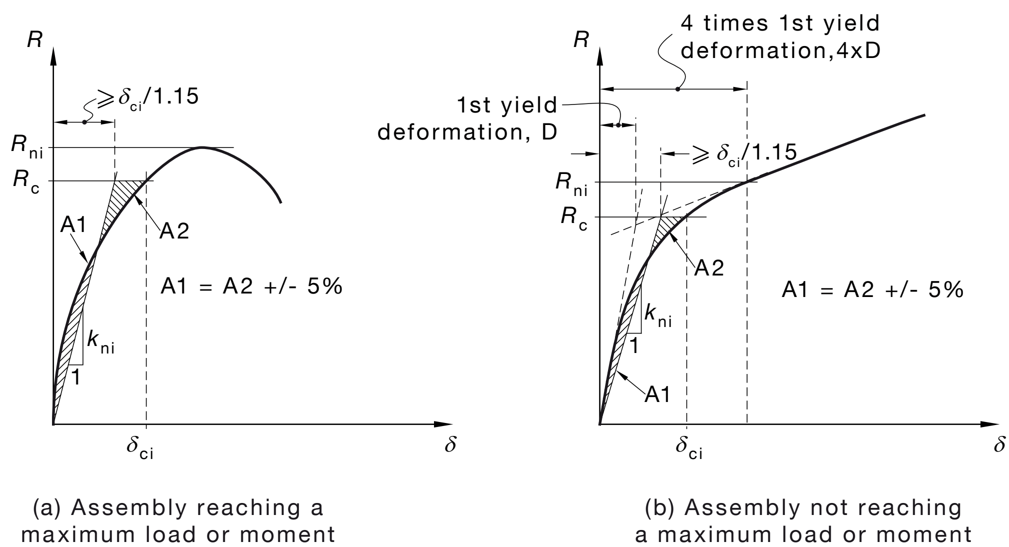


FIGURE 7.2.2.2 DERIVATION OF AN ASSEMBLY STIFFNESS

The design value, k_d, of the stiffness shall be taken as the average value, k_m, where:

$$k_m = \frac{1}{n}\sum_{i=1}^{n} k_{ni} \qquad \text{. . . 7.2.2.2 (2)}$$

where

n = number of tests performed.

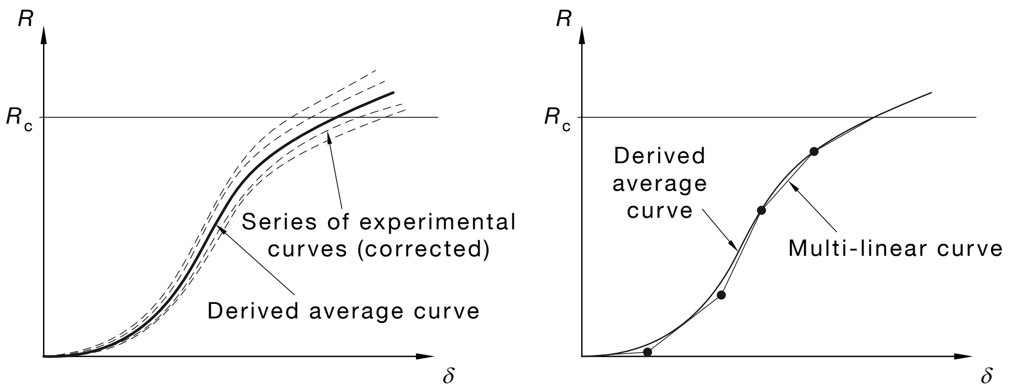
7.2.2.3 *Procedure to derive a multi-linear curve*

For a multi-linear load or moment-deformation curve, the first step is to derive an 'average' curve from the results of a group of tests.

The average curve shall be obtained by plotting the mean value of the deformation at each load or moment increment up to the design value R_c using the load or moment-deformation curves after correction in accordance with Clause 7.2.1.2.

This yields a single curve linking the load or moment and deformation as shown by the solid line in Figure 7.2.2.3 (a).

The multi-linear curve may be obtained by replacing the average curve by a series of straight lines which always lie below the curved line, as illustrated by Figure 7.2.2.3 (b).



(a) Experimental and average curves (b) Average and multi-linear curves

FIGURE 7.2.2.3 DERIVATION OF A MULTI-LINEAR CURVE

7.3 TESTS ON UPRIGHTS

7.3.1 Stub column tests

7.3.1.1 *Test specimen and procedure*

The form factor (Q) of perforated compression members shall be determined by stub upright tests as described in AS/NZS 4600. The ends of the stub upright shall be milled flat (to tolerance of +0.025 mm) and perpendicular to the longitudinal axis of the upright. The axial load shall be applied by flat plates bearing (not welded or otherwise connected) against the milled ends. For the purpose of determining Q, only the ultimate strength of the stub upright needs to be determined.

7.3.1.2 *Evaluation of test results*

The form factor (Q) shall be calculated as follows:

$$Q = \frac{R_m}{f_t A_{\text{net min}}} \qquad \text{. . . 7.3.1.2(1)}$$

where

f_t = measured yield stress of the upright material if no cold work effects are to be considered, or the weighted average yield stress f_{ya}, calculated in accordance with AS/NZS 4600, if cold work effects are to be considered

$A_{net\ min}$ = minimum cross-sectional area based on measured dimensions obtained by passing a plane through the upright normal to the axis of the upright

R_m = mean compressive strength of the stub column tests.

In no case shall the form factor (Q) be greater than 1.0.

Where a series of sections with identical cross-sectional dimensions, identical hole dimensions and locations, and identical material grade are produced in a variety of thicknesses, stub upright tests need only to be made for the largest and the smallest thicknesses (t_{max} and t_{min}). Q values for intermediate thicknesses shall then be determined by linear interpolation according to the following equation:

$$Q = Q_{min} + \frac{(Q_{max} - Q_{min})(t - t_{min})}{(t_{max} - t_{min})} \qquad \text{. . . 7.3.1.2(2)}$$

where

Q = form factor for the intermediate thickness t

Q_{min}, Q_{max} = form factors obtained by test in accordance with this Clause for the smallest (t_{min}) and largest (t_{max}) thicknesses, respectively.

This interpolation is permissible only if the yield stresses of the two test specimens do not differ by more than 25% and if the yield stresses of the intermediate thicknesses fall between or below those of the t_{min} and t_{max} test specimens.

7.3.2 Compression tests on uprights—Determination of buckling curves

7.3.2.1 *Purpose of test*

The purpose of the test shall be to determine the axial load capacity of the upright section for a range of effective lengths in the down-aisle direction, taking into account out-of-plane buckling effects and the torsional restraint provided by the bracing and its connection to the uprights. This Clause also specifies a test to determine the influence of the distortional buckling mode.

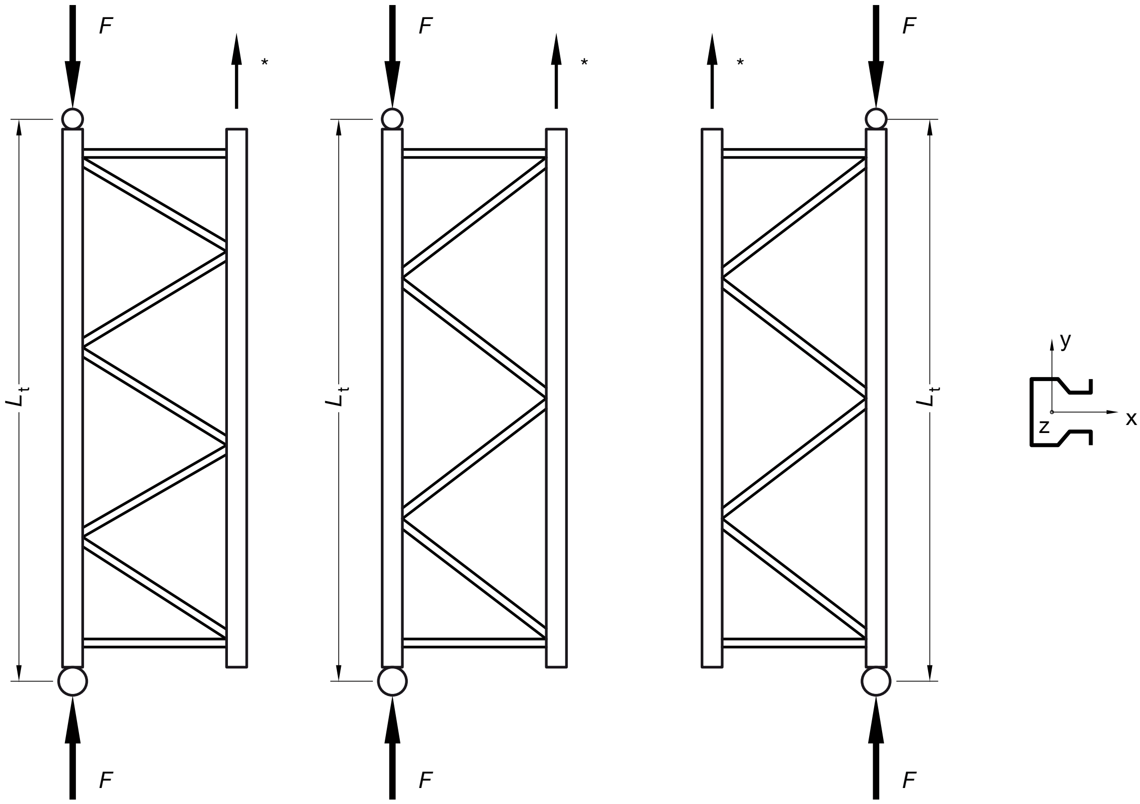
NOTE: The results of this test series provide a column curve which is a plot of the buckling reduction factor χ against the non-dimensional slenderness ratio λ. The value of λ is obtained from the slenderness corresponding to the out-of-plane buckling mode = L/r_x, irrespective of whether the failure mode is a distortional, flexural-torsional or in-plane buckling mode. The purpose is to allow the column curve to be used in the design and relating buckling loads to down-aisle buckling lengths alone. It may be noted that the procedure is conservative in that no account is taken of the restraining effects of the beam end connectors.

7.3.2.2 *Test arrangement*

The test arrangement comprises a frame assembly using the maximum frame width specified for the product, in which one of the two uprights is loaded axially (see Figure 7.3.2.2(a)). The bracing pattern, bracing sections and bracing connections shall be the standard ones used with the product. The upright shall be loaded through ball bearings and fitted with base and cap plates, the upright section shall not be adjusted for springback in any way. The position of the ball bearings in relation to the cross-section shall be the same at both ends of the upright, but may be adjusted to give the maximum failure load.

As an alternative to this test method, a complete frame assembly may be tested in compression as shown in Figure 7.3.2.2(b).

NOTE: If a particular upright may be used with a variety of bracing arrangements, the test should be carried out with the least effective arrangement that is likely to be used in practice. If this is not done, the test results may be taken to be valid only for the tested arrangement and more effective alternatives.



* Independent support of weight of upright frame

FIGURE 7.3.2.2(a) COMPRESSIVE TEST ON UPRIGHTS

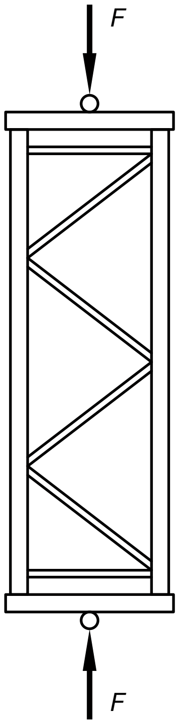


FIGURE 7.3.2.2(b) ALTERNATIVE COMPRESSIVE TEST ON UPRIGHTS

7.3.2.3 *Test method*

The upright shall be tested in a range of lengths, the smallest of which shall just allow a single bracing panel; the longest length shall correspond to a slenderness ratio λ of 1.50 for down-aisle buckling, and a minimum of three other test lengths shall be chosen equally spaced between these two extremes. In the test, the load shall be increased to failure. The failure mode shall be recorded.

For short lengths, care shall be taken to ensure that the chosen bracing arrangements give the worst case.

7.3.2.4 *Corrections to observations*

Corrections to each measured value of failure load shall be made in accordance with the provisions of Clause 7.2.1.2, in which the measured failure loads shall be adjusted to take account of the actual thickness and yield stress of the sample, so that—

$$R_{\mathrm{ni}} = R_{\mathrm{ti}} C^{\alpha} \left(\frac{t}{t_{\mathrm{t}}} \right)^{\beta} \quad \ldots 7.3.2.4(1)$$

where

$$C = \frac{f_{\mathrm{y}}}{f_{\mathrm{t}}} \quad \text{for } 0 \leq \lambda \leq 0.2 \quad \ldots 7.3.2.4(2)$$

$$= \frac{\lambda - 0.2 + \left(f_{\mathrm{y}} / f_{\mathrm{t}} \right)(1.5 - \lambda)}{1.3} \quad \text{for } 0.2 \leq \lambda \leq 1.5 \quad \ldots 7.3.2.4(3)$$

$$= 1.0 \quad \text{for } 1.5 \leq \lambda \quad \ldots 7.3.2.4(4)$$

$$\lambda = \frac{L_t/r_x}{\pi\sqrt{\frac{E}{f_y}}} \qquad \ldots 7.3.2.4(5)$$

Other terms are E and f_y as defined in Clause 7.2.1.2 and L_t and the x-axis in Figure 7.3.2.2(a).

7.3.2.5 *Derivation of the column curve*

All compression test results shall be considered in this Clause.

The procedure shall be as follows:

(a) For each test, the values of the stress reduction factor χ_{ni}, and the non-dimensional slenderness ratio λ_{ni} shall be computed, where:

$$\chi_{ni} = \frac{R_{ni}}{QA_{net\ min}f_y} \qquad \ldots 7.3.2.5(1)$$

and

$$\lambda_{ni} = \frac{L_{ti}/r_x}{\pi\sqrt{E/f_y}}\sqrt{\frac{QA_{net\ min}}{A_g}} \qquad \ldots 7.3.2.5(2)$$

where

R_{ni} = the adjusted failure load for test number i

L_{ti}/r_x= slenderness ratio for flexural buckling about the major axis for test i

Q and $A_{net\ min}$ are in accordance with Clause 7.3.1.2.

