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Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks

Rack Manufacturers Institute

Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks - 1997 Edition

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Specification for the Design,
Testing and Utilization of
Industrial Steel Storage Racks

The Alliance of Material Handling Equipment, Systems, and Service Providers

PREFACE

RACK MANUFACTURERS INSTITUTE

The Rack Manufacturers Institute (RMI) is an independent incorporated trade association affiliated with the Material Handling Industry. The membership of RMI is made up of companies which produce the preponderance of industrial storage racks.

MATERIAL HANDLING INDUSTRY

Material Handling Industry (Industry) provides RMI with certain services and, in connection with this Specification, arranges for its production and distribution. Neither Material Handling Industry, its officers, directors, nor employees have any other participation in the development and preparation of the information contained in the Specification.

All inquiries concerning the Specification should be directed in writing to the RMI Engineering Committee, 8720 RED Oak Boulevard, Suite 201, Charlotte, NC 28217

SPECIFICATION - HISTORY

In the interest of improved uniformity of rack performance and enhanced public safety, the RMI published in 1964 its first "Minimum Engineering Standards for Industrial Storage Racks." and now publishes this Specification.

Since 1964, mechanized storage has developed very rapidly in size and height of installations and new or modified types of racks have been developed. To reflect this rapid development and to assure adequate safety and performance of modern rack installations, the RMI decided early in 1971 to replace its original standards by a more detailed and comprehensive specification. Professors George Winter and Teoman Pekoz of Cornell University were retained to assist the Rack Standard Development Project Committee in producing such a document. The members of The Material Handling Institute, Inc. were the sponsors.

In 1972, the "Interim Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks" was adopted by the Rack Manufacturers Institute at their annual fall meeting. The specification was then submitted to the American National Standards Institute for their review and acceptance. In 1974, the Interim Specification with minor changes was accepted as an American National Standard and was identified as ANSI MH 16.1-1974.

The Rack Manufacturers Institute together with its sponsors from the Material Handling Institute, Inc., retained Professors Winter and Pekoz to test rack components as well as to perform full scale tests of storage racks at Cornell University. A number of the test results have been analyzed, and it was considered necessary to rewrite the 1972 Interim Specification to include the knowledge gained from the analysis of those tests. The 1972 Interim Specification was rewritten by the Rack Standards Subcommittee with the assistance of Professors Winter and Pekoz. Specifications relating to drive-in and drive-through racks have been removed from the Specification until drive-in and drive-through rack test results can be analyzed more thoroughly; perhaps more testing may be required. Movable shelf racks have been included.

As a result of additional testing and analytical research, RMI revised the 1972 Specification resulting in the 1979 Edition. ANSI MH 16.1-1974 was withdrawn in deference to the 1979 Edition. Another revised edition of the RMI Specification was published in 1985.

Subsequent testing and research by Professor Pekoz sponsored by the RMI was the basis of the changes in this 1990 Edition. It was developed and promulgated by the RMI with the sole intent of offering information to the parties engaged in the engineering, manufacturing, marketing, purchasing, installation or use of such racks.

To update the 1990 edition, specifically as it relates to seismic and model code issues, the Specification Advisory Committee, the Seismology Committee and the RMI Engineering Committee worked again with Pekoz and several highly regarded members of the code community. Various other members of similar groups throughout the world were consulted to support this work.

Over the period 1990-1997, RMI continued to conduct extensive physical testing and parametric analysis. The result was the development of the 1997 edition.

In addition to the state-of-the-art benefit from the ongoing testing/analysis, the 1997 edition was expanded to include complete treatment of seismic design considerations more easily allowing its incorporation by reference into various code documents.

The objective is to move the 1997 edition forward as an American National Standard via the canvass process.

SPECIFICATION - 1997 EDITION

The use of this Specification is permissive, not mandatory. Voluntary use is within the control and discretion of the user and is not intended to, and does not in any way limit the ingenuity, responsibility or prerogative of individual manufacturers to design or produce industrial steel storage racks which do not comply with this Specification. RMI has no legal authority to require or enforce compliance with the Specification. The advisory Specification provides technical guidelines for the user to specify his application. Following the Specification does not assure compliance with applicable federal, state, or local regulations and codes. The Specification is not binding on any person and does not have the effect of law.

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In the interest of safety, all users of storage racks are advised to regularly inspect and properly maintain the structural integrity of installed storage rack systems by assuring proper operational, housekeeping and maintenance procedures

Users of the Specification must rely on competent advice to specify, test or design applications or uses. This Specification is offered as a guideline. If a user refers to, or otherwise employs, all or any part of the Specification, the user is agreeing to follow the terms of indemnity, warranty disclaimer, and disclaimer of liability.

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SPECIFICATION FOR THE DESIGN, TESTING AND UTILIZATION OF INDUSTRIAL STEEL STORAGE RACKS

1997 EDITION

1. GENERAL

1.1 SCOPE.

This standard applies to industrial pallet racks, movable shelf racks, and stacker racks made of cold-formed or hot-rolled steel structural members. It does not apply to other types of racks, such as drive-in or drive-through racks, cantilever racks, portable racks, etc. or to racks made of material other than steel.

1.2 MATERIALS.

This standard assumes the use of steel of structural quality as defined in general by the specifications of the American Society for Testing and Materials (ASTM) that are listed in the American Iron and Steel Institute (AISI) Specification for the Design of Cold-Formed Steel Structural Members [1][1], and the American Institute of Steel Construction (AISC) Specification for the Design, Fabrication and Erection of Structural Steel for Buildings [2].

Steels not listed in the above specifications are not excluded provided they conform to the chemical and mechanical requirements of either reference [1] or [2], or other published specifications which establish their properties and structural suitability, and provided they are subjected either by the producer or the purchaser to analyses, tests, and other controls in the manner prescribed by either reference [1] or [2] as applicable.

1.3 APPLICABLE DESIGN SPECIFICATIONS.

Except as modified or supplemented in this standard, the AISI Specification [1] and the AISC Specification [2,3], as respectively applicable, apply to the design and testing of industrial steel storage racks.

1.4 INTEGRITY OF RACK INSTALLATIONS.

1.4.1 Owner Maintenance

The owner shall maintain the structural integrity of the installed rack system by assuring proper operational, housekeeping, and maintenance procedures including, but not limited to, the following:

(1) Prohibit any overloading of any pallet positions and of the overall rack system.

(2) Regularly inspect for damage. If damage is found, immediately unload the affected area and replace or repair any damaged columns, beams, or other structural components.

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[1] Numbers in brackets refer to corresponding numbers in Section 10, References to the Text.

(3) Require all pallets to be maintained in good, safe, operating condition.

(4) Ensure that pallets are properly placed onto pallet load support members in a properly stacked and stable position.

(5) Require that all goods stored on each pallet be properly stacked and stable.

(6) Prohibit double-stacking of any pallet position, including the top-most position, unless the rack system is specifically designed for such loading.

1.4.2 Plaque

All rack installations should display in one or more conspicuous locations a permanent plaque and each plaque shall have an area of not less than 50 square inches. Plaques shall show in clear, legible print the maximum permissible unit load and/or maximum uniformly distributed load per level, the average unit load ($PL_{Average}$, see Section 2.7.2) and maximum total load per bay. The unit load is usually a single pallet or container and its contents mechanically transported. Storage levels having multiple tiering of unit loads shall be so identified. It is the responsibility of the owner to insure that the rack system is not altered so that the plaque information is invalidated.

1.4.3 Conformance

All rack installations produced in conformity with this standard shall be so identified by a plaque having the same characteristics as specified in 1.4.2. The same plaque may be used to show permissible unit loads.

1.4.4 Load Application and Rack Configuration Drawings

Load application and rack configuration drawings shall be furnished with each rack installation. One copy should be retained by the owner and another by the dealer or other local representative for use by an inspecting body.

1.4.5 Multiple Configurations

If use of a pallet rack or stacker rack is permissible in more than one configuration, the drawings are to include either all permissible configurations or limitations as to the maximum number of shelves, the maximum distance between shelves and the maximum distance from the floor to the bottom shelf. This information may be furnished in a table presented to the owner with the drawings referred to in 1.4.4. If drawings are provided, a notice is to be included in conspicuous text on the drawings that deviations from the limitations may impair the safety of the rack installation.

1.4.6 Movable Shelf Rack Stability

The stability of movable shelf racks is not to depend on the presence, absence or location of the movable-shelves. Those components which do provide stability, such as the permanently bolted or welded top shelves and the longitudinal and transverse diagonal bracing, are to be clearly indicated on the rack drawings specified in Section 1.4.4. In specific movable-shelf rack installations where rack height requires it, a conspicuous warning is to be placed in the owners utilization instruction manual of any restrictions on shelf placement or shelf removal. Such restrictions also are to be permanently posted in locations clearly visible to forklift operators.

1.4.7 Bearing Plates and Anchors

The bottom of all columns shall be furnished with bearing plates, as specified in Section 7.2. All racks should be anchored to the floor by anchors capable of resisting the forces caused by the horizontal and vertical loads on the rack.

1.4.8 Small Installations

For installations not exceeding 12 feet in height to the top shelf, covering a floor area less than 3,000 square feet (not including aisles), and having a unit load not exceeding 2,500 pounds (no multiple stacking on top shelf), the provisions given in 1.4.2 through 1.4.5 may be waived.

1.4.9 Resistance to Minor Impact

In those applications where the bottom portion of frames or columns are exposed to potential minor impacts from forklift trucks or other moving equipment during normal operations, consideration shall be given to furnishing collision protection devices and/or increasing the impact resistance of the exposed column members to reduce the effect of these minor impacts. When impact resistance is deemed to be required by the user, the collision protection and/or impact resistance provisions shall be determined based on the operational features and equipment used in the specific application.

Upon any visible damage, the pertinent portions of the rack shall be unloaded immediately by the user and the damaged portion shall be adequately repaired or replaced.

1.4.10 Racks Connected to the Building Structure

If the racks are connected to the building structure, the maximum possible horizontal and vertical forces imposed by the rack on the building are to be calculated for the effects listed in 2.1 or 2.2 as appropriate and the owner of the building advised of these forces and their locations.

1.4.11 Plumbness

To assure adequate plumbness, the maximum tolerance from the vertical is 0.5 inches in 10 feet of height.

2. Loading

Design shall be made in accordance with the provisions for Load and Resistance Factor Design (LRFD), or the provisions for Allowable Stress Design (ASD). Both methods are equally acceptable although they may not produce identical designs. However, the two methods shall not be mixed in designing the various elements of a storage rack structure.

2.1 LOAD COMBINATIONS FOR THE ASD DESIGN METHOD

When the ASD design method is used, all load combinations shall be as stated in the AISI Specification [1] and AISC Specification [2] as modified below for racks.

1. DL	Dead Load Critical
2. DL + LL + (SL or RL) + PL	Gravity Load Critical
3. DL - (WL or EL) + PLapp	Uplift Critical

4. DL + LL + (SL or RL) + (WL or EL) + PL — Gravity Plus Wind/Seismic Critical

For load support beams and their connections only:

5. DL + LL+ 0.5(SL or RL) + 0.88PL+ Imp — Shelf Plus Impact Critical

where:

DL = Dead Load
LL = Live Load other than the pallets or products stored on the racks. (Example, floor loading from work platforms)
SL = Snow Load
RL = Load from rain
WL = Wind Load
EL = Seismic Load
Imp = Impact loading on a shelf, see section 2.4
PL = Maximum Load from pallets or products stored on the racks.
PLapp = That portion of pallet or product load that is used to compute the seismic base shear. When checking for wind uplift, if loads must be present, to develop calculated wind force, their minimum weight may be included in PLapp. See commentary.

Cases 3 and 4 may be multiplied by 0.75. In addition, when checking cases 3 and 4 and seismic forces determined from section 2.7 or another limit state base code was used EL may be multiplied by 0.67.

2.2 Load Factors And Combinations For The LRFD Design Method

When the LRFD design method is used, all load factors and combinations shall be as stated in the AISI Specification [1] or AISC Specification [3] with the following modifications:

For all rack members:	limit state
1. 1.4DL + LL + 1.2PL	Dead load
2. 1.2DL + 1.6LL + 0.5(SL or RL) + 1.4PL	Live/Product load
3. 1.2DL + 1.6(SL or RL) + (0.5LL or 0.8WL) + 0.85PL	Snow/Rain
4. 1.2DL + 1.3WL + 0.5LL + 0.5(SL or RL) + 0.85PL	Wind load
5. 1.2DL + 1.5EL + 0.5LL + 0.2SL + 0.85PL	Seismic load
6. 0.9DL - (1.3WL or 1.5EL) + 0.9PL	Uplift

For load support beams and their connections only:

7. 1.2DL + 1.6LL+ 0.5(SL or RL) + 1.4PL + 1.4 * Imp — Product/Live/Impact (for shelves and connections)

All load symbols, DL, LL, PL SL, RL, WL, EL and Imp are as defined in Section 2.1.

Note: There is no required increase in the live load factors when the live load exceeds 100 psf as required in the AISI. The load factor for EL in load cases 5 and 6 may be 1.0 for the seismic loading determined by section 2.7 or another Limit State based code.

For load combination #6 when checking for wind uplift, only pallet loading that must be present to develop the wind forces can be considered in PL. This value will be zero for an unloaded rack or a rack with cladding.

All resistance factors are to be as given in the latest edition of the AISI Specification [1] or AISC Specification [3]. The resistance factors for anchor bolts are determined as follows:

For Wind uplift: ϕ =0.45
For Seismic: ϕ =0.55
For overturning forces in Section 8:
ϕ =0.400

2.3 Design Loads.

Racks are to be designed for the most critical combination of dead loads, live loads, product loads, vertical impact loads, horizontal loads, and wind or seismic induced loads as applicable.

2.4 Vertical Impact Loads.

Load support beams, supporting arms (if any), and end connections of beams or arms to columns are to be designed for an additional vertical impact load of 25 per cent of one unit load, placed in the most unfavorable position for the particular determination (moment or shear force). When designing by test (see 9.3), due allowance must be made for the additional impact load as specified in this section. No impact load need be applied when checking beam deflections (see 9.3 and 9.3.4) and no impact loads need be considered in designing upright frames, columns, and other vertical components.

2.5 Horizontal Forces.

2.5.1 Beam support connections, frame bracing, and frame bracing to column connections are to be designed for the horizontal forces in this section.

The amount of horizontal force that a rack must resist varies with the application. The beam connections and frame bracing and connections must be designed for the more critical of:

1. Earthquake Forces (see Section 2.7).

2. For Allowable Stress Design -1.5% of the dead load plus the product load at all connections based on maximum loading.

For Load and Resistance Factor Design - 1.5% of the factored dead load and factored product load.

These horizontal forces include the effect of out-of-plumbness in Section 1.4.11. These forces shall be assumed to be applied separately, not simultaneously, in each of the two principal directions of the rack.

The horizontal forces shall be assumed to be applied simultaneously with the full vertical live load, product load and dead load. The beam support connection moments shall be checked against the permissible moments (both positive and negative) determined from the Cantilever Test (Section 9.4.1) and/or the Portal Test (Section 9.4.2).

2.5.2 Stacker racks or racks fully or partially supporting moving equipment shall meet the requirements of 2.5.2.1, 2.5.2.2, and 2.7.

2.5.2.1 The moving equipment manufacturer is responsible for supplying to the rack manufacturer information on the maximum static and dynamic live loads and their locations, transmitted from moving equipment to racks, and the applicable longitudinal and transverse impact factors.

2.5.2.2 Forces described in 2.5.2.1 need not be assumed to act concurrently with those described in 2.5.1 and 2.7.

2.6 Wind Loads.

Racks exposed to the wind shall be designed for the wind loads acting on the rack plus the loaded pallets. For stability, consideration is to be given to loading conditions which produce large wind forces combined with small stabilizing gravity forces.

The forces described in 2.5.1, except that portion of horizontal loading resulting from out-of-plumb installation, and 2.7 need not be assumed to act concurrently with wind loads. The forces described in 2.5.2 shall be assumed to act concurrently with wind forces.

2.7 Earthquake Forces.

2.7.1 General.

Where customer specifications require or local building codes dictate that provisions be made for earthquake effects and associated lateral forces, customers or their representatives shall bring such requirements to the attention of the rack manufacturer. For each such installation, the storage rack shall be designed, manufactured, and installed in accordance with such provisions. Storage racks, which are not connected to buildings or other structures, shall be designed to resist seismic forces in conformance with this section.

Adequate clearance shall be maintained around the storage rack to avoid damaging contact with other structures.

2.7.2 Minimum Seismic Forces.

The total minimum lateral force shall be determined using the following considerations:

At-Grade Elevation: Storage rack installed at or below grade elevation shall be designed, fabricated and installed in accordance with the following requirements:

The seismic design forces shall not be less than that required by the following equation for the determination of seismic base shear:

$$V = C_s I_p W_s$$

where:

C_s = the seismic response coefficient determined in Section 2.7.3 below.

I_p = system importance factor that varies from 1.00 to 1.50 for the following:
- $I_p = 1.5$ if the system is an essential facility;
- $I_p = 1.5$ if the system contains material that would be significantly hazardous if released;
- $I_p = 1.0$ for all other structures;

however, for storage rack in areas open to the general public, e.g., in warehouse retail stores, $I_p = 1.5$.

$W_s = (0.67 \mathrm{x} PL_{RF} \mathrm{x} PL) + DL + 0.25 \mathrm{x} LL$

where:

PL_{RF} = Product Load Reduction Factor

Seismic Force Direction	PL_{RF}
Cross-Aisle	1.0
Down-Aisle	$PL_{Average} / PL_{Maximum}$

$PL_{Average}$ is the maximum total weight of product expected on the shelves in any row divided by the number of shelves in that row.

$PL_{Maximum}$ is the maximum weight of product that will be placed on any one shelf in that row.

Above-Grade Elevation: Storage rack installed at elevations above grade shall be designed, fabricated and installed in accordance with the following requirements:

Storage racks shall meet the force and displacement requirements required of building structures including the force and displacement effects caused by amplifications of upper-story motions.

