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

Rack Manufacturers Institute

RACK MANUFACTURERS INSTITUTE

Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks - 2002 Edition

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Testing and Utilization of
Industrial Steel Storage Racks

MATERIAL HANDLING INDUSTRY OF AMERICA®

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, reaffirmed to include accumulated editorial clarifications as the 2002 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 - 2002 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

2002 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 rack columns 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 (278.7 m^2) (not including aisles), and having a unit load not exceeding 2,500 pounds (1134 kg) (no multiple stacking on top shelf), the provisions given in 1.4.2 through 1.4.5 may be waived.

1.4.9 Rack Damage

Preventing damage to rack is beyond the scope of this specification. See commentary for a broader discussion of this topic.

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 (1.25 cm in 3.0 m 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{xe "DL"} = Dead Load
LL{xe "LL"} =Live Load other than the pallets or products stored on the racks.
(Example, floor loading from work platforms)
SL{xe "SL"} =Snow Load
RL{xe "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 (488 kg/m^2) 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:

$\phi = 0.400$

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, $I_p = 1.5$.

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

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

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:

$$C_s = \frac{1.2 C_v}{R T^{2/3}}$$

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 (2.44 m) 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, $\bar{v}_s > 5{,}000\text{ft} / \text{sec}$ (1,500 m/s)

B Rock with measured shear wave velocity, $2{,}500\text{ft} / \text{sec} < \bar{v}_s \leq 5{,}000\text{ft} / \text{sec}$ ($760\text{ m/s} < \bar{v}_s \leq 1500\text{m/s}$)

C Very dense soil and soft rock with measured shear wave velocity, $1{,}200\text{ft/sec} < \bar{v}_s \leq 2{,}500\text{ft/sec}$ $(360\text{m/s} < \bar{v}_s \leq 760\text{m/s})$ or with either $\bar{N} > 50$ or $\bar{s}_u \geq 2{,}000\text{psf}$ (100 kPa)

D Stiff soil with measured shear wave velocity, $600\text{ft/sec} < \bar{v}_s \leq 1{,}200\text{ft/sec}$ $(180\text{m/s} < \bar{v}_s \leq 360\text{m/s})$ or with either $15 \leq \bar{N} \leq 50$ or $1{,}000\text{psf} \leq \bar{s}_u \leq 2{,}000\text{psf}$ $(50\text{ kPa} \leq \bar{s}_u \leq 100\text{kPa})$

E A soil profile with $\bar{v}_s < 600\text{ft/sec}$ (180 m/s) or any profile with more than 10 ft (3048 mm) of soft clay defined as soil with $PI > 20$, $w \geq 40\ \%$, and $s_u < 500$ psf (25 kPa)

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 (3048 mm) of peat and/or highly organic clay where H = thickness of soil ~~)~~]

3. Very high plasticity clays ~~(~~[H > 25 ft (7620 mm) with PI > 75~~)~~]

4. Very thick soft/medium stiff clays ~~(~~[H > 120 ft (36580 mm~~)~~)]

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.

