AS 4084 Supp1—1993

Australian Standard®

Steel storage racking—Commentary

(Supplement to AS 4084-1993)

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- Australian Institute of Steel Construction
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- Metal Trades Industry Association of Australia
- Queensland University of Technology
- University of Sydney
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Steel storage racking—Commentary

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PREFACE

The Supplement was prepared by the Standards Australia Committee for Steel Storage Racking as a Commentary on AS 4084, *Steel storage racking*.

The Supplement provides background and explanatory material to the requirements of the Standard.

The clause numbers of this commentary are prefixed by the letter ‘C’ to distinguish them from references to the Standard clauses to which they directly relate. The commentary clause numbers are not sequential because if no explanation of the Standard clause is necessary that clause number is not included.

The Standard is predominantly based on the American Rack Manufacturers Institute Specification (RMI) when used at its lower tier. However, information has been taken from the British Storage Equipment Manufacturers Association (SEMA) and the European Racking Code FEM 10.2.02 for the higher tiers of the Standard.

It also provides advice and guidance in certain areas which cannot at this stage be covered by the Standard requirements.

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STANDARDS AUSTRALIA

Steel storage racking—Commentary

(Supplement to AS 4084—1993)

SECTION C1 SCOPE AND GENERAL

C1.1 SCOPE The Standard specifically limits its application to the design of adjustable static pallet racking (in permissible stress method) made of cold-formed or hot-rolled members. The Standard does not cover materials other than steel.

It is the intention of the Standard to cover such racking commonly referred to as 'adjustable pallet racking' and 'stacker' racking (both 'open' and 'closed' faced) and 'shelf' and other racking, all of which are static.

Other commonly known racking such as 'drive-in' and 'drive-through' racking, cantilever and mobile racking are excluded from the Standard. Due to their nature and special requirements for stability and effective length criteria, these types of racking are not addressed by the Standard.

C1.2 REFERENCED DOCUMENTS The Standard references existing Australian Standards as applicable. The Standard will supplement those Standards and, where applicable, design rules have been applied that deal with the special provisions for 'adjustable static pallet racking'.

C1.3 DEFINITIONS

C1.4 NOTATION

For the purpose of the Standard, a definitions clause and a notation clause have been adopted to specify both commonly used terminology within the industry and the current design notation. Sample illustrations of typical structures have been provided to explain these definitions.

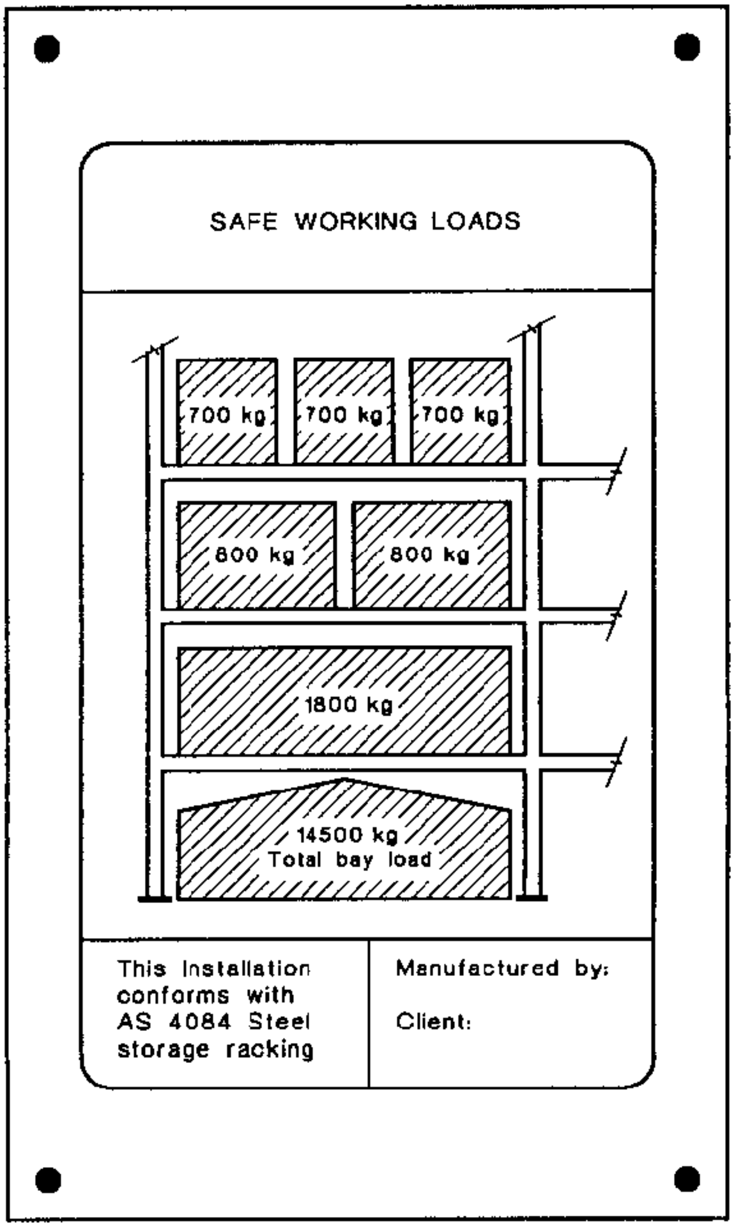
C1.5 USE OF ALTERNATIVE MATERIALS OR METHODS The use of alternative materials and methods is not prohibited, but they should comply with the design procedures set out in Section 3. Appropriate Standards should be applied in the design of materials or methods other than those specified, so as to ensure the safety of racking installations and operating personnel.

C1.6 GENERAL REQUIREMENTS FOR RACKING INSTALLATIONS

C1.6.1 General The following requirements should be taken into consideration:

(a) *Racking installation* It is a requirement of the Standard that each racking structure, including functionality changes within a racking installation, show at one or more conspicuous locations a plaque of specified size stating manufacturer's name and safe working unit loads (for each shelf beam and total bay load).

For an example of plaque, see Figure C1.



NOTE: This Figure is for example only.

FIGURE C1 CORROSION-RESISTANT PLAQUE FOR SAFE WORKING LOADS

(b) *Configuration drawings and design* For each racking installation, a set of racking configuration drawings, as originally ordered or subsequently modified, should be made available.

This information should be retained so that it is available for reference over the lifetime of the installation in cases of inspection, checking or failure of racking installations. This will greatly increase the ease with which problems may be solved.

Because of the nature of racking, commonly bolted together frames with adjustable shelf beams can be altered from the original design. Moving beam levels affects the design capacity by altering the effective length criteria, for example—

(i) raising the beam levels and frame brace nodes, increases the likelihood of flexural torsional buckling of the frame, thus lowering its carrying capacity and increasing instability; and

(ii) lowering beams can produce the reverse effects up until a point where other design failures modes occur.

For both of these reasons, consultation should be sought with the manufacturer of the racking installation.

(c) *Different configuration* Commonly, many installations contain different racking configurations. For each different configuration, the requirements of Clause 1.6.1, Items (a) and (b) are to be met.

(d) *Maximum allowable racking damage* The maximum allowable racking damage specified by the Standard is shown in Figure 7. Where the design specification of the racking differs from that shown in Figure 7, detailed drawings and specifications stating the extent of permissible racking damage should be made available.

(e) *Exemption on small installations* The Clause offers an exemption for relatively small racking installations of the size specified such as non-industrial, supermarkets, low-level order-pick operations and others that meet the criteria. Although all of the detailed specification and design is not required, it is common however to provide order confirmations which briefly outline the basic design for operating loads that the particular racking is designed for, as well as the manufacturer's own specification. Any of the information specified in Items (a) to (d) should also be made available on request.

C1.6.2 Resistance to minor impact One of the most common sources of structural damage and failure of racking occurs from collisions of forklift trucks and other moving unit load handling equipment impacting the racking uprights.

The Standard provides the following methods for protecting the racking from collapse due to minor impact:

(a) *Collision protection devices* Typically, most damage occurs on the aisle side columns approximately 300 mm from the floor level. This is due to the nature of operating equipment, installation and design.

Collision protection devices may be installed at these locations subjected to damage and be designed in accordance with Clause 1.6.2, Items (i) to (iv). These provide a safeguard against minor impact loads. However, note that the extent to which they extend into the aisle may inhibit the recommended operating aisle for mechanical equipment. Check this with the supplier of the equipment.

Although these are typically not designed for a full impact, many are shaped to deflect damage away from the columns.

(b) Those members exposed to minor impact may be designed to resist minor horizontal impact forces as specified in Clause 1.6.2(b) but should not exceed the permissible stresses factored by 1.25.

The following situations may result from this analysis:

(i) Uprights may deform but stay fixed to the ground due to floor fixings and the extent of impact. The extent of deformation on the upright will impair the performance of the upright and should be maintained in accordance with the damage limits shown in Figure 7.

(ii) Uprights may lose contact with the ground thus making the upright ineffective.

C1.6.3 Bracing to building structures When intending to connect the racking structure to part of a building, guidance should be sought to ascertain the viability of such a connection.

For structures commonly referred to as 'racking clad buildings', the racking actually forms the base structure, the roofing and walls being attached to the outside envelope of the racking. These are subject to other significant forces such as wind loads and should be designed accordingly.

SECTION C2 LOADS

C2.1 DESIGN LOADS The Standard includes, in addition to the design vertical loads, vertical impact loads and horizontal loads that a normal racking installation is considered to experience. Forces that could not reasonably be considered to act together need not be considered.

C2.2 VERTICAL IMPACT LOADS An additional load due to vertical impact of individual unit loads during loading is applied to the members and connections immediately supporting the unit loads. The additional load is intended to cover vertical impacts due to normal operation only. It should not be regarded as providing for loadings due to accidental dropping of unit loads from unit load handling equipment.

Since the impact loads relate to the loading into the racking structure of a single unit load only, impact need not be considered for upright frames, uprights and other vertical components which are primarily designed for cumulative loads resulting from a number of individual unit loads.

In testing of beams, supporting arms and end connections, due allowance should be made for a single increased unit load, placed in the most unfavourable position for each particular determination, e.g. moment, shear and reaction. It may be necessary to place the single increased unit load at different locations to check moment, shear and end connections.

C2.3 HORIZONTAL LOADS

C2.3.1 General Horizontal loads due to loading and off-loading, and horizontal forces arising from the interaction of initial out-of-plumb of the racking with gravity loads should be considered for stability, strength and serviceability of the racking structure.

Racking is designed for the greater of horizontal loads due to loading and off-loading and out-of-plumb. These loads are not applied concurrently, since design for out-of-plumb forces is considered to provide adequate resistance to horizontal loads due to loading and off-loading in typical overall racking installations. The horizontal loads are considered to act concurrently with vertical loads including impact and wind loads.

The horizontal forces are considered to act separately, not simultaneously, in each principal direction of the racking. In reality, loads of a somewhat lesser value are likely to act concurrently in each principal direction.

Racking should be designed for any specific horizontal force resulting from unit load handling equipment which is supported wholly or partially on the racking structure.

C2.3.2 Loading and off-loading (placement loads) Horizontal forces arise due to placing of individual unit loads. These loads occur primarily due to the application of dynamic or static thrusts from the unit load handling equipment. The loads are also intended to provide for minor impact at elevated portions of the racking structure. These horizontal forces should be applied to all components which can be approached by the unit load handling equipment.