As above, $W_s = (0.67 \mathrm{x} PL_{RF} \mathrm{x} PL) + DL + 0.25 \mathrm{x} LL$

2.7.3 Calculation of Seismic Response Coefficient:

When the fundamental period of the rack structure is computed, the seismic response coefficient, C_s, shall be determined in accordance with the following equation:

$$C_s = \frac{12C_v}{RT^{2/3}}$$

where:

C_v = the seismic coefficient based upon the Soil Profile Type and the value of A_v as determined from Table 2.7.2 in Section 2.7.3.1.

R = for rack structures more than 8 ft in height, R = 4.0 in the braced direction and R = 6.0 in the unbraced direction. Higher values may be used if substantiated by tests.

T = the fundamental period of the rack structure in each direction under consideration shall be established using the structural properties and deformation characteristics of the resisting elements in a properly substantiated analysis.

Alternatively, the seismic response coefficient need not be greater than the following:

$$C_s = \frac{2.5C_a}{R}$$

where:

R is as above

C_a = the seismic coefficient based upon the Soil Profile Type and value of A_a as determined from Table 2.7.1 in Section 2.7.3.1.

2.7.3.1. Seismic Coefficients C_a and C_v:

Seismic coefficient C_a based on Soil Profile Type and A_a is determined from Table 2.7.1.

Table 2.7.1

Seismic Coefficient C_a

Soil Profile Type	$A_a < 0.05$	$A_a = 0.05$	$A_a = 0.10$	$A_a = 0.20$	$A_a = 0.30$	$A_a = 0.40$
A	A_a	0.04	0.08	0.16	0.24	0.32
B	A_a	0.05	0.10	0.20	0.30	0.40
C	A_a	0.06	0.12	0.24	0.33	0.40
D	A_a	0.08	0.16	0.28	0.36	0.44
E	A_a	0.13	0.25	0.34	0.36	0.36

NOTE: For intermediate values, the higher value or straight-line interpolation shall be used to determine the value C_a.

Seismic Coefficient C_v based on Soil Profile Type and A_v is determined from Table 2.7.2.

Table 2.7.2

Seismic Coefficient C_v

Soil Profile Type	$A_v < 0.05$	$A_v = 0.05$	$A_v = 0.10$	$A_v = 0.20$	$A_v = 0.30$	$A_v = 0.40$
A	A_v	0.04	0.08	0.16	0.24	0.32
B	A_v	0.05	0.10	0.20	0.30	0.40
C	A_v	0.09	0.17	0.32	0.45	0.56
D	A_v	0.12	0.24	0.40	0.54	0.64
E	A_v	0.18	0.35	0.64	0.84	0.96

NOTE: For intermediate values, the higher value or straight-line interpolation shall be used to determine the value C_v.

Note that where A_a and A_v are less than 0.05, $C_a = A_a$ and $C_v = A_v$.

Soil Profile Types are defined as follows:

A Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000 \text{ft} / \sec$

B Rock with measured shear wave velocity, $2{,}500\text{ft/sec} < \bar{v}_s \le 5{,}000\text{ft/sec}$

C Very dense soil and soft rock with measured shear wave velocity, $1{,}200\text{ft/sec} < \bar{v}_s \le 2{,}500\text{ft/sec}$ or with either $\bar{N} > 50$ or $\bar{s}_u \ge 2{,}000\text{psf}$

D Stiff soil with measured shear wave velocity, $600\text{ft/sec} < \bar{v}_s \le 1{,}200\text{ft/sec}$ or with either $15 \le \bar{N} \le 50$ or $1{,}000\text{psf} \le \bar{s}_u \le 2{,}000\text{psf}$

E A soil profile with $\bar{v}_s < 600\text{ft/sec}$ or any profile with more than 10 ft of soft clay defined as soil with PI > 20, w ≥40 %, and $s_u < 500$ psf

F Soils requiring site-specific evaluations:

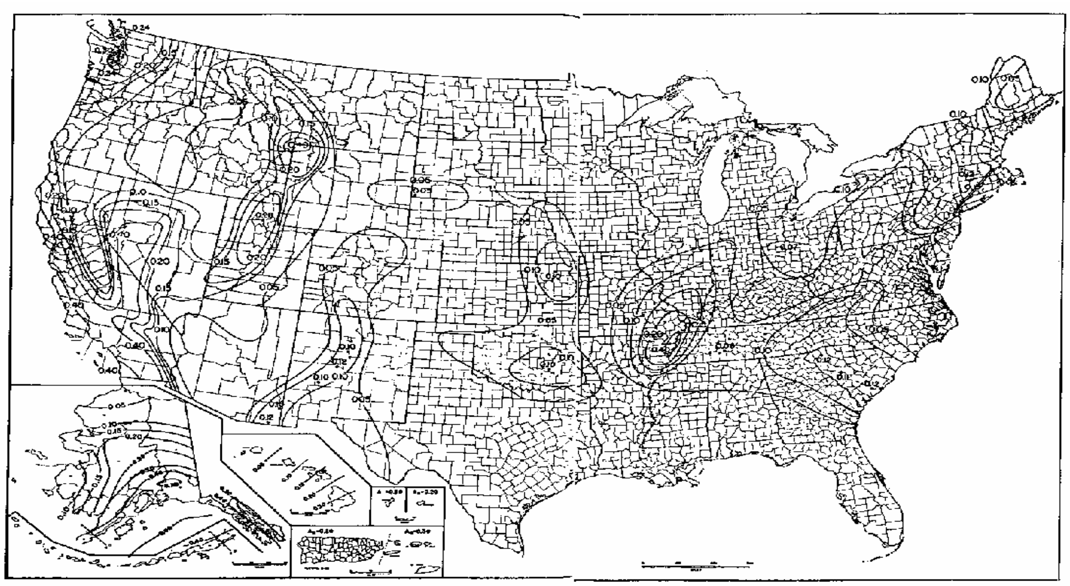
1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.

2. Peats and/or highly organic clays (H > 10 ft of peat and/or highly organic clay where H = thickness of soil)

3. Very high plasticity clays (H > 25 ft with PI > 75)

4. Very thick soft/medium stiff clays (H > 120 ft)

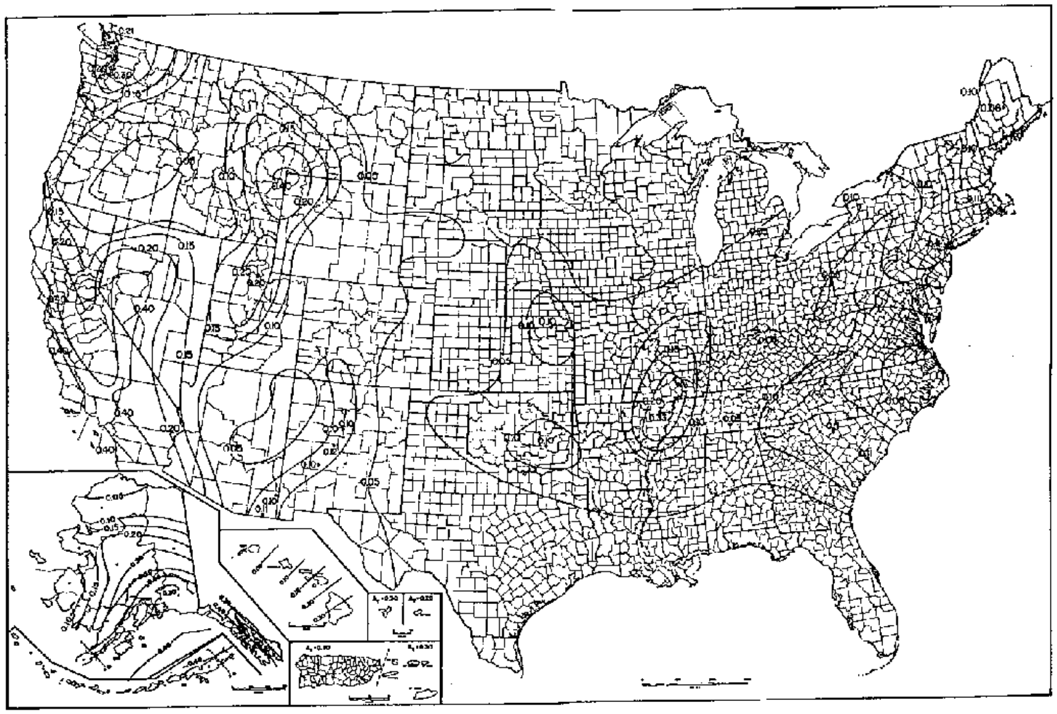
Exception: When the soil properties are not known in sufficient detail to determine the Soil Profile Type, Type D shall be used. Soil Profile Types E or F need not be assumed unless the regulatory agency determines that Types E or F may be present at the site or in the event that Types E or F are established by geotechnical data.



MAP 9-1. *Contour Map for* Coefficient A_a

48 49

Figure 2.7.1 Contour Map for Coefficient A_a



MAP 9-2. Contour Map for Coefficient A_v

50 51

Figure 2.7.2 Contour Map for Coefficient A_v

2.7.4 Vertical Distribution of Seismic Forces:

The lateral force, F_x at any level shall be determined from the following equations:

If the centerline of the first shelf level is 12” above the floor or less:

$$F_1 = C_s I_p w_1 \qquad \text{For the first shelf level}$$

and

$$F_x = \frac{(V - F_1) w_x h_x^k}{\sum_{i=2}^{n} w_i h_i^k} \qquad \text{For levels above the first level}$$

If the centerline of the first shelf level is greater than 12”:

$$F_x = \frac{V w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{For all levels}$$

where:

V = total design lateral force or shear at the base of the rack w*i* or w_x = the portion of the total gravity load (including live load, dead load and product load **x** product load reduction factor, see 2.7.2) of the rack, located or assigned to the bottom shelf level, level i or x

h_i or h_x = the height from the base to level *i* or x

k = an exponent related to the structure's period

period ≤ 0.5	k = 1
period ≥ 2.5	k = 2

For racks having a period between 0.5 and 2.5 seconds, k shall be 2 or shall be determined by linear interpolation between 1 and 2. If the base shear is based on the default C_s then the k shall be taken as 1.

2.7.5 Horizontal Shear Distribution:

The seismic design shear at any level, V_x, shall be determined from the following equation:

$$V_x = \sum_{i=x}^{n} F_i$$

where F_i = the portion of the seismic base shear, V, induced at level i. The seismic design shear, V_x, shall be distributed to the various vertical elements of the seismic force resisting system in the level under consideration based on the relative lateral stiffnesses of the vertical resisting elements.

2.7.6 Overturning.

Safety against overturning moment shall be provided when only the top level of the rack is loaded, in which case it is assumed that the force acts through the center of gravity of the top load.

2.7.7 Concurrent Forces.

Forces described in Sections 2.5.1, 2.5.2, and 2.6 need not be assumed to act concurrently with seismic forces.

3. DESIGN PROCEDURES

All computations for safe loads, stresses, deflections, and the like shall be made in accordance with conventional methods of structural design as specified in the latest edition of the AISI Specification [1] for cold-formed steel components and structural systems and the latest edition of the AISC Specification [2,3] for hot-rolled steel components and structural systems except as modified or supplemented by this specification. In cases where adequate methods of design calculations are not available, designs shall be based on test results obtained in accordance with this specification or Section F of the AISI Specification [1].

No slenderness limitations shall be imposed on tension members which are not required to resist compression forces under any of the loading conditions specified in this specification.

4. DESIGN OF STEEL ELEMENTS AND MEMBERS

The effect of perforations on the load-carrying capacity of compression members is accounted for by the modification of some of the definitions of the AISI Specifications [1] and the AISC Specifications [2,3] as described below.

4.1 Elements of Cold-Formed Steel Members {The AISI Specification [1] Section B}

Effective width calculated for an element shall not exceed the total net width of the element. Net effective section properties shall be calculated as specified in Section 4.2.

4.2 Cold-Formed Steel Members {The AISI Specification [1] Section 3}.

4.2.1 Properties of Sections {The AISI Specification [1] Section C1}

Exceptions to the provisions of the AISI Specification [1] for computing the section properties are given in Section 4.2.2.

4.2.2 Flexural Members {The AISI Specification [1] Section C3}.

S_e = Elastic section modulus of the net section times $(0.5+\frac{Q}{2})$ for the extreme compression fiber.

S_f = Elastic section modulus of the full unreduced gross section for the extreme compression fiber.

S_c = Elastic section modulus of the net section for the extreme compression fiber times $1-\frac{(1-Q)}{2}\left(\frac{M_c/S_f}{F_y}\right)^Q$.

In the calculation of M_e, (σ_{ex},σ_{ey} and σ_t the section properties shall be based on full unreduced gross section considering round corners. Furthermore, j, r_o, and C_w may be computed assuming sharp corners.

Inelastic reserve capacity provisions of the AISI Specification [1] Section C3.1.1 (b) shall not be considered for perforated members.

4.2.3 Concentrically Loaded Compression Members. {The AISI Specification [1] Section C4}.

A_e = Effective area at the stress F_n determined according to Section 4.1 when applicable. Where Section 4.1 is not applicable, A_e shall be calculated as:

$$A_e=\left[1-(1-Q)\left(\frac{F_n}{F_y}\right)^Q\right]A_{net\,min}$$

where the Q factor shall be determined by the procedure specified in Section 9.2 and $A_{net\ min}$ is defined in Section 9.2.

4.2.3.1 Sections Not Subject to Torsional Flexural Buckling {AISI Specification [1] Section C4.1}

Radius of gyration is based on gross section properties computed considering rounded corners.

4.2.3.2 Doubly- or Singly-Symmetric Sections Subject to Torsional or Torsional-Flexural Buckling. {The AISI Specification [1] Section C4.2}

σ_{ex}, σ_{ey}, and σ_t shall be the stresses calculated as specified in Section 4.2.2.

L_x, L_y and L_t are the unbraced lengths defined in Section 6.3 for bending about x- and y-axes and twisting.

4.3 Hot-Rolled Steel Columns

Designs shall be made according to the provisions for Load and Resistance Factor Design as given in Section 4.3.1 or to the provisions of Allowable Stress Design as given in Section 4.3.2.

4.3.1 Load and Resistance Factor Design: {The AISC LRFD Specification [2] Chapter E and Appendices B and E}

All hot-rolled steel columns shall be designed according to Chapter E, Appendices B and E with the exceptions stated herein.

4.3.1.1 Design Compressive Strength for Flexural Buckling

The design strength for flexural buckling of compression members is $\phi_c P_n$ where

$$P_n = A_e F_{cr}$$

F_{cr} shall be determined by equations of Appendix B, Section B5.3d. A_e is defined in Section 4.2.3. The value of Q shall be determined according to Section 9.2.2

4.3.1.2 Design Compressive Strength for Flexural-Torsional Buckling

The design strength for flexural-torsional buckling of compression members is $\phi_c P_n$ where

$$P_n = A_e F_{cr}$$

F_{cr} shall be determined by equations of Appendix E, Section E3. A_e is defined in Section 4.2.3. The value of Q shall be determined according to Section 9.2.2.

4.3.2 Allowable Stress Design: {The AISC ASD Specification [3] Chapter E and Appendix B}

All hot-rolled steel columns shall be designed according to Chapter E, Appendix B with the exceptions stated herein.

4.3.2.1 Design Compressive Strength for Flexural Buckling

The allowable axial force for flexural buckling of compression members is P_a where

$$P_a = A_e F_a$$

F_a shall be determined by equations of Appendix B, Section B5c. A_e is defined in Section 4.2.3. The value of Q shall be determined according to Section 9.2.2

5. BEAMS

5.1 Calculations.

The bending moments, reactions, shear forces, and deflections shall be determined by considering the beams as simply supported, or by rational analysis for beams having partial end-fixity. Where the shape of the beam cross section and the end-connection details permit, permissible loads of pallet-carrying beams shall be determined by conventional methods of calculation according to the AISI Specification [1] or the AISC Specification[2,3].

5.2 Cross Section.

Where the configuration of the cross section precludes calculation of allowable loads and deflections, the determination shall be made by tests according to Section 9.

5.3 Deflections.

At working load (excluding impact) the deflections shall not exceed 1/180 of the span measured with respect to the ends of the beam.

6. UPRIGHT FRAME DESIGN.

6.1 Definition.

The upright-frame consists of columns and bracing members.

6.2 General.

6.2.1 Upright-frames and multi-tiered portal frames shall be designed for the critical combinations of vertical and horizontal loads for the most unfavorable positions as specified in Section 2. All moments and forces induced in the columns by the beams shall be considered. In lieu of the calculation, frame capacity may be established by tests according to Section 9.5.

6.2.2 Connections that cannot be readily analyzed shall be capable of withstanding the moments and forces in proper combinations as shown by test.

6.3 Effective Lengths.

Effective lengths for columns are those specified in 6.3.1 through 6.3.4 or as determined by rational analysis or tests.

Guidance for using effective length method is given in the following subsections. It is not intended to preclude the use of other design methods. Other rational methods, consistent with AISC and AISI may be used. Design methods may not be mixed within one structure.

6.3.1 Flexural Buckling in the Direction Perpendicular to the Upright Frames.

Lx is the distance from the centerline of one beam to the centerline of the next beam or the distance from the floor to the centerline of the first beam.

6.3.1.1 Racks Not Braced Against Sidesway.

For the portion of the column between the bottom beam and the floor as well as between the beam levels, the effective length factor K shall be taken as 1.7 or as otherwise determined by an analysis properly accounting for the member stiffnesses, the semi-rigid nature of the beam to column connections and the partial fixity of the base, allowing for average load reduction, as applicable.

If K of 1.7 is used without analysis, then no reduction of this value shall be made.