(b) A graph shall be plotted of χ_{ni} against λ_{ni}.

(c) A suitable algebraic expression shall then be chosen for χ_{cu} (= $\chi_{cu}(\lambda_{ni})$) to represent the locus of mean values of the test results, χ_{ni}. This expression shall not have more than five independent coefficients. This may be done by using a least squares curve fitting process or by sketching a best fit smooth curve in by hand.

(d) The individual values, χ_{ni}, should be normalized by dividing each one by the corresponding mean value, χ_{cu}. The standard deviation, s, of these normalized values may then be calculated.

(e) The characteristic value of the stress reduction factor, χ, shall then be determined using:

$$\chi = \chi_{cu}(1 - k_s s) \qquad \ldots 7.3.2.5(3)$$

where

k_s = given in Table 7.2.1.3 based on the total number of test results.

This column curve is valid over the range of column lengths tested. For column lengths outside the tested range, the axial capacity should be calculated according to Clause 5.2.3, but before this is done, the distortional buckling check in Clause 7.3.3 should be carried out using the results from the tests on the single panel frames.

7.3.3 Test for the effects of distortional buckling

7.3.3.1 *Purpose of the test*

The purpose of the test is to determine the influence of the distortional buckling mode on the axial load capacity of the upright section. The test result provides a means of correcting the theoretically determined axial load capacity according to Clause 5.2.3.4.

NOTE: If compression tests of uprights according to Clause 7.3.2 are carried out over the complete range of upright lengths, the effects of distortional buckling are included in the buckling curves so that tests according to this Clause need not be carried out.

7.3.3.2 *Test arrangement and method*

At least three tests should be carried out on nominally identical single uprights with cap plates, as described in Clause 7.3.2.2. The length L of the upright shall be equal to the length of the single bracing panel closest to 1 m. Where a frame has a variable panel length then each panel length per upright combination shall be tested. The test may also be made on a single panel frame using appropriate lateral restraints as part of a series of tests made to determine the buckling curve for the upright, described in Clause 7.3.2.

Care shall be taken to centre the specimen on the axis of the test machine to ensure that the applied load is uniformly distributed over the specimen end surfaces. The column ends shall rest on flat steel plates, as per Clause 7.3.11, or on a spherical surface with a point contact, or on pins in mutually perpendicular directions, such that the resultant of the axial load is applied through the centroid of the gross section.

If significant twisting is observed at the ends of the specimen, the ends should be restrained in order to resist this twisting; this restraint should not offer any additional resistance to distortion of the section.

7.3.3.3 *Derivation of the test results*

The results of these tests shall be corrected for yield stress and thickness in accordance with Clause 7.3.2.4.

The characteristic failure load, R_k, shall then be derived as described in Clause 7.2.1.3 and the corresponding design strength, R_c, calculated as per Clause 7.2.2.1. This test value is used in Clause 5.2.3.4 to check for the effects of distortional buckling.

7.3.4 Bending tests on upright sections

7.3.4.1 *Purpose of the test*

The purpose of the test is to determine the moment of resistance of an upright section about its major and minor axes of bending.

7.3.4.2 *Test arrangement*

The test shall be carried out by loading the upright section in bending as shown in Figure 7.3.4.2. The span, L, of the upright shall be such that:

$$L \geq 30d_u \qquad \text{. . . 7.3.4.2}$$

where

d_u = depth of the upright being tested.

The test shall be made to measure the bending strength of the upright about either the major or minor axis. When the test is made to determine the bending strength about the axis of symmetry, then a complete frame shall be tested with the two upright sections linked together by the normal bracing system, with the section free to twist at the supports, as shown in Figure 7.3.4.2. This test arrangement permits flexural torsional buckling effects to occur that are similar to those developed by the upright in its normal mode of use. The applied loads and their reactions for each upright shall always be in the same vertical plane. This plane may be defined by the shear centre or the centroid of the section.

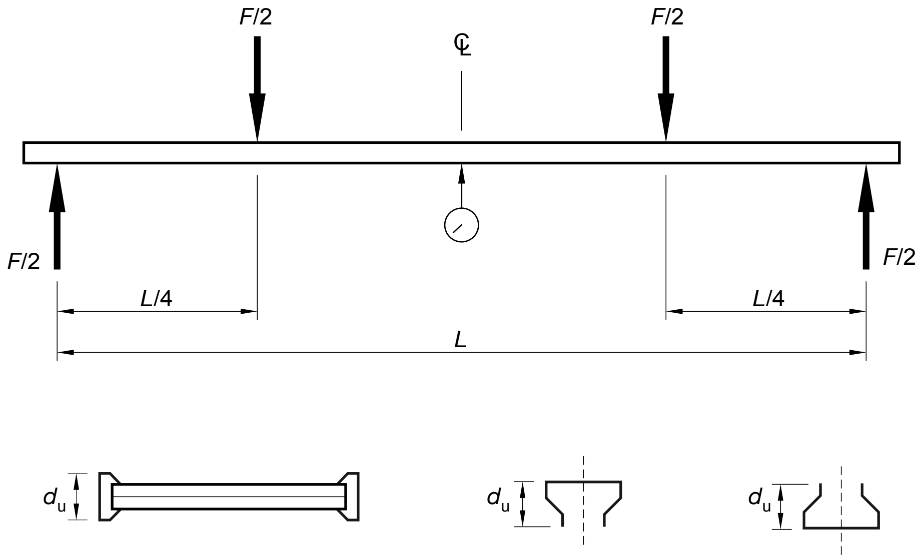


FIGURE 7.3.4.2 TEST ARRANGEMENTS

7.3.4.3 *Test method*

The load shall be applied in increments up to failure to the quarter points of the span through spreaders large enough to prevent any local crushing of the section.

7.3.4.4 *Corrections to the observations*

The failure moment measured in the test shall be adjusted to take account of variations in thickness and yield stress of the uprights in accordance with Clause 7.2.1.2.

7.3.4.5 *Derivation of results*

The design value of the moment of resistance shall be calculated in accordance with Clause 7.2.2.1.

7.4 PALLET BEAM TESTS

7.4.1 Simply supported pallet beam tests

7.4.1.1 *General*

These tests are acceptable only for beams that are not subject to significant torsional stresses or distortions.

The simply supported pallet beam test shall be made only if the flexural behaviour parameters such as yield moment, ultimate moment and the effective rigidity (EI) are to be determined. If lateral restraints are required, the beams shall be tested in pairs as they would be used in completed assemblies.

7.4.1.2 *Test set-up*

The test set-up consists of a beam test specimen simply supported at each end (not connected to uprights). The test load is applied to a load distribution beam which in turn imposes a load at two points on the beam specimen. Each load point on the beam test specimen is set at a distance of S/C from the support, where S is the span, in millimetres, and C is a numerical value between 2.5 and 3.

Plates shall be used to prevent local failure at supports or at load points.

7.4.1.3 *Test procedure*

After alignment, a small initial load of about 5% of the expected ultimate load shall be applied to the test assembly to ensure firm contact between the specimen and all load and support components. At this load, initial readings shall be taken from all gauges. Loads shall then be applied in increments no larger than about one fifth of the expected design load. Readings shall be taken for all load increments.

NOTES:

1 It is good practice to plot load versus mid-span deflection readings at each load increment during testing.

Noticeable deviation from straightness of such a plot will indicate incipient inelastic behaviour or local buckling or crippling. When such is the case, load increments shall be reduced to no more than half the initial increments.

2 It is good practice to measure permanent set for loads within the interval of 25% of the expected design load by reducing, within this interval, the ratio of the applied load to the initial load after each increment. Appropriate gauge readings are to be taken at this reduced load to determine permanent set.

When deflection increments for given load increments increase rapidly, this indicates the approach of ultimate failure load. If sudden failure is possible because of the nature of the specimen, and if such sudden failure may damage the gauges, the gauges shall be removed.

On the other hand, if a gradual failure is expected, such as by simple yielding, it is desirable to measure at least centre-line deflections right up to and past the maximum or ultimate load, to obtain some part of the descending portion of the load-deflection curve.

All specific events noticeable by visual inspection, such as local buckling, crippling or failure of connections, shall be recorded at the loads at which they occur.

7.4.1.4 *Evaluation of test results*

The parameters investigated shall be determined from the test results derived from Clauses 7.2.1 and 7.2.2.

7.4.2 Bending tests on beams

7.4.2.1 *Purpose of test*

The purpose of the test shall be to measure the bending strength of a beam and the beam rotation about its longitudinal axis under the service action. The test for beam strength shall be designed primarily for beams with only one axis of symmetry which may be susceptible to flexural torsional buckling.

7.4.2.2 *Test arrangement*

The test assembly shall comprise a pair of beams supported on frames using standard connectors as shown in Figure 7.4.2.2. For the beam rotation test, the beam span shall be at least equal to 50 times the width of the beam section. The beams shall be linked together by pallet support beams, fork entry bars, beam ties or any other component which is incorporated in the least favourable arrangement specified by the manufacturer. The load pattern shall be that which occurs in practice, as exemplified in Figure 7.4.2.2. Alternatively, as a standard test to determine the stability of the section, the load shall be applied at the quarter points of the span as shown in Figure 7.3.4.2. In this case, the load shall be applied through platens of maximum width 100 mm to reduce the tendency for web crippling.

Where necessary, the horizontal movements of the supports shall be taken into account in the interpretation of the test.

If the test is being conducted to evaluate the effects of possible flexural torsional buckling in a beam section with only one axis of symmetry, either the normal operating conditions for lateral restraint of the compression flange shall be utilized in the test or, if a range of conditions is to be covered by the test results, the most disadvantageous of the range shall be tested. Where pallets are deemed to provide lateral support to the compression flange, it shall be permissible to apply the action through pallets or using an equivalent substitute arrangement. The test shall be carried out on a range of spans corresponding to the range in which the beam is supplied.

One frame shall be supported on a pinned support at its base and held in position, whilst the other frame shall be supported on rollers so that it is free to move horizontally so that no horizontal force, and hence no moments, can develop in the upright.

NOTES:

1 It is important that the load media interact with the beams in the same way as in practice. For instance, flexible pallets, or stored products such as tyres, may tend to load the beams horizontally as well as vertically. This situation should be accurately modelled in such tests.

2 Loading devices should be free to sway with the structure under test.

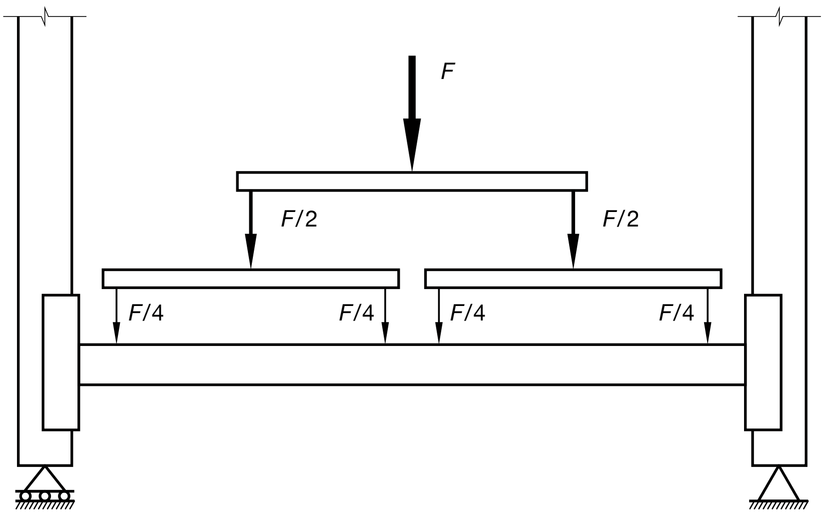


FIGURE 7.4.2.2 ARRANGEMENT FOR BENDING TEST ON A BEAM

7.4.2.3 *Test method*

The load shall be increased to the service action for the beams, and the absolute twist rotation (θ_{ti}) of the beam about its longitudinal axis shall be measured at centre span. This measurement shall be made using an inclinometer or displacement transducer mounted independently of the test structure. The central deflection of the beams shall be measured to estimate the flexural rigidity of the beam taking into account the actual distribution of action in the span. Once measurements of beam deflection and load have been made, the load shall be increased until failure occurs and the failure moment in the beam (M_{ti}) calculated.

7.4.2.4 *Corrections to observations*

Corrections to the measured value of the beam twist rotation (θ_{ti}) shall be made as follows:

$$\theta_{ni} = \theta_{ti}\left(\frac{t_t}{t}\right)^3 \quad \text{for open sections} \qquad \ldots 7.4.2.4(1)$$

$$\theta_{ni} = \theta_{ti}\left(\frac{t_t}{t}\right) \quad \text{for closed sections} \qquad \ldots 7.4.2.4(2)$$

and $\theta_{ni} \geq \theta_{ti}$

where

θ_{ti} = measured value of the central twist rotation in the serviceability limit state

θ_{ni} = corrected value of the central twist rotation.

Corrections to the measured failure moment (M_{ti}) shall be calculated in accordance with Clause 7.2.1.2.

7.4.2.5 *Derivation of test results*

The design value of the beam twist rotation shall be taken to be the average from a minimum of three tests. The characteristic value of the moment of resistance shall be calculated in accordance with Clause 7.2.2.1.

7.5 PALLET BEAM TO UPRIGHT CONNECTION TESTS

7.5.1 Cantilever test

7.5.1.1 *General*

This test shall be to determine the connection moment capacity and stiffness.

7.5.1.2 *Test set-up*

The test set-up shall be as follows:

(a) A short length of upright shall be connected to a stiff testing frame at two points with a clear distance, h, between them, where

$h \geq$ beam connector length + 2 × column face width.

Over this distance there shall be no contact during the test between the upright and the testing frame. A short length of beam shall be connected to the upright by means of the connector to be tested, and beam safety locks shall be in place. A typical example of a suitable test set-up is shown in Figure 7.5.1.2.

(b) Sideways movement and twisting of the beam end shall be prevented by a lateral restraint guide which, however, allows the beam component to move freely in the direction of the load. Alternatively, a pair of connectors may be tested in parallel.

(c) The load shall be applied at a lever arm, a, of 400 mm from the face of the upright by an actuator and loading bar at least 750 mm long between pinned ends, as shown in Figure 7.5.1.2.