The design and operation of the unit load handling equipment will dictate the most unfavourable position at which the horizontal force acts.

The following two typical situations arise:

(a) Horizontal and frictional loads being applied on the top flange of the load supporting beams or runners when placing or retrieving loads.

(b) Forces arise from direct impact on the column or load supporting beams due to misalignment of the unit load.

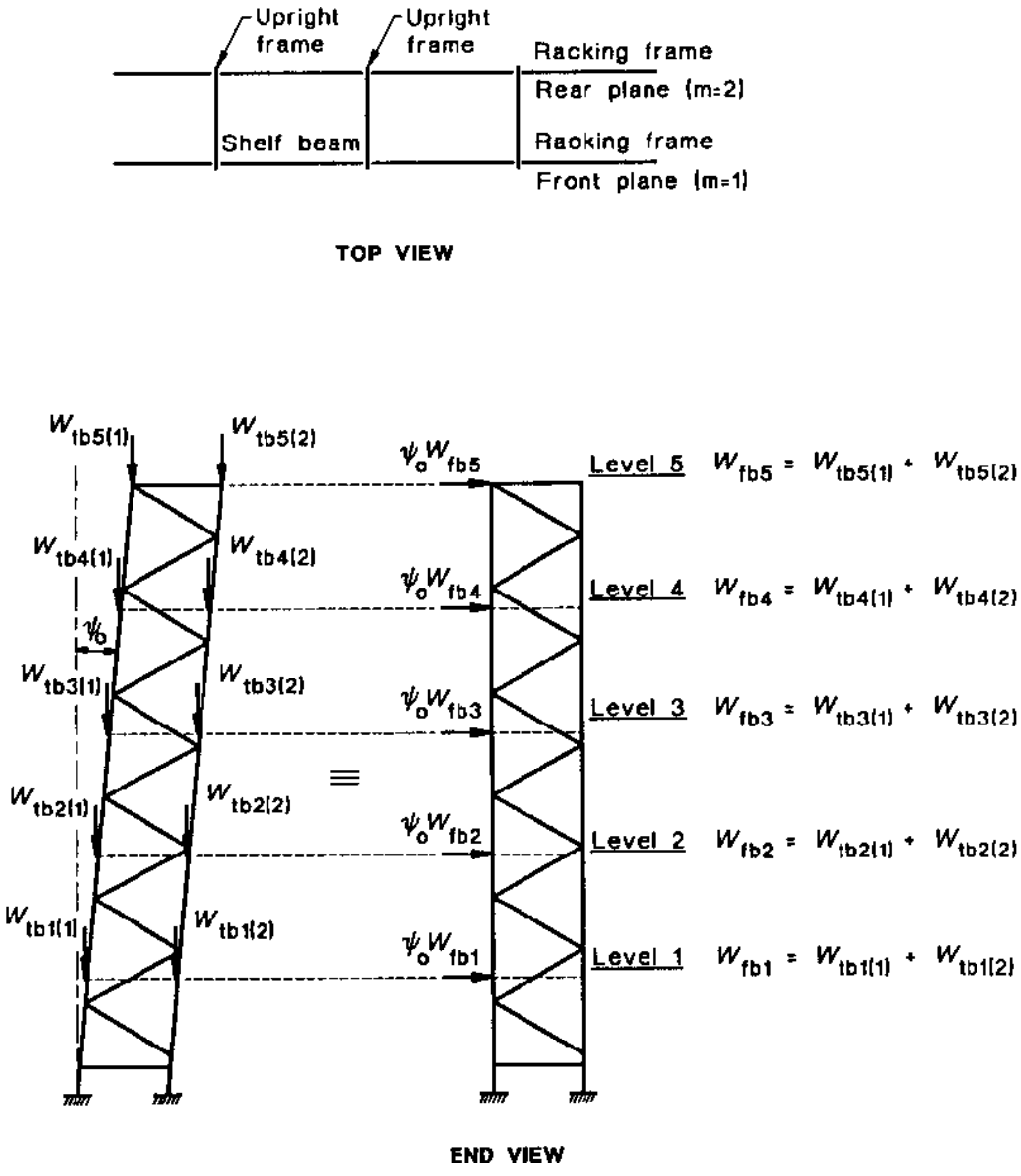
These loads are intended to cover impacts due to normal operation only. They should not be regarded as providing resistance to accidental damage from major impacts (see also Clause 9.3 for inspection of damage due to minor impact and Clause 1.6.2 for provision of minor impact at column bases).

Commonly, due to the adjustable nature of racking, beam-to-column connections have angular rotation (ϕ_1). In the case of bracing members which are typically bolted and where the beam connections are also bolted, the design may dictate rigid connections. If the connections are designed with suitable holes (not oversize), it can be shown without test to be a rigid connection. In these cases, ϕ_1 should be nominated as appropriate to the connection used, and may even be considered to be as low as zero.

C2.3.3 Initial out-of-plumb The effects of initial out-of-plumb due to construction tolerance and connector looseness are allowed for in the analysis of the frames. These take the form of the following equivalent horizontal forces in each direction of the racking:

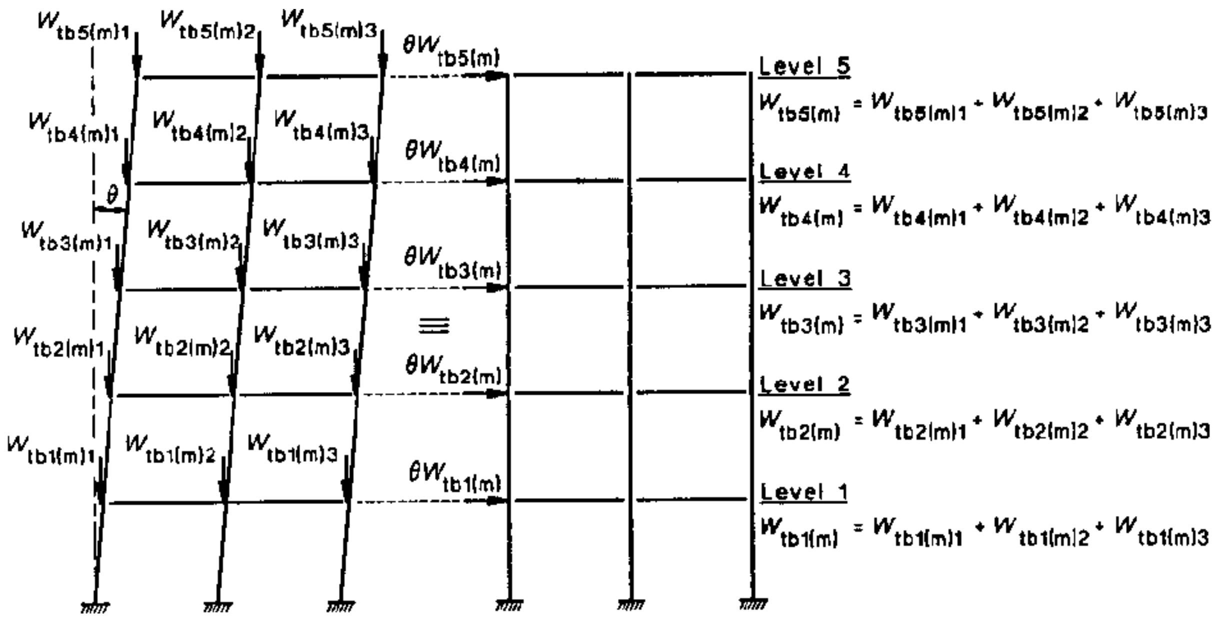
(a) For upright frames: Forces ($\psi_o W_{fb}$) are applied at each loaded level (see Figure C2(a)).

(b) For racking frames in the plane of the beams: Forces (θW_{tb}) are applied at each loaded level (see Figure C2(b)).



(a) For upright frames

FIGURE C2 (in part) INITIAL OUT-OF-PLUMB FORCES



FRONT VIEW

(b) For racking frames

FIGURE C2 (in part) INITIAL OUT-OF-PLUMB FORCES
(In plane m)

SECTION C3 DESIGN PROCEDURES

C3.1 GENERAL The design aspects of the Standard are based exclusively on the permissible stress method. The design of cold-formed members is based on AS 1538 (Reference 1) with modification to account for the effect of perforations (slots) in the upright members. In addition, distortional buckling is considered in Clause 4.2.3.4.

For situations within the racking structure where adequate design methods do not exist, testing of the racking structure should be carried out using Section 8. These procedures provide a series of uniform methods of testing which are appropriate for racking structures. For other situations, the testing requirements of AS 1538 may be used.

C3.2 METHODS OF STRUCTURAL ANALYSIS

C3.2.1 Upright-frame design The Standard allows two methods of structural analysis, namely, the linear elastic method which analyses the structure in its undeformed configuration and the non-linear (second order) elastic method which accounts for the displaced geometry of the structures. This is in line with the elastic methods of the new Steel Structures Code (AS 4100) and overseas codes of racking design, e.g. Section 4 of FEM 10.2.02. Being a permissible stress Standard, the plastic method of analysis is not considered at this stage.

C3.2.1(b) Non-linear (second order) elastic With the availability of the more sophisticated computer programs to the design profession, non-linear (second order) elastic analysis is no longer restricted to the research institutions. These programs not only allow for the displaced configuration of the loaded structure, but also account for the effects of the partial end-fixity on the deformations of the joints. With a little extra computation, the secondary P-Δ effect on the usually flexible structure is accounted for in the analysis.

The computer input in this method is roughly the same as the linear elastic method of Clause 3.2.1(a). The partial end-fixity of the joints (obtained from test as specified in Clause 8.4.2) are input as spring constants. This method gives more realistic behaviour of the structure than previous methods.

Although not stated explicitly in the Standard, the axial capacity alone should be checked using the full effective length of the upright, in addition to the check using the results of a second order elastic analysis and a column effective length factor of 1.0. The axial capacity check alone ensures that frames which have very small horizontal forces resulting from out-of-plumb do not fail by sway instability of the frame.

The use of the factor 1.67 takes into account the fact that non-linear analyses are essentially for the strength limit state and that the Standard is a permissible stress Standard.

C3.2.2 Beams Most of the beam/upright connections are semirigid joints. In line with the European Racking Code (FEM 10.2.02), the Standard allows for the influence of the partial end-fixity in the calculations of the flexural strength and deflection. Guidance in these calculations allowing for the end 'rotational spring constant' effect is given below.

The following equation can be derived for determining the maximum mid-span moment ($M_{max.}$) of a pallet beam considering semirigid end connections:

$$M_{max.} = \left(\frac{WL}{8}\right) r_m \qquad \text{. . . . C3.2.2(1)}$$

where

W = total load on each beam (including vertical impact loads), in kilonewtons

L = span (centre-line to centre-line of the columns), in millimetres

$$r_m = 1 - \frac{LF}{3EI_b\lambda} \quad \text{.... C3.2.2(2)}$$

and where

F = joint spring constant determined either by the cantilever test described in Clause 8.4 or by the pallet beam in upright frames assembly test described in Clause 8.3.2.