6.3.1.2 Racks Braced Against Sidesway.

The effective length factor for pallet racks, stacker racks, and movable-shelf racks is K = 1 provided that all such racks have diagonal bracing in the vertical plane and that such racks have either a rigid and fixed top shelf, or diagonal bracing in the horizontal plane of the top fixed shelf. Increased column capacity may be achieved by additional rigid and fixed shelf (or shelves) or bracing in the horizontal plane. The unsupported length is defined as the distance from floor to fixed top shelf or bracing; or, in the case of additional rigid fixed shelf (or shelves) or fixed shelf with diagonal bracing in its horizontal plane, the unsupported length is the distance between fixed shelves or between braced shelves. The effective length factor is K = 1. If there is no bracing in the vertical plane of the rack, the K values are the same as for racks in 6.3.1.2, Racks Not Braced Against Sidesway.

6.3.2 Flexural Buckling in the Plane of the Upright Frame.

6.3.2.1 L_y is defined as the distance between the intersection of the neutral axis of the column with the neutral axis of either two adjacent diagonals or a diagonal and a horizontal.

6.3.2.2 For upright frames having diagonal braces or a combination of diagonal and horizontal braces that intersect the columns, the effective length factor K for the portion of the column between braced points shall be taken as 1.0, provided that the maximum value of the ratio of L_{short} or L_{long} does not exceed 0.15.

L_{short} or L_{long} is defined as the distance between the intersection of the neutral axis of the column with the neutral axis of either two adjacent diagonals or a diagonal and a horizontal.

In an upright frame with diagonals and horizontals, L_{short} and L_{long} refer to the minimum and maximum distances between two adjacent segments between two adjacent horizontals. In an upright frame with only diagonal L_{short} and L_{long} refer to two adjacent segments. All distances are measured along the neutral axis of the column.

6.3.2.3 For upright frames having diagonal braces that intersect the horizontal braces, the effective length factor K for the portion of the column between braced points shall be taken as 1.0 providing the ratio of L_{short} to L_{long} does not exceed 0.12.

L_{short} is defined as the shortest distance between the intersection of the neutral axis of one of the two diagonal braces with the neutral axis of the horizontal brace, or the shortest distance between the intersection of one diagonal brace with the neutral axis of the horizontal brace with the neutral axis of the column.

L_{long} is defined as the length of the horizontal brace measured between the neutral axis of the columns.

All measurements are along the neutral axis of the horizontal brace.

6.3.2.4 For upright frames having bracing patterns not included above, the effective length factor K of the column shall be determined by rational analysis or by upright frame test.

6.3.3 Torsional Buckling.

6.3.3.1 L_t is the length of the member unsupported against twisting.

6.3.3.2 The effective length factor K_t for torsional buckling shall be taken as 0.8 provided that the connection details between the columns and the braces are such that the twisting of the column is prevented at the brace points. If the connection details do not prevent twist, K_t can be larger and shall be determined by rational analysis or test.

6.3.4 Compression Diagonals and Horizontals.

For compression diagonals and horizontal members of trussed upright frames, the effective length is the full unsupported length of the member.

6.4 Stability of Trussed-Braced Upright Frames.

To prevent tall and narrow trussed-braced upright frames from becoming unstable and buckling in their own plane, the columns of such upright frames shall be designed using the appropriate provisions of the AISI Specification [1] or the AISC Specification [2,3] for a value KL/r or Kl/r, respectively, equal to:

$$\sqrt{\frac{\pi^2 EA}{P_{cr}}}$$

where for P_{cr} the following apply:

1. For upright frames braced with diagonals and horizontals

$$P_{cr} = \frac{\pi^2 EI}{k^2 \boldsymbol{l}^2} \frac{1}{1 + \frac{\pi^2 I}{k^2 \boldsymbol{l}^2}\left(\frac{1}{A_d \sin\phi \cos^2\phi} + \frac{b}{aA_b}\right)}$$

2. For upright frames braced with diagonals

$$P_{cr} = \frac{\pi^2 EI}{k^2 \boldsymbol{l}^2} \frac{1}{1 + \frac{\pi^2 I}{k^2 \boldsymbol{l}^2} \frac{1}{A_d \sin\phi \cos^2\phi}}$$

3. For upright frames braced with horizontals only, and with fully rigid connections

$$P_{cr} = \frac{\pi^2 EI}{k^2 \boldsymbol{l}^2} \frac{1}{1 + \frac{\pi^2 I}{k^2 \boldsymbol{l}^2}\left(\frac{ab}{12\, I_{br}} + \frac{a^2}{24\, I_c}\right)}$$

where:

- a Vertical distance between the horizontal brace axis.
- A Sum of the minimum net area ($A_{net\ min.}$) of the columns of the upright frame.
- A_b Cross-sectional area of a horizontal brace.
- A_d Cross-sectional area of a diagonal brace.
- B Horizontal distance between neutral axes of the columns.
- E The modulus of elasticity of steel.
- I Minimum net moment of inertia of the columns about the gravity axis of the upright frame perpendicular to the plane of the upright frame
- I_{br} Moment of inertia of the horizontal brace about its own axis perpendicular to the plane of the upright frame.
- I_c Minimum net moment of inertia of one column about its own major axis perpendicular to the plane of the upright frame.
- k =1.1 if the center of gravity of the loads along the upright frame is below midheight.
  =1.6 if the center of gravity is below the upper third-point of the height.
  =2.0 if the center of gravity is above the upper third-point of the height.
- ***l*** Total height of the upright frame.
- ϕ Angle between horizontal and diagonal braces.

7. CONNECTIONS AND BEARING PLATES.

7.1 Connections.

7.1.1 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 9.

7.1.2 Beam Locking Device.

Except for movable-shelf racks, beams shall have connection locking devices (or bolts) capable of withstanding an upward force of 1,000 pounds per connection without failure or disengagement.

7.1.3 Movable Shelf Racks.

For movable shelf racks, the top shelf and other fixed shelves are to include support connections capable of supporting an upward force of 1,000 pounds per connection without failure.

The movable shelves are generally constructed of a set of front and rear longitudinal beams connected to each other rigidly by transverse members. The movable shelves are to be connected in such a way to prevent forward displacement when lifting out the front beam of the shelf.

7.2 Bearing Plates.

Provision shall be made to transfer column forces and moments into the floor. These forces and moments shall be consistent in magnitude and direction with the rack analysis. Unless otherwise specified, the maximum allowable bearing stress F'_p(ASD) or design bearing loads ϕcPp (LRFD) on the bottom of the plate shall determined as follows:

for ASD

$$F'_p = 0.7f'_c$$

for LRFD

$$P_p = 1.7f'_c A_{\text{Effective Base Bearing Area}}$$

$$\phi_c = 0.60$$

where f'_c = the minimum 28-day compression strength of the concrete floor which, unless otherwise brought to the attention of the rack fabricator, shall be assumed to be 3,000 psi.

Once the required bearing area has been determined from the allowable bearing stress F'_p, the minimum thickness of the base plate is determined by rational analysis or by appropriate test using a test load 1.5 times the ASD design load or the factored LRFD load. Upon request, information shall be given to the owner, or the owner’s agent on the location, size, and pressures under the bearing plates of each type of upright frame in the installation.

8. SPECIAL RACK DESIGN PROVISIONS.

8.1 Overturning.

Overturning is to be considered for the most unfavorable combination of vertical and horizontal loads. Stabilizing forces provided by the anchors to the floor are not considered in checking overturning, unless anchors and floor are specifically designed and installed to meet these uplift forces (see 2.6 and 2.7).

Unless all columns are so anchored, the ratio of the restoring moment to overturning moment shall not be less than 1.5.

The height-to-depth ratio of a storage rack shall not exceed 6 to 1 measuring to topmost beam position unless the rack is properly anchored or braced externally.

Rack that exceed the 6 to 1 ratio defined above, shall also be designed to resist a 350 pound side force applied to any single frame at the top shelf level in a direction perpendicular to the aisle. For LRFD design method, the load factor applied to this force shall be 1.6. This force is to be applied to an empty frame and divided into as many frames as are interconnected in the direction of the force. Anchors and base plates will be designed to resist uplift forces from this force when applied to an empty frame. Frame columns need not be designed for the additional axial load from this force.

Unless it can be shown to be unnecessary because of such factors as soil, slab and frame stiffness, single rows of rack exceeding a height to depth ratio of 8 to 1 must be tied externally to the building or cross-aisle to another rack. Stabilizing a single rack with a height to depth ratio of over 8 to 1 with anchoring alone is not recommended unless designed and certified by an engineer.

The 350 pound side force in this section need not be applied concurrently with the horizontal forces of sections 2.5, 2.6 and 2.7.

8.2 Connections to Buildings.

Connections of racks to buildings, if any, shall be designed and installed to prevent reactions or displacements of the buildings from damaging the racks or the reactions or displacements of the racks from damaging the building (see also 1.4.10).

8.3 Interaction with Buildings.

Storage rack located at levels above the ground level (as described in Section 2.7.2), rack buildings, or racks which depend upon attachments to buildings or other structures at other than floor level for their lateral stability, shall be designed to resist seismic forces that consider the responses of the building and storage rack to seismic ground motion and their interaction so as not to cause damage to one another.

9. TEST METHODS.

9.1 GENERAL.

Material properties as determined in accordance with the applicable ASTM A370 test procedures and Section F3 of the AISI Specification apply. For this purpose, tensile coupons are taken, after the completion of testing, from flat portions of the specimen at regions of low bending moment and shear force.

If the effect of cold-work is being accounted for by test, the test specimens must be formed by the same procedure as is used or contemplated in the prototype. This is essential because different manufacturing methods produce different amounts of cold working (e.g., cold working of a specimen by press-braking is less than that in a cold-roll-formed prototype).

Test specimens are to be fully described prior to testing and any dents or defects shall be noted and the condition of welds, if any, inspected and described. All cross-sectional dimensions of each specimen are to be measured prior to testing at several points along the length and photographs of specimens should be taken prior to, during, and after testing whenever it seems advisable. (The purpose of these tests is for design and not for purchase acceptance-tests).

9.1.1 Testing Apparatus and Fixtures.

These tests should be carried out in a testing machine or by means of hydraulic jacks in a test frame or by application of properly measured weights. The testing machine or load-measuring apparatus must meet the requirements prescribed in the ASTM Methods E4, Verification of Testing Machines.

The weights of load distribution beams and other fixtures are to be measured and included in evaluating the test data.

9.1.2 Instrumentation

Dial gages or other deflection measuring devices are required at appropriate points to obtain proper alignment and to measure load-deflection behavior accurately. The deflections should be measured and reported to an accuracy of ±0.03 inches.

Strain gages may be used if behavior characteristics other than ultimate loads and load-deflection relations are desired. In general, for coupon tests, extensometers are used.

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

9.1.3 Reduction and Presentation of Test Data.

For each test, the report is to include:

1. A sketch of the specimen with all dimensions.
2. A sketch of the test set-up with all dimensions, including locations and kinds of gages, loading and support arrangements and an identification of the loading apparatus (testing machine, jacks, etc.) with information on the range used and the smallest increment readable for that range.
3. The results of the coupon tension tests should be presented in the form of a table of elongations vs. loads or, alternatively, strains vs. stresses. Yield stress and ultimate strength shall be determined by any of the accepted ASTM methods. (It is desirable to include stress-strain curves in the data presentation.)
4. For presentation of the results of the test, all load, deflection, and other recorded data shall be properly reduced to actual values by correcting, where appropriate, for initial readings, weights of loading apparatus (e.g., loading beams), etc.

These reduced measurements shall be presented in tables showing load vs. the particular measured quantity (deflection, strain, etc.) In the same tables, observations of special events (flange buckling, connection failure, etc.) 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 tests and on inspection of tabulated data, graphic presentation of selected or all other load-deformation data is desirable, but optional as dictated by judgment.

9.1.4 Evaluation of Tests for Determining Structural Performance.

Tests are to be evaluated in accordance with Sec. F1 of the AISI Specification [1].

9.2 STUB COLUMN TESTS FOR COLD-FORMED AND HOT-ROLLED COLUMNS.

9.2.1 Test Specimen and Procedure.

The Q values of perforated compression members for use in Section 4 are determined by stub column tests as described in Part VIII of the AISI Cold-Formed Steel Design Manual [4]. The ends of the stub column must be milled flat (preferably to a tolerance of ±0.001 inch) and perpendicular to the longitudinal axis of the column. The axial load is to be applied by flat plates bearing (not welded or otherwise connected) against the milled ends.

For the purposes of determining Q, only the ultimate strength of the stub column needs to be determined.

9.2.2 Evaluation of Test Results.

Q is calculated as follows:

$$Q = \frac{\text{ultimate compresive strength of stub column by test}}{F_y A_{\text{net min}}}$$

where

F_y = actual yield stress of the column material if no cold work effects are to be considered; or the weighted average yield to point F_y, calculated in accordance with appendix A 5.2.2 of the AISI Specification [1], if cold work effects are to be considered.

$A_{\text{net min}}$ = minimum cross-sectional area obtained by passing a plane through the column normal to the axis of the column. In no case shall Q be greater than 1.

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

$$Q = Q_{min} + \frac{(Q_{max} - Q_{min})(t - t_{min})}{(t_{max} - t_{min})}$$

where Q is the value for the intermediate thickness t, and Q_{max} and Q_{min} are the values obtained by test for the largest and smallest thicknesses, respectively. This interpolation is permissible only if the yield stresses of the two specimens do not differ by more than 25 percent and if the yield points of the intermediate thicknesses fall between or below those of the test specimens.

9.3 PALLET BEAM TESTS.

9.3.1 Simply Supported Pallet Beam Tests.

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

The simply supported pallet beam test is to be made only if the flexural behavior parameters such as the yield moment, ultimate moment and the effective flexural rigidity (EI) are to be determined. For the latter parameter, tests are to be conducted on two identical specimens unless a third test is required as specified in Section 9.3.1.3. If lateral restraints are required, the beams are to be tested in pairs as they would be used in completed assemblies.

9.3.1.1 Test Setup.

The test set-up consists of a beam test specimen simply supported at each end (not connected to columns). The test load is applied to a load distribution beam which in turn imposes a load at two points on the 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 and C is a numerical value between 2.5 and 3. Plates can be used to prevent local failure at supports or at load points.

9.3.1.2 Test Procedure.

After alignment, a small initial load of about 5% of the expected ultimate load shall be applied to the test assembly to insure firm contact between the specimen and all loading and support components. At this load, initial readings are to be taken from all gages. Loads shall then be applied in increments no larger than about one-fifth of the expected design load. Readings are taken for all load increments. (It is good to plot load verses mid-span deflection readings at each load increment during testing). Noticeable deviation from straightness of such a plot will indicate incipient inelastic behavior or local buckling or crippling. When such is the case, load increments are reduced to no more than half the initial increments.(It is good practice, though not required, 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 the increment. Appropriate gage 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 by the nature of the specimen, and if such sudden failure could damage the gages, they should be removed. On the other hand, if a gradual failure is expected, such as by simply yielding, it is desirable to measure the last center 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, failure of connections, etc., are to be recorded at the loads at which they occur.

9.3.1.3 Evaluation of Test Results.

The parameters investigated shall be determined by test results by conventional methods.

The flexural rigidity shall be calculated on the basis of the results of two tests of identical specimens, provided that the deviation from the average value does not exceed 10%. If the deviation from the average exceeds 10%, then a third identical specimen is to be tested. The average of the two lower values obtained from the tests shall be the result from the series of tests.

9.3.2 Pallet Beam in Upright Frames Assembly Test.

This test is intended to simulate the conditions in the actual rack as closely as possible.

9.3.2.1 Test Setup.

The test assembly shall consist of two upright frames not bolted to the floor and two levels of pallet beams with front-to-back ties when specified.

The upright frame may be as high as desired. However, the bottom level beams shall be tested and shall be located so there will not be less than 24 inches clear between the test beams and the floor or between the test beams and the top-level beams.

The end connections shall be those used in the prototype.

The location of the test loads perpendicular to the beams shall simulate actual loading.

If loads are to be applied by pallets or other devices resting on beams, it is important that friction between pallet and beams be reduced to the minimum possible amount by greasing or other means. (This is suggested 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.)

The minimum instrumentation for such tests consists of devices for measuring the deflections of both beams at mid-span relative to the ends of the beams. One way of doing this is to attach a scale graduated to 0.01 inch at mid-span of each beam and to stretch a tight string (usually a string with a rubber band at one end) or wire attached to each end of the beam. Another way is to use dial gages at mid-span and at each end of the beams. Transits may also be used to read scales located at mid-span and at the end of the beams.

Additional instrumentation, such as strain gages or additional dial gages at the ends of the beam, is needed only if special problems are to be considered. For highly unsymmetrical beams, e.g., deep channels or C-sections, it may be advisable to measure rotation under load. This is most easily done by rigidly attaching a protractor of sufficient size to the beam at or close to mid-span. A vertical string weighted at the end and acting as a plumb is then read against the protractor at every load increment.

9.3.2.2 Test Procedure.

The test procedures specified in Section 9.3.1.2 of this report shall be used.

9.3.2.3 Evaluation of Test Results.

The design load shall be the smallest of the following:

1. Strength determined according to the applicable provisions of the AISI Specification [1] Section F and its subsections.
2. Two-thirds of the load at which harmful or objectionable distortions are observed in the connections or elsewhere. These distortions include rotations of such magnitude as to render the beam unserviceable.
3. The load (not including impact) at which maximum vertical deflections attain 1/180 of the span, measured with respect to the ends of the beams.

9.3.3 The number of tests for determining design loads shall be as specified in Section F of the AISI Specification [1].

9.3.4 Once the design load has been determined as specified in 9.3.1 through 9.3.2.3, an additional test shall be made using a new set of specimens. An initial load equal to the design load shall be applied, reduced to zero and the deflection read; this deflection reading shall be the zero reference reading. A load equal to 1.5 times the design load shall then be applied and the deflection read. The load shall then be held constant for one-quarter of an hour and the deflection read again. This deflection reading shall not exceed the previous reading by more than 5 percent. The load shall then be reduced to zero and the residual or permanent deflection read. The net residual deflection of the beam shall not exceed 15 percent of the final deflection measured at 1.5 times the design load. If these limitations are not met, the design load shall be reduced accordingly or the source of residual deflections determined and remedied, and the test repeated with new specimens.