(d) The rotation shall be measured by two displacement transducers (transducers 1 and 2 in Figure 7.5.1.2) symmetrically clamped to a plate fixed to the pallet beam and with the tips of the transducers resting on brackets fixed to the upright, so as to record the rotation of the beam relative to the upright.

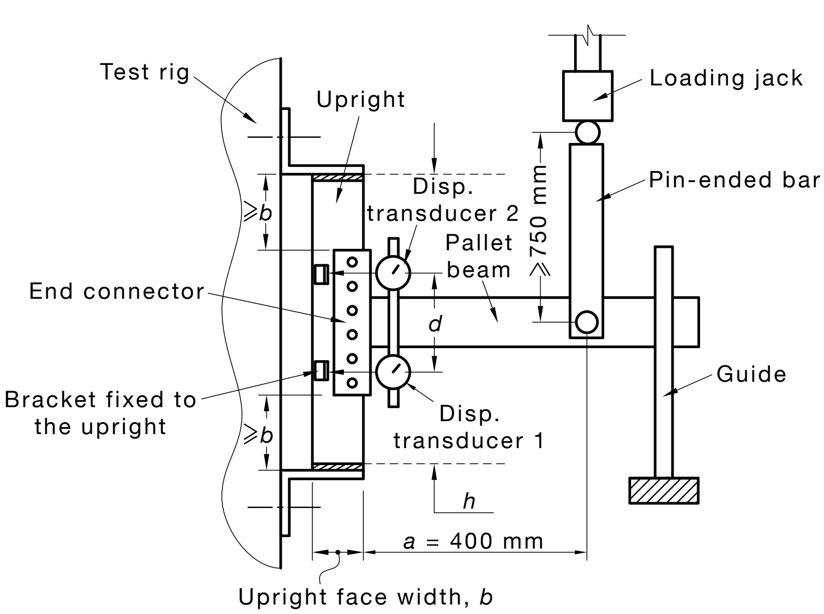


FIGURE 7.5.1.2 SET-UP FOR BEAM END CONNECTOR CANTILEVER TEST

7.5.1.3 *Test procedure*

The test set-up described in Figure 7.5.1.2 loads the connector vertically downwards. Separate values for the stiffness and strength shall be obtained for both right and left hand connectors and the mean value used in design.

An initial load, F, equal to 10% of the anticipated failure load may be applied to the assembly and then removed as a preload in order to bed in the components. The transducers should be reset thereafter. The load, F, shall then be increased gradually until the maximum load is reached and the connection fails. The rotation of the connection shall be measured and, for each test, a plot of the moment M_t versus the rotation θ_t shall be made, in which:

$$M_t = a\,F \qquad \ldots 7.5.1.3\ (1)$$

and

$$\theta_t = \frac{\delta_1 - \delta_2}{d} \qquad \ldots 7.5.1.3\ (2)$$

where

a = lever arm for the load F

d = distance between the transducers 1 and 2 as shown in Figure 7.5.1.2

δ_i = displacement of transducer number i.

7.5.1.4 *Corrections to the observations*

The yield stress and thickness of the materials of the beam, upright and connector shall be determined and the correction factor C_m calculated where

$$C_m = \left(\left(\frac{f_y}{f_t}\right)^{\alpha}\left(\frac{t}{t_t}\right)\right)_{max} \text{ but } C_m \leq 1.0 \qquad \ldots 7.5.1.4$$

where

f_t = the measured yield stress for the relevant component

f_y = the nominal yield stress for the relevant component

t_t = the measured thickness for the relevant component

t = the nominal thickness for the relevant component

α = 0 when $f_y \geq f_t$

= 1.0 when $f_y < f_t$.

Unless the beam fails or the beam yield strength is more than 1.25 times the guaranteed strength, the relevant component is either the beam end connector or the upright. Of these two components, the one that gives the largest correction to the test values, irrespective of which component was observed to fail, shall be used. If the beam fails, the correction relating to beam materials shall also be taken into account. Corrections of 15% and below may be ignored.

In order to make corrections to the observations, the moment-rotation (M_t-θ_t) curve for each test shall be separated into two components, one to represent the elastic deformations and the other the inelastic deformations of the connection.

The procedure shall be as follows:

(a) Plot the unadjusted test results as moment-rotation curve (M_t-θ_t).

(b) Measure the slope of this curve (k_0) at the origin.

(c) From the measured rotations, θ_t, subtract the elastic rotations M_t/k_0 to obtain the plastic rotations θ_p.

(d) Calculate the corrected moments $M_n = CM_t$, where $C = 0.15 + C_m$ and C is less than or equal to 1.0.

(e) Add back the elastic rotations, M_n/k_0, to give new rotations $\theta_n = \theta_p + M_n/k_0$.

(f) Plot the adjusted moment-rotation curve (M_n-θ_n).

NOTES:

1 The adjusted moment-rotation curve has the same initial slope, k_0, as the original measured curve.

2 An algebraic expression may be used to represent the plotted curve of the moment as a function of the rotation. This expression should not have more than five independent coefficients. This may be achieved by using a least squares curve fitting process.

7.5.1.5 *Evaluation of test results*

The connection design moment capacity shall be determined from the test results according to Clause 7.2.2.1.

NOTE: When deriving a multi-linear curve according to Clause 7.2.2.3, if the looseness is omitted from the calculation of the frame imperfections in Clause 3.3.2, the looseness measured in accordance with Clause 7.5.3 shall be added to the average moment-rotation curve, obtained as a horizontal or nearly horizontal line.

7.5.2 Portal test

7.5.2.1 *General*

This test shall be used to obtain the connection stiffness needed for a semi-rigid frame analysis.

7.5.2.2 *Test set-up*

The test set-up shall consist of two upright frames supported on four half-round bars, one under the base of each upright, and two beams, the top of which are installed at a distance of at least 600 mm from the floor, and including front-to-back ties when specified. The half-round bars shall be located on the centroidal axes of the uprights perpendicular to the beams. The bases of the uprights shall be held against lateral displacement but not against rotation.

7.5.2.3 *Test procedure*

After the racking is assembled, a load equal to the design load of the beams shall be placed on the beams, simulating usual vertical action. Horizontal forces equal to the horizontal design load and corresponding to the vertical load on the assembly shall be applied to the assembly in increments, equally distributed between the two uprights of an upright frame on one side, at the level of the top of the beams, and in the direction of the beams. For each upright frame, deflection due to the horizontal action shall be measured by two displacement transducers (transducers 1 and 2 in Figure 7.5.2.3) at the level of the top of the beam. Figure 7.5.2.3 illustrates the complete test set-up.

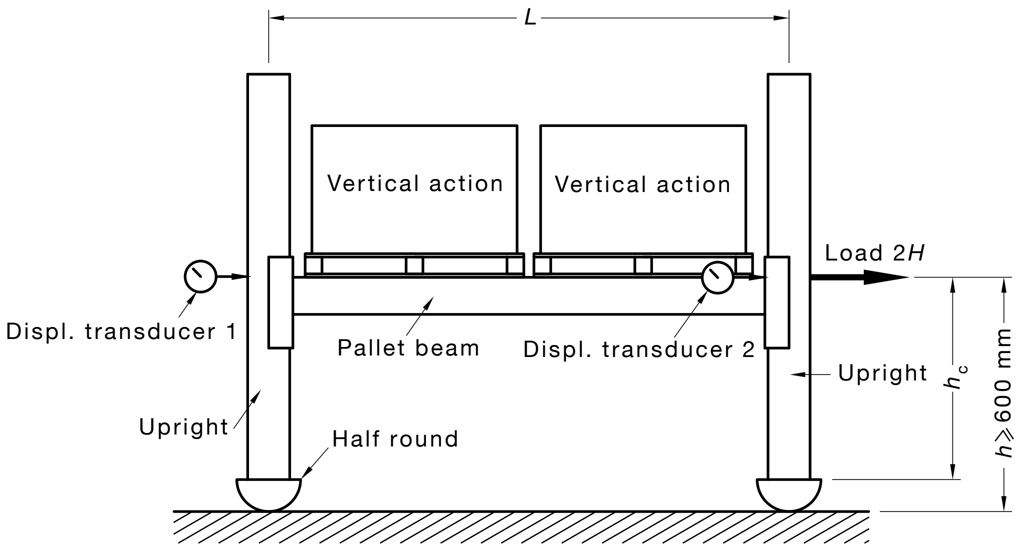


FIGURE 7.5.2.3 PORTAL TEST SET-UP

7.5.2.4 *Evaluation of test results*

Considering two portal frames of height h and span L with second moments of area of the uprights and beams designated I_c and I_b, respectively, an expression for the average pallet beam to upright connection stiffness, k, corresponding to a horizontal load of $2H$ applied to the assembly is as follows:

$$k = \frac{1}{\frac{2}{Hh^2}\left[\Delta - \frac{H(3h-2h_c)(h-h_c)}{12k_c}\right] - \frac{1}{6k_c}\left(\frac{3h-h_c}{h}\right) - \frac{1}{6k_b}} \qquad \ldots 7.5.2.4$$

where

h = distance from the floor to the top of the pallet beam

h_c = distance from the top of the half round to the top of the pallet beam

H = horizontal load per pallet beam

Δ = average measured side-sway displacement of the frame at the top of the pallet beam

$= \frac{\delta_1 + \delta_2}{2}$

δ_i = displacement of transducer number i

k_c = upright stiffness

$$= \frac{EI_c}{h_c}$$

k_b = pallet beam stiffness

$$= \frac{EI_b}{L}$$

E = modulus of elasticity

I_c = second moment of area of the upright about the axis perpendicular to the plane of bending

I_b = second moment of area of the pallet beam perpendicular to the plane of bending

L = distance between the centroids of the two upright frames

Since the behaviour at both the design load and the ultimate load is of interest, portal tests are to be conducted at both load levels.

7.5.2.5 *Corrections to the observations*

Corrections to the observations are not required.

7.5.2.6 *Derivation of test results*

The design value of joint stiffness shall be calculated in accordance with Clause 7.2.2.1.

7.5.3 Looseness tests on beam end connectors

7.5.3.1 *Purpose of the test*

The purpose of this test shall be to obtain a value of the looseness of the connection (ϕ_ℓ) for use in the design calculations as specified in Clause 3.3.2.1.

7.5.3.2 *Test arrangement*

The same test arrangement as that used for the measurement of beam end connector strength and stiffness, specified in Clause 7.5.1, shall be used, except that the loading jack shall be double acting and capable of applying the load in the reverse direction, or counterbalancing dead weights shall be used to obtain the same effect.

7.5.3.3 *Test method*

The load (F) shall be slowly increased until the moment at the connector (equation 7.5.1.3(1)) reaches a value equal to 10% of the design moment (M_c) specified in Clause 7.5.1.5. The deflection shall be measured. The load shall then be reduced and reversed until a negative moment of $0.1M_c$ is applied. The load shall then be removed. Typical output from such a test is shown in Figure 7.5.3.3.

The looseness shall be measured by extrapolating the linear parts of the moment-rotation curves towards the origin until they intersect the rotation axis as shown in Figure 7.5.3.3. The difference between the two intersection points so obtained shall be equal to double the looseness of the connector.

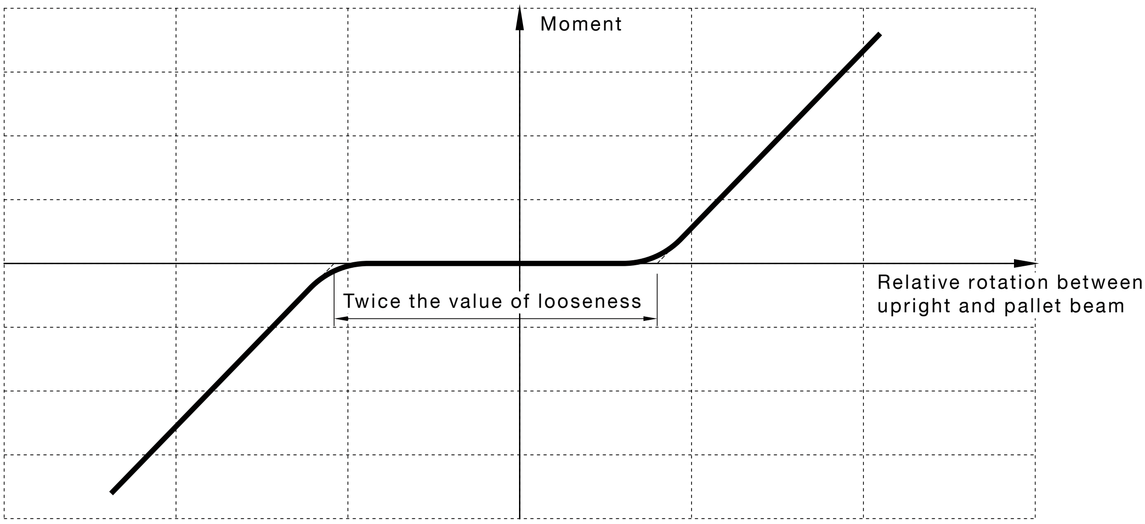


FIGURE 7.5.3.3 CANTILEVER RESULTS FOR CONNECTOR LOOSENESS DETERMINATION

7.5.3.4 *Corrections to the observations*

No corrections shall be made to the observations to account for thickness or strength variations.

7.5.3.5 *Derivation of test results*

The looseness (ϕ_ℓ) shall be taken to be the mean value of the test results.

7.5.4 Shear tests on beam end connectors and connector locks

7.5.4.1 *Purpose of the test*

The purpose of the test shall be to measure the shear strength of the connector and of the connector lock.

7.5.4.2 *Test arrangement*

The test arrangement shall comprise a short length of upright connected rigidly to a stiff frame, with a length of beam section attached to the upright by means of the connection to be tested as shown in Figure 7.5.4.2. The load shall be applied to the connection by a pin-ended jack, placed a distance (a) from the face of the upright and as close to the upright as possible. The free end of the beam shall be restrained by a pinned support 400 mm from the face of the upright, as shown in Figure 7.5.4.2. This support shall be mounted on a mechanical or hydraulic screw, and adjusted in the vertical direction so that the beam remains horizontal during the test. The load and screw jacks shall be aligned with the shear centre of the beam (as in Figure 7.5.4.2), and the action shall be applied across the full width of the top surface of the beam.