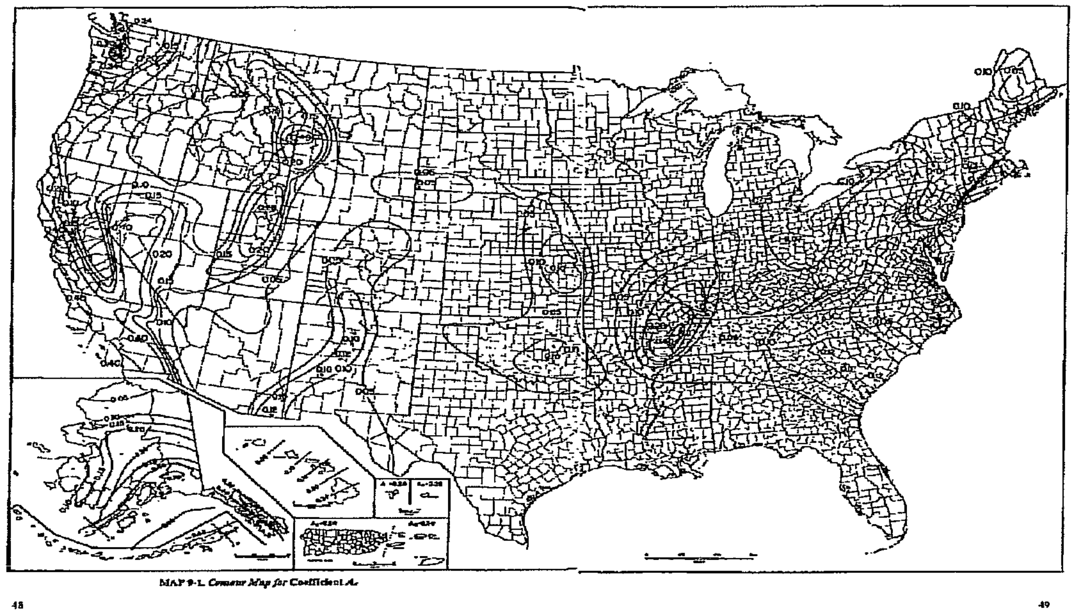

Figure 2.7.1 Contour Map for Coefficient A_a

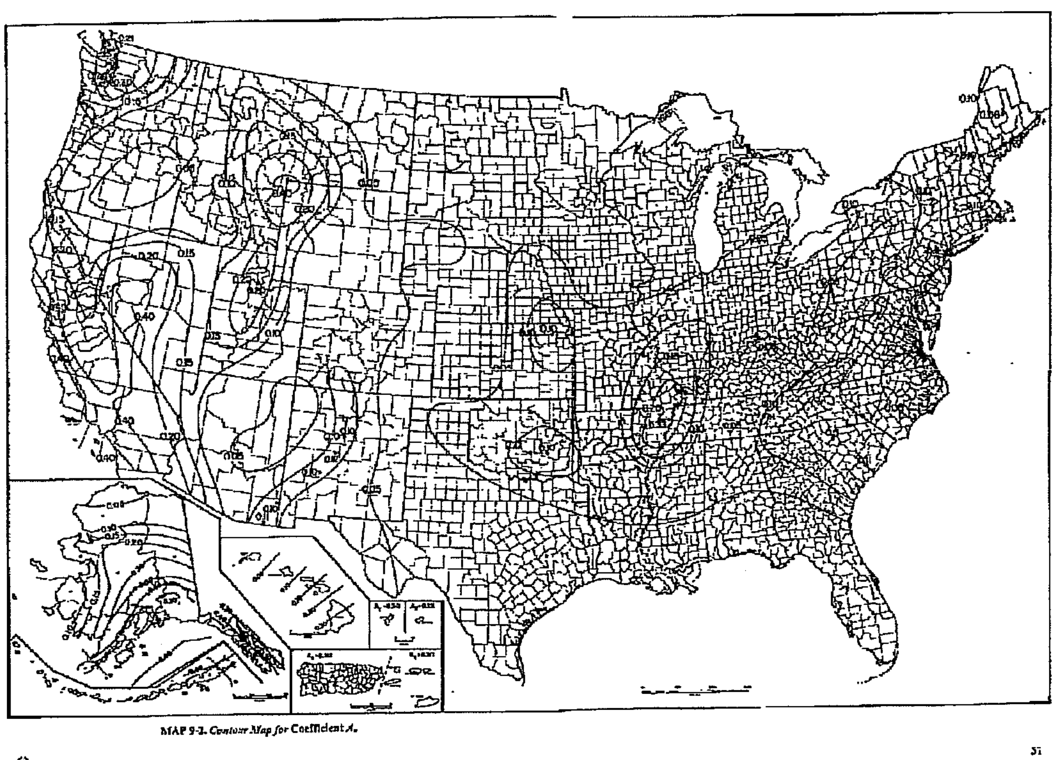

Figure 2.7.2 Contour Map for Coefficient A_v

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" (30.5 cm) 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" (30.5 cm):

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

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 $\leq$ 0.5 k = 1
period $\geq$ 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[2] 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.

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[2] The Commentary and the illustrative examples for this specification contain suggestions for a rational analysis in accordance with conventional methods of structural design in many of the cases where rational analysis is either permitted or required.

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

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

2. For upright frames braced with diagonals

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

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

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

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~~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 (453.6 kg) 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 (453.6 kg) 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 $\phi c P p$ (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 (2.1 x 10^6 kg/m^2).

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, which is loaded and unloaded by powered handling equipment, that exceed the 6 to 1 ratio defined above, shall also be designed to resist a 350 pound (159 kg) 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 (159 kg) 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 (0.76 mm).

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 [0.00254 mm]) 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 (61 cm) 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 (0.0254 mm) 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 (66 mm) in length connected to the center of a column at least 30 inches (76 cm) 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 (61 cm) 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 (61 cm) 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.