9.4 PALLET BEAM-TO-COLUMN CONNECTION TESTS.

9.4.1 The Cantilever Test.

This test is for determining the connection moment capacity.

9.4.1.1 Test Setup.

The test setup shall consist of a pallet beam at least 26 inches in length connected to the center of a column at least 30 inches in length. Both ends of the column shall be rigidly connected to rigid supports. The load shall be applied to the pallet beam at 24 inches from the face of the column. At this load application point, a dial gage shall be mounted to measure deflections.

9.4.1.2 Test Procedure.

The test procedure specified in Section 9.3.1.2 shall be used.

9.4.1.3 Evaluation of Test Results.

The design moment shall be determined in a manner similar to that specified in provisions 1 and 2 of Section 9.3.2.3.

9.4.2 The Portal Test.

This test is be used to obtain a joint spring constant needed for a semi-rigid frame analysis.

9.4.2.1 Test Setup.

The test setup shall consist of two upright frames supported on four half-round bars, one under the base of each column, two beams the top of which is installed at a distance of 24 inches from the floor, and including front-to-back ties when specified. The half-round bars shall be located at the centroidal axes of the columns perpendicular to the beams. Extra plates may be placed between the base plates and the half-round bars, if necessary. The bases of the columns shall be held against lateral displacement but not against rotation.

9.4.2.2 Test Procedure.

After the rack is properly assembled, a load equal to the design load of the beams shall be placed on the beams, simulating usual loading. A horizontal force equal to the horizontal design load corresponding to the vertical load on the assembly shall be applied to the assembly, equally distributed between the two columns on one side, at the level of the top of the beams, and in the direction of the beams. Deflection due to the horizontal loading shall be measured at the level of the top of the beams.

The procedure shall be repeated at a load twice the design load.

9.4.2.3 Evaluation of Test Results.

The spring constant is to be determined by rational analysis.

9.5 UPRIGHT FRAME TEST.

The frame tests specified in this section are intended to simulate the conditions in the actual rack as closely as possible. The purpose of the test is to determine the upright frame loads for an expected column failure that takes place between the floor and the bottom beam or between the two lower beams in a three beam-level test setup.

The test will account for vertical and horizontal loads as specified in Section 2.5.1 as well as the effects of semi-rigid connections. This procedure is also applicable to Section 2.6 and 2.7 with adjustments to take into account modified loads and increased allowable stresses for Allowable Stress Design.

9.5.1 Horizontal Load in the Direction Perpendicular to the Upright Frame.

9.5.1.1 Symmetrical Loading Condition.

9.5.1.1.1 Test Setup.

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 rack. 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 vertical spacing of the beams shall be the same as in the actual application. The upright frame may be as high as desired; however, its construction consisting of a column and truss web members shall be of the same cross section, pattern and spacing as in the actual application. The top beam level and its column connection may 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 loading from two or more beam levels.

Horizontal loads shall be applied perpendicular to one outside upright frame at the centerline of the beam connection by means of either hydraulic cylinder(s) or by ropes and pulleys with hanging weights attached. The load at each beam level shall be applied equally to each column of the upright frame.

To measure horizontal displacements, one scale shall be located at the centerline of each beam level, and another scale at midheight between the bottom beam level and the floor. All scales may be placed on one column.

9.5.1.1.2 Test Procedure.

1. Align the rack structure so that it is level and plumb and so that all components are properly seated.
2. Take initial scale readings.
3. Place a vertical load equal to 1.5 times beam design load on each of the lower beam levels.
4. Take scale readings for horizontal movement.

5. Apply a horizontal load to the upright frame at each beam level. The horizontal load shall be determined per Section 2.5.1.
6. Take scale readings for horizontal movement.
7. Apply one additional unit of vertical load to the reinforced top level beams only and take scale readings for horizontal movement.
8. Apply one additional unit of horizontal load to the reinforced top level beams only. Take scale readings for horizontal movement. (If hydraulic cylinders are used, be sure the hydraulic cylinder at the bottom beam level is always applying the proper force to the upright frame.)
9. Repeat steps (7) and (8) until failure occurs in the upright frame.

9.5.1.1.3 Evaluation of Test Results.

The vertical design load for an upright frame shall be determined according to the applicable provisions of the AISI Specification [1] Section F and its subsections. The tested ultimate load must be the last set of test data which has an equal number of both vertical and horizontal load increments. The tested ultimate load should be the lowest of the three tested conditions, namely symmetrical loading in 9.5.1.1, unsymmetrical loading in 9.5.1.2, or for the horizontal load in the direction parallel to the upright frame.

9.5.1.2 Unsymmetrical Loading Condition.

Test setup and test procedure are the same as 9.5.1.1 for symmetrical loading condition above, except that no load should be placed on one beam level in one bay directly adjacent to the expected column failure location. The direction of the horizontal load should be in the direction of sidesway.

9.5.2 Horizontal Load in the Direction Parallel to the Plane of Upright Frame.

9.5.2.1 Test Setup.

The test setup is the same as in Section 9.5.1.1.1, except that the locations of horizontal loads and scales shall be changed so that the horizontal loads and displacements are in the plane of the upright frame.

9.5.2.2 Test Procedure.

The test procedure is the same as the procedure in Section 9.5.1.1.2 above, except in step (5) the distribution of the horizontal load on each beam level on each upright frame shall be as determined in Section 2.5.1.

In order to compensate for the effect of the longer moment arm of the upper beam levels in the actual application, the applied test loads shall be modified such that the effect of the loads in the upper beam levels of the rack are properly accounted for both in overturning and shear force.

9.5.2.3 Evaluation of Test Results.

See 9.5.1.1.3 above.

10. REFERENCES TO THE TEXT

1. Specification for the Design of Cold-Formed Steel Structural Members. American Iron and Steel Institute, Washington, DC, 1996 Edition.

2. Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design. American Institute for Steel Construction, Chicago, Ill., June 1, 1989.

3. Load and Resistance Factor Design Specification for Structural Steel Buildings. American Institute for Steel Construction, Chicago, Ill., December 1, 1993

4. Cold-Formed Steel Design Manual, American Iron and Steel Institute, Washington, DC, 1996 Edition

Part I - Appendix

Special Provisions for Racks for

Automated and Manual Storage and Retrieval Systems

(Stacker Racks)

1.1 Scope. Though most of the general provisions in the main part of the Specification do apply to Racks for Automated and Manual Storage and Retrieval Systems, special additional provisions for such racks are provided herein. Such racks may be “Drive-in-Type” or “Beam Column Type” and can be used in “Rack Supported Structures”.

The provisions of this Appendix are given the same numbers as the corresponding parts of the Specification. When no special provision or exception is given in the Appendix, it is implied that the main body of the Specification shall be followed where applicable.

1.4.10 Resistance to Minor Impact. Rack structure need not be designed for accidental collision from the captive aisle crane or stacker crane.

1.4.12 Tolerances. Installation and design tolerances shall be supplied by the user of the installation based on the requirements of the equipment manufacturer.

2.4 Vertical Impact Loads. The moving equipment manufacturer is responsible for supplying to the rack manufacturer information on maximum vertical static and dynamic loads for the design of racks; the rack structures shall be designed for these loads.

2.5 Horizontal Loads. Horizontal loads specified in Section 2.5.1 and 2.5.2 of the Specification shall be used in the design of racks.

2.6 Wind and Snow Loads. Wind (including uplift) and snow loads shall be considered in the design of rack during erection and use. In determining the total force on a rack structure, forces in all members of the structure shall be accounted for with proper consideration of shielding effects, the shape effect, and other applicable forces.

The forces specified in Section 2.5.1 and 2.7 need not be assumed to act concurrently with wind loads, but forces described in Section 2.5.2 shall be assumed to act concurrently with wind forces.

5.3 Deflections. Deflections shall not exceed the limits set by the requirements of the equipment operation.

8.4 Rack Compatibility with the Equipment. Horizontal and vertical deflections shall be calculated and reviewed with the crane equipment supplier for compatibility.

Rack design shall be compatible with the equipment. The basic considerations shall include the height of the first shelf, clearance from the top shelf to the cross-aisle tie, shuttle window height, and sprinkler system.

PREFACE

Any structural design specification is the product of extensive research and development work combined with accumulated engineering experience. Rack structures differ in many respects from more familiar types of structures, such as buildings and bridges. It follows that the generally recognized principles and methods of design and testing of steel structures must be, modified and supplemented in those features peculiar to rack structures. This can be done adequately only by extensive analytical and experimental research on rack structures, combined with engineering experience in this field.

It is important to bear in mind that the Specification and the Commentary should not be used without first obtaining competent engineering advice with respect to suitability for any given application.

This Commentary to the Specification, like those in the AISC and AISI Specifications referred to in section 10, attempt to serve two purposes: (1) they give explanations of, and reasons for, the various provisions of the Specification, and (2) where advisable, they suggest specific procedures with regard to engineering design, calculation or testing, which satisfy the particular requirements of the Specification.

It should be emphasized that, while the provisions of the Specification are meant to be explicit, recommendations and suggestions made in the Commentary are not. In many cases they represent one way of interpreting the Specification provisions, but do not preclude other ways of doing so.

COMMENTARY

on the

SPECIFICATION FOR THE DESIGN, TESTING AND UTILIZATION OF INDUSTRIAL STEEL STORAGE RACKS

1997 EDITION

1.1 SCOPE.

The scope limits the applicability of the Specification to pallet racks, movable shelf racks, and stacker racks made of hot-rolled or cold-formed steel. Although only these three types of rack are explicitly mentioned, the Specification is also intended to be applied to any freestanding rack having a three dimensional structural system comprised of braced frames in one direction and moment frames in the other. In other words, any rack system that is constructed with beams and frames. Such rack types include push back rack, pallet flow rack case flow rack and order picking modules. The Specification is also intended to be applied to the design of the storage rack portion of rack supported buildings.

The rack systems that are excluded from this Specification (such as cantilever and drive-in) are excluded for two reasons. First, certain sections contained in the Specification do not apply to these rack types. For example, the upright frame and effective length provisions of Section 6 and the beam design provisions in Section 5 are not applicable to these rack types. Second, the Specification does not include the necessary design provisions for these rack types. For example, effective length factors and deflection limits for cantilever uprights would need to be included. Additional analytical work and testing is planned by the committee that will enable the addition of comprehensive design provisions for these rack types in the future. Some of the design sections and special test provisions of this Specification are applicable, and therefore helpful, in the design and testing of other rack types. For example, Section 4 Design of Steel Elements and Members is applicable to any hot-rolled or cold-formed steel column of other rack types such as cantilever or drive-in racks.

1.2 MATERIALS.

The intent of this section is to ensure that a reliable quality of steel is used in the fabrication of racks, without limiting the type of steel to any particular strength or rolling characteristics.

1.3 APPLICABLE DESIGN SPECIFICATIONS.

This provision states that the Rack Specification merely contains such relatively minor supplements or modifications of the nationally accepted AISI and AISC Specifications in Section 10 as are necessitated by the special nature of rack structures, as distinct from regular framing for steel buildings.

This edition of the specification allows the use of either Allowable Stress Design (ASD) or Load Resistance Factor Design (LRFD).

1.4 INTEGRITY OF RACK INSTALLATIONS.

1.4.1 OWNER MAINTENANCE.

This section stresses the importance of planning in the initial design process, controlling the use of the rack to that initially intended, and scheduling regular inspection to maintain the integrity of rack structures. Users are directed to “American National Standard For the Use of Industrial and Commercial Steel Storage Racks – Manual of Safety Practices / A Code of Safety Practices” [1] regarding safety practices in the use of storage racks for further information.

1.4.2 PLAQUE

In industrial and commercial warehouses, allowable unit floor loads are generally posted in easily visible locations, and such posting is often required by law. The Specification provides for similar posting of maximum permissible unit load for each given rack installation. For racks designed to receive loads on standard sized pallets, a unit load means the combined weight of product and pallet unless the installation provides for more than one unit load being stacked on top of each other. Load beams may be separately identified. A sample plaque is illustrated in Fig. 1.4.2a and b. The figures are not intended to limit the plaque details, but rather are presented as a possible example. It is the intent of the Specification for the plaque to inform the storage facility manager of the safe rack capacity and any plaque that transmits the required information is acceptable. The manager of the storage facility shall have the responsibility to be cognizant of this load limit and to instruct all operating personnel to see to it that the permissible load is not exceeded.

Figure 1.4.2a Example of Load Capacity and Compliance Plaque.

Figure 1.4.2b Example of Load Capacity and Compliance Plaque.

The plaques should not be transferred to any reconfigured or relocated rack without first verifying the applicability of the information on the plaque to the new configuration or location.

1.4.3 CONFORMANCE.

For racks designed in accordance with this Specification, it is important for building and safety inspectors to know whether they were produced and erected following this Specification. To this end, Sec. 1.4.3 states that a plaque should be displayed indicating conformance with the Specification for racks so produced. The intent is that such a statement of conformance will greatly facilitate and simplify approval of rack installations by local, regional or federal inspecting authorities.

1.4.4 LOAD APPLICATION AND RACK CONFIGURATION DRAWINGS.

For purposes of safety inspection, complete data should be available on engineering design and capacity of the racks as originally ordered, delivered and installed.

For this reason Sec. 1.4.4 provides that such information, in the form of rack configuration drawings with load magnitude and application indications be furnished by the rack dealer or manufacturer's local representative involved in procuring and erecting the particular rack installation. The provision that both these parties retain such information on file is important because both the owner of the rack installation and the local dealer may change over the lifetime of the installation. The safekeeping of such information by both parties will greatly increase the probability that such information will be available if and when needed.

1.4.5 MULTIPLE CONFIGURATIONS.

Most racks are produced so that they are adjustable and can be assembled in configurations different from the one originally ordered and installed. Consequently, it is possible to install or modify a rack into an alternate configuration which is unsafe. For example, while using the original components [beams and upright frames] the rack could be rearranged to reduce the vertical distance between the upper beams, which would increase the unbraced length of the bottom portion of the columns. Its increased slenderness ratio would reduce the carrying capacity of the columns as compared to the original configuration. Alternately, racks can be modified by installation of additional components; e.g., greater number of shelf beams at smaller vertical spacing with the original upright frames. This would reduce the slenderness ratios of the individual column segments and increase their load capacities. However, the additional loads, which can now be placed on the greater number of shelves, could increase the load on the column by an amount greater than the increased capacity resulting from the reduction of the unbraced length. These are just two examples of changed configurations which could make an originally adequate rack unsafe.

The owner or user of the rack installations generally will not have the engineering capability to establish the safety of his changed configuration.

It is for these reasons that Sec. 1.4.5, in essence, provides that the owner be given comprehensive guidelines as to those alternate configurations which can be used safely. If changes other than those detailed in the guidelines must be made the original manufacturer or competent storage rack engineer should be contacted.

1.4.6 MOVABLE SHELF RACK STABILITY.

These racks differ from standard storage racking in that a majority of shelves are designed to be removed. In standard storage racks, shelves (beams) are readily adjustable, but cannot be removed without unloading the rack and re-assembling the components. For this reason, movable shelf racks are fitted with one or more permanent shelves and/or braces that provide the needed stability to the structure. This section specifies the provisions for identifying those stabilizing components, and for posting warnings and restrictions for removal.

1.4.7 BEARING PLATES AND ANCHORS.

It is the function of bearing plates to receive the concentrated forces at the bottom ends of the columns and to distribute them with adequate uniformity over a large enough bearing area. Provisions for the dimensioning of bearing plates on concrete floors are given in Sec. 7.2. Adequate connection of the column to the bearing plate is required to properly transfer loads.

This section also specifies that all racks should be anchored to the floor. The anchor bolts should be installed in accordance with the anchor manufacturer's recommendations.

Anchors serve several distinct functions:

1.) Anchors fix the relative positions of, and distances between, neighboring columns.

2.) Anchors provide resistance against horizontal displacements of the bottom ends of the columns. A tendency for such horizontal displacement may result from external lateral forces (earthquake, wind, impact, etc.) or from the horizontal reactions (shear forces) resulting from the rigid or semi-rigid frame action of the rack. If such shear forces would in fact cause horizontal displacements of the bottoms of the columns, this would reduce the carrying capacity of the rack as compared to computed values.

3.) For particularly tall and narrow racks, anchors may significantly increase the stability against overturning (see Specification Section 8.1).

1.4.8 SMALL INSTALLATIONS.

This section offers an exemption for small rack installations from the documentation provisions of Secs. 1.4.2 through 1.4.5. These requirements would represent an excessive hardship for the management of such installations. However, in all other respects, the design, testing and utilization provisions of the Specification apply to all racks including the small installations as defined in this section.

1.4.9 RESISTANCE TO MINOR IMPACT.

Collisions of forklift trucks or other moving equipment with front columns are the single most important source of structural distress of storage racks.

This section is concerned with the protection of those bottom portions of columns which are exposed to such collisions. At what exact level such collisions can occur depends on the detailed configuration of the particular forklift truck. It seems to be general experience that with existing equipment, collision occurs and the column damage is confined to below the first level of beams. When the lowest beam is located at some distance, say 2 feet to 4 feet from the floor, the rear counterweight of some trucks can impact the beam imposing a very significant horizontal load on the beam or frame bracing. In this case impact protection of a special nature should be considered.

While it is not practical to design racks to resist the maximum possible impact of storage equipment, this section addresses two possible ways to safeguard racks against the consequences of minor collisions. Users should contact the rack supplier for recommendations on products available.

The first way is the provision for protective devices that will prevent trucks from hitting the exposed columns. Fenders or bumpers can and have been used for this purpose. Also, deflectors which, while not designed to withstand the full impact of the truck, are shaped to deflect it away from collision with the columns. No specific data is available regarding the force for which such protective devices must be designed. It is the responsibility of the owner to specify, in the contract documents, the design requirements of the deflector. They will, of course, depend on the weight and velocity of the particular truck and also on such energy absorbing bumpers as may be provided on the truck itself. It is not necessary, that such devices fully maintain their own integrity in such collisions, but merely that they protect the columns from collision, even at considerable damage to themselves. Therefore such devices should be made to be easily replaceable or repairable in case of collision damage.