To test the connector lock, the test piece shall be installed in the inverted position and, in addition to the load shown in Figure 7.5.4.2, a constant action of 500 N shall be applied to the top surface of the beam in a direction normal to the face of the upright, such as to pull the beam end connector away from the face of the upright. The purpose of this action shall be to take away horizontal looseness in the assembly and thereby create the worst condition for the connector lock.

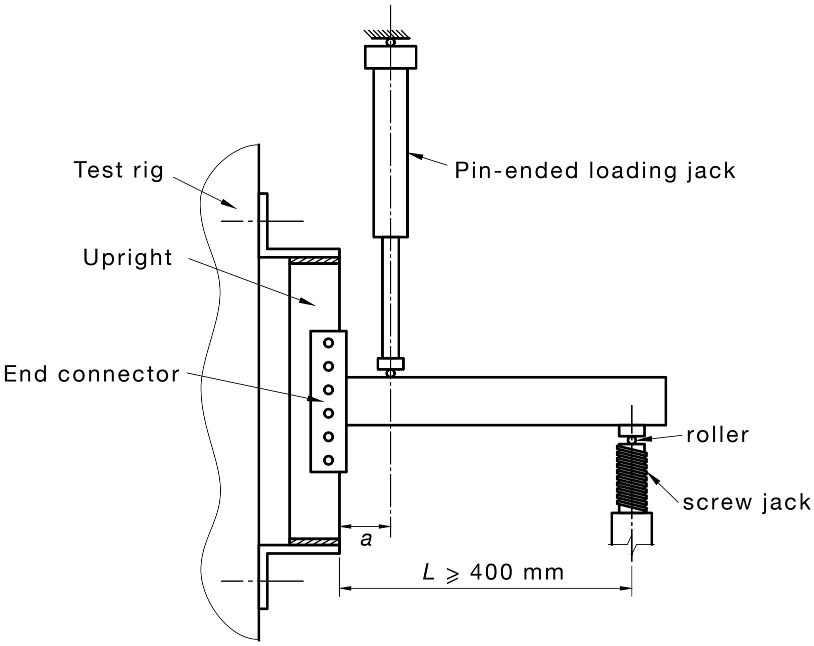


FIGURE 7.5.4.2 ARRANGEMENT FOR THE BEAM END CONNECTOR SHEAR TEST

7.5.4.3 *Test method*

To measure the strength of the beam end connector or the lock, the beam shall be loaded as shown in Figure 7.5.4.2 until the maximum load (F_{ti}) is reached. The strength of the connector (R_{ti}) shall be calculated as follows:

$$R_{ti} = F_{ti}\left(1 - \frac{a}{L}\right) \qquad \text{. . . 7.5.4.3}$$

7.5.4.4 *Corrections to the observations*

The results of these tests shall be corrected for yield stress and thickness in accordance with Clause 7.2.1.2.

7.5.4.5 *Derivation of test results*

The design value of the shear strength of the shear connector and the connector lock shall be determined in accordance with Clause 7.2.2.1.

7.6 UPRIGHT FRAME TEST

7.6.1 General

The upright frame tests are intended to simulate the conditions in the actual racking as closely as possible. The purpose of the test is to determine the ultimate upright frame capacity for an expected upright failure that takes place between the floor and the bottom beam or between the two lower beams in a three beam-level test set-up. The test shall account for vertical and horizontal loads as specified in Clauses 2.2, 2.3 and 2.4 as well as the effects of semi-rigid connections.

7.6.2 Horizontal load in the direction perpendicular to the upright frame

7.6.2.1 *Test set-up for symmetrical loading condition*

The test assembly shall consist of three upright frames not bolted to the floor, and at least two levels of beams connecting the frames together to make two bays of pallet racking. The vertical spacing of the beams shall be the same as in the actual application. When the distance from the floor to the first beam is smaller than the distance between beams, then three levels of beams shall be used.

The upright frame may be as high as desired. However, its construction, consisting of upright and truss web members, shall be of the same cross-section, pattern and spacing as the actual application. The top beam level and its upright connection shall be heavier or reinforced to the degree necessary to carry the test load to the point where the frame fails. The remaining beams and their connections shall be as in the actual application. This test load represents the load from two or more beam levels.

Horizontal loads shall be applied perpendicular to one outside upright frame at the centre-line of the beam connection by means of either hydraulic jacks or by ropes and pulleys with hanging weights attached. The load at each beam level shall be applied equally to each upright of the upright frame.

To measure horizontal displacements, one transducer shall be located at the centre-line of each beam level, one transducer mid-height between each beam level, and another transducer mid-height between the bottom beam level and the floor. All transducers shall be placed on one upright.

7.6.2.2 *Test procedure for symmetrical loading condition*

The test procedure for upright frames shall be as follows:

(a) Align the racking structure so that it is level and plumb, and all components are properly seated.

(b) Take initial transducer readings.

(c) Place a vertical load equal to three quarters times the factored ultimate beam design load on each of the lower beam levels.

(d) Take transducer readings for horizontal movement.

(e) Apply a horizontal load to the upright frame at each level of beams. The horizontal load shall be determined in accordance with Clauses 2.3.4.3 and 2.4.3.

(f) Take transducer readings for horizontal movement.

(g) Apply one additional unit of vertical load to the reinforced top level beams only and take transducer readings for horizontal movement.

(h) Apply one additional unit of horizontal load to the reinforced top level beams only. Take transducer readings for the horizontal movement. If hydraulic jacks are used, be sure the jack at the bottom beam level is always applying the proper load to the upright frame.

(i) Repeat Steps (g) and (h) until failure occurs in the upright frame.

7.6.2.3 *Evaluation of test results for symmetrical load condition*

The tested ultimate load for an upright frame based on combined vertical and horizontal loads shall be the last set of test data which has an equal number of both vertical and horizontal load increments.

The tested ultimate load shall be the lowest of the three tested conditions, namely symmetrical actions as specified in Clause 7.6.2.1, unsymmetrical actions as specified in Clause 7.6.2.4, or for the horizontal action in the direction parallel to the upright frame as specified in Clause 7.6.3.

7.6.2.4 *Test set-up and test procedure for unsymmetrical load condition*

The test set-up and test procedure for the unsymmetrical load condition shall be as specified in Clauses 7.6.2.1 and 7.6.2.2, except that no load shall be placed on one beam level in one bay directly adjacent to the expected upright failure location. The direction of the horizontal load shall be in the direction of the side-sway.

7.6.3 Horizontal load in the direction parallel to the plane of upright frame

7.6.3.1 *Test set-up*

The test set-up shall be as specified in Clause 7.6.2.1 except that the locations of horizontal actions and transducers shall be changed so that the horizontal loads and displacements are in the plane of the upright frame.

7.6.3.2 *Test procedure*

The test procedure shall be as specified in Clause 7.6.2.2, except that in Step (e), the distribution of the horizontal load on each beam level on each upright frame shall be determined in accordance with Clauses 2.3.4.4 and 2.4.3.

7.6.3.3 *Evaluation of test results*

Test results shall be evaluated as specified in Clause 7.6.2.3.

7.7 TESTS FOR SHEAR STIFFNESS OF UPRIGHT FRAMES

7.7.1 Shear test

7.7.1.1 *Purpose of the test*

The purpose of the test shall be to determine the transverse shear stiffness per unit length of upright frame in order to be able to assess the stability and shear strength of the frame.

NOTE: In lieu of testing, the reduction in transverse shear stiffness may be accounted for by reducing the axial stiffness (EA) of the bracing members of the upright frame to 5% of the unreduced value. This procedure is allowed for stability and strength calculations but not for earthquake design or natural frequency calculations.

7.7.1.2 *Test arrangement*

The test sample shall be an upright frame assembly with a number of bracing panels loaded in the manner shown in Figure 7.7.1.2. At least two panels should be used. One upright of the frame shall be pinned at one end so that it is prevented from moving horizontally and the transverse displacements shall be restrained at the two ends of the same upright so that the frame is prevented from rotating. Load is applied along the centroid of the other upright.

Where the product utilizes a range of frame widths, this test shall be made on the most commonly used frame width. Where the product utilizes a range of angles between the upright and the bracing member, the average angle in the tests may be used.

The horizontal deflection of the frame δ shall be measured at the free end of the upright on which load is applied, as shown in Figure 7.7.1.2.

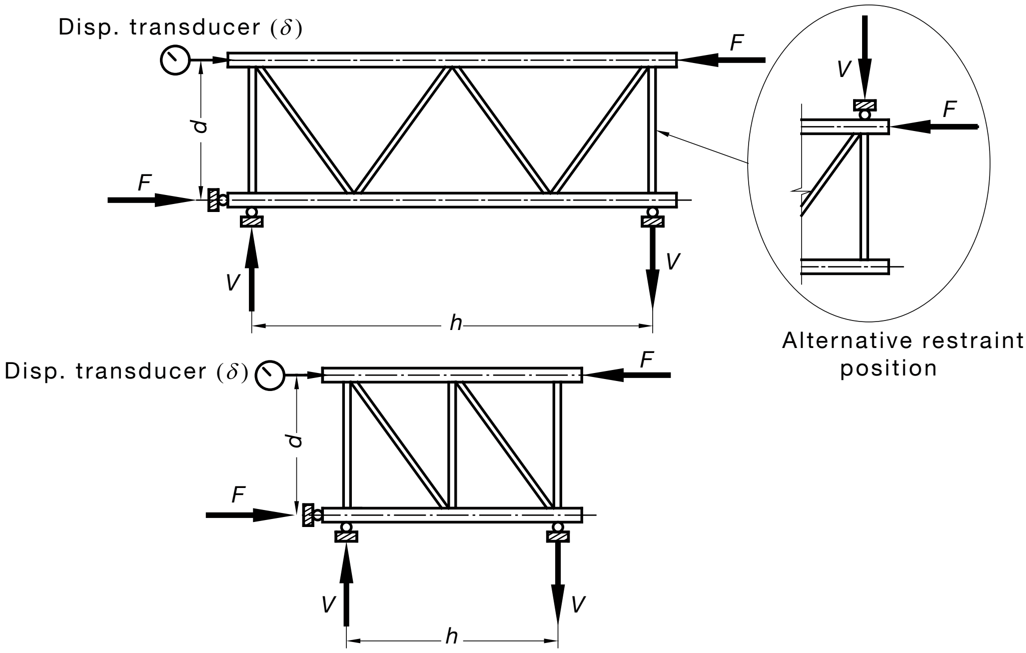


FIGURE 7.7.1.2 TEST ARRANGEMENTS FOR MEASURING THE SHEAR STIFFNESS OF BRACED FRAMES

7.7.1.3 *Method of test*

The load (F) shall be increased in increments up to a sufficient level to provide a minimum of three points on the essentially linear portion of the load-deflection curve. The corresponding deflection (δ) shall be measured during the test and a graph plotted of F against δ (see Figure 7.7.1.3).

The slope, k_{ti}, of the linear portion of the load-deflection curve shall be measured as shown in Figure 7.7.1.3 and the transverse shear stiffness of the frame (S_{ti}) shall be calculated as follows:

$$S_{ti} = \frac{k_{ti} d^2}{h} \qquad \ldots 7.7.1.3$$

where

h = length of the frame (see Figure 7.7.1.2)

d = distance between the centroidal axes of the upright sections (see Figure 7.7.1.2).

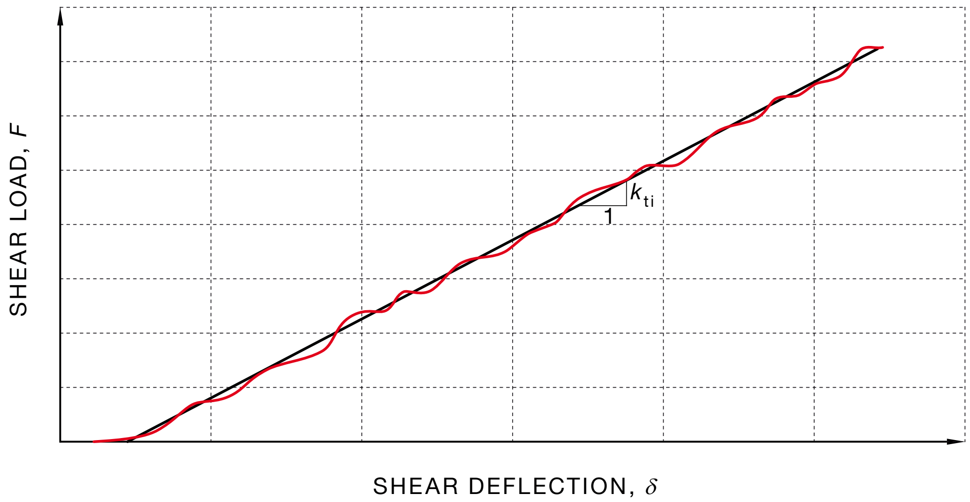


FIGURE 7.7.1.3 LOAD-DEFLECTION CURVE

7.7.1.4 *Corrections to the observations*

Corrections to the observations are not required.

7.7.1.5 *Derivation of the test results*

The design value of the transverse shear stiffness for the frame shall be taken to be the average value from a minimum of three tests.

7.7.2 Bending and shear test

7.7.2.1 *General*

This test may be used as an alternative to the shear test in Clause 7.7.1 to determine the transverse combined bending and shear stiffness of the upright frame.

7.7.2.2 *Purpose of the test*

The purpose of the test shall be to determine the overall sway stiffness per unit length of the upright frame in the transverse direction.

7.7.2.3 *Test arrangement*

A frame assembly, with a minimum number of two panels, shall be connected to a rigid testing frame using pin-supports for the upright ends. Figure 7.7.2.3 illustrates the test set-up.

The top of the frame shall be restrained from lateral displacements, and a load (F) shall be applied in the transverse direction at the elevation of the top horizontal bracing member.

The transverse displacements (δ_1 and δ_2) of the frame at the elevation of the load application point and at bottom horizontal member shall be recorded using a displacement transducer as shown in Figure 7.7.2.3.