E = modulus of elasticity, in megapascals

I_b = beam second moment of area about the bending axis

$$\lambda = \frac{F}{E}\left(\frac{h}{12I_c} + \frac{L}{2I_b}\right) + 1 \quad \text{.... C3.2.2(3)}$$

and where

I_c = column second moment of area about the bending axis

In the above derivation, the load is assumed to be uniformly distributed. For a value of F equal to zero, $M_{max.} = WL/8$. The specification requires applying a vertical impact factor of 25% to one unit load. For a pair of pallet beams supporting two pallets, this would mean that the load on one half of one beam will be 25% more than the load on the other half. The maximum moment will not occur at midspan in that case. However, it can be shown that the magnitude of the maximum moment thus computed will be within 1% of the moment computed on the basis of distributing the total load uniformly.

Again if one considers semirigid joints, the following expression for maximum deflection ($\delta_{max.}$) can be derived:

$$\delta_{max.} = \delta_{ss} r_d \quad \text{.... C3.2.2(4)}$$

$$\delta_{ss} = \frac{5WL^3}{384EI_b} \quad \text{.... C3.2.2(5)}$$

$$r_d = 1 - \frac{2FL}{5EI_b\lambda} \quad \text{.... C3.2.2(6)}$$

The rotational spring constant of the beam/upright connection should be determined by test methods described in Section 8. In the absence of these test, a conservative approach assuming a pinned end support should be used.

Beams in racking systems are commonly formed from two separate channel sections which are fitted together to form a box-like section. The sections are interlocked together along their length and welded at the ends and, depending on the application, at further intervals along the beam element (see Figure C3).

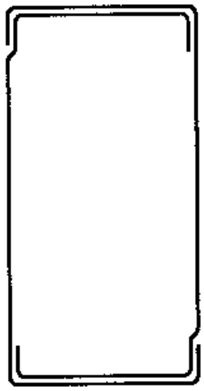


FIGURE C3 INTERLOCKED CHANNEL SECTIONS

While for effective width calculations, the effect of interlocking is negligible and therefore neglected, it can be very conservative to neglect this interlocking action for lateral buckling considerations. Provided the sections are effectively interlocked, i.e. both parts have to interact to balance the rotations about the shear centres, lateral torsional buckling capacity can be significantly greater than would be indicated by conventional analysis of the separate sections, depending on the effectiveness of the interlocking. The influence of pallets in restraining lateral torsional buckling can also lead to significantly greater lateral torsional buckling capacity. This is also true for pairs of single open-channel sections used as beams within typical racking systems.

It is therefore recommended that these beam sections be tested in accordance with Section 8, to allow any increased lateral buckling resistance to be incorporated into the design of the pallet racking system. It is important that beams be tested in pairs to ensure that those items which are present in the racking system and assist in resisting lateral torsional buckling (e.g. pallet support bars and fork entry bars) are included in the test.

Cold-formed tubular sections, also frequently used as beam sections in racking structures, do not suffer from lateral torsional buckling limitations, provided that the section and shape criteria of AS 1250 (Reference 2) are satisfied.

The deflection limit of 1/180 of the span is an industry consensus figure based on visual appearance and operational clearance considerations.

SECTION C4 DESIGN OF COLD-FORMED STEEL ELEMENTS AND MEMBERS

INTRODUCTION The Section deals only with cold-formed steel elements and members. In conformity with Clause 3.1, Clause 4.1 merely repeats that permissible stresses and effective widths shall be as specified in AS 1538 (Reference 1), except where the peculiarities of members used in racking structures necessitate additions or modifications to these specification.

Neither AS 1538 (Reference 1) nor AS 1250 (Reference 2) make provisions for perforated members, particularly of the type routinely used for uprights and other components of racking. The characteristics of these members are that they contain a number of relatively small, frequently odd-shaped perforations (slots) in repetitive patterns of considerable variety depending on the particular manufacturer's configurations. The purpose of these upright perforations is to permit mechanical connections of beams to be made at a great variety of levels, thus permitting wide latitude in racking configurations.

The effect of perforations on the load carrying capacity of compression and flexural members is accounted for by replacing the maximum permissible axial compression stress and the maximum permissible bending stress by modified values which allow for the effects of perforations. The approach is to use the effective section properties based on the net section whereas AS 1538 bases the effective section properties on the gross section. Further information on the development of AS 1538 can be found in Reference 3.

C4.1 ELEMENTS (see Clause 2.4 of AS 1538) The intent of the Clause is to avoid having an effective width larger than the net width. The intent of the modifications of AS 1538 is to use a consistent net section approach. Within this approach, the effective net section and the effective section is never to be greater than the net section.

C4.2 MEMBERS

C4.2.2 Flexural members (see Clause 3.3 of AS 1538) The Standard approach involves a modification of AS 1538 for unperforated flexural members. The procedure consists of obtaining the permissible bending capacity by multiplying the maximum permissible bending stress ($F_{b,net\ min.}$) by the section modulus of the full unreduced gross section. This is equivalent to multiplying the permissible bending stress (F_b) specified in AS 1538 by the section modulus of the net effective section ($Z_{x,net\ min.}$).

$$F_b\, Z_{x,net\ min.} = F_{b,net\ min.}\, Z_{x\ min.} \qquad \text{. . . . C4.2.2}$$

Effective width equations do not exist for the type of perforations that are common in racking uprights. For this reason, approximate approaches need to be formulated. For example, the effective width for the flange of the lipped channel section shown in Figure C4 can, in general, be determined by checking in two ways—

(a) assuming the flange consists of two unstiffened elements of b_{u1} and b_{u2}; or

(b) checking the effective width of the entire flange of width (b).

Item (b) may be necessary if the hole is wide and the width-to-thickness ratio of the element is relatively high.

If the width of the perforation (H_b) and the length of the perforation (L_b) are too large then the expressions for the adequacy of the lip stiffener and the application of the effective width to a possible failure mechanism become questionable. Failure mechanisms observed in stub upright tests on the same section should be kept in mind in formulating an effective width for the flange.

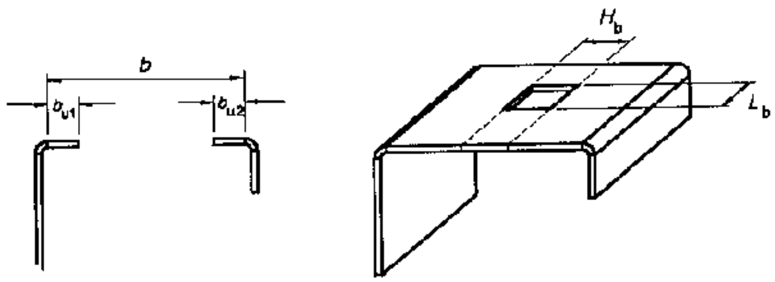


FIGURE C4 PERFORATED FLANGE GEOMETRY

It should be noted that for the lipped channel upright section in bending, the effective net width is to be used for the compression flange and the net width for the tension flange.

It can be shown that for all practical racking upright sections in bending, webs remain fully effective and the effective width for the web does not need to be checked. However, again the width and the length of the perforation in the web should not be too excessive. The failure mechanism in the stub column test would indicate conservatively if the web would be a problem in bending.

In the calculation of the elastic critical stress for flexural-torsional buckling (F_{ob}), the section properties are to be based on the full unreduced gross section considering round corners except for J, β_x, r_{o1} and I_w which should be based on the full unreduced gross section because the calculation of these parameters for the net section is extremely tedious. Furthermore, β_x, r_{o1} and I_w may be computed assuming sharp corners because considering rounded corners in the calculation of these parameters is extremely cumbersome and the differences resulting from this assumption are not very significant.

The extent of inelastic reserve capacity for perforated elements needs further study and is hence excluded in the Standard.

C4.2.3 Axially-loaded compression members (see Clause 3.6 of AS 1538) Compression members, such as uprights, can buckle in either of two ways—

(a) purely flexurally, i.e. by simple bending about one of the principal axes without twist; or

(b) torsional-flexurally, i.e. bending accompanied by twisting of the member.

Some types of members which buckle purely flexurally are—

(i) all closed box-type members; and

(ii) sections whose shear centre and centroid coincide, which is true for doubly-symmetrical members, e.g. I-sections and equal-flange Z-sections.

Many other open thin-walled shapes can be subject to torsional-flexural buckling, such as singly-symmetrical channel, C, hat, and plain or lipped angle-sections. In all these shapes, centroid and shear centre do not coincide. However, whether such members actually will buckle torsional-flexurally or just flexurally in the direction of the axis of symmetry depends not only on the type of cross-section but also on its relative dimensions. Thus, channels with wide flanges tend to buckle torsional-flexurally, while narrow-flanged channels generally buckle only flexurally.

The treatment of concentrically-loaded perforated compression members is based on a modification of the AS 1538 approach for unperforated compression members. The procedure consists of obtaining the permissible axial load capacity by multiplying the maximum permissible axial compression stress ($F_{\mathrm{a,net\ min.}}$) by the gross cross-sectional area (A). This is equivalent to multiplying the permissible axial compression stress (F_{a}) specified in AS 1538 by the minimum net area ($A_{\mathrm{net\ min.}}$).

$$F_{\mathrm{a,\ net\ min.}}\ A = F_{\mathrm{a}}\ A_{\mathrm{net\ min.}} \qquad \text{. . . . C4.2.3}$$

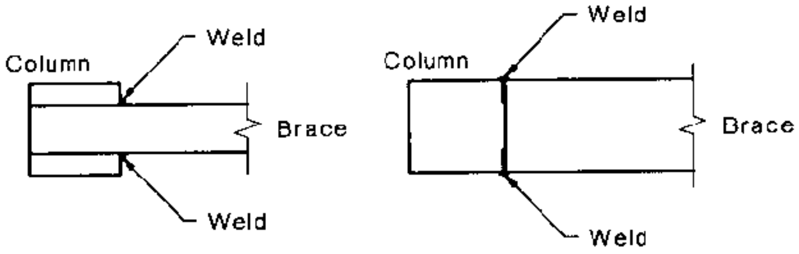
The value of F_{a} is based on the Q factor determined from a stub column test as set out in Clause 8.2. The Q factor allows for the effect of local plate buckling of plates with perforations which cannot be assessed theoretically.

The reduction of the imperfection parameter (η) to half its value specified in Clause 3.6.1 of AS 1538 is based on tests of racking uprights which are generally straighter than conventional hot-rolled columns, and also to allow the Standard to reasonably accurately reproduce the design values in the American Rack Manufacturers Institute Specification on which the Standard was based.

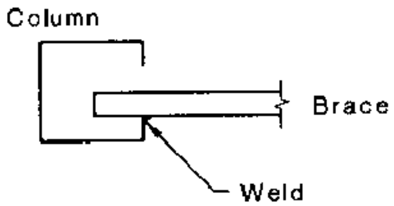
C4.2.3.2 *Sections not subject to torsional-flexural buckling* For members subject only to flexural buckling, provisions of AS 1538 apply with some modifications. For perforated members, it is necessary to spell out which I and which A are to be taken. The Clause stipulates that the second moment of area ($I_{\mathrm{min.}}$) should be calculated for that net section which gives the minimum second moment of area about the axis about which buckling will occur. In general, this is the minor axis if the unbraced length is the same in both principal directions. However, in assemblies such as upright frames, the unbraced lengths may be very different in the two directions and it is possible that buckling in that direction is much larger than for buckling about the minor axis. The Clause further stipulates that the area to be used in connection with $I_{\mathrm{min.}}$ is the net area of the same cross-section which has been used to calculate $A_{\mathrm{min.}}$. Incidentally, it will be found that in many perforated sections, the difference in radius of gyration between net and gross section is not very large because I and A may be reduced in about equal proportions by the presence of perforations. This will facilitate at least preliminary design.