A second method of safeguarding the rack upright is to reinforce the bottom portion of the front column and/or bracing in the frame. Common methods include welding an angle deflector to the front of the aisle side column, doubling the section strength by welding two columns together, using heavier horizontal and diagonal bracing to provide alternate load paths, or using larger baseplates and anchors with the aisle side column.

These methods are intended to aid in avoiding collapse of the frame due to minor impacts (not major collisions) and limit the damage caused. Users must perform regular inspections to ensure damaged racks are not used to store loads, and that adequate repairs are made promptly in consultation with the rack supplier.

1.4.10 RACKS CONNECTED TO THE BUILDING STRUCTURE.

It is common practice to connect certain racks to the building structure for added stability, such as single rows adjacent to a wall. It is important – particularly in seismic applications – to consider the forces that can be applied to each of the structures as well as considering the structural interactions due to those forces. This section requires that the building owner be advised of the possible force imposed by the rack so that he can notify the building architect.

1.4.11 PLUMBNESS.

Out-of-plumb installation of racks creates additional stress in the uprights that may not be accounted for in the design of the components. Commonly shims are used under the column baseplates to account for uneven floors, and to maintain the needed tolerance. The previously specified tolerance of 1 inch in 10 feet of height has been changed to 0.5 inches in this edition of this specification to reflect common industry practice.

2 LOADING

The purpose of this section is to clarify the design methods used in the AISI and the AISC Specifications as they apply to storage racks. Storage racks differ from building structures in that their dead loads are a very small percentage of the total load when compared to buildings. Also, racks have product loads in addition to dead load and live load. Product load has been defined for racks as the products or pallet loads stored in the rack. This load is given the symbol, PL, in the load combinations. Live loads could still be present in racks. Examples of live loads would be floor loading from work platforms or the moving equipment loads of Section 2.5.2.

Since the last edition of the RMI Specification LRFD design has become much more commonplace for cold-formed and structural steel. The AISC has recognized both methods of design [2, 3]. The AISI has a combined specification [4] that contains both methods. The two methods of analysis should give results that are similar but they will not be exactly the same. The RMI allows the designer to use either method but the analysis must be consistent, that is the ASD and LRFD methods must not be mixed. The designer may see some benefit to the LRFD method due to the product load factor that has been incorporated in the load combinations.

2.1 LOAD COMBINATIONS FOR THE ASD DESIGN METHOD

The ASD design method is to use unfactored applied loads and then compare them with the allowable force, which is the ultimate load divided by a factor of safety. All of the loads have no factors except for combination #5. The 0.88 value is applied to the shelf plus impact critical because impact is a short duration load and for the two pallet case where the impact effects are not large, the beam design will result in the traditional factor of safety of 1.65 to 1. All loads resulting from these combinations must be checked against allowable loads from the AISC – ASD Specification [3] or AISI – ASD Specification [4].

The load PL_{app} represents the product loading that must be present for the WL or the EL to be possible. It is recommended that this be the percent of the product load that was used to compute the base shear for the seismic analysis. For outdoor racks or rack buildings with cladding PL_{app} is zero for the wind uplift case because the racks may be required to resist the full wind force when they are empty.

Combination #3 and #4 may be multiplied by 0.75. This is the same as using the 33% stress increase that has been historically allowed when checking for wind or seismic cases. The EL is allowed to be multiplied by 0.67 when the code used to derive the seismic loading is limit states based (such as section 2.7 of this specification). This is because the limit states based codes give higher applied seismic forces by about 50 percent. These codes have been written to be used with the LRFD design method.

2.2 LOAD FACTORS AND COMBINATIONS FOR THE LRFD DESIGN METHOD

As stated above, product loads are the loads that are placed on storage racks. Product load has been differentiated from the live load so it can be factored differently. It is necessary to differentiate between these two types of loading because their treatment under seismic conditions is also different. The maximum product load is generally well known for a typical installation and more predictable because the weight and density of the products to be stored is known. The potential for overload may also be reduced due to the lifting limitations of the fork truck. For this reason a smaller load factor than that used for a live load is justified. However the probability of a high product load being present during an earthquake is greater than the probability of the high live load being present, so for some of the loading combinations the product load factor is higher.

The purpose of these modifications is to make the load combinations more realistic for the rack structures. These loads are to be compared with the nominal strength for the member or connection, multiplied by the appropriate resistance factor from the AISC – LRFD Specification [2] or the AISI Specification [4].

Product load has been added to the uplift case because, for racks, the product loads must be present in order for the prescribed seismic forces to act. It is possible to get an irregular loading that will produce seismic uplift on an unloaded column for an interconnected section of rack. The unloaded frames, in this case, would be tied to frames with pallet loading that would resist uplift. The seismic forces would, in turn, be less for the under-loaded areas. The conservatism here is that the product load not used to compute W is still present and resisting uplift.

The modification of the LRFD approach is a reduced load factor, for product loads, of 1.4. As mentioned above, this is justified due to better predictability of product loads than live loads. The designer is reminded that this change only applies to product loading only and does not apply to other live loading from roof, mezzanines and so on. The load factors for all of the combinations were derived by averaging the LL factor and the DL factor. This will result in a safety factor for the gravity load case of 1.65 for the entire range of column lengths with respect to product loading. The resistance factor (ϕ) for compression members is 0.85.

Load combination #7 in the LRFD and load combination #5 in the ASD have been added to give a more realistic treatment of impact loading for shelves. This combination will

$$(1.2 \times 0.01 \times PL) + (1.4 \times PL) + (1.4 \times (0.125 \times PL)) = 1.587\ PL$$

usually govern the design of the shelf. For a two pallet wide shelf, which is most common, the impact effect is about 1/8 of the beam load so the margin of safety for this combination (with the DL equal to 1 percent of the product load) would be:

For $\phi = 0.95$

This corresponds to the traditional 1.67 factor of safety. A resistance factor (ϕ_b) of 0.9

$$1.587/0.95 = 1.67$$

would result in a higher factor of safety. This load combination would govern over combination #2 because combination #2 includes no impact. For ASD, combination #2 could govern on a shelf with many loads applied, for example a shelf with 50 boxes hand stacked. Combination #7 will always govern for LRFD.

There is no need to change live load factors for racks when the area floor loading exceeds 100 psf as required in some codes and specifications. This is covered in the notes within section 2.2. of the RMI Specification. Also, when the method used to derive the seismic lateral forces is limit states based (such as section 2.7 of this specification) the load factor for EL in combinations #5 and #6 may be reduced to 1.0. This is consistent with other codes.

The resistance factors for the anchor bolts have been derived to give a factor of safety of 4 as recommended by most anchor bolt manufacturers and accounting for the 33% Allowable Stress increase, where applicable.

2.3 DESIGN LOADS.

The Specification includes, in addition to the vertical load, provisions for vertical impact and horizontal loads that a normal rack installation will experience during its use. It is important to include all forces that could reasonably act together. For instance, one could reasonably expect that a forklift truck would not be placing the load on the rack during an earthquake. Therefore, it is not necessary to consider both shelf impact and earthquake loading acting concurrently

2.4 VERTICAL IMPACT LOADS.

Handling of pallets being placed on and being removed from shelves is responsible for most beam damage. Considering the magnitude of the forces possible, no beam can be designed and guaranteed not to be damaged by a pallet being dropped onto the rack. An allowance for impact can therefore be no substitute for proper lift truck operation. How the lift truck is operated is the sole responsibility of the owner. The owner must make sure that his drivers are properly trained and responsible, and that no one else can operate the trucks at any time. It must also be recognized that it is not possible to load a pallet without applying some impact to the shelf. When a pallet is loaded onto the rack, the impact force will be transmitted by the pallet being loaded. The pallet position should be chosen to ensure that the minimum safety margin exists for loading pallets at any location, Section 2.4 requires the impact force to be on one shelf distributed along the width of the pallet which causes the greatest stresses.

When determining allowable loads by test, the impact load must be included in checking compliance with Section 2.4. The impact load should be applied by loading one pallet 125% of the test weight with all of the other pallets at the test weight. This will give an additional 25% of the test pallet load on each shelf. The heavy pallet may have to be placed in different locations to check bending moment, shear force and end connections. When testing or designing for deflection in accordance with Section 5.3, the inclusion of impact is not required.

This impact provision is included to add extra safety to the design of the shelves and their connections due to vertical impact of loads being placed by the lift truck or other device. When 25% of one pallet load is added for impact on a two load wide shelf, the margin of safety is about 1.67 as shown in the Commentary Section 2.2. This is equal to the traditional margin of safety. If there is one load per shelf the margin of safety will be higher. For the shelf with many small boxes the margin of safety will be less and could approach $1.4/\phi$ or 1.47 minimum

2.5 HORIZONTAL FORCES

There are few true horizontal loads imposed on a storage rack system. There are cases where horizontal forces may be generated that are addressed in other parts of this specification, such as Section 2.6, Wind Loads and Section 2.7 Earthquake Forces and the design of the storage rack components must be checked for those forces when applicable. Other horizontal loads are generally balanced out in long rack rows, such as plumbness or member out of straightness, or isolated, such as fork truck impacts, and it is not generally necessary to check the overall rack system for these loads. The local effects of possible fork truck impacts are addressed in Section 1.4.9 and, if columns are exposed to potential impacts, careful attention should be paid to the impact resistance.

In past RMI specifications, an artificially high horizontal force was prescribed to be imposed in both the down-aisle and the cross-aisle direction of the rack. In the down-aisle direction the column members were required to be checked for axial load from the pallets and bending moments from this horizontal force. The horizontal force was a $P\Delta$ force generated if the storage rack row leaned, in the down-aisle direction, 0.015 of the distance to the first shelf. It was found, in subsequent investigations, that this force had a severe impact on the capacity of an individual rack column. However, when many columns are installed in a row and interconnected the effect was balanced out. Further, thousands of storage racks systems have been designed and installed without the $P\Delta$ forces and have performed well.

Other specifications NEHRP [5], UBC [6] specify a drift limit for storage racks of 0.0125 h_x and 0.0036 h_x respectively. These specifications do not require $P\Delta$ analysis for drifts below the indicated limits. These codes state that if an analysis of the storage rack shows that the drift is within these limits, no analysis of the main force resisting components for $P\Delta$ forces is required.

The drift calculation for a column segment is straight forward. However, much of the down-aisle drift in a storage rack comes from the flexibility of the beam-to-column connection. The effect on the system of the various manufacturers' beam to column connectors is generally difficult to analyze. If the connections are strong enough, generally, the overall rack system will also be sufficient. It is for that reason that a separate check of the strength of the connections is needed. Since the strength of many connectors can not be analyzed, the connection test in Section 9.4 is recommended.

In the cross-aisle direction there are not generally the quantity of members necessary to balance out the horizontal forces. The usual configuration is a back-to-back rack row with two frames attached with back-to-back ties. Additionally, fork truck impact will have a greater effect in the cross-aisle direction. In the cross-aisle direction the frame bracing can generally accommodate a force of 1.5% of the frame vertical load. Similarly, in the cross-aisle direction, the connections of the bracing to the columns should also be checked.

2.5.2.1 Some forms of storage rack also provide guidance for the top of the material handling equipment. In that case the equipment manufacturer will specify the top horizontal force and the frequency of that force. It is necessary that the force be included in the rack design in proper combination with the other forces on the system.

2.6 WIND LOADS

There are instances where racks will be the main wind resisting structural system. Storage racks may be installed outdoors or they may be designed as a part of a rack-supported structure.

When walls do not protect the rack system the wind will exert force primarily on the surface area of the pallet loads in the stored locations. Consideration should be given to unit loads of less than maximum weight but the same size as the posted unit load. Consideration should also be given to partially loaded rack where, for instance, a load is placed only in the top position and no others. The effects of wind acting on the rack components when empty, or during construction should be considered.

When a rack system supports a wall, consideration should be given in the design, especially for overturning, of racks that may be subjected to wind loading whether or not pallets loads are placed in the racks.

2.7 EARTHQUAKE FORCES

2.7.1 GENERAL

It is important that rack systems be engineered, manufactured, installed, and utilized in a manner that such systems can perform adequately under all known loading conditions. Many geographic regions have building codes which are known to require that building and non-building structures, including rack systems, be designed to accommodate earthquake loads. The analytical approach to the seismic behavior of rack structures developed within this Specification is intended to reflect the current thinking within the Building Seismic Safety Council (BSSC) and their current provisions of the National Earthquake Hazards Reduction Program NEHRP [7] , as well as the national model codes promulgated by the Building Officials and Code Administrators International, BOCA [8]; the International Conference of Building Officials, ICBO [6]; the Southern Building Code Congress International, SBCCI [9], American Society of Civil Engineers, ASCE [10].

Should the rack structure be connected to another structure in a manner which significantly modifies the free field ground motions, then this structural interaction must be made part of the analysis and resulting design of both the rack system and the supporting structure.

The principle advantage of mass-produced steel storage rack systems is their modular design, which allows considerable flexibility of configuration and installation. This advantage also presents a serious challenge to competent seismic performance. The initial installation of a rack system should be in accordance with an engineered design. Subsequent modifications should be made only with guidance by a registered design professional to avoid compromising the seismic integrity of the system. Further, storage rack systems are often subject to rough use and damage. It is the owner's responsibility to maintain the integrity of the rack to insure adequate structural performance during an earthquake.

2.7.2 MINIMUM SEISMIC FORCES

The base of a rack system supported by a floor slab at or below grade experiences the ground accelerations directly, and the design should proceed accordingly. For a rack system supported by another structure (e.g., an upper story of a multi-story building structure) the structural analysis must consider the interaction between the structures.

The system importance factors with magnitudes greater than one are intended to result in a higher performance level for certain rack installations under seismic conditions, viz., those within systems deemed to be essential facilities that should continue to perform following a seismic event; those which might release hazardous materials in such a seismic event; and those installations located in warehouse retail stores where the rack system is located in an area open to the general public. In such a warehouse retail store, unlike a sparsely populated typical warehouse and distribution center, large numbers of the shopping public can be expected to be within the rack system during business hours. The consequences of a rack failure, in this environment, dictate a higher level of performance for such systems.

To properly account for the fact that the product loads placed on shelves are often less than the capacity for which the shelves are designed, the product load reduction factor (PL_{RF}) is introduced. Thus, in the longitudinal (or down-aisle) direction, where there are numerous repetitious pallet positions, $PL_{average}$ is defined as the maximum total weight of product expected on the shelves in any row divided by the number of shelves in that row. $PL_{maximum}$ is defined as the maximum weight of product that will be placed on any one shelf in that row, this being usually the design capacity for the pallet positions. With $PL_{average}$ and $PL_{maximum}$, the Product Load Reduction Factor (PL_{rf}) becomes simply the quotient of the two. This reduction is not permitted in the cross-aisle direction.

The factor of 0.67 applies to the loading considerations under seismic events. Research has shown that there is some friction inducing, energy dissipating, relative movement between the rack and the stored product during seismic motions. The 0.67 factor represents the fraction of the dynamically active load on a fully-loaded system that is likely to be felt by a structure in a normal application, and that needs to be taken into account in the determination of lateral loads under seismic events. If the designer knows that for a particular installation the dynamic portion of the load is likely to be greater than 67 percent, then such a higher magnitude should be used in the determination of lateral forces.

2.7.3 CALCULATION OF SEISMIC RESPONSE COEFFICIENT.

The seismic response coefficient is intended to be a site-specific value; the magnitude of this coefficient is affected by the characteristics of the structural system through the values of R and T, and also by the characteristics of the soil underlying the building on whose floors the rack system is founded, through the values assigned to the various soil profile types. T is the fundamental period of the rack structure. The factor R is an empirical response reduction factor intended to account for both damping and the ductility inherent in the structural system at displacements great enough to surpass initial yield and approach the ultimate load displacement of the structural system. Magnitudes of the acceleration components A_v and A_a are to be taken from the accompanying contour maps or as specified by the building code authority.

There are several ways for estimating the fundamental period of vibration for a pallet rack in the down aisle direction. One method that is sometimes used is the Rayleigh Equation:

$$T = 2\pi \sqrt{\frac{\sum W_i \Delta_i^2}{g \sum F_i \Delta_i}}$$

where:

W_I = DL + PL (used to determine the seismic lateral forces) + 0.25LL at each level *i*.
For RMI Specification Section 2.7: DL + 0.67PL + 0.25LL

F_i = Seismic lateral force at level *i*. The force at each level must be computed from the force distribution equation required by the seismic design code. For the RMI Specification, these formulas are given in Section 2.7.4.

g = acceleration due to gravity (386.4 in/sec^2)

T = the fundamental period of vibration.

Δ_i = total lateral displacement at level *i* relative to the base, as computed using F_i.

In order to use the Rayleigh Equation it is necessary to be able to compute the story lateral displacements. These values can be found by a rigorous frame analysis or by approximation. More accurate computations of the lateral displacements will result in a more accurate T value. If the second order lateral displacements are ignored or the drifts are otherwise underestimated the resulting T value will be conservative. The Horne-Davis method for frame analysis provides a simple method for computing lateral displacements at the beam levels. This method computes displacements as a function of P_{cr} which is the elastic critical story buckling load of the column span. A summary is

$$\Delta_p = \frac{H \cdot L}{P_{cr}} + \Delta_{i-1}$$

shown here:

where:

Δ_p = primary story drift not including PΔ effects.

H = total lateral force above the shelf elevation being evaluated.

L = column span length.

Δ_{i-1} = Primary deflection just below the level being evaluated.

P_{cr} = critical elastic buckling load of the column span

One method that is sometimes used to compute the P_{cr} value is to calculate it using the value K_x for the column span. In this sense K_x is being used as a tool to approximate the effect of story buckling on the critical elastic buckling load of the column. P_{cr} could also be figured from a rigorous frame analysis or other equally acceptable methods. Computation of P_{cr} using the K method is shown below:

$$P_{cr} = \frac{\pi^2 E I_x}{(K_x L)^2}$$

where:

K_x = Effective length factor for story buckling in the down aisle direction.

I_x = Column Moment of inertia perpendicular to the plane of the frame.