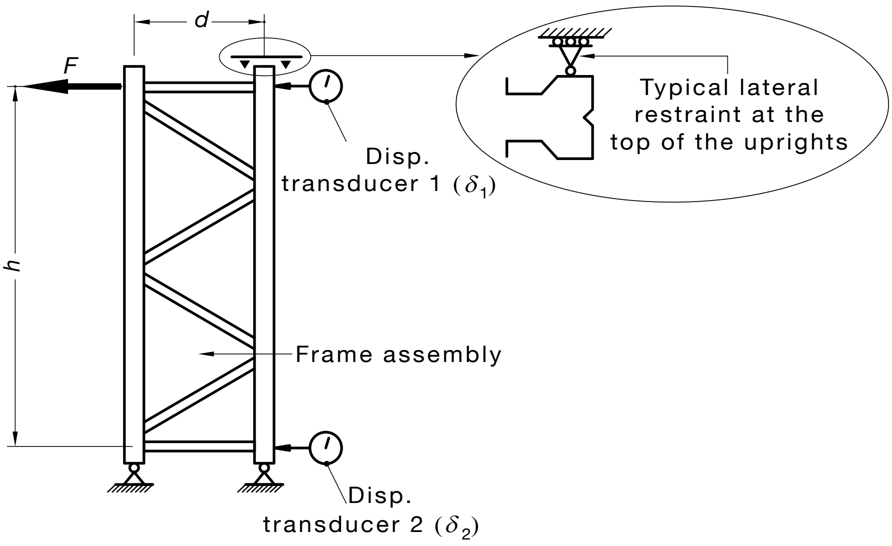


FIGURE 7.7.2.3 TEST ARRANGEMENTS FOR MEASURING THE COMBINED BENDING AND SHEAR STIFFNESS OF BRACED FRAMES

7.7.2.4 *Method of test*

The test method described in Clause 7.7.1.3 shall be followed.

The slope, kc_{ti}, of the linear portion of the load-deflection curve shall be calculated as follows:

$$kc_{ti} = \frac{F}{\delta_1 - \delta_2} \qquad \text{. . . 7.7.2.4 (1)}$$

where δ_1 and δ_2 are the displacements of transducers 1 and 2 respectively. The transverse combined bending and shear stiffness of the frame, Sc_{ti}, shall be calculated as follows:

$$Sc_{ti} = kc_{ti} \times h \qquad \text{. . . 7.7.2.4 (2)}$$

h = length of the frame (see Figure 7.7.2.3)

7.7.2.5 *Corrections to the observations*

Corrections to the observations are not required.

7.7.2.6 *Derivation of the test results*

The design value of the transverse combined bending and shear stiffness for the frame shall be taken to be the average value from a minimum of three tests.

7.8 TESTS ON UPRIGHT SPLICES

7.8.1 Purpose of the test

The purpose of the test is to determine the stiffness and strength of splices between upright sections. When the stiffness and strength of the splice is required in the cross-aisle direction, a single splice assembly shall be tested.

7.8.2 Test arrangement

This test is most commonly made to determine the stiffness and strength of a splice in the down-aisle plane. In this case, bending of the splice is about the axis of symmetry of the upright and may cause twisting. To eliminate such effects, a pair of splices may be tested together, face to face or back to back, and mounted on common base plates. They may be connected together away from the splice to reduce twisting of the sections. When testing uprights in pairs, the applied load shall be twice that specified in Clause 7.8.3.

The test arrangement is shown in Figure 7.8.2 and comprises two uprights connected together by the splice under investigation. This test sample is loaded axially with a force (F_1) through pin joints at its ends. The load (F_1) shall be applied along the centroidal axis of the test sample. The assembly is loaded horizontally with a force (F_2) at mid-span at shown in Figure 7.8.2.

Each splice assembly shall comprise two lengths of upright section, each at least four times the width of the upright section, plus the length of the splice. Measurement devices shall be fitted to the ends of the upright and on the splice, as shown in Figure 7.8.2.

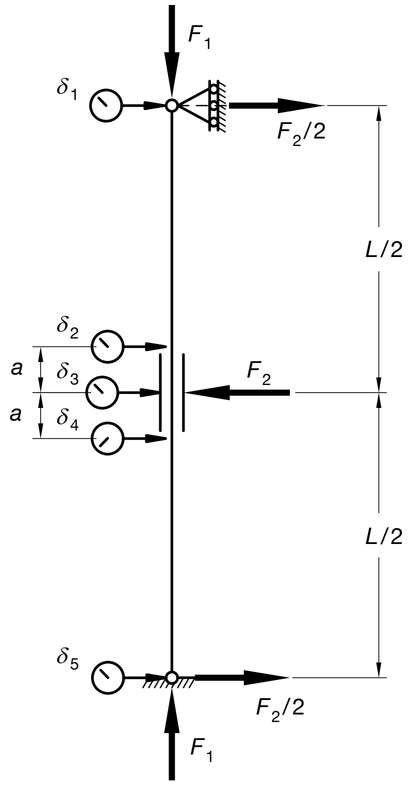


FIGURE 7.8.2 TEST ARRANGEMENT

7.8.3 Test method

Tests shall be made at a range of values of the axial load, F_1, approximately equal to $0.25F_{sd}$, $0.5F_{sd}$, $0.75F_{sd}$ and $1.0F_{sd}$ where F_{sd} is the maximum design load expected for the upright. At least one test shall be made at each value of the axial load.

The load on the upright (F_1) is first applied at a chosen value and kept constant at that value as the horizontal load (F_2) is applied. The load F_2 is then gradually increased until failure of the splice occurs and no further load can be applied.

The displacements shall be measured at the points shown in Figure 7.8.2.

A graph of the moment (M) applied to the splice against the rotation (θ) shall be plotted, for which:

$$M = F_2\frac{L}{4}+F_1\left(\delta_3-\left(\frac{\delta_1+\delta_5}{2}\right)\right) \qquad \text{. . . 7.8.3 (1)}$$

and

$$\theta = \frac{2}{a}\left(\delta_3 - \left(\frac{\delta_2 + \delta_4}{2}\right)\right) \qquad \text{. . . 7.8.3 (2)}$$

where

L = distance between pins

a = distance between measuring transducers 2 and 3 and transducers 3 and 4 in Figure 7.8.2

F_1 = axial load

F_2 = transverse load

δ_i = displacement of transducer number i.

7.8.4 Corrections to observations

Corrections need not be applied to the results of this test.

NOTE: The designer should consider the consequences of any significant variations in the mechanical and geometric properties of the test assembly from the nominal values.

7.8.5 Derivation of results

The design values for the stiffness and moment resistance of the splice for each value of the axial load (F_1) shall be derived in the manner described for floor connections in Clause 7.9.5.

If the variation in stiffness or design ultimate moment with axial force is not greater than ±10% of the mean value for the range of axial loads (F_1) up to the design load for the upright, the mean value may be assumed and used in the analysis and design of the structure. Where there is greater variation in the failure moment and the stiffness of the splice, then appropriate values corresponding to the design axial force shall be used.

7.9 TESTS ON FLOOR CONNECTIONS

7.9.1 Purpose of the test

The purpose of the test is to measure the moment-rotation characteristics of the connection between the upright and floor for a range of axial loads up to the maximum design strength of the upright.

7.9.2 Test arrangement

The test arrangement is shown in Figure 7.9.2(a). Alternatives may be used, provided they accurately model the real structural conditions.

The test arrangement comprises two lengths of upright section, each at least four times the width of the upright section, fitted with base plates, and bearing onto a concrete cube to represent the floor surface, as shown in Figure 7.9.2(a). Standard base plates shall be used in this test and they shall be connected to the concrete cube using the fixings adopted for the structure they are representing. If the base plates have floor fixings, then the cube strength of the concrete used in the test shall be the same as that used in the floor in practice. Tests carried out using concrete in strength class 20/25 may be used for any sound concrete floor when the concrete strength is not known. The test may be conducted using other materials corresponding to the actual floor material when it is not concrete, provided the test conditions represent those in practice.

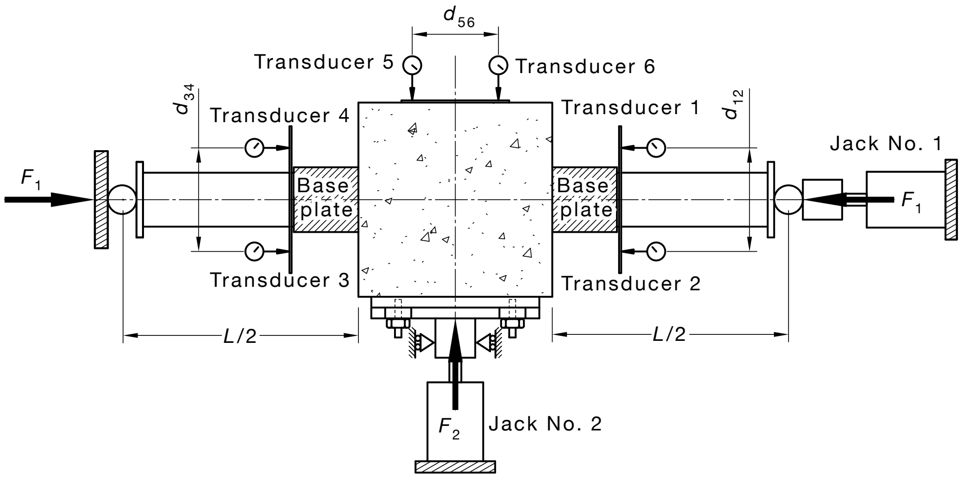


FIGURE 7.9.2(a) TEST ARRANGEMENT FOR FLOOR CONNECTIONS (PLAN)

Two jacks apply load to the assembly. Jack No. 1 simulates the axial load in the upright, while Jack No. 2 applies a lateral force on the concrete block to create a moment in the base plate assembly, as shown in Figure 7.9.2(a).

The concrete cube shall have parallel faces and shall allow a clearance of at least 50 mm all round the base plate. It shall be mounted on rollers, ball bearings or a well lubricated surface so that it is free to move in the horizontal plane, but restrained from rotating about the vertical axis. The retraining device shall allow the concrete to slide relative to Jack No. 2 so as not to transfer shear between the concrete block and Jack No. 2. Special care shall be taken to restrain the rotation of Jack No. 2. Additionally, it is preferable for the connection between Jack No. 2 and the concrete block to allow the transfer of both compressive and tensile forces in order to reach and pass the maximum moment applied to the base plate and avoid catastrophic failure.

Measurement devices shall be fitted to measure the horizontal movement of the concrete cube and the rotation of the column bases relative to the surface of the concrete. A suitable arrangement is shown in Figure 7.9.2(a).

To capture the total rotation of the upright, transducers 1 to 4 in Figure 7.9.2(a) shall be located just above the boundary between the base plate assembly and the upright. Figure 7.9.2(b) shows an appropriate position for the transducers.

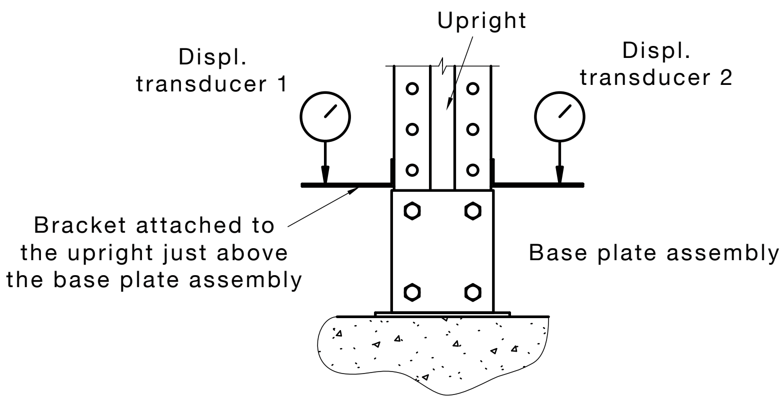


FIGURE 7.9.2(b) APPROPRIATE POSITIONS FOR TRANSDUCERS

The uprights shall be cut normal to their longitudinal axes, and the faces of the cube on which the uprights bear shall be parallel, so that the axes of both uprights coincide with the line of action of the load.

7.9.3 Test method

Tests should be performed over a range of axial loads up to the estimated maximum design load of the upright.

At least six tests shall be made for each upright.

The load in Jack No. 1 shall be set at a nominal value of 5% of the estimated maximum design load which keeps all the components in contact, and the transducers zeroed. The load in Jack No. 1 shall then be increased to its full value and held constant at that value. The displacements shall be measured, and then the load in Jack No. 2 shall be increased and further displacement observations shall be made until failure or until the base rotation exceeds a value which is deemed to be excessive.

NOTE: Failure may not have occurred when the load in Jack No. 2 reaches a maximum and it is essential for Jack No. 2 to be driven in displacement control to reach and pass the maximum bending moment applied to the base plate.

The system of forces is shown in Figure 7.9.3.

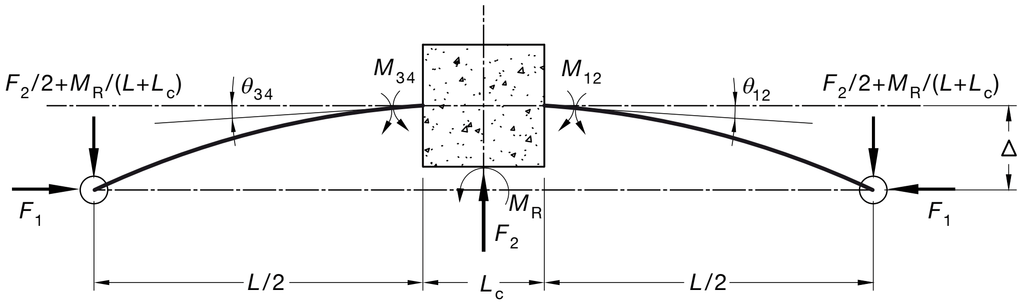


FIGURE 7.9.3 FORCES AND DEFLECTIONS IN THE TEST ON FLOOR CONNECTIONS

The average moment applied to the base plate, M_b, and the average rotation of the base plate, θ_b, shall be calculated as follows:

$$M_b = \frac{F_2}{4}L + F_1\Delta \qquad \text{. . . 7.9.3 (1)}$$

$$\theta_b = \frac{1}{2}\left(\frac{\delta_1 - \delta_2}{d_{12}} + \frac{\delta_4 - \delta_3}{d_{34}}\right) \qquad \text{. . . 7.9.3 (2)}$$

where

F_1 = load applied by Jack No. 1

F_2 = load applied by Jack No. 2

δ_i = displacements of transducer number i in Figure 7.9.2(a), i = 1,2,3,4,5 or 6

Δ = displacement of the concrete block

$$= \frac{\delta_5 + \delta_6}{2} \qquad \text{. . . 7.9.3 (3)}$$

d_{ij} = distance between transducers i and j in Figure 7.9.2(a), i, j = 1,2,3,4,5 or 6

L = twice the length of an upright section.