In designing uprights for flexural buckling without torsion, the effective length factors (K) shall be taken as specified in Clause 5.1.

C4.2.3.3 *Doubly symmetric or monosymmetric sections subject to torsional-flexural buckling* For singly symmetric shapes, these methods, while somewhat lengthy, are quite straightforward, provided that the effective length is the same for bending about the axis of symmetry (x axis) and for twisting. This is generally the case for building-type frames, but need not be so for racking structures. For instance, for a pallet racking with channel- or C-columns placed so that the x axis is in the plane of the upright frame, the unsupported length (L_{x}) for buckling about the x axis is the length from the floor to the bottom beam, or between successive beams, as the case may be. This is the unsupported length (L_{x}), not the effective length ($K_{\mathrm{x}}L_{\mathrm{x}}$). However, for torsion it can be assumed, as given in Reference 4, that even light members, such as the diagonal or horizontal struts of upright frames, will prevent twisting at the point where they are connected to the uprights, provided the connection itself does not permit twist. Typical connection details between the uprights and the bracing which are expected to inhibit twist, and those that are not, are shown in Figure C5. For those racking with proper connection details, the unsupported length (L_{z}) for torsion will be the free length between adjacent connections to any members which counteract torsion. For instance, if a diagonal of the upright frame meets the upright column somewhere between the floor and the lowest beam, then the longer of the two lengths, from the diagonal connection to either the floor or the beam, represents the unsupported length for torsion (L_z).



(a) For restrained twist



(b) For unrestrained twist

FIGURE C5 CONNECTION DETAILS

The effect of different effective lengths for torsion and flexure is accounted for by taking K_xL_x in the expression for F_{ox}, and K_zL_z in the expression for F_{oz}. The effective length factors (K_x) and (K_z) are given in Clauses 5.1.3 and 5.1.4, respectively.

C4.2.3.4 *Monosymmetric sections subject to distortional buckling* Distortional buckling involves both membrane and flexural deformations of the cold-formed section as shown in Figure C6. Since the American Rack Manufacturers Institute Specification on which the Standard is based does not allow for distortional buckling, principally because bolted racking with rear flanges are not commonly used in North America, the additional Clause for distortional buckling has been included.

Justification for the method of design is given in References 5 and 6. The design procedure adopted is that specified as Australian Alternative II in both references. The calculation of the allowable stress for distortional buckling can be regarded as independent of those for flexural or flexural-torsional buckling and so provides a separate check to be performed by the designer.

The elastic distortional buckling stress can be computed using the design charts in the Standard (see Figure 3(b)) which were taken from Reference 7 or the design equation specified in Reference 8, or a computer program of the type specified in Reference 9.

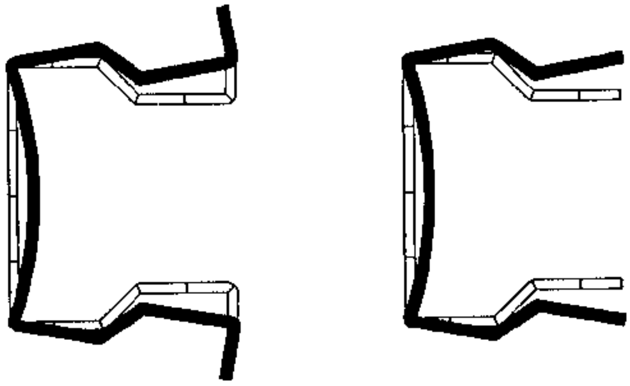


FIGURE C6 DISTORTIONAL BUCKLING

SECTION C5 UPRIGHT FRAME STABILITY

C5.1 EFFECTIVE LENGTH FACTORS It is the common structural engineering practice to use the effective length concept in determining the load carrying capacity of a member subjected to an axial load alone or in combination with bending moments. Such a member is usually part of a frame. The effective length factor accounts for the restraining effect of the end conditions or the effect of the members framed into a particular member. General discussions of the effective length concept can be found in References 10 and 11.

Basically, the effective length factor times the unsupported length (L) gives the length of a simply-supported column which would have the same elastic buckling load as the particular member which is part of a frame or which has other end conditions. Though the effective length is computed on the basis of elastic frame behaviour, it is general practice to use the effective length approach to find the inelastic load carrying capacity. This is the approach taken in AS 1538 and AS 1250 as well as in the Standard. As discussed in connection with Clause 4.2.2, the effective length approach is extended to torsional-flexural buckling as well.

The behaviour of racking structures and hence the effective length factors depends on various peculiarities of racking such as the rigidity of the connection between the columns and beams. Due to the wide variety of details and cross-sectional dimensions in racking structures, the effective length factors vary within a very broad range. For example, for a simple portal frame with pinned column bases, the effective length factor approaches infinity as the connection between the beam and the columns approaches a pinned condition due to the connection details. The values of the effective length factors given in the Standard are by no means maximum values. They are average values assuming the racking to be designed according to good engineering practice and judgment. In all cases, rational analysis would indicate whether the stipulated values are too conservative or very unconservative for the particular racking. Possible rational analysis procedures are presented below.

C5.1.2 Flexural buckling in the direction perpendicular to the upright frames The buckling considered here is parallel to the aisle. In general, racking have single symmetric sections for uprights and, also in general, the axis of symmetry is perpendicular to the aisle. The buckling of such sections parallel to the aisle, namely about the axis of symmetry, takes the form of torsional-flexural buckling. For such cases, the effective length factor is intended to be used in computing F_{ox} in Clause 4.2.3. F_{ox} is in turn used in computing the torsional-flexural buckling load.

C5.1.2.1 *Racking not braced against side-sway* The Clause is applicable to racking that do not meet the bracing requirements of Clause 5.1.2.2. The effective length factor was a typical value that was reached on the basis of analytical computation and test results. The analytical computations were based on the rational analysis procedure given below.

A possible rational analysis procedure for determining effective length coefficients would be to use the graphs and procedures described in Appendix E of AS 1250 with the following exceptions. These exceptions are to account for the semirigid nature of the connection of the uprights to the floor and to the pallet beams. The floor is assumed to be a beam with the following stiffness (I_f/L_f):

$$\frac{I_f}{L_f} = \frac{bd^2}{720} \qquad \text{. . . . C5.1.2.1(1)}$$

where

I_f/L_f = the floor stiffness

b = the width of the upright (parallel to the flexure axis), in millimetres

d = the depth of the upright (perpendicular to the flexure axis), in millimetres

The floor is assumed to be concrete, and the upright connection to the floor should be adequate to develop base moments consistent with this stiffness. For other floor material, a modification of the equation is noted below.

In the analysis, the stiffness of the pallet beams is taken to be reduced to $(I_b/L_b)_{red}$ due to the semirigid nature of the joints and is calculated as follows:

$$\left(\frac{I_b}{L_b}\right)_{red} = \frac{I_b/L_b}{1 + \dfrac{6EI_b}{L_b F}} \qquad \text{. . . . C5.1.2.1(2)}$$

where

I_b = actual second moment of area of the pallet beams

L_b = actual span of the pallet beams, in millimetres

E = modulus of elasticity, in megapascals

F = joint rigidity determined by the portal test in accordance with Clause 8.2.2

The analysis for the effective length factor for the portion of the upright from the floor to the first beam level would involve the following stiffness ratio (G) values as shown in Figures E1 and E2 of Reference 2:

$$G_A = \frac{I_c(1/L_{c2} + 1/L_{c1})}{2(I_b/L_b)_{red}} \qquad \text{. . . . C5.1.2.1(3)}$$

$$G_B = \frac{I_c/L_{c2}}{I_f/L_f} \qquad \text{. . . . C5.1.2.1(4)}$$

where

I_c = upright second moment of area

L_{c2} = distance from the floor to the first beam level, in millimetres

L_{c1} = distance from the first beam level to the second beam level, in millimetres

The effective length factor is then found directly from Reference 2 on the basis of G_A and G_B.

The expression used above for I_f/L_f is based on an interpretation of Reference 12 given in References 13 and 14. The expressions given in References 13 and 14 are modified to reflect the situation for the racking uprights which in general have thin base plates. This expression is a crude representation of the base fixity. The base fixity depends, among other parameters, on the ratio of the base moment to the axial load, namely the eccentricity of the axial load. A general formulation would be quite complex. Tests done with high eccentricities, namely an M/P ratio of more than one half the upright depth ($d/2$) show that the above expression overestimates the fixity given in Reference 15. Though direct test data is not available, it seems reasonable to expect that the above equation would estimate the fixity rather closely for eccentricities corresponding to design load and horizontal loads as specified in Clause 2.3. The eccentricities in this case are about one-third of what was used in the tests of Reference 15. It should be noted, however, that the base fixity is just one of many properties of the racking that affect the structural behaviour.

The expression for I_f/L_f given above assumes that the floor is concrete. For other types of floors, similar expressions can be derived from Reference 13.

The joint rigidity (F) is to be determined by the portal test. As the frame sways, as the type of buckling under consideration implies, the beams of the frame will have different joint rigidities at each end. This is due to the fact that at one end, the rotation is increased while at the other end the rotation is decreased. The portal method yields an intermediate value between the values of the rigidities of the two ends.

C5.1.2.2 *Racking braced against side-sway* A racking structure, in order to be treated as braced against side-sway, should have diagonal bracing in the vertical plane for the portion under consideration. This would restrain the uprights in the braced plane. In order to restrain the uprights in other planes, there needs to be shelves which are rigid or have diagonal bracing in their horizontal plane as specified in the Clause. Some of the terms used above are illustrated in Figure C7(a). The function of this rigid or braced shelf is to assure restraint for the other row of uprights against side-sway with respect to the braced row of uprights. All bracing should, of course, be tight and effective for its intended use.

Horizontal movement, or translation, of the front upright relative to the rear upright of a racking with bracing in the rear vertical plane can, in some cases, be prevented by the presence of pallets on the load beams. To prevent translation of the front upright, the frictional forces between the pallets and the load beams should be capable of resisting a horizontal force perpendicular to the plane of the upright. The magnitude of this force at a bracing point should be at least 2% of the upright load immediately below the beam acting as the horizontal brace. Whether or not sufficient force exists to prevent translation should be determined by rational analysis giving full consideration to factors such as, but not limited to, lighter than normal loads and the absence of any or all loads.

Under typical warehouse conditions, the coefficient of friction between a wood or metal pallet and its supporting beams can be taken as 0.10. Special consideration is necessary in cold storage freezers where operational procedures may produce ice on the connecting surfaces. Representative tests are recommended in this and other conditions, such as greasy or oily environments, where they would likewise be warranted.

In order to cut down the unsupported lengths of the columns, the diagonal bracing should divide the braced plane as shown in Figures C7(b) and (c). At the same time, rigid or braced fixed shelves are to be provided at levels AA in order to have unsupported lengths of h as shown in the figures. If such shelves are not provided at levels AA, then the column should be designed in accordance with Clause 5.1.2.1.