For the total drift at level i.

$$\Delta_i = \frac{\Delta_p}{1 - P/P_{cr}} = \frac{HL}{P_{cr} - P}$$

This method will be very accurate if the value of K_x is accurately determined. K_x for this method is a measure of the lateral stiffness of the story. If K_x is underestimated, the T value will be conservative. The designer should use the same K_x value to check column members as is used to determine T. The value of K_x used should not be more than is used for the member check.

The period in the cross-aisle direction is usually much shorter.

2.7.4 VERTICAL DISTRIBUTION OF SEISMIC FORCES

The calculation of the vertical distribution of the lateral forces F which are being resisted by the base shear V results in a linearly increasing or triangular distribution for values of k=1, and a nonlinearly increasing value of F for values of k greater than one.

It is appropriate to account fairly for the contribution of the shelf-loading pattern on the development of the lateral forces, their distribution, and the resulting behavior of the rack structure. Thus, it is felt that when the bottom most pallet beam is within twelve (12) inches of the floor, such a shelf loading contributes little to the lateral deflections and resulting lateral force distribution along the height of the structure. However, when such a bottom shelf is located at an elevation greater than twelve (12) inches above the floor, the contributions will begin to be significant and should be considered in the same manner as the remaining loading on all the upper shelves.

2.7.5 HORIZONTAL SHEAR DISTRIBUTION

The magnitude of the lateral shear force at any level is determined simply by the equations of equilibrium applied to the particular section of the structure. The story shear in any story is the sum of the lateral forces acting at all levels above that story.

2.7.6 OVERTURNING

In an effort to represent an extreme case which might result in an unstable rack system, an analysis must be made and resulting design implemented for the condition where only the top-most level of the rack is loaded; that load must be the applicable design load and the lateral force caused by a seismic event shall be determined accordingly.

This overturning check is intended for only anchor uplift and floor reactions. When calculating the load combination for seismic uplift in Section 2.1 and 2.2, PL is the top load level only.

2.7.7 STORAGE RACK NEED NOT BE CHECKED FOR OVERTURNING WITH MORE THAN JUST THE TOP LOAD IN PLACE. THE FULLY LOADED RACK HAS TO DEFLECT SUBSTANTIALLY MORE THAN THE TOP LOADED CONDITION TO MOVE THE CENTER OF GRAVITY TO THE CRITICAL OVERTURNING LOCATION. CONCURRENT FORCES

Considering the probabilities, it is reasonable to expect that the effects of out-of-plumbness, impact, wind forces, and seismic events will not occur simultaneously. The design shall proceed accordingly.

3 DESIGN PROCEDURES.

This section specifies that engineering design calculations are to be made in accordance with accepted principles and conventional methods of structural design. This means among other things, that the basic concepts of structural analysis must be observed. This section also refers to the AISI [4] and AISC [2, 3] Specifications as modified in various specifics in this Specification.

The following is just one example of what is meant by “conventional methods of structural analysis”. Depending on types of connections, cross sections and relative capacities of beams and columns, pallet racks may function and be analyzed either as elastic rigid frames or as frames with semi-rigid connections. Regardless of what methods are used, the basic laws of equilibrium and compatibility must be satisfied in all parts of the structure. For example in the design of shelf beams, advantage can be taken of negative end moments up to values that can be developed by the specific connections, as determined by test (Section 9.4). However, if this is done, the column must be designed for the end moments which they must develop in order to create the end restraint used in the beam design. For instance, the upper end of a corner column has to support the full end moment of the abutting uppermost shelf beam, and the column must be designed for its axial load plus indicated moment. Unless this is done, the basic law of equilibrium has been violated. The same holds true at all other beam and column joints, except that the unbalanced end moment of two adjacent beams, is jointly resisted by both columns framing in to that joint and possibly also by the unloaded beam, if its connection can resist an appropriate moment. This is so regardless of whether the negative beam moments have been calculated on the basis of conventional rigid frame analysis, or on the basis of semi-rigid analysis (i.e., using test values of connection capacities). By the simple law of equilibrium, no negative moment can act on the end of a beam unless the abutting members can develop this moment, and are designed for it.

There may be situations in rack structures for which adequate design methods do not exist. This is the case where configurations of sections are used which cannot be calculated by established methods, where connections of a non-standard character are employed, etc. In these cases, design calculations of member and connection capacity, shall be replaced by appropriate tests. Several of these tests, peculiar to rack construction, are spelled out in later parts of the Specification. Tests not spelled out are to be conducted according to the general test procedure requirements of Sec. F1 of the AISI Specification [4].

Tests are not permitted to be used in lieu of design calculations except in those situations which cannot be calculated by available methods. The AISI Specification [4] is quite specific about this in Sec. F1. It should be noted that confirmatory tests have a different nature and are covered in the AISI Specification [4] Section F2.

No slenderness limitations are imposed on tension members. Indeed the AISC Specification [2,3] limitations themselves are not mandatory, but are only suggested as good practice.

4 DESIGN OF STEEL ELEMENTS AND MEMBERS.

Neither the AISI [4] nor the AISC Specifications [2, 3] make provisions for perforated members, particularly of the type routinely used for columns and other components of racks. The effect of perforations on the load carrying capacity of compression members is accounted for by the modification of some of the definitions of these Specifications. The approach is to use the effective section properties based on the net section whereas the AISI Specification [4] bases the effective section properties on the unperforated section. Further information on the development of the AISI Specifications [4] can be found in Reference 12.

4.2 COLD-FORMED STEEL MEMBERS

4.2.2 FLEXURAL MEMBERS. {THE AISI SPECIFICATION [4] SECTION C3}.

The RMI Specification approach involves the replacement of the section properties used in the AISI Specification [4] by the effective net section properties. The effective net section is the effective section determined for the net section. Effective width equations do not exist for the type of perforations that are common in rack columns. For this reason approximate approaches need to be formulated.

The area of the effective section for axial loading is determined by means of stub column tests according to Section 9.2. There are no test procedures for determining the effective section properties for bending. The approximate approach of this section was developed assuming that when the section is in tension local buckling does not reduce the capacity thus Q = 1 for the tension region. This assumption implies that the cold forming effects do not increase the axial tensile strength. In flexure approximately half of the section is in compression and the other half is subjected to tension. Of course the effective section is not symmetric and thus this is an approximation. The effective area of the portion of the section in compression can be approximated conservatively by using the result of stub column tests. This is conservative because the web has a more favorable stress gradient when the section is in flexure. Thus the reduction factor for the area to account for local buckling when the section is in flexure is taken as the average of 1.0 for the tension portion and Q for the compression portion, namely $0.5 + Q/2$. Thus, S_e, the elastic section modulus of the effective net section at design yield stress, is determined by multiplying the net section elastic modulus by this reduction factor.

The term S_c is the elastic section modulus of the effective net section at the lateral buckling stress of the gross section M_c / S_f. The reduction factor at the lateral buckling stress of the gross section is derived on the basis of the approach described in Reference 12 as:

$$1 - \frac{1-Q}{2}\left(\frac{M_c / S_f}{F_y}\right)^Q .$$

This approach gives conservative (lower) values of the reduction factor compared with the more complicated rational analysis procedures described in the 1990 Edition of the RMI Specification [13] Commentary.

In the calculation of M_e, σ_{ex}, σ_{ey}, and σ_t the section properties are to be based on full unreduced gross section considering round corners except for J, j, r_o and C_w which shall be based on the full unreduced gross section using sharp corners because the calculation of these parameters using rounded corners for the net section is extremely tedious.

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

4.2.3 CONCENTRICALLY LOADED COMPRESSION MEMBERS. {The AISI Specification [4] Section 4}.

Compression members can buckle in either of two ways: purely flexurally, i.e., by simple bending about one of the principle axes without twist; or torsional-flexurally, i.e., by bending accompanied by twisting of the member. Some types of members which buckle purely flexurally are: all closed box-type members, sections whose shear center and centroid coincide, which is true for doubly-symmetrical members [e.g., I-sections], equal flange Z-sections, and others. 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, and others. In all these shapes, centroid and shear center 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. For some of the more common shapes, Part VII of the AISI Cold-Formed Steel Design Manual [14] contains curves which permit one to decide whether or not a member of given dimensions will buckle torsional-flexurally. Another way is to compare elastic torsional flexural buckling stress Eq C4.2-1 of Reference 4 with the elastic flexural buckling stress Eq C4.1-1 of Reference 4.

In designing columns for flexural buckling without torsion, the effective length factors K shall be taken as specified in Section 6.3 of this specification. For singly symmetrical shapes these methods 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 rack structures. For instance, for a pallet rack with channel or C-columns placed so that the x-axis is in the plane of the upright frame, the unbraced length L_x for buckling about the x-axis is the length from the floor to the center line of the bottom beam, or between successive beam center lines, as the case may be. (This is the unbraced length L_x, not the effective length K_xL_x.) However, for torsion it can be assumed 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 columns, provided the connection itself does not permit twist. Typical connection details between the columns and the bracing which are expected to inhibit twist and those that are not are shown in Fig. 4.2.3. For those racks with proper connection details, the unbraced length L_t for torsion will be the free length between adjacent connections to any members which counteract torsion. For instance, if a diagonal of an upright frame meets the 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 unbraced length for torsion, L_t.

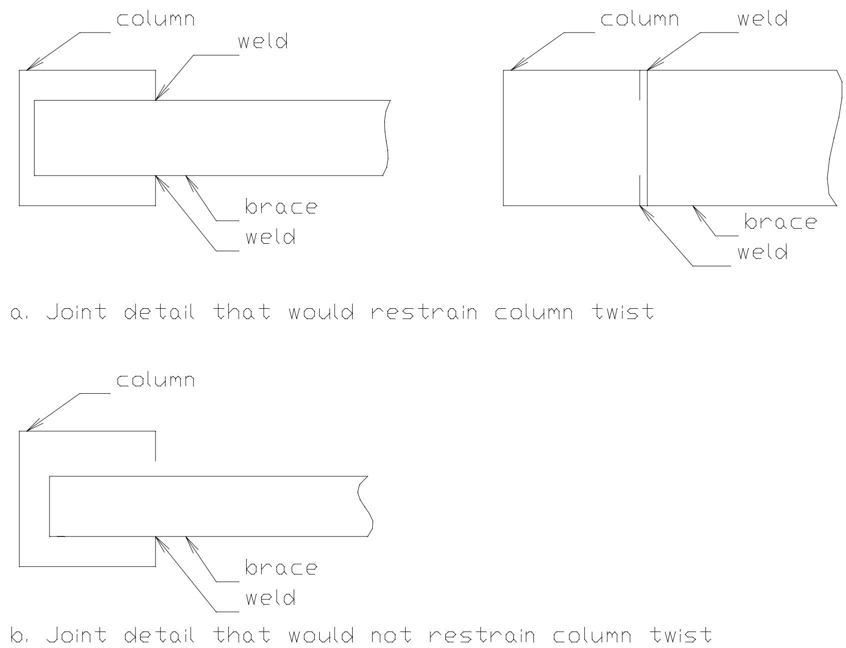


Fig. 4.2.3-1 Joint details

Different effective lengths for torsion and flexure are accounted for by taking K_xL_x in the expression for σ_{ex}, and K_tL_t in the expression for σ_t. The effective length factors K_x and K_t are given in Sections 6.3.1 and 6.3.3, respectively.

The treatment of concentrically-loaded perforated compression members is based on a modification of the AISI Specification [4] approach for unperforated compression members. The modification is based on the studies reported in Reference15. The procedure consists of obtaining the nominal axial load capacity by multiplying the nominal failure stress obtained for the gross section by the effective net area obtained at the nominal failure stress. In general, the effective net area cannot be calculated for column sections with the types of perforations typical in rack structures. For this reason the effective net section area is to be determined through the use of the following formula which was developed in Ref.12:

$$A_e = \left[1 - (1 - Q)\left(\frac{F_n}{F_y}\right)^Q\right] A_{NetMin}$$

where the Q factor is determined by the procedure specified in Section 9.2.

5 BEAMS

5.2 CROSS SECTION

For pallet rack and stacker rack beams, this section states that the load effects shall be determined by conventional methods of calculation if the shape of the cross section permits. In general, the usual simple formulas for stresses and deflections of beams apply only if the cross section is symmetrical about the loading direction, i.e., if the section has a vertical axis of symmetry. Beams of any other cross sectional shape may twist under load. Such twist can reduce the carrying capacity of the beams, and/or result in deflections larger than that determined by conventional computations. Examples of such sections are channels, particularly those with wide flanges, and wide flanged C-shapes when placed with web vertical. Since calculations that include the twist are fairly complex and not always reliable Section 5.2 calls instead for test determination.

It is worth noting that closed box shapes, even if they have no vertical axis of symmetry, are much less subject to twist than open shapes. Thus, in many cases of closed unsymmetrical box beams, determination by conventional calculations may prove adequate.

It can be shown that the following equation can be used to account for the effect of end fixity in determining the maximum midspan moment M_{max} of a pallet beam considering semi-rigid end connections:

$$M_{Max} = \frac{WL}{8} r_m$$

where:

$$r_m = 1 - \frac{2FL}{6EI_b + 3FL}$$

E = the modulus of elasticity

F = the joint spring constant determined either by the Cantilever Test described in Sec. 9.4 or by Pallet Beam in Upright Frames Assembly Test described in Section 9.3.2.

I_b = the beam moment of inertia about the bending axis

L = the span of the beam

W = the total load on each beam [including vertical impact loads]

where:

$$M_e = \frac{wL}{8}\left(1 - r_m\right)$$

M_e = the beam end moment

In the above derivation the load is assumed to be uniformly distributed. For a value of F equal to zero, M_{max}=WL/8 is obtained. 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 the 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.

If one considers semi-rigid joints, the following expression for maximum deflection δ_{max} can be derived.

$$\delta_{Max} = \delta_{ss} r_d$$

where:

$$\delta_{ss} = \frac{5WL^3}{384EI_b}$$

$$r_d = 1 - \frac{4FL}{5FL + 10EI_b}$$

5.3 DEFLECTIONS.

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

6 UPRIGHT FRAME DESIGN

6.3 EFFECTIVE LENGTHS.

The AISI [4] and the AISC [2, 3] 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 method is not the only available technique for determining the axial capacity of a compression member. Alternative methods, consistent with AISC and AISI are equally acceptable. Where large lateral load requirements already exist (such as the higher seismic zones) a method employing the lateral load may dominate the instability considerations in the design and a K factor approach may not be required. The effective length factor accounts for the restraining effect of the end conditions or the effect of the members framed into a particular member.

The effective length concept is one method for estimating the interaction effects of the total frame on a compression member being considered. The RMI has chosen to use the K factor approach but does not preclude the use of other properly substantiated methods. Several references are available concerning alternatives to effective length factors for multilevel frames under combined loads or gravity loads alone. Work has been done for hot-rolled members and the RMI has co-sponsored, with AISI ongoing research for cold-formed members.

General discussions of the effective length concept can be found in references[16, 17, 18, 19]. Basically, the effective length factor K times the unbraced 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 connections. Though the effective length is computed on the basis of elastic frame behavior, it is general practice to use the effective length approach to find the inelastic load carrying capacity. This is the approach taken in the AISI and the AISC Specification [4, 2 and3] as well as in this specification. As discussed in connection with Section 4.2.2, the effective length approach is extended to the torsional-flexural buckling mode as well.

The behavior of rack structures and hence the effective length factor depends on the unique design of racks such as rigidity of the connection between columns and beams. Due to the wide variety of details and cross sectional dimensions in rack structures, the effective length factors vary within a very broad range. For example, 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 this specification are by no means maximum values. They are average values assuming the racks 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 too unconservative for the particular rack. Possible rational analysis procedures are presented later in this commentary.

6.3.1 FLEXURAL BUCKLING IN THE DIRECTION PERPENDICULAR TO THE UPRIGHT FRAMES.

The buckling considered here is parallel to the aisle. In general, racks have singly symmetric sections for columns 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 σ_{ex} in Section 4.2.2; σ_{ex} is in turn used in computing the torsional-flexural buckling load.

6.3.1.1 Racks Not Braced Against Sidesway.

This section is applicable to racks that do not meet the bracing requirements of Section 6.3.1.2. The side-sway failure of several columns in a down-aisle direction is quite catastrophic. Portions of rows or entire rows collapse. A value of K_x greater than 1.0 is used to design against this type of failure. The theoretical lower limit of K is 1.0 in braced framing or for full fixity at the top and the bottom of an unbraced column. Since full fixity is never achieved and the unbraced columns are free to translate, K will always be greater than 1.0 for unbraced frame design. The actual value of K depends on the rotational restraint at the top and the bottom of the column. Pallet racks that use semi-rigid connections will have K_x values much greater than 1.0 and may even exceed 2.0.

The Specification allows the use of $K_x = 1.7$ as a default value. Numerous typical rack assemblies were researched. These rack assemblies had K_x values ranging from as low as 1.3 to as high as 2.4. The racks with high K values had lighter beams and heavy columns. A larger number of bays tend to increase the K values because the supporting action of lighter loaded end frame columns diminishes. As the number of bays increases the probability of having all the bays fully loaded decreases. Thus as the number of bays increases the probability of getting a higher K may not increase. A three bay rack has a greater probability of being fully loaded than racks with more bays. Thus practice has shown that a three bay rack may be more likely to fail by sidesway.

The number of levels also has an influence on the value of K. As the number of fully loaded levels increase the value of K also increases. This is because the difference in loads in the lowest level and the second level columns decreases as the number of stories increases. When the difference in the loads decreases the value of K increases.

A value of K equal to 1.7 was chosen to give a reasonable amount of protection against sidesway for most common rack configurations. The designer should be aware that K may actually be greater than or less than the default value of 1.7. If the default value of 1.7 is used no further reductions may be taken based on utilization because utilization has already been considered in the selection of this value. K values other than 1.7 may be used if they can be justified on the basis of rational analysis. The rational analysis must properly consider column stiffness, beam stiffness, semi-rigid connection behavior and base fixity. The common approaches to evaluate K are frame analyses that compute the frame buckling loads directly and alignment charts. The latter approach will be discussed below.