7.9.4 Corrections to observations

Corrections need not be applied to the results of this test.

NOTE: The designer should consider the consequences of any significant variations in the mechanical and geometric properties of the test assembly from the nominal values.

7.9.5 Derivation of results

For each value of axial load, the design values of the ultimate moment of resistance and the stiffness of the base plate connection shall be calculated in the manner indicated in Clause 7.2.2. The test results shall be plotted on charts of resistance and stiffness versus axial load. In each case, a smooth design curve or series of straight lines, all of which lie below all the test results, shall be drawn. As an alternative, a single stiffness may be chosen for all values of the axial load and the corresponding resistances calculated in accordance with Clause 7.2.2.

7.10 CHARPY TYPE IMPACT TESTS

7.10.1 Purpose of the test

The purpose of the test is to determine the effect of low temperature on the absorbed impact energy of critical elements of the connection between the beam and the upright, such as the connector hook and the beam to connector weld. The result obtained from the test shall be used to determine whether the connection is suitable for use at low temperatures, such as in a cold store.

7.10.2 Test method

The test shall consist of a Charpy type impact test in which a sample is subjected to impact loading and the energy absorbed, as it fractured, shall be measured. The testing machine shall be capable of delivering enough energy on impact to fracture the sample, so that the residual energy shall be measured and the energy absorbed calculated. If this is not the case, and fracture does not occur, then the test arrangements shall be changed.

The test shall be made at different temperatures ranging from ambient temperature down to not less than 20°C below the service temperature for the connection. The minimum frequency of testing shall be a single test at each temperature, reducing from ambient in increments of 5°C.

Temperature control shall be achieved by immersing the test samples in a fluid bath at the appropriate temperature prior to mounting the test sample in the testing machine. The design of the mount for the test specimen shall be such that it can be quickly installed so that no significant change in temperature takes place before the impact test is made.

NOTES:

1 The test method cannot be precisely defined because of the variety of designs of the connection between the beam and upright. The test sample will normally be non-standard; however, the actual value of the energy absorbed is not significant.

2 Figure 7.10.2(a) shows a beam to upright connection. The elements to be tested may be a single hook or a part of the weld between the beam and connector (see Figures 7.10.2(b) and 7.10.2(c)) or, providing the testing machine has adequate capacity, it may be the complete assembly shown in Figure 7.10.2(a).

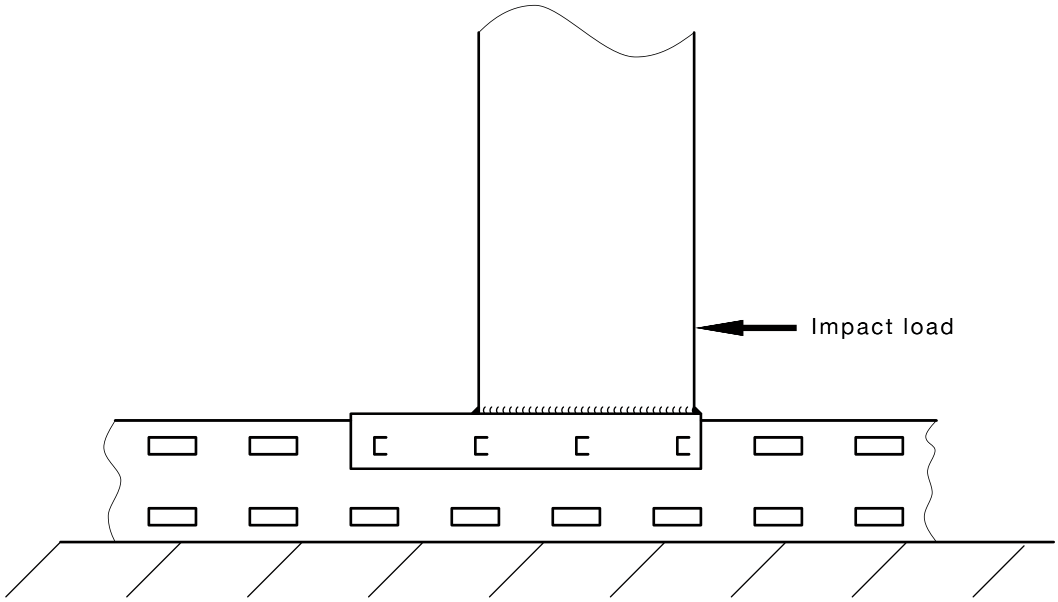


FIGURE 7.10.2(a) TEST ARRANGEMENT OF BEAM TO UPRIGHT CONNECTION

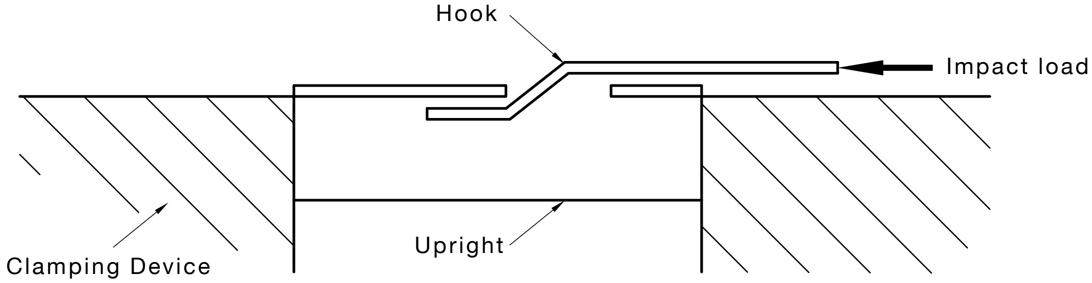


FIGURE 7.10.2(b) TEST ARRANGEMENT OF A SINGLE HOOK BETWEEN BEAM AND CONNECTOR

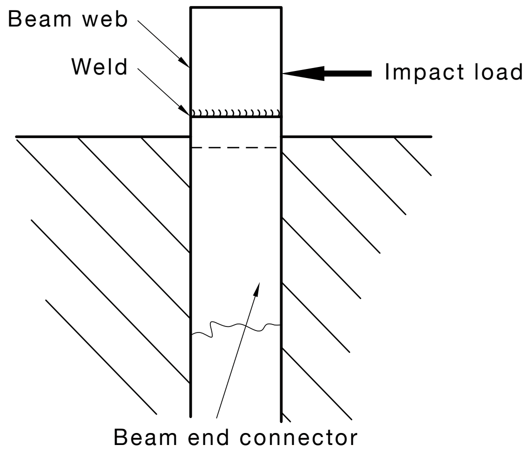


FIGURE 7.10.2(c) TEST ARRANGEMENT OF THE WELD BETWEEN BEAM AND CONNECTOR

7.10.3 Derivation of the transition temperature

It shall be assumed that the behaviour at ambient temperature is ductile, although fracture surfaces shall be inspected to confirm that this is the case, and that brittle behaviour, as the temperature reduces, will be characterized by a significant drop in the energy absorbed. If the temperature at which the behaviour changes is as sharp as that shown in Figure 7.10.3(a), it is possible to define the transition temperature (T_R) as shown. However, a less well defined characteristic may be expected in many instances (see Figure 7.10.3(b). For this reason, to remove uncertainty, the transition temperature shall be obtained by drawing a smooth curve below all the test results and measuring the temperature (T_R) at which the absorbed energy drops to 75% of that at ambient temperature (E_A) (see Figure 7.10.3(b)).

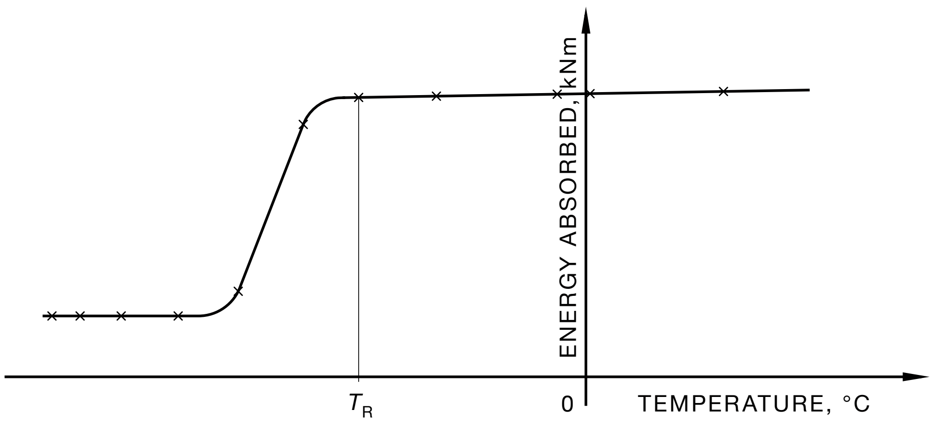


FIGURE 7.10.3(a) CHANGE OF TRANSITION TEMPERATURE (T_R)

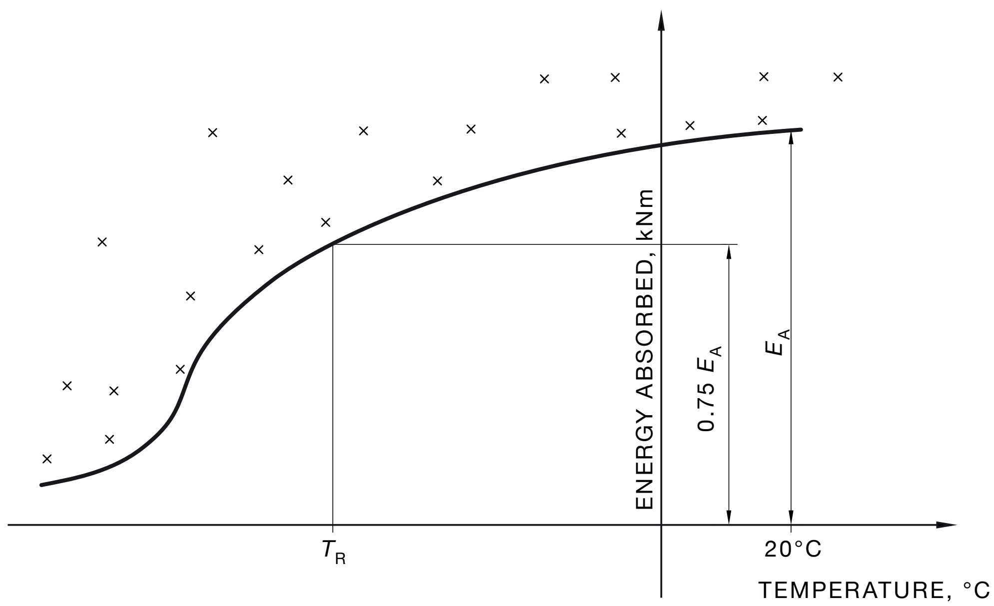


FIGURE 7.10.3(b) CHANGE OF ABSORBED ENERGY

SECTION 8 OPERATION AND MAINTENANCE OF ADJUSTABLE PALLET RACKING

8.1 GENERAL

8.1.1 Safety management

The racking supplier shall provide user manuals and appropriate training to the end users for correct usage and maintenance of the racking. The safety of the storage equipment shall be maintained. This shall include training of equipment operators, inspection, documentation of damage reports, and appropriate action when damage occurs. The equipment supplier shall be liaised with for advice on training, inspection, repair or replacement of the equipment, as appropriate.

8.1.2 Working load limits

The working load limit or the total working load limit per bay for the racking installation shall not be exceeded. The permissible unit load limit and permissible unit load limit per bay shall be displayed in accordance with Item (a) of Clause 1.6.1.

8.1.3 Alteration of the racking installation

The racking installation shall not be altered to deviate from the load application and configuration furnished for the racking installation.

Physical alterations to uprights, bracings, beams or components, such as welding on additional cleats or bearers, shall not be made.

In addition, change of use, such as from timber pallets to post pallets, shall not be permitted.

When designing the racking, the nature of the load transfer between unit load (e.g. uniformly distributed action and concentrated action) and the immediate supporting elements shall be considered.

8.1.4 Operating instructions

Racking installation shall have in one or more conspicuous locations, a permanent corrosion resistant plaque not less than 125 mm long and 250 mm high with minimum 25 mm lettering and shall be mechanically secured to the racking structure at 2 m above floor level. Operating instructions on the plaque shall include but not be limited to the following:

(a) The correct application and use of the equipment.

(b) The working load limits to be adhered to.

(c) Prohibitions on unauthorized alterations.

(d) The requirement to report any damage incurred due to impact so that its effect can be assessed in accordance with Clauses 8.3 and 8.4.

8.1.5 Hazardous situations

Any hazardous situations which may exist in relation to the operation or maintenance of the racking installation shall be reported.

8.1.6 Damage report

Any damage incurred, however minor, shall be reported so that its effect on safety can be immediately assessed.

8.2 INSPECTIONS

Inspections shall be carried out on a regular basis, and at least once every twelve months to—

(a) ensure the correct application and use of equipment;

(b) ensure that the working load limits are adhered to;

(c) ensure that the racking installation has not been altered. A copy of the load application and configuration drawings shall be retained for this purpose;

(d) examine the extent of damage due to impact in the racking installation;

(e) examine the out-of-plumb of the racking;

(f) examine for any dislocation and deformation of sections and connections for uprights and beams;

(g) examine connectors for deformation or signs of cracking of the welds; and

(h) examine base plates and floor anchors.

8.3 DAMAGE DUE TO IMPACT

8.3.1 General

The extent of damage due to impact in the racking installation, determined from regular inspections and reports of damage incurred, shall be assessed in accordance with Clauses 8.3.2 to 8.3.6. Damage levels shall be specified and appropriate actions shall be taken in accordance with Clause 8.5.

8.3.2 Uprights

For uprights, the visible damage shall not exceed that shown in Figure 8.3.2.