The bottom and top portions of uprights in Figure C7(d) are to be designed as uprights in an unbraced racking whereas those in the mid-portion as uprights in a braced racking.

A rational analysis similar to that described in Clause C5.1.2.1 can also be used for racking braced against side-sway. In this case, the following changes need to be made:

(*a*) $$\frac{I_f}{L_f} = \frac{bd^2}{240} \qquad \text{. . . . C5.1.2.2(1)}$$

(*b*) $$\left(\frac{I_b}{L_b}\right)_{\mathrm{red}} = \frac{I_b/L_b}{1 + \dfrac{2EI_b}{L_bF}} \qquad \text{. . . . C5.1.2.2(2)}$$

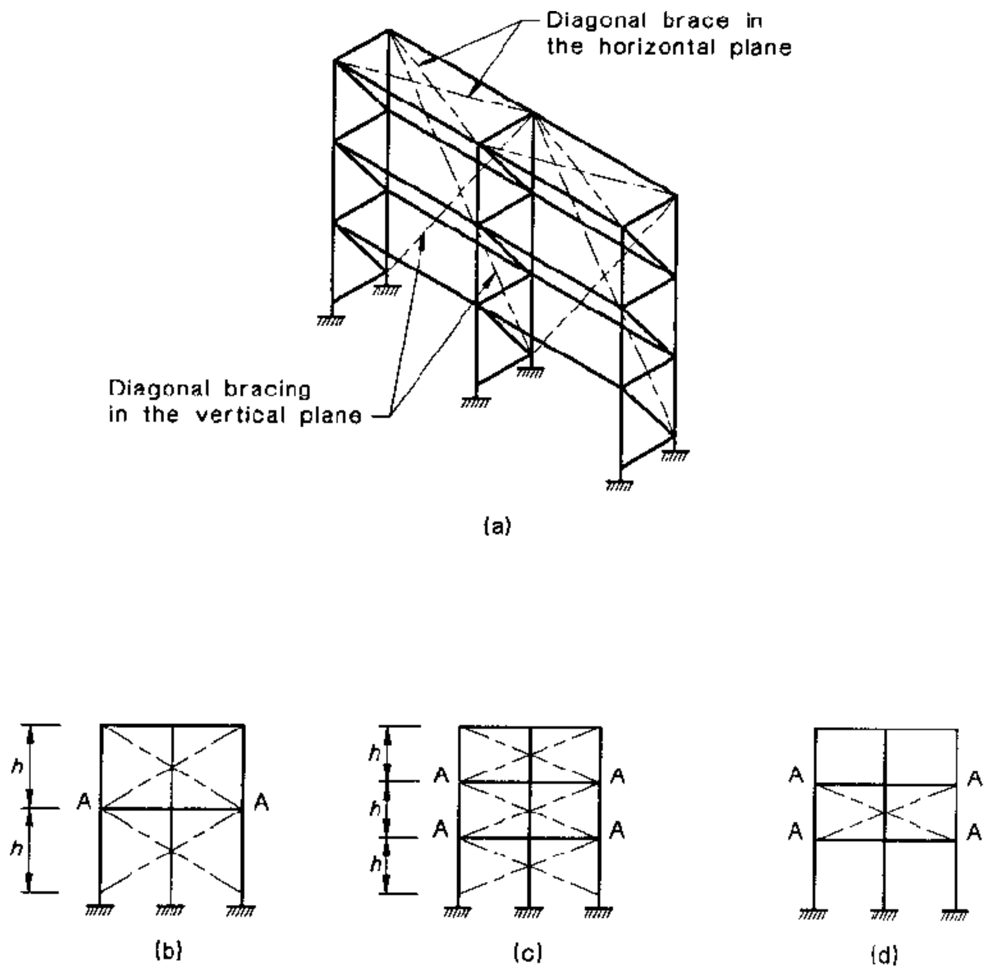


FIGURE C7 RACKING BRACED AGAINST SIDE-SWAY

C5.1.3 Flexural buckling in the plane of the upright frames In racking structures, the uprights are in general either singly symmetric shapes with the axis of symmetry in the plane of the upright frames or doubly symmetric shapes. Because of this, buckling in the plane of the uprights is in general flexural. Upright frames have a wide variety of bracing patterns. The most effective bracing pattern is one where the centre-lines of braces and the uprights intersect at one point as shown in Figure C8(a). This is so because the braces restrain the uprights by virtue of their axial stiffness. On the other hand, the bracing action in the system shown in Figure C8(b) depends on the flexural rigidities of the braces and the connections between the uprights and the braces. Thus this type of bracing is not as effective.

The effective length factor for the frame of Figure C8(a) can be taken in general as one. This assumes that the braces are adequate and the connection between the braces and the uprights are sufficiently rigid in the axial direction of the braces. The effective length factor for the frame of Figure C8(b) is in general greater than one and can be found by rational analysis.

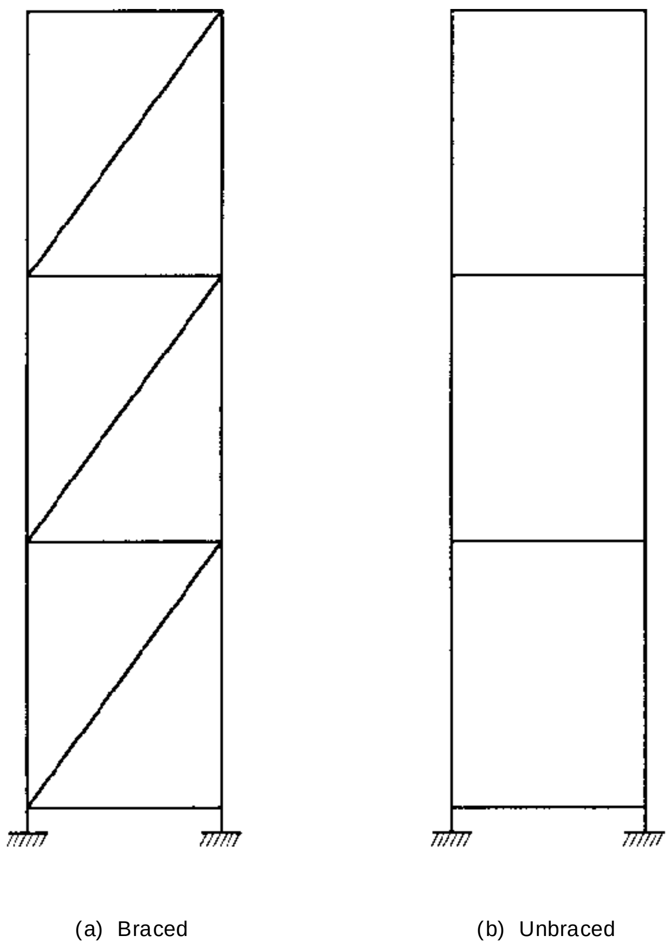
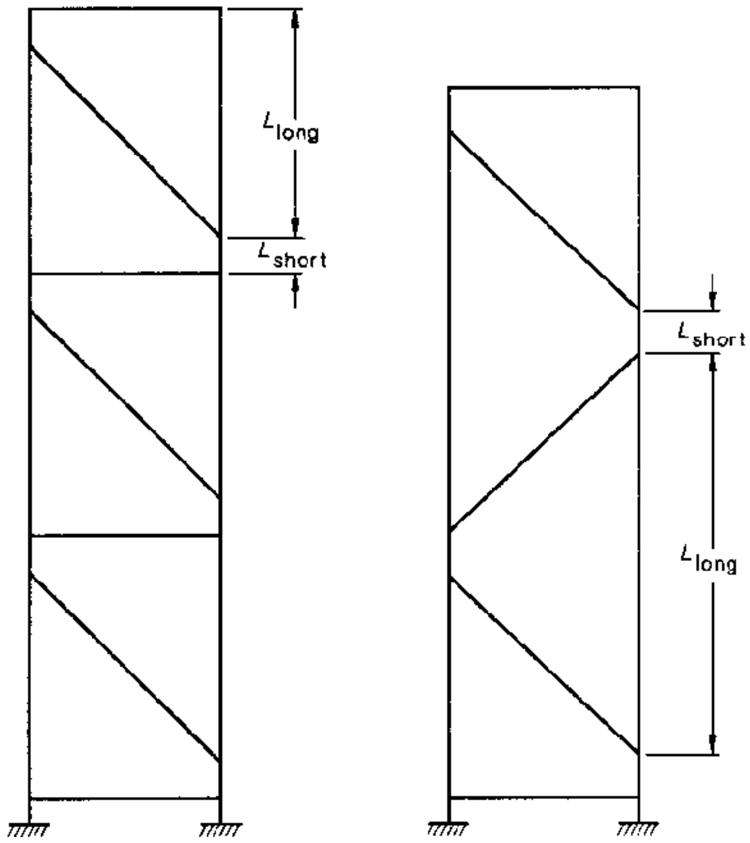


FIGURE C8 BRACED AND UNBRACED FRAMES

In racking structures, frequently the centre-lines of the horizontal and diagonal braces and the centre-line of the upright do not meet at one point. Thus, the bracing arrangement falls between the extremes illustrated in Figures C9(a) and (b). The following bracing configuration are considered:

(a) *Upright frames and diagonal braces or a combination of diagonal and horizontal braces that intersect the uprights* These cases are illustrated in Figures C9(a) and (b). These figures also define the terms L_{long} and L_{short}. As the ratio L_{short} and L_{long} increases, the frame approaches the case shown in Figure C9(b) and hence, the effective length factor can be greater than one.

The stability of the frame is dependent on not only the relative axial and flexural stiffnesses of the members but also the details of connections between the members. The axial stiffnesses at the connection in the direction of the braces is dependent on the details of the connection. For example, the details shown in Figure C9(a) are quite good while the detail shown in Figure C9(b) may not be desirable. In the latter figure, the connection in the axial direction of the brace may be too flexible.



(a) (b)

FIGURE C9 BRACING TYPES WITH DIAGONAL BRACES WHICH INTERSECT COLUMNS

(b) *Upright frames with diagonal braces that intersect horizontal braces* The implied geometry and the definitions of various terms are illustrated in Figures C10(a) and (b). The discussions in connection with Clause 5.1.3 are also quite pertinent to this Clause as well. As the ratio L_{short} / L_{long} increases, the basic behaviour of the frame approaches that of Figure C10(b) and hence the effective length factor can be greater than one.

For uprights having bracing patterns such as the configurations shown in Figure C10(b), no typical effective length factors are recommended. Rational analysis is to be used for such cases to determine the effective length factor. Alternatively, the load carrying capacity may be determined by test.

C5.1.4 Torsional buckling A general discussion of the torsional and torsional-flexural buckling can be found in Clause C4.2.3.3. Though torsional buckling is not likely to happen in racking structures, torsional-flexural buckling is quite usual. The torsional buckling effective length factor is a parameter in the analysis of torsional-flexural behaviour as discussed in Clause C4.2.3.3. The provision of the Clause is based on References 4, 16 and 17. The value of K_z given in the Clause assumes an effective connection between the uprights and the braces as shown in Figure C5.