The use of alignment charts to determine effective length coefficients is described in References 16 and 17. The procedures described in these references need to be modified as described below to account for the semi-rigid nature of the connection of the columns to the floor and to the pallet beams. The floor is assumed to be a beam with the following stiffness:

$$\frac{I_f}{L_f} = \frac{bd^2}{1440}$$

where:

b = the width of the column [parallel to the flexure axis]

d = the depth of the column [perpendicular to the flexure axis]

The floor is assumed to be concrete, and the column connection to the floor must be adequate to develop base moments consistent with this stiffness. For other floor material the equation should be modified.

In the analysis the stiffness of the pallet beams is taken to be reduced to $(I_b/L_b)_{red}$ due to the semi-rigid nature of the joints.

$$\left(\frac{I_b}{L_b}\right)_{red} = \frac{I_b / L_b}{1 + 6\left[{(EI_b)}/{(L_b F)}\right]}$$

where

I_b = the actual moment of inertia of the pallet beams

L_b = the actual span of the pallet beams

F = the joint rigidity determined by the Portal Test of Section 9.4.2

E = the modulus of elasticity

The analysis for the effective length factor for the portion of the column from the floor to the first beam level would involve the following G values as defined in the commentary of Ref. 3 and 4.

$$G_a = \frac{I_c\left({1}/{L_{c1}} + {1}/{L_{c2}} \right)}{2\left({I_b}/{L_b} \right)_{red}}$$

$$G_b = \frac{{I_c}/{L_{c1}}}{{I_f}/{L_f}}$$

where

I_c the column moment of inertia

L_{c1} the distance from the floor to the first beam level

L_{c2} the distance from the first beam level to the second beam level

The effective length factor is then found directly from references 16 and 17 on the basis of G_a and G_b.

The expression used above for I_f/L_f is based on References 20 and 21. The expression given in these references are modified to reflect the situation for rack columns 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. 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 1.5% lateral loads. This reference using the above procedure reaches reasonably satisfactory correlation between the computed and the observed test results. It must be noted, however, that the base fixity is just one of many properties of the rack that affect the structural behavior.

The expression for I_f/L_f given above assumes that the floor is concrete. The joint rigidity F is to be determined by a portal test. As the frame sidesways 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 the rotation is decreased at the other end. The portal method yields an intermediate value between the values of the rigidities of the two ends.

Table 6.3.1.1-1 shows the results of the rational analysis for various configurations. Depending on the rack configuration and the values of F, it is seen that the value of K may be unconservative or conservative. This table is for the case of L_{c1}=L_{c2} and b=d=3 in. A similar table can be developed for other L_{c1}, L_{c2}, b and d values.

		Beam - Column Connection Spring Constant (F in kip/rad)							
Ic/Lc	**Ib/Lb**	**200**	**400**	**600**	**800**	**1000**	**1200**	**1400**	**1600**
0.005	0.005	1.54	1.43	1.38	1.35	1.33	1.32	1.30	1.30
0.010	0.005	1.76	1.66	1.60	1.56	1.54	1.52	1.50	1.49
0.015	0.005	1.92	1.82	1.76	1.72	1.70	1.68	1.66	1.65
0.020	0.005	2.05	1.95	1.90	1.86	1.83	1.81	1.80	1.78
0.025	0.005	2.16	2.07	2.01	1.98	1.95	1.93	1.91	1.90
0.050	0.005	2.63	2.55	2.49	2.46	2.43	2.41	2.39	2.38
0.100	0.005	3.34	3.26	3.21	3.17	3.14	3.12	3.10	3.08
0.005	0.010	1.53	1.40	1.34	1.30	1.28	1.26	1.25	1.24
0.010	0.010	1.75	1.62	1.55	1.50	1.47	1.45	1.43	1.41
0.015	0.010	1.90	1.78	1.71	1.66	1.62	1.60	1.58	1.56
0.020	0.010	2.03	1.92	1.84	1.79	1.76	1.73	1.70	1.68
0.025	0.010	2.15	2.04	1.96	1.91	1.87	1.84	1.82	1.80
0.050	0.010	2.62	2.51	2.44	2.39	2.35	2.31	2.29	2.26
0.100	0.010	3.33	3.23	3.15	3.10	3.05	3.01	2.98	2.95
0.005	0.025	1.51	1.38	1.31	1.27	1.24	1.22	1.21	1.19
0.010	0.025	1.74	1.59	1.51	1.46	1.42	1.39	1.37	1.35
0.015	0.025	1.89	1.76	1.67	1.61	1.57	1.53	1.50	1.48
0.020	0.025	2.02	1.89	1.80	1.74	1.69	1.66	1.62	1.60
0.025	0.025	2.14	2.01	1.92	1.86	1.81	1.77	1.73	1.71
0.050	0.025	2.61	2.49	2.40	2.33	2.27	2.23	2.19	2.15
0.100	0.025	3.32	3.20	3.11	3.03	2.96	2.91	2.86	2.82
0.005	0.100	1.51	1.37	1.29	1.25	1.22	1.20	1.18	1.17
0.010	0.100	1.73	1.58	1.49	1.43	1.39	1.36	1.33	1.31
0.015	0.100	1.89	1.74	1.65	1.58	1.53	1.49	1.46	1.44
0.020	0.100	2.02	1.88	1.78	1.71	1.65	1.61	1.58	1.55
0.025	0.100	2.13	2.00	1.90	1.82	1.77	1.72	1.68	1.65
0.050	0.100	2.60	2.47	2.37	2.29	2.22	2.17	2.12	2.08
0.100	0.100	3.32	3.19	3.08	2.99	2.91	2.84	2.78	2.72

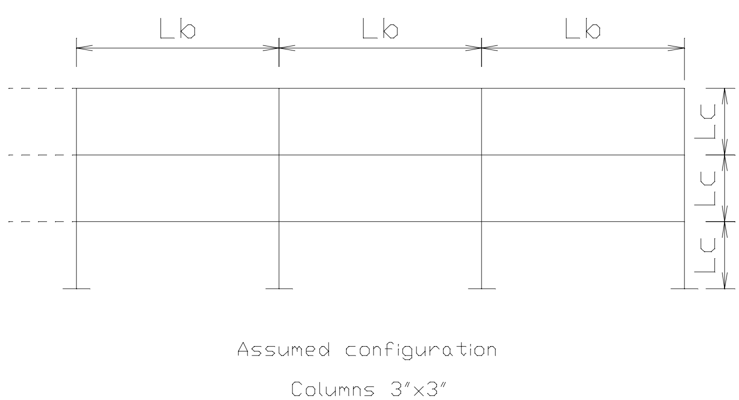


Fig. 6.3.1.1-1 Assumed overall configuration for Table 6.3.1.1-1

6.3.1.2 Racks Braced Against Sidesway.

A rack structure, in order to be treated as braced against sidesway, must have diagonal bracing in the vertical plane for the portion under consideration. This would restrain the columns in the braced plane. In order to restrain the columns in other planes, there need to be shelves which are rigid or have diagonal bracing in their horizontal plane as specified in this section. [Some of the terms used above are illustrated in Fig. 6.3.1.2a]. The function of this rigid or braced shelf is to ensure restraint for the other row of columns against sidesway with respect to the braced row of columns. All bracing should, of course, be tight and effective for its intended use.

Horizontal movement, or translation, of the front column relative to the rear column of rack 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 column, the frictional forces between the pallets and the load beams must be capable of resisting horizontal force perpendicular to the plane of the upright. The magnitude of this force at a bracing point should be at least 1.5% of the column load immediately below the beam acting as the horizontal brace. Whether or not sufficient force exists to prevent translation must 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 has been the subject of many tests and can conservatively be taken as 0.10. Special consideration is necessary in cold storage freezers where operational procedures can produce ice on the contact 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 brace plane as shown in Fig. 6.3.1.2[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 will be designed in accordance with Section 6.3.1.1.

The bottom and top portions of columns in Fig 6.3.1.2d are to be designed as columns in an unbraced rack whereas those in the mid-portion as columns in a braced rack.

A rational analysis similar to that described in 6.3.1.1 of this commentary can also be used for racks braced against sidesway. In this case the following changes need to be made:

$$\frac{I_f}{L_f} = \frac{bd^2}{240}$$

and

$$\left(\frac{I_b}{L_b}\right)_{red} = \frac{I_b/L_b}{1+2\left((EI_b)/(L_bF)\right)}$$

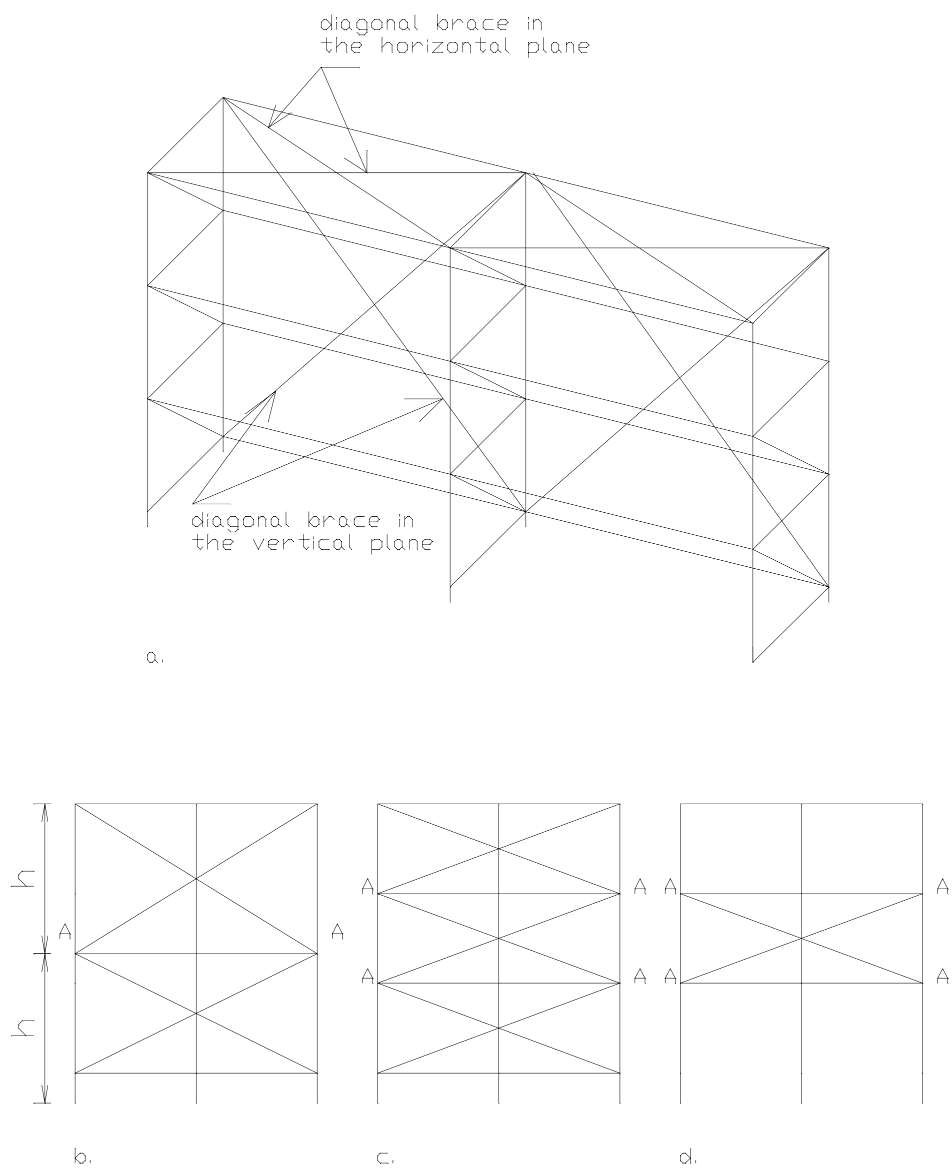


Figure 6.3.1.2-1 Racks Braced Against Sidesway

6.3.2 FLEXURAL BUCKLING IN THE PLANE OF THE UPRIGHT FRAME.

In rack structures the columns are in general either singly symmetrical shapes with the axis of symmetry in the plane of the upright frames or doubly symmetric shapes. Because of this, buckling in the planes 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 centerlines of braces and the columns intersect at one point as shown in Fig. 6.3.2-1 [a]. This is so because the braces do restrain the columns by virtue of their axial stiffness. On the other hand, the bracing action in the system shown in Fig 6.3.2-1 [b] depends on the flexural rigidities of the braces and the connections between the columns and the braces. Thus this type of bracing is not as effective.

The effective length factor for the frame of Fig. 6.3.2-1 [a] can be taken in general as 1.0. This assumes that the braces are adequate and the connection between the braces and columns are sufficiently rigid in the axial direction of the braces. The effective length factor for the frame of Fig 6.3.2-1 [b] is in general greater than one and can be found by rational analysis.

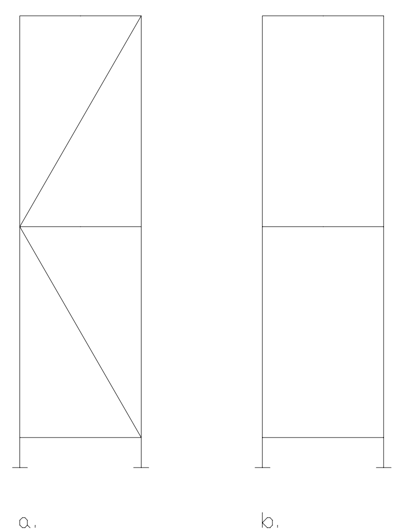


Figure 6.3.2-1 Braced and Unbraced Frames

In rack structures, frequently the centerlines of the horizontal and the diagonal braces and the centerline of the column do not meet at one point. Thus, the bracing arrangement falls between the extremes illustrated in Figs. 6.3.2-1 [a] and 6.3.2-1 [b]. The following three subsections treat various bracing configuration possibilities.

6.3.2.2 Upright Frames with Diagonal Braces or a Combination of Diagonal and Horizontal Braces that intersect the Columns are illustrated in Figs. 6.3.2-2[a] and [b]. These figures also define the terms L_{long} and L_{short}. As the ratio L_{short}/L_{long} increases, the frame approaches the case shown in Fig 6.3.2-2[b] and hence, the effective length factor can be greater than one.

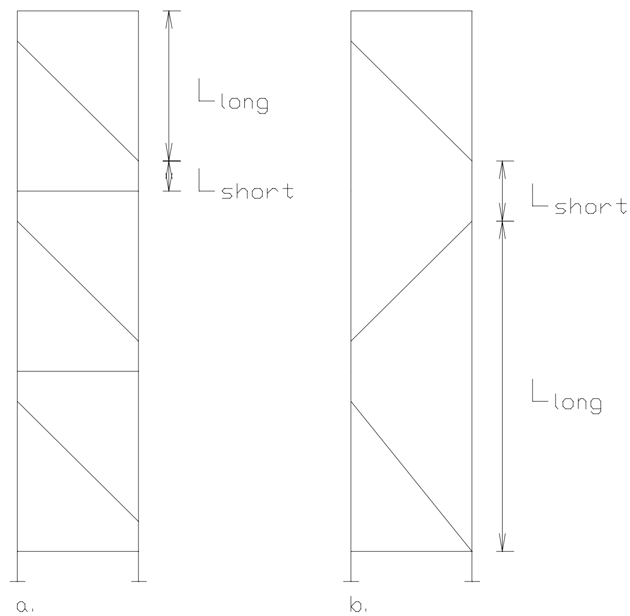
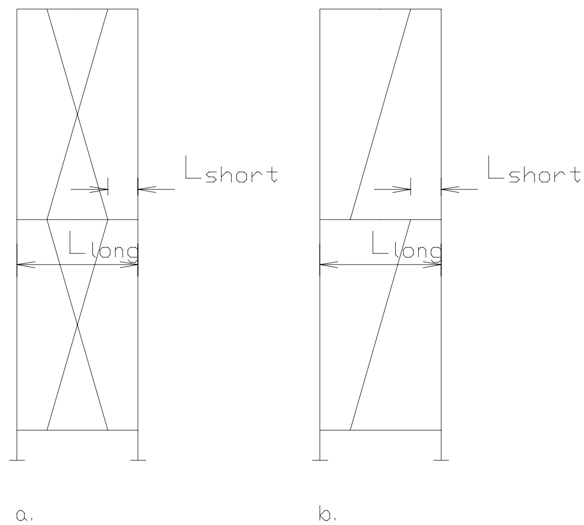


Fig. 6.3.2-2 Upright Frames with Diagonal Braces that intersect the Columns

The stability of the frame is quite dependent on not only the relative axial and flexural stiffness of the members but also the details of the connections between the members. The axial stiffness at the connection in the direction of the braces is dependent on the details of the connection.

6.3.2.3 Upright Frames with Diagonal Braces that Intersect Horizontal Braces are illustrated in Fig. 6.3.2-3[a] and [b]. As the ratio L_{short}/L_{long} increases, the basic behavior of the frame approaches that of Fig 6.3.2-3[b] and hence the effective length factor can be greater than one.



Figs. 6.3.2-3 Upright Frames with Diagonal Braces that intersect the Horizontal Braces

6.3.2.4 For uprights having bracing patterns such as the configuration shown in Fig. 6.3.2-1(b) no typical effective length factors are recommended. Rational analysis is to be used for such cases to determine the effective length factor. Alternately, the load carrying capacity may be determined by test.

6.3.3 TORSIONAL BUCKLING.

Though torsional buckling is not likely to happen in rack structures, torsional-flexural buckling is usually the governing critical buckling mode. The torsional buckling effective length factor is a parameter in the analysis of torsional-flexural behavior. The provision of the Section is based on References. 14 and 22. The value of K_t given in this section assumes an effective connection between the columns and the braces as shown in Fig.6.3.2(a).

6.4 STABILITY OF TRUSSED-BRACED UPRIGHT FRAMES.

The provisions of this section are based on Reference23 with the exception of the value of K. The expressions given in the reference were 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 this section are more likely to govern for high rise racks.