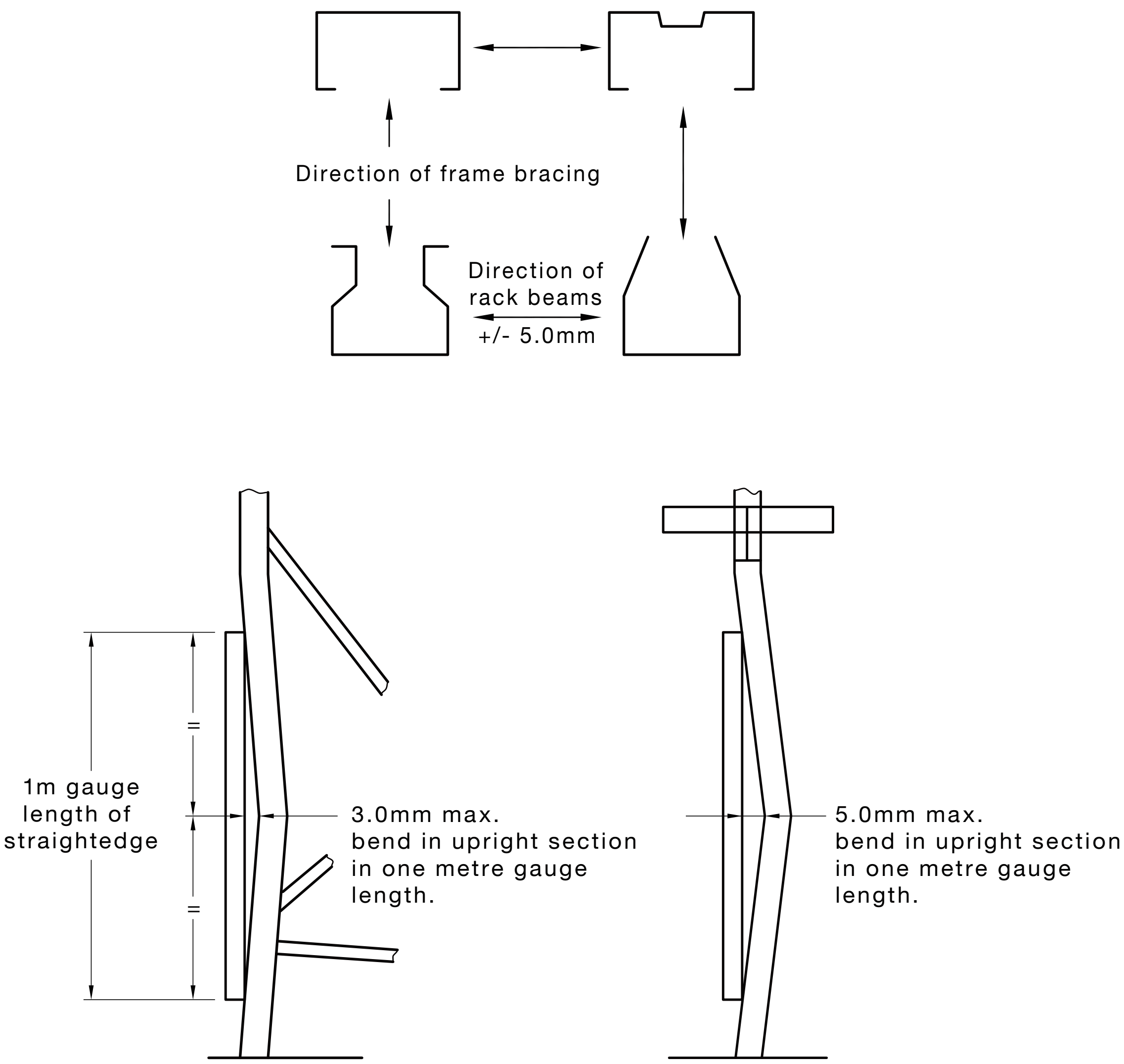


FIGURE 8.3.2 TYPICAL UPRIGHT SECTIONS AND METHOD OF MEASUREMENT

8.3.3 Bracing

For bracing, the member deviation from a 1 m long straightedge in either plane shall not exceed 10 mm.

NOTE: For bracing members less than 1 m long, damage may be assessed pro-rata.

8.3.4 Beams

The residual vertical deformation when unloaded shall not exceed 20% of the allowed vertical deflection under serviceability action specified in Clause 4.2.5.

The residual horizontal deformation measured at the top or bottom edge of the beam shall not exceed 50% of the allowed vertical deflection under serviceability action specified in Clause 4.2.5.

When these limits are exceeded, the beam shall be unloaded and advice shall be sought from the racking supplier.

8.3.5 Connectors

Connectors shall not show visible permanent deformation or signs of cracking of welds. Damage of this type shall require the beam to be unloaded and advice shall be sought from the racking supplier.

8.3.6 Connector safety lock

The presence of the necessary locking devices shall always be checked during maintenance and repair. Any missing locking devices shall be replaced immediately to prevent accidental dislodging of beams. A supply of beam locking devices shall be stocked on site.

8.4 OUT-OF-PLUMB OF RACKING

The out-of-plumb of unloaded racking caused by impact shall not exceed the finished tolerances given in Table 1.7.1(a), factored by 1.5. When this limit is exceeded, advice shall be sought from the racking supplier for structural evaluation.

8.5 DAMAGE CLASSIFICATION AND RISK MANAGEMENT

8.5.1 General

The assessment applies only to damage that produces an overall bend in a member in accordance with Clause 8.3.2 to 8.3.4. These limits do not apply to highly localized damage such as dents, buckles, tears and splits. Members subjected to tears and splits shall be replaced.

8.5.2 Acceptable damage (Green)

When the level of damage does not exceed the permissible limits specified in Clauses 8.3.2 to 8.3.4, the racking shall be considered serviceable and does not require either unit load reduction or immediate unloading. This constitutes green level damage.

There shall be a method of recording damage to components. There shall also be a method, such as the use of stick-on tags on the components, to show that these components have been inspected and that the racking is suitable for further service until the next inspection, when the components have been re-inspected and re-assessed (Section 8.2). For example, use could be made of coloured dated adhesive labels, which indicate that racking or components are suitable for service and need to be re-inspected at future inspections.

Exceeding the green level shall be considered hazardous or very serious damage and causes risk to the racking system.

8.5.3 Hazardous damage (Amber risk)

When the level of damage exceeds the permissible limits specified in Clauses 8.3.2 to 8.3.4 by up to a factor of 2, the damage section shall be clearly marked and isolated until remedial work has been carried out before the racking can be reloaded. If remedial work cannot be carried out within four weeks, the level of damage shall be redesignated as ‘very serious damage’ (red risk).

There shall be a method of isolating such racking to ensure that it will not come back into use until the necessary repairs have been carried out and the equipment certified as safe. For example, dated coloured adhesive labels could be used to indicate that the racking is not to be reloaded until rectified.

8.5.4 Very serious damage (Red risk)

When the level of damage is in excess of the permissible limits specified in Clauses 8.3.2 to 8.3.4 by more than a factor of 2, the damage section and appropriate adjacent section(s) shall be immediately unloaded and isolated from further use until remedial work has been carried out and certified as safe.

There shall be a method of isolating areas of racking to ensure that they do not come back into use prior to the repair work being carried out. For example, a particular bay could be immediately off-loaded and roped off to prevent further use.

Any repair works shall be in consultation with the racking supplier or a qualified structural engineer.

NOTE: Remedial work usually involves replacement.

APPENDIX A

AMPLIFIED SWAY METHOD FOR DOWN-AISLE STABILITY ANALYSIS

(Normative)

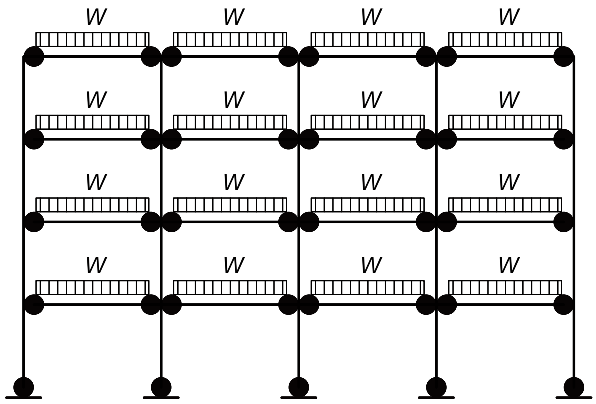
A1 GENERAL

The amplified sway method is an approximate linear buckling analysis method. It provides a close approximation to the value of the elastic critical load ratio (N_{cr}/N^*) of a plane frame. It then allows the increase in the bending moments and deflections due to second order effects to be estimated.

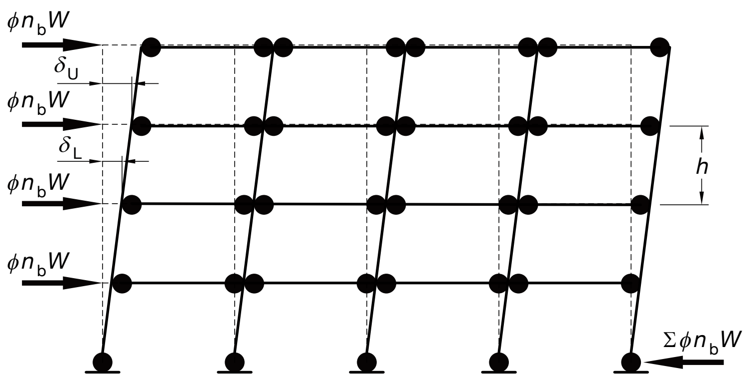
The principles are described with reference to Figure A1:

(a) Actual frame with semi-rigid connections and loading (see Figure A1(a) where W is the resultant vertical action per beam).

(b) Equivalent imperfection horizontal actions and resulting deflections (see Figure A1(b)).



(a) Actual frame and loading



(b) Imperfection horizontal actions and resulting deflections, n_b is the number of bays

FIGURE A1 BASIS OF THE AMPLIFIED SWAY METHOD

A2 LINEAR ELASTIC ANALYSIS

A linear elastic analysis (LA) of the complete frame may be carried out in order to determine the internal actions and deflections due to the imperfection horizontal actions as shown in Figure A1(b). The frame imperfection (ϕ) is defined in Clause 3.3.2.1 and Figure 3.3.2.1.

The flexibility of the semi-rigid beam to upright connection shall be taken into account.

Allowance shall also be made for the stiffness of the upright to floor connections (see Clause 7.9).

A3 ELASTIC CRITICAL LOAD RATIO

The elastic critical ratio of the vertical load (N_{cr}/N^*) for buckling in a sway mode shall then be determined from:

$$\frac{N_{cr}}{N^*} = \frac{\phi}{\phi_{max}} \qquad \text{. . . A3(1)}$$

where

N^* = design value of the vertical action on the frame

ϕ = frame imperfection defined in Clause 3.3.2.1

ϕ_{max} = largest value of the sway index (ϕ_s) of any storey

$$\phi_s = \frac{\delta_U - \delta_L}{h} \qquad \text{. . . A3(2)}$$

where

δ_U = horizontal deflection at the top of the storey (see Figure A1(b))

δ_L = horizontal deflection at the bottom of the storey (see Figure A1(b))

h = storey height.

A4 AMPLIFICATION FACTOR

At the required limit state, the design internal forces and deflections in any sway mode are amplified by the factor (β) where,

$$\beta = \frac{N_{cr}}{N_{cr} - N^*} = \frac{N_{cr}/N^*}{N_{cr}/N^* - 1} \qquad \text{. . . A4}$$

where N_{cr}/N^* is given by equation A1.

APPENDIX B

SIMPLIFIED EQUATIONS FOR THE DESIGN OF A REGULAR STORAGE RACK IN THE DOWN-AISLE DIRECTION

(Normative)

B1 SIMPLIFIED EQUATIONS FOR REGULAR RACK CONSTRUCTION

The equations that follow are applicable to any rack of reasonably regular construction and do not introduce any significant assumptions.

N_s = number of storeys

N_b = number of bays

I_c = I of upright

K_c = base stiffness

W_c = additional action on top of racking, if any

I_b = I of beam

K_b = joint stiffness at end of pallet beam

W_b = design action per beam

h_1 = height of first pallet beam from floor

h = height of second pallet beam above first pallet beam (taken as typical)

L = pallet beam span

ϕ = frame imperfection (or ratio of notional horizontal action to vertical action)

E = Young's modulus

$\sum(W)$ = $N_b(N_sW_b + W_c)$ total action on racking

S_2 = $\sum(W) - N_bW_b$ action above first storey.

Moment about base of vertical actions applied horizontally—

$$\sum(W\mathrm{h}) = \frac{N_s(N_s - 1)}{2}N_bW_bh + N_sN_bW_bh_1 + (N_sh + h_1 - h)N_bW_c$$

I_{cc} = $(N_b + 1)\,I_c$ total column I

K_{cc} = $(N_b + 1)\,K_c$ total base stiffness

$$F = \frac{12N_bEI_bK_b}{6EI_b + K_bL}$$

$$C = F + \frac{EI_{cc}K_{cc}}{EI_{cc} + K_{cc}h_1} + \frac{EI_{cc}}{h}$$

$$A = \frac{\sum(W)h_1}{2C}\,\frac{K_{cc}h_1 + 2EI_{cc}}{K_{cc}h_1 + EI_{cc}} + \frac{S_2h}{2C}$$

$$B = \frac{EI_{cc}}{hC}$$

$$D = (N_s - 1 + B)F + \frac{EI_{cc}K_{cc}B}{EI_{cc} + K_{cc}h_1}$$

$$G = \frac{\sum (W) h_1}{2}\left[\frac{K_{cc} h_1}{K_{cc} h_1 + EI_{cc}}\right] + \frac{EI_{cc} K_{cc} A}{K_{cc} h_1 + EI_{cc}} - \sum (Wh) + FA$$

$$\theta_1 = A - \frac{BG}{D}$$

Elastic critical load factor (λ_{cr})

$$\lambda_1 = \frac{K_{cc} h_1 + EI_{cc}}{\left[\frac{K_{cc} h_1}{2} + EI_{cc}\right]\theta_1 + \frac{\sum (W) h_1^2}{12 EI_{cc}}\left(K_{cc} h_1 + 4EI_{cc}\right)}$$

$$\lambda_3 = -\frac{D}{G}$$

$$\theta_2 = \frac{S_2 h_2}{12 EI_{cc}} + 0.5\left[\frac{1}{\lambda_3} + \theta_1\right]$$

$$\lambda_2 = -\frac{1}{\theta_2}$$

$$\lambda_{cr} = \begin{cases} \min(\lambda_1, \lambda_2, \lambda_3) & \text{for } h_1 \geq h \\ \left(0.8 + 0.2\frac{h_1}{h}\right)\min(\lambda_1, \lambda_2, \lambda_3) & \text{for } h_1 < h \end{cases}$$

This method of analysis is only valid if $\lambda_{cr} \geq 3.333$

Magnification factor for second order effects $\beta = \dfrac{1}{1 - \dfrac{1}{\lambda_{cr}}}$

Storey sways Requirement in any storey $\phi_{max} \leq 0.02$ at ultimate

Bottom storey $\phi_1 = \dfrac{\beta\phi}{\lambda_1}$

Second storey $\phi_2 = \dfrac{\beta\phi}{\lambda_2}$

Upper storey $\phi_u = \dfrac{\beta\phi}{\lambda_3}$

NOTE: The correction factor to the elastic critical action, namely ($0.8+0.2h_1/h$), applies no correction if the lower pallet beam is at a distance above the ground similar to the pallet beam spacing elsewhere in the racking. If the lower pallet beam is near the ground, the critical action given by the equation is reduced by 20%. There is a linear transition between these two extreme cases. If $h_1 \geq h$, no correction is necessary. This procedure is conservative relative to the available calibration values.