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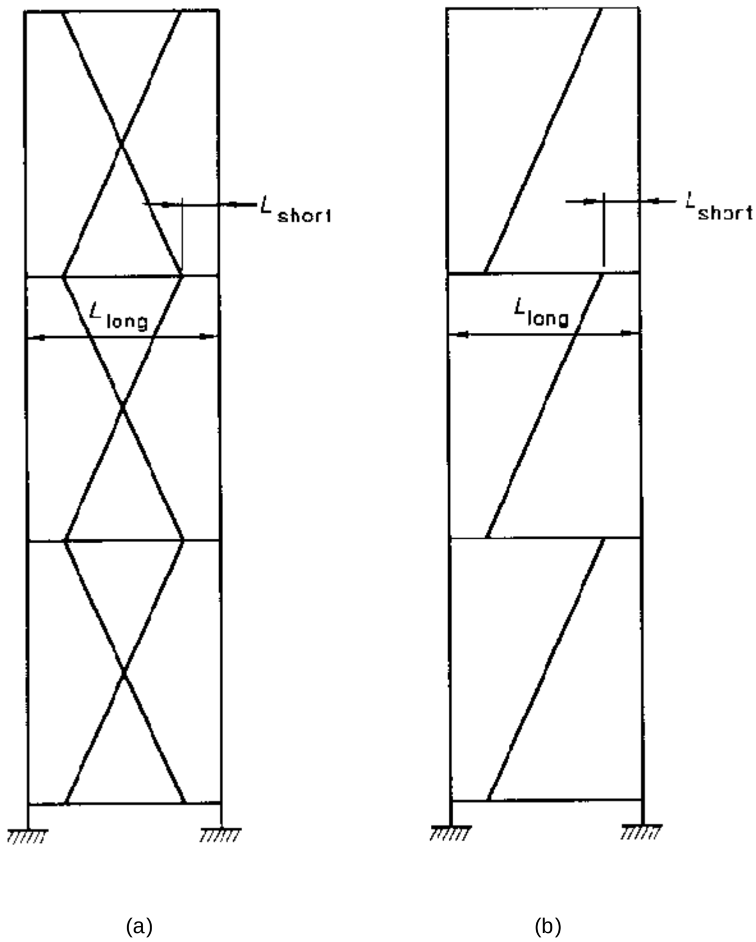

FIGURE C10 BRACING TYPES WITH DIAGONAL BRACES THAT INTERSECT HORIZONTAL BRACES

C5.2 STABILITY OF TRUSSED-BRACED UPRIGHT FRAMES The provisions of the Clause are based on Reference 18 with the exception of the value of *K*. The expressions given in the Reference are for members that have constant axial force throughout their entire length. The effective length factor (*K*) is intended to modify these expressions for the case of non-uniform distribution of axial forces. The provisions of the Clause are more likely to be applicable for high-rise racking.

SECTION C6 CONNECTIONS AND BEARING PLATES

C6.2 BEAM SUPPORT CONNECTIONS The upward load is specified to prevent accidental disengagement of the beam connection. The upward force should be applied to an unloaded beam.

Distortion of the locking device that prevents reapplication of upwards force or reduces the carrying capacity should be considered failure.

C6.3 BASE PLATES A base (bearing) plate should be provided at the bottom of each upright to spread the high concentrated load to the supporting floor. The base (bearing) plate and its anchor bolts should also be designed to transfer any uplift, shear and moment imposed by the upright to the floor.

Actual field experience and limited testing have shown that base (bearing) plates thinner than those normally provided by hot-rolled structural shapes, designed in accordance with AS 1250, with the maximum permissible bearing stress (F'_p) determined in accordance with Clause 6.3 may be acceptable.

In lieu of a rational analysis, appropriate testing may be carried out using a test load of 1.5 times the design load. Under the test load, visible deformation of the base (bearing) plate and local damage of the concrete floor are not permitted. To reduce the probability of local buckling at the base, fixing of the upright to the base (bearing) plate either by means of welding or bolting should be adequate to transfer the test load.

Guidance should be sought to ascertain the adequacy of the supporting floor to carry the imposed load from the racking.

C6.4 CONNECTIONS TO BUILDINGS Many building structures are more flexible than the racking structure. Any attachment should be made with provision for vertical and horizontal building movements. This could be achieved using such attachments which are proportioned so that the attachment would fail prior to causing damage to the building structure or racking.

C6.5 UPRIGHT SPLICES Where full upright contact areas can be achieved, splices are required to keep the connecting parts in accurate positions and to transfer any tensile forces due to bending or uplift across the joint. If the full contact areas are not achievable, the joint should be designed to transmit the imposed loading. Splices are not permitted below the lowest beam level on the front face of a racking due to potential impact from unit load handling equipment.

SECTION C7 TOLERANCES AND CLEARANCES

C7.1 FINISHED TOLERANCES IN UNLOADED CONDITION The finished tolerances in the unloaded condition are recommended to facilitate a smooth operation of the unit load handling equipment in addition to safety. The recommended figures are permissible maximum deviations from nominal dimensions, resulting from manufacture and erection.

C7.2 UNIT LOAD CLEARANCES The clearances are required for nominal distances between field and moving parts and which, all individual tolerances considered, prevent collisions.

SECTION C8 TEST METHODS

C8.1 INTRODUCTION

C8.1.1 General Many factors affecting the design of racking are difficult to account for analytically. Section 8 spells out a series of optional tests that may be used to evaluate the effects of components on the overall behaviour.

Except as either modified or supplemented herein, the rules of AS 1538 and AS 1250 apply to the testing of components.

Engineers involved in racking design are probably familiar with the test procedures stipulated in the Standard. However, some comments bear reiterating here. The important factor that should be kept in mind is that a test procedure should be such that the test results are repeatable. Anyone using the same test procedure on the same specimen should arrive at the same results.

It is also important that tensile coupons are taken from each specimen to determine the actual yield stress. Generally, the actual yield stress of the steel is higher than the specified minimum yield stress. It is important to know the actual yield stress in order to analyse the test results. It is also essential to have a complete report spelling out the procedures, the results and the analysis of the results.

C8.2 STUB COLUMN TESTS Because of the interplay of three influences which affect a cold-formed perforated compression member, i.e. local buckling, perforations, and the cold-work of forming, recourse should be taken to determination by tests. This is done by stub column tests, i.e. by careful concentric compression testing of pieces of the member short enough so as not to be affected by column buckling. The details of such testing are spelled out in Clause 6.1 of AS 1538.

C8.2.2 Evaluation of test results Q is a factor used in Clause 4.2.3.1. The column equation, as well as the test determination of Q, utilise the yield stress of the material. It is therefore essential that the value of F_Y used in the column equation be connected with the yield stress (F_Y) used when determining Q. This is elaborated below.

The basic definition of Q is:

$$Q = \frac{\text{Actual stress of stub column}}{\text{Hypothetical maximum stress without weakening influences}} \quad \text{.... C8.2.2(1)}$$

In turn, this hypothetical strength in the case of non-perforated sections is A_{full} times F_Y. For shapes with Q less than 1, AS 1538 permits the cold-work in the flats to be utilised, but not that of the corners.

For perforated members, the Standard assumes the hypothetical maximum stress to be governed by the minimum net section ($A_{\text{net min.}}$) of a plane appropriately passed through the perforations. Correspondingly, Q is defined as:

$$Q = \frac{\text{Ultimate stress of stub column by test}}{F_Y \, A_{\text{net min.}}} \quad \text{.... C8.2.2(2)}$$

In regard to the yield stress (F_Y) to be used in determining Q by test, and the value F_Y for calculating the stress of columns in accordance with Clause 3.6 of AS 1538, the following needs attention:

(a) In calculating column stress according to Clause 3.6 of AS 1538, F_Y is the specified minimum yield stress to which the steel is ordered by the fabricator. On the other hand, the yield stress of the particular coil from which the stub column test specimens will have been made, will be different and in general somewhat greater than the ordered minimum yield stress. In order for the determination of Q to be sufficiently accurate, it is necessary that the virgin yield stress of the stub column test material, before forming, be as close as possible to the specified stress.

(b) The yield stress (F_Y) should not deviate from the specified minimum yield stress by more than −10% to +20%. With this proviso, the Standard, in conjunction with Clause 3.1.3 of AS 1538, allows the determination of F_Y in the equation for calculating Q and consistent values of F_Y for calculating column strength in accordance with Clause 3.6 of AS 1538.

For a series of columns having different thicknesses, the thickest and the thinnest may be tested. For any intermediate thickness, the value Q so determined should be used in column stress calculations in accordance with Clause 3.6 of AS 1538 in conjunction with a value Q, obtained by similar interpolation, i.e.:

$$Q = Q_{min.} + \frac{(Q_{max.} - Q_{min.})(t - t_{min.})}{(t_{max.} - t_{min.})} \qquad \text{. . . . C8.2.2(3)}$$

where $Q_{min.}$ is for the stub column with thickness $t_{min.}$ and $Q_{max.}$ is for the stub column with thickness $t_{max.}$ both determined as above. Note that $Q_{min.}$ is not the smaller of the two Q-values, but the Q-value for the stub column of the smaller thickness.

This method is sufficiently accurate only if the actual virgin yield stresses of the two stub columns with $t_{max.}$ and $t_{min.}$ are not too different. For this reason, the Standard limits this difference to 25%.

It is acceptable to interpolate the Q-values for a series of shapes with identical cross-section and perforation dimensions, but with a variety of thicknesses. For this purpose, $Q_{max.}$ and $Q_{min.}$ should be determined from stub column tests on specimens made with the maximum and minimum thicknesses of material from which the stub column is made. This correction is necessary in order to avoid unsafe design in case the virgin yield stress, before forming, of the specimens was significantly higher than the specified minimum.

Using the procedures above, it is possible to obtain Q-values greater than 1. This is so if the neglected strengthening effects of the cold-work outweigh the weakening effects of perforations. However, it is basic to the use of Q in AS 1538 that it can be equal to or less than, but not greater than 1. Correspondingly, the Standard provides that if the selected procedure for determining Q results in a Q-value greater than 1, a Q-value of 1 should be used.

C8.3 PALLET BEAM TESTS In the Clause, depending upon the information required, two different types of tests are specified, i.e. simply-supported pallet beam tests and pallet beam in upright frame assembly test. The loading in these tests is applied by means of a test machine or jacks. These loading means restrain the torsional distortions and hence may lead to unconservative results for members subject to such distortions.

The beam test methods illustrated do not account for impact. However, in practice, the test results will have to be adjusted to consider the added impact effect.

C8.3.1 Simply-supported pallet beam tests This test can also be used in the design of beams, in general, when the end restraint is deemed not to lead to significant increase in the load carrying capacity. In the determination of yield and ultimate moments, the number of tests needed should be determined in accordance with Clause 6.2.2 of AS 1538.

C8.3.1.2 *Test set-up* The test set-up illustrated in Figure C11 should be used.