7 CONNECTIONS AND BEARING PLATES

The provisions of this section include the field connections and the connections between the various parts of the shop assemblies.

7.1 CONNECTIONS

7.1.1 GENERAL

The beam end connections must be designed to resist the forces and moments obtained from the structural analysis.

The effects of eccentricity of the connection and the effect of rotation of an attachment to the edge of an unstiffened flange must be evaluated. The influence of these connections on the overall behavior is significant. [Refer to 5.3]. Particular attention should be directed to the column-to-bracing connections.

7.1.2 BEAM LOCKING DEVICE

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

Failure of the locking device is defined as the distortion of the locking device that prevents reapplication of upward force, removal, reinstallation, or reduces the carrying capacity.

7.1.3 MOVABLE SHELF RACKS

The phrase “connected to each other rigidly” indicates that the beams are connected such that skewing of transverse members will be prevented in normal use.

7.2 BEARING PLATES

The column base connections must be designed to resist the forces and moments obtained from the structural analysis.

To reduce the probability of local buckling at the base, welds from the base plate to the column should be adequate to properly transfer loads. When analysis indicates, the bearing plate and welds to the rack column shall be designed for uplift forces. For bearing surfaces other than concrete, special design is required.

Actual field experience and limited testing has shown that base plates thinner than those normally provided under hot rolled structural shapes, designed to AISC Specifications, may be acceptable. The owner should ensure that the strength of the floor including but not limited to the strength of the concrete, the thickness of the floor slab, the method of reinforcement, and the quality of the subgrade is adequate for storage rack loading.

This specification is for the design of storage racks only. Floor slab design is a separate issue not within the scope of this Specification.

The owner shall bring up special bearing plate considerations to the attention of the rack supplier.

8 SPECIAL RACK DESIGN PROVISIONS

8.1 OVERTURNING

A very important aspect of rack design is to provide stability against overturning of the rack structure when the rack is subjected to horizontal forces. Horizontal forces on the rack structure can be due to wind (Section 2.6), earthquake (Section 2.7) or the force described in this section.

The designer is cautioned not to consider the stabilizing forces provided by ordinary anchorage to maintain rack alignment. However, if forces on anchors are analyzed and the anchors designed for these forces with appropriate safety factors, then the anchorage forces may be considered in the stability analysis.

A limit on the height to depth ratio of the rack is imposed. This ratio is defined as the height to the topmost beam divided by the frame width (or the combined width of interconnected frames). While it is recommended that all frames be anchored (Section 1.4.7), here it states that if the 6:1 ratio is exceeded, the rack must be analyzed for overturning even in the absence of seismic and wind forces. A 350 pound lateral force, which could result from moving equipment servicing the rack, is applied at the topmost shelf level for the purpose of designing the anchorage. This short duration load need not be considered in the design of the column.

A further limit on the height to depth ratio is given as 8:1. Stabilizing a single row of rack that exceeds this ratio with floor anchors alone is not generally recommended. Under certain circumstances, this may be feasible but such cases should be thoroughly analyzed and certified by an engineer.

The provisions of this section apply to frames of constant depth over their height. Other configurations such as offset or sloped legs requires more detailed analysis..

8.2 CONNECTIONS TO BUILDINGS

The relative stiffness of racks and buildings vary significantly. Therefore, any attachment between the rack and the building shall be made with provisions for vertical and lateral building movements. Such attachments shall be proportioned so that the attachment would fail prior to causing damage to the building structure. Care should be taken that roof loads are not transferred to the racks.

8.3 INTERACTION WITH BUILDINGS

This section recognizes that building structures and rack structures are likely to have different structural characteristics. During an earthquake, this could have a magnifying effect for structures that are interconnected but which have differing periods of vibration.

Thus, the connections must be designed to ensure that neither structure causes damage to the other during a seismic event.

9 TEST METHODS

9.1 GENERAL

Many factors affecting the design of rack are difficult to account for analytically. Sec 9 spells out a series of optional tests that may be used to evaluate the effects of components on the overall behavior.

Except as either modified or supplemented in this Specification, AISI [4] and AISC [2, 3] shall apply to the testing of components.

The engineers involved in rack design are probably familiar with the test procedures stipulated in the Specification. However, some comments bear reiterating here. The important factor that must 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 be 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 analyze the test results. It is also essential to have a complete report spelling out test procedures, the results and the analysis of the results.

9.2 STUB COLUMN TESTS FOR COLD-FORMED AND HOT-ROLLED COLUMNS.

Because of the interplay of three influences which affect a cold-formed perforated compression member, [i.e., local buckling, perforations, and cold-work of forming] recourse must 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 Part VIII of the AISI Cold-Formed Steel Design Manual [14].

9.2.2 EVALUATION OF TEST RESULTS.

Q is a factor used in Section 4.2.2 and 4.2.3. The column formulas, as well as the test determination of Q, both utilize the yield strength of the material. It is, therefore, essential that the value of F_y used in the column formulas be connected with the yield strength F_y used when determining Q. This is elaborated below.

The basic definition of Q is:

$$Q = \frac{\text{actual strength of stub column}}{\text{hypothetical maximum strength without weakening influences}}$$

In turn, this hypothetical strength in the case of nonperforated sections, is $A_{full}\,F_y$. For shapes Q<1 the AISI Specification permits the cold work in the flats to be utilized, but not that of the corners.

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

$$Q = \frac{\text{ultimate strength of stub column}}{F_y A_{net\ min}}$$

In regard to the yield strength F_y to be used by determining Q by test, and the value F_y for calculating the strength of columns according to AISI Sec. C4 the following needs attention: In calculating column strength according to AISI Sec. C4, F_y is the specified minimum yield strength to which the steel is ordered by the fabricator. On the other hand, the yield strength of the particular coil from which the stub column test specimens will have been made, will be different and in general somewhat larger than the ordered minimum yield point. In order for the determination of Q to be adequately accurate, it is necessary that the virgin yield point of the stub column test material [before forming] be as close as possible to the specified strength; it should not deviate from it by more than -10% to +20%. With this proviso, the Specification in conjunction with the quoted AISI Specification [4] Appendix A5.2.2 allows the determination of F_y in the formula for calculating Q and consistent values of F_y for calculating column strength according to the AISI Specifications Sec C4.

For a series of columns having different thicknesses, the thickest and the thinest may be tested. For any intermediate thickness, the Q so determined should be used in column strength calculations according to the AISI Specification [4]Sec. C4 in conjunction with a value Q obtained by similar interpolation. That is,

$$Q = Q_{min} + \frac{(Q_{max} - Q_{min})(t - t_{min})}{(t_{max} - t_{min})}$$

where Q_{min} is for the stub column with the thickness t_{min}, 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 adequately accurate only if the actual virgin yield strengths of the two stub columns with t_{max} and t_{min} are not too different. For this reason the Specification limits this difference to 25%.

It is acceptable to linearly 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 coil from which stub column was 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.

By the procedures above, it is possible to obtain Q-values larger than 1 [one]. This is so if the neglected strengthening effects of cold-work outweigh the weakening effects of the perforations. However, it is basic to the use of Q in the AISI Specifications that it can only be equal to or smaller than, but not larger than 1.0. Correspondingly, the Specifications provides that if the selected procedure for determining Q results in a Q-value larger than 1.0, Q = 1.0 shall be used.

9.3 PALLET BEAM TESTS.

In this section, depending on the information required, two different types of tests are specified, [i.e. simply-supported pallet beam tests and pallet beam in upright frame assembly.]

The loading in these tests is applied by means of a test machine or jacks. This loading may 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, test results will have to be adjusted to consider the added impact effect.

9.3.1 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 and yield moments, the number of tests needed shall be determined according to the AISI Specification [4].

9.3.1.1 Test Setup.

The test setup illustrated in Fig. 9.3.1-1 shall be used.

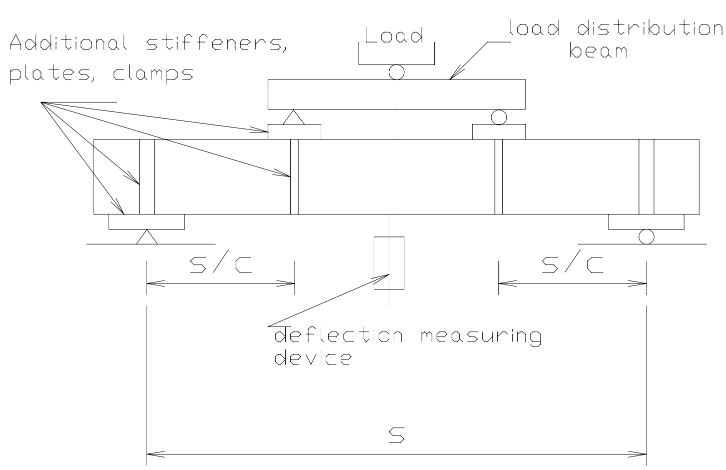


Fig. 9.3.1-1 Simply-Supported Pallet Beam Tests.

The value of C shown in the figure above shall 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 of stiffeners at the supports most likely not be needed.

9.3.1.2 Test Procedure

General guidelines given in Sec. 9.1.3 shall be used in addition to the particular requirements specified herein.

9.3.2 PALLET BEAM IN UPRIGHT FRAMES ASSEMBLY TESTS.

This test is intended to simulate the conditions in the actual rack as close as possible to determine the allowable load.

This test may also be used to determine the magnitude of the joint spring constant F defined in the commentary to Sec. 9.4. For vertical loads this test may reflect the actual behavior of the connections more accurately than the test described in Sec. 9.4.1.

9.3.2.1 Test Setup.

It is specified that the upright frame not be bolted to the floor even if the actual racks are. The test is intended to represent the behavior of the rack between the inflection points. Therefore, any restraint at the column bases other than 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.

9.3.2.3 Evaluation of Test Results.

General guidelines given in Sec. 9.1.3 shall be used in addition to the following three particular requirements or criteria for determining allowable load. The first of these is the determination of the factor of safety or the resistance factor according to Section F of the AISI Specification.

The second criterion by which to determine allowable loads from the test results prescribes a safety factor of 1.5 against excessive load distortion.

The third and last criterion limits deflection of beams under design load to 1/180 of the span. To satisfy this requirement, the load that results in this amount of deflection should be read from the load deflection curve plotted from the test results. If this load is smaller than those obtained from the first two requirements, it governs.

9.4 PALLET BEAM-TO-COLUMN CONNECTION TESTS.

The tests specified in this section 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 below 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 columns and the pallet beam do not in general behave as rigid. 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 behavior very significantly. The connection details vary 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 test.

The change in angle between the column and the connecting beam θ [in radius] can be idealized as follows:

$$\theta = \frac{M}{F}$$

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

9.4.1 THE 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 [that is the vertical reaction] to moment at the joint is not well represented. For typical rack connections this ratio is probably higher than it is in the cantilever test as spelled out in the Specification.

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 according to Sec. 9.3.2.

9.4.1.1 Test Setup.

This test setup illustrated in Fig. 9.4.1.1-1

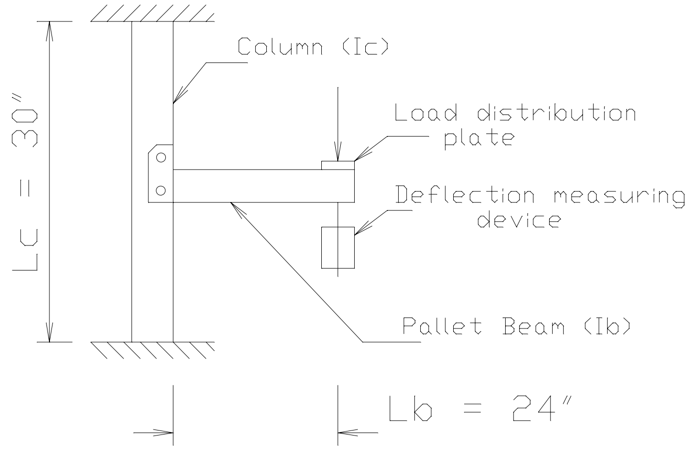


Figure 9.4.1.1-1 Cantilever Test

9.4.1.3 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.

$$F = \frac{(R.F.)}{\frac{\delta_{0.85}}{P_{0.85} L^2{}_b} - \frac{L_c}{16EI_c} - \frac{L_b}{3EI_b}}$$

where $P_{0.85}$ is 0.85 times the ultimate load and $\delta_{0.85}$ is the deflection of the free end of the cantilever at load $P_{0.85}$, L_c, L_b, I_c, I_b are the same lengths and moments of inertias of the columns and the beam, respectively. (R.F.) is a reduction factor to provide safety considering scatter of test results. Since a lower F means a higher design moment for the beam, an (R.F.)=2/3 should be taken in the design of the beam. However, in determining bending moments for the columns a higher F leads to a more conservative value of the bending moment. It is therefore recommended to take (R.F.) = 1.0 for this case.

It is suggested that the spring constant F be calculated on the basis of the average results on two tests of identical specimens provided that the deviation from the average results of two tests 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.

9.4.2 THE PORTAL TEST.

The portal test is desirable when the value of F obtained is to be used in a sidesway analysis either for lateral deflections or stability. Under vertical loads the connections in general “tighten up”. Subsequently, under sidesway, 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 the process. Thus it is more desirable to use the portal test for evaluating sidesway behavior, namely, the effective lengths and horizontal deflections.

9.4.2.1 Test Setup.

A schematic of the test setup is shown in the figure 9.4.2.1. According to the Specification, h=24 in.

Dial gage #1 shall be used to measure the lateral deflection δ of the rack. Dial gages #2 and #3 indicate whether the column bases are properly restrained or not. In lieu of dial gages other deflection measuring devices may be used. In general the friction between concrete and the half round bars is enough for this restraint.

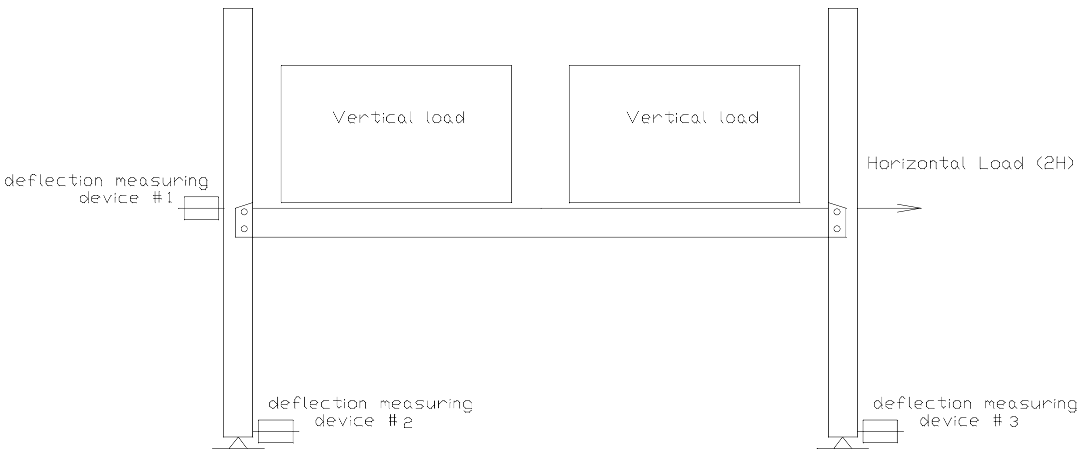


Figure 9.4.2.1-1 Portal test

9.4.2.3 Evaluation of Test Results.

The following is a possible rational analysis for evaluating test results . Considering a portal height h and span L with moments of inertia of the columns and beams designated I_c and I_b respectively, and expression for maximum sidesway deflection δ corresponding to a lateral load of 2H combination as follows:

$$\delta = \frac{Hh^3}{3EI_c} + \frac{Hh^2L}{6EI_b} + \frac{Hh^2}{F}$$

Solving this equation for F, the following is obtained:

$$F = \frac{R.F.}{2\dfrac{\delta}{Hh^2} - \dfrac{h}{3EI_c} - \dfrac{L}{6EI_b}}$$

[R.F.] is a reduction factor that should be taken equal to 2/3.

E = the modulus of elasticity.

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

H = the horizontal load per beam.

I_b = the moment of inertia of the beam about the axis parallel with the floor.

I_c = the moment of inertia of the column about the axis parallel with the upright frame.

L = the distance between the centroid of the two columns parallel with the shelf beam.

δ = Sway deflection corresponding to a lateral load of 2H.

Since the behavior 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 the commentary on Sec. 9.4.1.3 are also recommended here.

9.5 UPRIGHT FRAME TEST.

The hazard of collapse of a full scale high rise rack system poses severe safety problems. Therefore, the testing procedures proposed herein are geared to a reduced scale 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 racks as closely as possible.

9.5.1.1.1 Test Setup for Horizontal Load in the Direction Perpendicular to the Upright Frame.

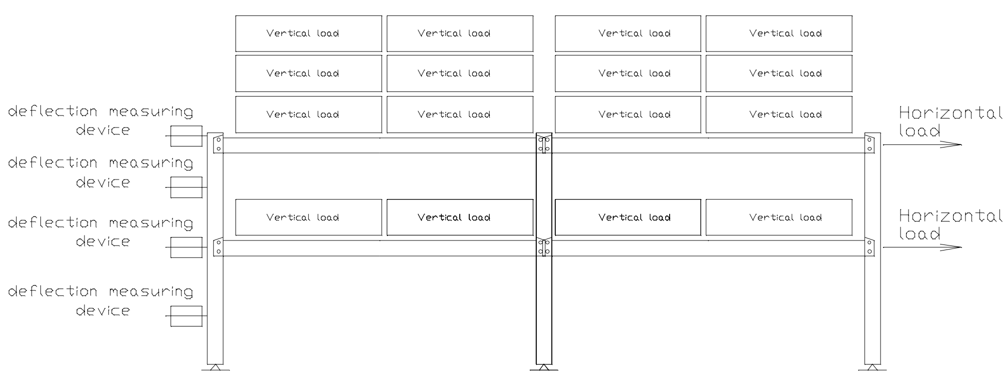


Figure 9.5.1.1.1 Test Setup

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