B2 ADDITIONAL BENDING MOMENTS DUE TO PATTERN ACTION

Fixed end moment in beam-upright connection due to pallet action (kN/mm)—

$$M_p = \frac{W_b L}{12}\left[\frac{K_b L}{2EI_b + K_b L}\right]$$

$$K_{b1} = \frac{4EI_b K_b (K_b L + 3EI_b)}{(K_b L + 2EI_b)(K_b L + 6EI_b)}$$ pallet beam stiffness (general case)

$$K_{b2} = \frac{2EI_b K_b}{(K_b L + 2EI_b)}$$ pallet beam stiffness (symmetrical case)

$$K_{c1} = \frac{4EI_c}{h_1}\frac{K_c h_1 + 3EI_c}{K_c h_1 + 4EI_c}$$ stiffness of lower length of upright

$$K_{c2} = \frac{4EI_c}{h}$$ stiffness of upper length of upright

$$S_k = K_{b1} + K_{b2} + K_{c1} + K_{c2}$$ total stiffness at joint

B3 DESIGN MOMENTS

Moment in beam upright connection due to side-sway and pattern action

$$M_c = \frac{6EI_b K_b \beta\phi\theta_1}{6EI_b + K_b L} + M_p\left[1 - \frac{K_{b_1}}{S_k}\right]$$

Moment in column below first pallet beam level due to side-sway

$$M = \frac{-\beta\phi\sum(W)h_1}{2N_b}\left[\frac{K_c h_1 + 2EI_c}{K_c h_1 + EI_c}\right] + \frac{EI_c K_c \beta\phi\theta_1}{K_c h_1 + EI_c}\frac{N_b + 1}{N_b}$$

Moment below first pallet beam level due to vertical pattern action

$$M_{c1} = M_p \frac{K_{c1}}{S_k}$$

Total moment below first beam level

$$M_1 = M - M_{c1}$$

Moment at base plate due to side-sway (per upright)

$$M_H = \frac{-\beta\phi\sum(W)h_1}{2(N_b + 1)}\left[\frac{K_c h_1}{K_c h_1 + EI_c}\right] - \frac{EI_c K_c \beta\phi\theta_1}{K_c h_1 + EI_c}$$

Moment at base plate due to pattern action (per upright)

$$M_{c11} = M_{c1}\frac{K_c h_1}{2(K_c h_1 + 3EI_c)}$$

Total moment at base plate

$$M_2 = M_H - M_{c11}$$

Moments in second storey of upright

$\theta_2 = \phi_u$ (magnified)

Moments above first pallet beam level due to side-sway

$$M_{bc} = \frac{-\beta\phi S_2 h}{2N_b} + \left[\frac{EI_c\beta\phi\theta_1}{h} - \frac{EI_c\theta_2}{h}\right]\frac{N_b+1}{N_b}$$

Moment above first pallet beam level due to vertical pattern action

$$M_{c2} = M_p \frac{K_{c2}}{S_k}$$

Total moment above first pallet beam level

$$M_3 = M_{bc} - M_{c2}$$

Moment below second pallet beam level due to side-sway

$$M_{cb} = \frac{-\beta\phi S_2 h}{2N_b} - \left[\frac{EI_c\beta\phi\theta_1}{h} - \frac{EI_c\theta_2}{h}\right]\frac{N_b+1}{N_b}$$

Moment below second pallet beam level due to pattern action

$$M_{c22} = 0.5M_{c2}$$

Total moment above second pallet beam level

$$M_4 = M_{cb} - M_{c22}$$

B4 DESIGN ACTIONS IN OUTER UPRIGHTS

Axial forces in upright

$$N = \frac{\sum(W)}{N_b}$$

As pattern action has been included in the design of the internal uprights, separate consideration of the outer columns need not be carried out.

B5 SAMPLE CALCULATION

For the following rack data:

N_s	=	5	N_b	=	5
I_c	=	735 000 mm^4	K_c	=	90 000 kN mm/radian
W_c	=	0 kN			
I_b	=	577 500 mm^4	K_b	=	70 000 kN mm/radian
W_b	=	6			
h	=	1 500 mm	h_1	=	1 500 mm
L	=	2 700 mm			
ϕ	=	0.01 radian			
E	=	200 000 MPa			

the following results are obtained using the equations set out in Paragraphs B1 to B4:

λ_1	=	3.576	λ_2	=	4.039
λ_3	=	5.435	λ_{cr}	=	3.576
β	=	1.388			
ϕ_1	=	0.00388	ϕ_2	=	0.00344
ϕ_u	=	0.00255			
M_1	=	-512.118 kN mm	M_2	=	-322.228 kN mm
M_3	=	427.114 kN mm	M_4	=	-525.076 kN mm
N	=	30 kN			

APPENDIX C

SIMPLIFIED METHOD FOR CROSS-AISLE STABILITY ANALYSIS

(Normative)

C1 GENERAL

The elastic critical action (N_{cr}) for sway instability shall first be estimated. The amplified sway method shall then be used to enhance the internal forces and displacements to take account of second order effects.

C2 GLOBAL BUCKLING OF UPRIGHT FRAMES

The elastic critical load (N_{cr}) of an upright frame is given as:

$$N_{cr} = \frac{1}{(1/N_{cr0}) + (1/S_D)} \quad \text{. . . C2(1)}$$

where

N_{cr} = total vertical action on the frame causing elastic sway buckling

N_{cr0} = critical load neglecting the shear flexibility of the bracing system

$$= \frac{\pi^2 E A_u D^2}{2H_b^2} \quad \text{. . . C2(2)}$$

A_u = cross-sectional area of one upright

D = upright frame width

H_b = buckling length of the frame

$$= 2H\sqrt{\frac{1 + 2.18(W_o/W_1)}{3.18}} \quad \text{for the unpropped frame (see Figure C2(a))} \quad \text{. . . C2(3)}$$

$$= H\sqrt{\frac{1 + 1.65(W_o/W_1)}{5.42}} \quad \text{for the propped frame (see Figure C2(b))} \quad \text{. . . C2(4)}$$

H = distance from the floor to the top horizontal frame bracing member (see Figure C2(a))

W_o = action applied at top of racking (see Figure C2(c))

W_1 = total action on racking (see Figure C2(c))

S_D = shear stiffness of upright frame per unit length determined in accordance with Paragraph C3.

If equal beam actions are applied at all levels of the upright frame:

$(W_1/W_o) = n_s$

= number of beam levels.

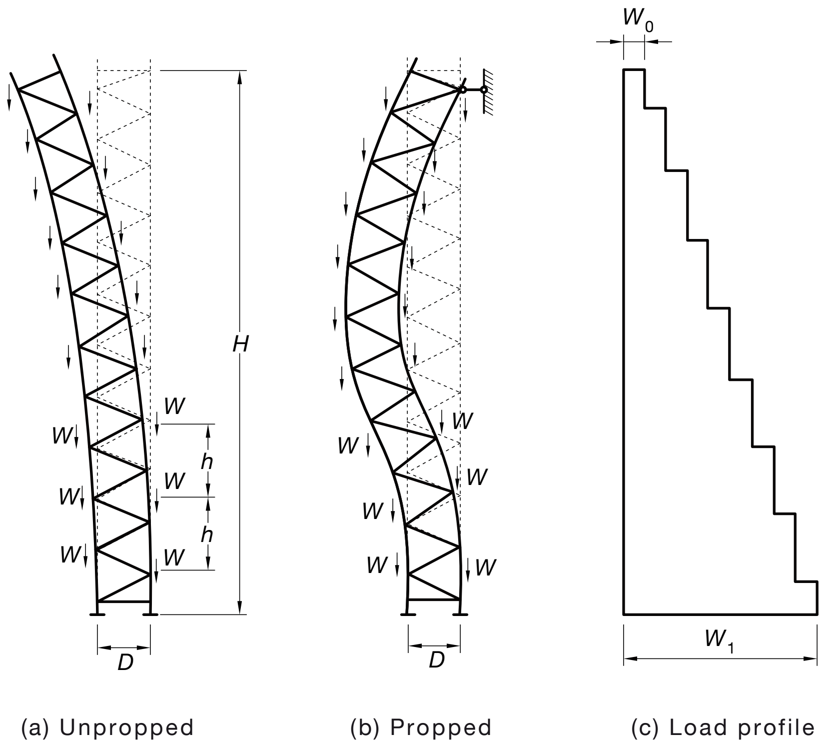


FIGURE C2 ASSUMPTION FOR SIMPLIFIED CROSS-AISLE STABILITY ANALYSIS

C3 SHEAR STIFFNESS OF UPRIGHT FRAME

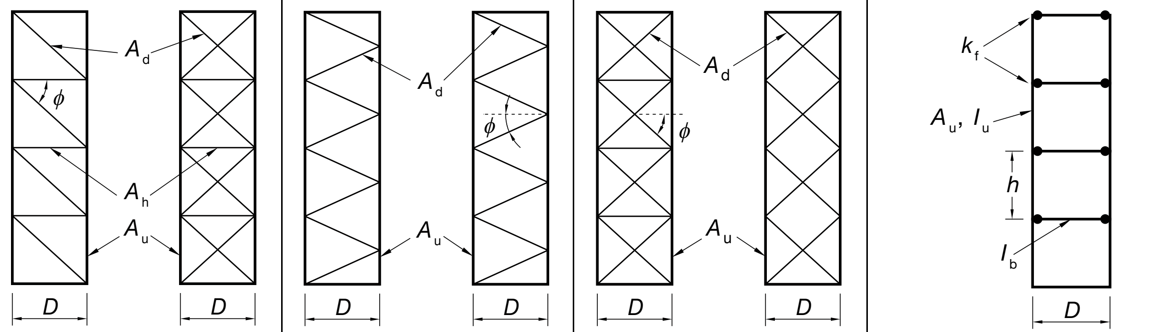
For a frame in which the joint flexibility can be shown to be negligible or may be allowed for within the given expressions, e.g. by using a reduced cross-sectional area for the bracing member, the shear stiffness per unit length (S_D) shall be calculated as follows:

$$\frac{1}{S_D} = \left(\frac{1}{S_{dd}}\right) + \left(\frac{1}{S_{dh}}\right) + \left(\frac{1}{S_{db}}\right) \qquad \text{. . . C3}$$

where

S_{dd}, S_{dh}, S_{db} = as shown in Figure C3 for a variety of different bracing systems.

When a reliable calculation of the shear stiffness cannot be carried out, it should be determined by test in accordance with Clause 7.7.



(a) Tension Bracing Only

$$\frac{1}{S_{dd}} = \frac{1}{A_d E \sin\phi \cos^2\phi}$$

$$\frac{1}{S_{dh}} = \frac{1}{A_h E \tan\phi}$$

$$\frac{1}{S_{db}} = 0$$

(b)

$$\frac{1}{S_{dd}} = \frac{1}{A_d E \sin\phi \cos^2\phi}$$

$$\frac{1}{S_{dh}} = 0$$

$$\frac{1}{S_{db}} = 0$$

(c)

$$\frac{1}{S_{dd}} = \frac{1}{2A_d E \sin\phi \cos^2\phi}$$

$$\frac{1}{S_{dh}} = 0$$

$$\frac{1}{S_{db}} = 0$$

(d)

$$\frac{1}{S_{dd}} = 0$$

$$\frac{1}{S_{dh}} = 0$$

$$\frac{1}{S_{db}} = \left[\frac{hD}{12EI_b}\left(1+\frac{6EI_b}{Dk_f}\right) + \frac{h^2}{24EI_u}\right]$$

FIGURE C3 SHEAR STIFFNESS FOR UPRIGHT FRAMES

The following symbols are used in Figure C3:

E = elastic modulus

A_d = area of diagonal bracing

A_h = area of horizontal bracing

ϕ = angle of diagonal bracing with horizontal

h = storey height

D = width of upright frame

I_b = second moment of area of beam

I_u = second moment of area of upright

k_f = stiffness of semi-rigid beam to upright connection

C4 AMPLIFICATION FACTOR (β)

If $(N^*/N_{cr}) < 0.1$, global second order effects may be neglected.

At the limit state, the sway component of the internal forces and deflections calculated using first order theory are enhanced due to second order effects by the multiplication factor (β), where:

$$\beta = \frac{N_{cr}}{N_{cr} - N^*} \qquad \text{. . . C4}$$

where

N^* = design value of the total vertical action on the frame (shown as W_1 in Figure C2(c)).

NOTE: The arrangement shown in Figure C2(b) should be used with care. Connecting the frames together at the top does not constitute an adequate prop because all of the frame may undergo sway buckling together. A prop can only be utilized if an independent structure of sufficient rigidity is available.

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NOTES

NOTES

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