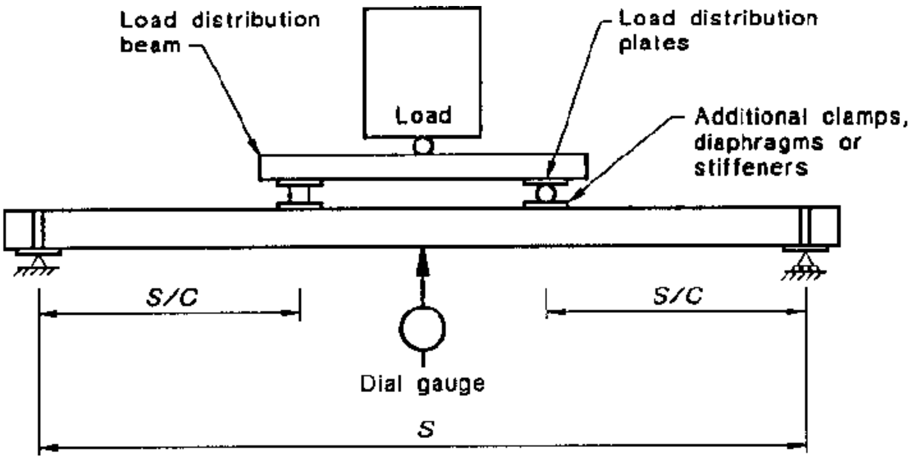

FIGURE C11 BEAM TEST SET-UP

The value of *C* used in Figure C11 should be between 2.5 and 3 and has been chosen to avoid shear failure and to ensure a sufficiently long portion with constant moment. For most pallet beams, the end connection detail is such that the beam can be placed directly on the supporting surface and have simply-supported end conditions. In this case, the clamps, diaphragms or stiffeners at the supports would normally not be needed.

C8.3.1.4 *Evaluation of test results* General guidelines given in Clause 8.1.4 should be used in addition to the particular requirements specified herein.

C8.3.2 Pallet beam in upright frames assembly tests This test is intended to simulate the conditions in the actual racking as closely as possible to determine the permissible load. This test may also be used to determine the magnitude of the joint spring constant (*F*) described in Clause C8.4. For vertical loads, this test may reflect the actual behaviour of the connections more accurately than the test described in Clause 8.4.1.

C8.3.2.2 *Test set-up* It is specified that the upright frame not be bolted to the floor even if the actual racking are. The test is intended to represent the behaviour of the racking between the inflection points. Therefore, any restraint at the column bases other than that due to the pressure, should be avoided.

It is important to minimize friction between beams and pallets because new, dry pallets on new, dry beams, when used in the test, could provide considerably more bracing than pallets and beams worn smooth in use and possibly covered with a film of oil.

C8.3.2.4 *Evaluation of test results* General guidelines given in Clause 8.1.3 should be used in addition to the following criteria for determining the permissible load:

(a) The first of these prescribes an overall safety factor of 2.0. The question is often asked why a factor of 2.0 is necessary when the basic safety factor for beam computations, both in AS 1538 and AS 1250 is 1.67. The answer to this is two-fold. Firstly AS 1250 prescribes, for test evaluation, a safety factor of 2.0 on live load and dead load. Safety factors applicable to load tests in all specification and codes are generally larger than those incorporated in design specifications for a variety of reasons. The material from which the test specimen has been made will, in general, be of somewhat higher yield stress and ultimate strength than specified for the particular steel, since furnished steel properties are generally, but not always, above specified minimum. Since the beams in an actual racking should show the necessary strength reserve even if made of steel exactly as specified rather than of steel overstrength to various degrees, a larger test safety factor is used to compensate for this. Secondly, a beam under load usually does not fail when yielding has just been reached in the most highly stressed fibre, because of subsequent plastic strength reserve. The strength criterion by calculation, however, is incipient extreme fibre yielding. This means that beams so calculated in actuality have some plastic strength reserve, i.e. have a real safety factor larger than 1.67 by amounts which depend on beam configuration. For these and similar reasons, test safety factors are universally kept somewhat higher than those on which design calculations are based.

(b) The second criterion by which to determine permissible loads from the test results, prescribes a safety factor of 1.5 against excessive local distortion. This requirement, again, is similar to that of AS 1538 where a safety factor of 1.3 is used.

(c) The third and last criterion limits deflections of beams under design load to 1/180 of the span. To satisfy this requirement, the load which results in this amount of deflection should be read from the load deflection curve plotted from the test results. If this load is less than those obtained from Items (a) and (b), the load obtained from Item (c) should be taken.

If this test is used to determine the joint spring constant (F), a rational analysis procedure can be followed. A conservative value of F can be calculated from the deflection equations given in Clause C3.2.2. The value of $M_{max.}$ calculated in accordance with Clause C3.2.2 can be obtained from the test at both the design load and 1.67 times the design load.

All the other terms given in Clause C3.2.2 are self-explanatory. The lesser of the F values, computed at both the design load and 1.67 times the design load, should be divided by 1.5 to account for possible test scatter. The number of tests can be specified as discussed in Clause C8.4.1.4.

C8.4 PALLET BEAM TO COLUMN CONNECTION TESTS The tests specified in the Clause have two objectives. One is to determine the moment capacity of the connection, the other is the determination of the joint spring constant (F) described in this Clause for use with the rational analysis approach.

In a rigid frame analysis, the members connected in a joint are assumed to maintain the angle between themselves while the frame deflected under applied loading. The joints between the upright column and the pallet beam do not in general behave as rigid joints. This is primarily due to the distortion of the walls of the columns at the joint and to a lesser extent due to the distortion taking place at the connectors themselves. This peculiarity influences the overall behaviour very significantly. The connection details vary very widely. Thus it is impossible to establish general procedures for computing joint stiffness and strength. It is therefore necessary to determine these characteristics by simple tests.

The change in angle (θ) between the column and the connecting beam, in radians, can be idealized as follows:

$$\theta = \frac{M}{F} \qquad \text{. . . . C8.4}$$

where M is the moment at the joint between the connecting members and F is the spring constant relating the moment to the rotation.

C8.4.1 Cantilever test The cantilever test provides a simple means of determining the connection moment capacity and rigidity. However, it has the disadvantage that the ratio of shear force, i.e. the vertical reaction, to the moment at the joint is not well represented. For typical racking connections, this ratio is probably higher than it is in the cantilever test as specified in the Standard. In general, a higher ratio would probably lead to a more rigid connection. However, bending moment and shear force would interact and lower the ultimate load of the connection. This effect should be studied by reducing the length of the cantilever to the distance between the end of the beam and the expected location of the inflection point.

This test is suitable for determining F for computing stresses due to vertical loads. A somewhat more tedious but more accurate determination of F can be achieved by tests in accordance with Clause 8.3.2.

C8.4.1.2 *Test set-up* The test set-up illustrated in Figure C12 should be used.

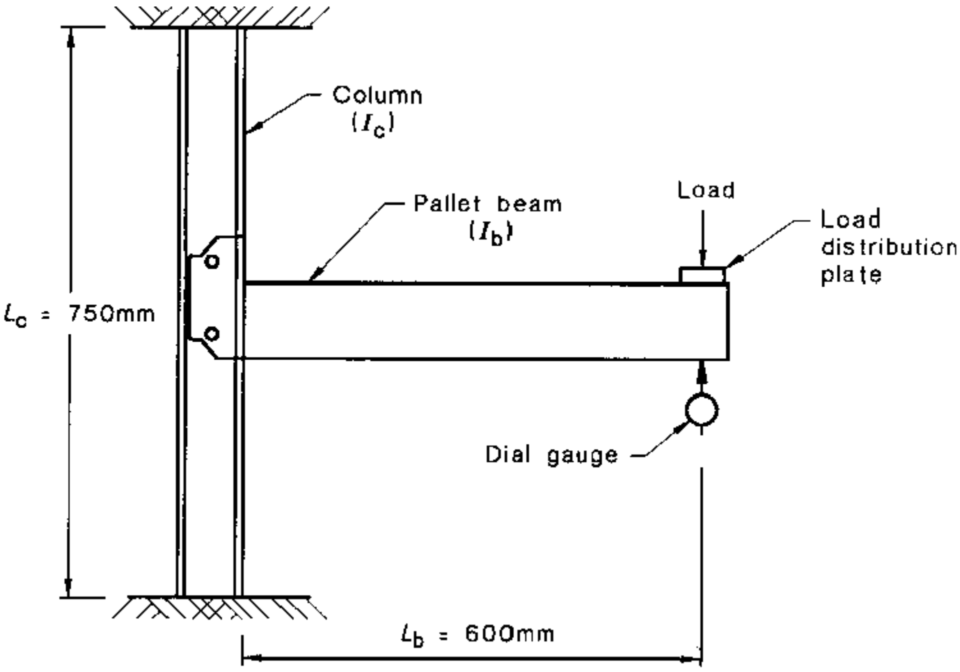

FIGURE C12 CANTILEVER TEST

C8.4.1.4 *Evaluation of test results* The relationship between the moment and the angular change at a joint is not linear. The following equation appears to be reasonable for determining a constant value of F to be used in a linear analysis:

$$F = \frac{RF}{\dfrac{\delta_{0.85}}{P_{0.85}L_b^2} - \dfrac{L_c}{16EI_c} - \dfrac{L_b}{3EI_b}} \qquad \text{. . . . C8.4.1.4}$$

where

RF = reduction factor to provide safety considering the scatter of test results. Since a lower *F* value means a higher design moment for the beam, an *RF* value equal to 0.67 may be taken in the design of the beam. However, in determining bending moments for the columns, a higher *F* value leads to a more conservative value of the bending moment. It is therefore recommended to take *RF* equals to 1.0 for this case.

$\delta_{0.85}$ = deflection of the free end of the cantilever at load $P_{0.85}$, in millimetres

$P_{0.85}$ = 0.85 times the ultimate load, in kilonewtons

L_c, L_b = length of the column and the beam, respectively, in millimetres

I_c, I_b = second moment of area of the column and the beam, respectively.

It is suggested that the spring constant (*F*) be calculated on the basis of the average results of two tests on identical specimens provided that the deviation from the average does not exceed 10%. If the deviation from the average exceeds 10%, then a third specimen is to be tested. The average of the two higher values is to be regarded as the result in the design of the columns.

C8.4.2 Portal test Based on observations, the portal test appears to be desirable when the value of *F* obtained is to be used in a side-sway analysis either for horizontal deflection or stability. Under vertical loads, the connections in general ‘tighten up’. Subsequently, under side-sway, the connection at one end of the beam ‘tightens up’ while the connection at the other end ‘loosens’. The portal test gives an approximate average value of the spring constants involved in this process. Thus it is more desirable to use the portal test for evaluating side-sway behaviour, namely, the effective lengths and horizontal deflections.

C8.4.2.2 *Test set-up* A schematic of the test set-up is shown in Figure C13. According to the Standard, *h* equals 600 mm.

Dial gauge 1 should be used to measure the horizontal deflections (δ) of the racking. Dial gauges 2 and 3 indicate whether the column bases are properly restrained or not. In lieu of dial gauges, other deflection measuring devices may be used. In general, the friction between concrete and the half round bars is enough for this restraint.

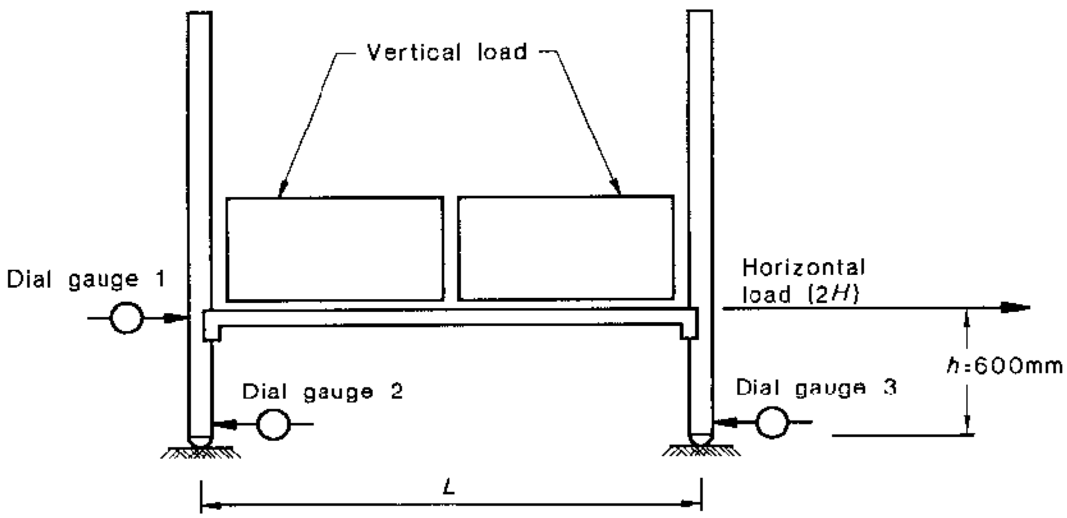

FIGURE C13 PORTAL TEST

C8.4.2.4 *Evaluation of test results* The following is given as a possible rational analysis. Considering a portal of height h and span L with second moments of area of the columns and beams designated I_c and I_b, respectively, an expression for maximum side-sway deflection (δ) corresponding to a horizontal load of $2H$ combination is as follows:

$$\delta = \frac{Hh^3}{3EI_c} + \frac{Hh^2L}{6EI_b} + \frac{Hh^2}{F} \quad \text{. . . . C8.4.2.4(1)}$$

Solving this equation for F, the following is obtained:

$$F = \frac{RF}{\dfrac{2\delta}{Hh^2} - \dfrac{h}{3EI_c} - \dfrac{L}{6EI_b}} \quad \text{. . . . C8.4.2.4(2)}$$

where

RF = reduction factor to be taken equal to 2/3

δ = sway deflection corresponding to a horizontal load of $2H$

H = horizontal load per beam, in kilonewtons

h = distance from the floor to top of the beam, in millimetres

L = distance between the centroid of the two columns parallel with the shelf beam, in millimetres

E = modulus of elasticity, in megapascals

I_c = second moment of area of the column about the axis parallel with the upright frame

I_b = second moment of area of the beam about the axis parallel with the floor

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. Multiple tests as recommended in Clause C8.4.1.4 are also recommended here.

C8.5 UPRIGHT FRAME TEST The hazard of collapse of a full-scale high-rise racking system poses severe safety problems. Therefore, the testing procedures proposed here are geared to a reduced-scale test that will, by simulating a full-scale test, establish the upright frame capacity in a safe manner. The tests are further intended to simulate the conditions in the actual racking as closely as possible.

C8.5.2 Horizontal load in the direction perpendicular to the upright frame The test set-up for horizontal load in the direction perpendicular to the upright frame illustrated in Figure C14 should be used.

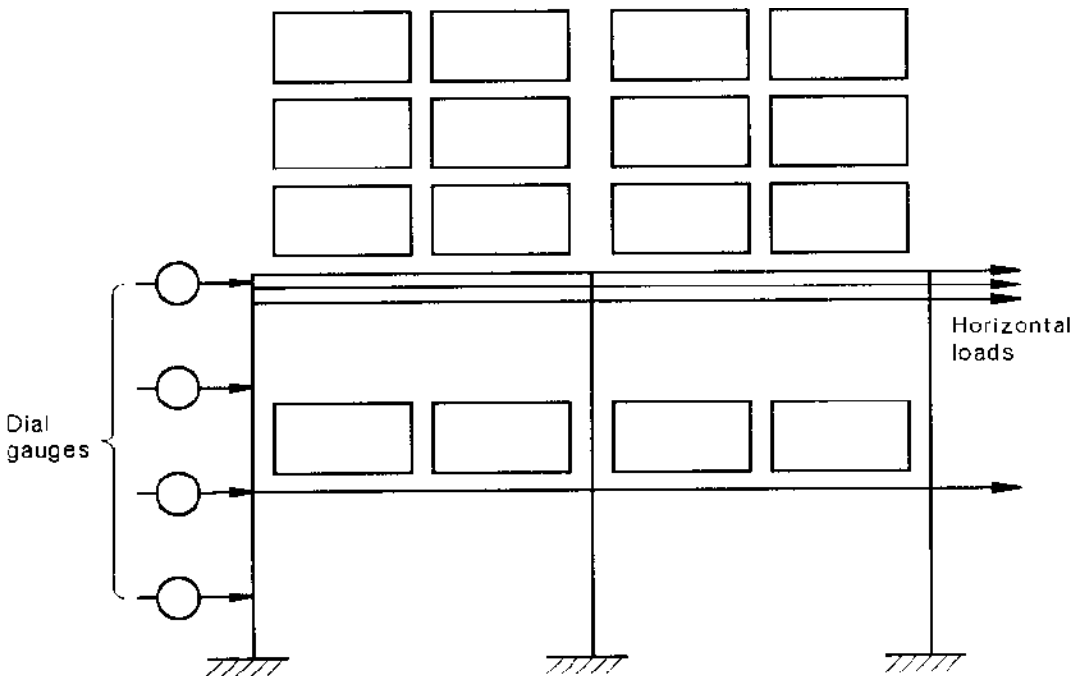

FIGURE C14 TEST SET-UP

SECTION C9 OPERATION AND MAINTENANCE OF ADJUSTABLE PALLET RACKING

C9.1 GENERAL The purpose of the Section is to ensure that the racking installation is operated safely and maintained correctly during its serviceable life.

C9.1.1 Safe working loads The safe working loads will generally be indicated on configuration drawings provided by the manufacturer of the racking installation and shown on a plaque fixed to the racking structure (see Clause 1.6.1(a)).

C9.1.2 Alteration of the racking installation Because pallet racking is by its very nature adjustable, it can be easily altered from its original configuration. However, any deviation from the original design of the racking installation may impair seriously the safety of the installation.

Proposed alterations from the safe working loads and configuration, supplied with the racking installation (see Clause 1.6.1(b)), may be referred to the manufacturer for guidance.

C9.1.3 Operating instructions A plaque similar to that shown in Figure C15, fixed securely to the racking structure, may be used to provide suitable operating instructions.

C9.1.4 Hazardous situations Adjustable pallet racking is particularly prone to damage when the following conditions exist:

(a) high product turn-around demands rapid loading and unloading operations;

(b) large access aisles are used to allow for reach or counter-balance forklift trucks;

(c) unit loads and pallets are in poor condition;

(d) access aisles are obstructed with debris or similar, making racking unloading and loading difficult; or

(e) operator skill level or forklift truck condition is poor.

Hazardous situations can usually be accommodated in a racking installation, provided they are identified at the design stage.

C9.1.5 Damage report Encouraging to report any damage to the racking as it occurs, allows early assessment and prevents compounding of potential problems.

C9.2 INSPECTIONS Regular inspections are necessary to maintain the racking in a serviceable condition. Load application, racking configuration drawings and specifications, should be retained to facilitate inspection and checking of the racking installation.

Visual inspection of the racking is generally sufficient to determine if the racking components have been damaged to a degree requiring further checking or replacement. Typical areas of concern includes damage that—

(a) significantly alters the original cross-sectional shape of a member;

(b) deforms the straightness of a load-bearing member;

(c) causes misalignment of the racking; or

(d) causes weakening of welds or distortion of bolted joints.

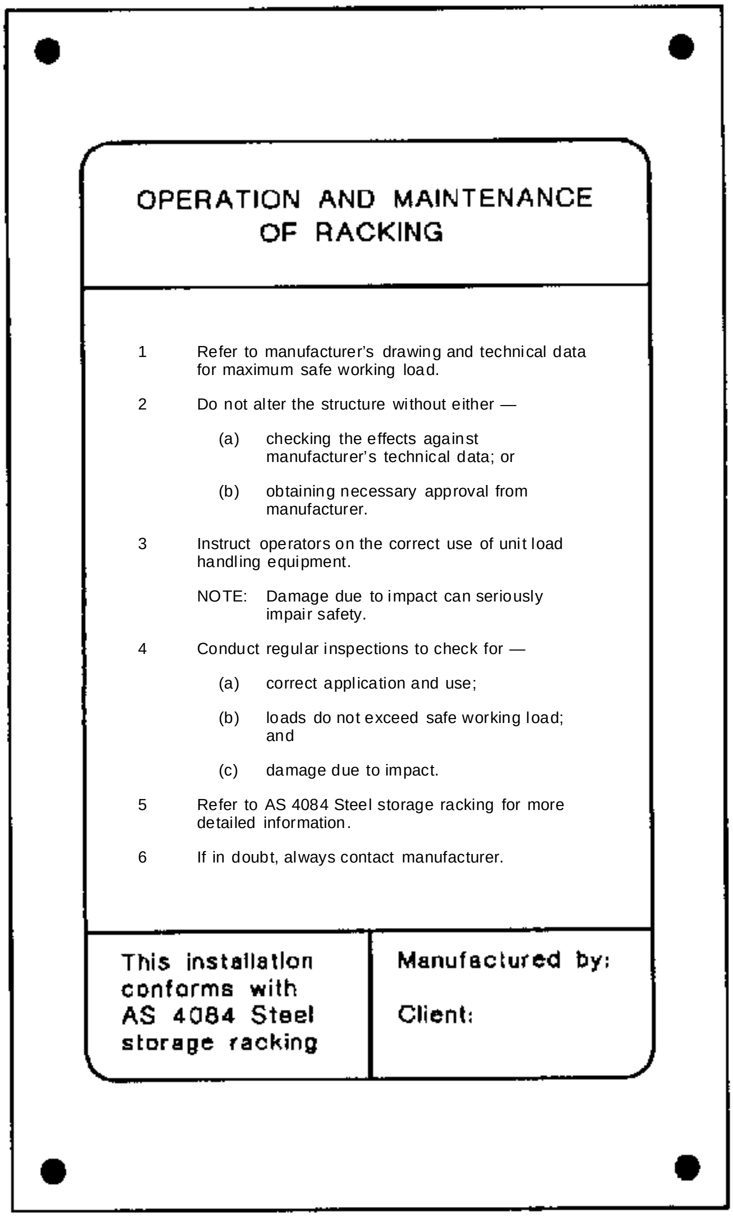

FIGURE C15 CORROSION-RESISTANT PLAQUE FOR OPERATION AND MAINTENANCE OF RACKING

C9.3 DAMAGE DUE TO IMPACT For economical reasons, the components of adjustable pallet racking are manufactured from light gauge material. This tends to render them liable to impact damage which may significantly impair the safety of the racking.

Visual inspection, following damage reports or through regular inspections, is used to assess if the racking components have been damaged to a degree requiring further checking or replacement.

Evidence of damage may be referred to the racking manufacturer who will generally provide assistance in evaluating damage and advice on the correct steps to be taken to ensure the racking is returned to its original design capacity.

C9.4 OUT-OF-PLUMB OF RACKING The factor of 1.5 allows for some permanent deformation from the finished tolerance which generally occurs in steel structures after the initial cycle of loading.

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