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

RACK MANUFACTURERS INSTITUTE

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American National Standard

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Approved April 21, 2008
American National Standards Institute, Inc.

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Foreword (This foreword is not part of American National Standard MH16.1:2008)

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 steel storage racks. This specification is the result of RMI's recognition of the need to develop a comprehensive safety specification and establish minimum design and performance criteria to ensure the safe application and utilization of racking, and was formulated under American National Standards Institute (ANSI) procedures.

At the date of approval of this specification, RMI consisted of the following member companies:

Adrian Fabricators, Inc./Cargotainer
Advance Storage Products, Div. of J.C.M. Industries Inc.
Atlas Material Handling, Inc.
AWP Industries, Inc./American Wire Products
Base Manufacturing
BITO Lagertechnik Bittmann GmbH
Bulldog Rack Company
Engineered Products
Excel Storage Products, LP
Frazier Industrial
Hannibal Material Handling, Inc.
ITC
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SpaceRak, Division of Tarpon
Speedrack Products Group, Ltd.
Steel King Industries, Inc.
Unarco Material Handling, Inc.
United Fixtures Interlake
Wireway Husky Corporation

RMI maintains a public website at www.MHIA.org/RMI that has information about storage racks and the RMI members including ordering information for literature and a section for frequently asked questions. All inquiries concerning the Specification should be directed in writing to the RMI Engineering Committee, 8720 Red Oak Boulevard, Suite 201, Charlotte, NC 28217

MATERIAL HANDLING INDUSTRY

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

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. 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.

Since 1964, mechanized storage systems have grown very rapidly both in size and height with new and modified types of racks having been developed. To reflect this rapid development and to assure adequate safety and performance of modern rack structures, 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 American National Standard 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 continue testing rack components plus perform full scale tests on typical storage rack structures. 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. Design parameters relating to drive-in and drive-through racks have been removed from the Specification until drive-in and drive-through rack test results could be analyzed more thoroughly; perhaps more testing would be required. Movable-shelf racks were added to the Specification.

As a result of additional testing and analytical research, the RMI revised the 1972 Specification. The ANSI MH 16.1: 1974 was withdrawn in deference to the 1979 Specification. More additions and revisions prompted the RMI to publish the 1985 Specification.

Subsequent testing and research by Dr. Pekoz was the basis of the changes resulting in the 1990 Specification.

From 1990 to 1997, due to continuing changes, specifically as they relate to seismic analysis and other model building code issues, the Specification Advisory Committee, the Seismology Committee and the RMI Engineering Committee working again with Dr. Pekoz and several highly regarded members of the code community and various other members of similar groups throughout the world, conducted extensive testing and parametric analysis. Findings resulted in the 1997 Specification.

In addition to the state-of-the-art benefit from the ongoing testing and analysis, the 1997 Specification was expanded to include complete treatment of seismic design considerations so that the Specification could be more easily incorporated by reference into various model building and design codes.

In 1999, the Membership of RMI acted to create a Voluntary Certification Program known as the R-MARK. The R-Mark is a license earned by a manufacturer following a rigorous review by Independent Professional Engineers of tests and load capacity calculations performed by the manufacturer consistent with the RMI/ANSI Specification.

Continued testing and parametric studies resulted in the 2002 Specification. In 2004 the 2002 RMI Specification and Commentary were adopted as American National Standard ANSI MH 16.1-2004

SPECIFICATION - 2008 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 that do not comply with this Specification. RMI has no legal authority to require or enforce compliance with the Specification. This advisory Specification provides technical guidelines to the user for his specific application. Following the Specification does not assure compliance with applicable federal, state, or local regulations and codes. This Specification is not binding on any person and does not have the effect of law.

The RMI and the Material Handling Industry do not take any position regarding any patent rights or copyrights which could be asserted with regard to this Specification and does not undertake to insure anyone using this Specification against liability, nor assume any such liability. Users of this Specification are expressly advised that determination of the validity of any such copyrights, patent rights, and risk of infringement of such rights is entirely their own responsibility.

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

Users of the Specification must rely on competent advice to specify, test and/or design the storage rack system for their particular application. 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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SYMBOLS

a	Vertical distance between the horizontal brace axes
A	Sum of the minimum net area (Anet min.) of the columns of the upright frame
$\mathbf{A_b}$	Cross-sectional area of a horizontal brace
$\mathbf{A_d}$	Cross-sectional area of a diagonal brace
$\mathbf{A_e}$	Effective area at the stress F_n
$\mathbf{A_{net\ min}}$	Minimum cross-sectional area obtained by passing a plane through the column normal to the axis of the column
b	Horizontal distance between neutral axes of the columns
$\mathbf{C_s}$	Seismic response coefficient
DL	Dead Load
E	Modulus of elasticity of steel
EL	Earthquake (seismic) Load
$\mathbf{f'_c}$	Minimum 28-day compression strength of the concrete
$\mathbf{F_1}$	Lateral force at the first shelf level
$\mathbf{F_a}$	Site coefficient defined in Table 2.6.3.2 (2). If site class is unknown, use site class D
$\mathbf{F_c}$	Critical buckling stress
$\mathbf{F_n}$	Nominal buckling stress
$\mathbf{F_v}$	Site coefficient defined in Table 2.6.3.2 (3). If site class is unknown, use site class D
$\mathbf{F_x}$	Lateral force at any level
$\mathbf{F_y}$	Yield point used for design
$\mathbf{h_i}$ **or** $\mathbf{h_x}$	Height from the base to level i or x
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
$\mathbf{I_{br}}$	Moment of inertia of the horizontal brace about its own axis perpendicular to the plane of the upright frame
$\mathbf{I_c}$	Minimum net moment of inertia of one column about its own major axis perpendicular to the plane of the upright frame
IL	Impact loading on a shelf
$\mathbf{I_p}$	System importance factor that varies from 1.00 to 1.50
k	Upright stability coefficient based on location of the center of load
l	Total height of the upright frame
LL	Live Load other than the pallets or products stored on the racks

Lr	Roof Live Load
L_{short}, L_{long}	Distance between column brace points
L_x, L_y, L_t	Unbraced lengths for column design, for bending about x- and y-axes and torsion
PL	Maximum Load from pallets or products stored on the racks
PL_{app}	Portion of pallet or product load that is used to compute the seismic base shear
$PL_{Average}$	Maximum total weight of product expected on the beam levels in any row divided by the number of beam levels in that row
$PL_{Maximum}$	Maximum weight of product that will be placed on any one beam level in that row
PL_{RF}	Product Load reduction factor ($PL_{Average}$ / $PL_{Maximum}$)
P_n	Nominal axial strength
Q	Capacity reduction factor for compressive members
R	Seismic response modification factor (Section 2.6.3)
RL	Load from rain including ponding
S_1	Mapped spectral accelerations for a 1-second period as determined per USGS
S_c	Elastic section modulus of the net section for the extreme compression fiber times $1-0.5(1-Q)(Fc/Fy)^Q$
S_{D1}	Design spectral response acceleration parameter for 1-second period (2/3) S_{M1}
S_{DS}	Design spectral response acceleration parameter for short period (2/3) S_M
S_e	Elastic section modulus of the net section for the extreme compression fiber times (0.5+Q/2)
S_f	Elastic section modulus of the full unreduced gross section for the extreme compression fiber
SL	Snow Load
S_{M1}	Maximum considered earthquake spectral response accelerations for 1-second period
S_{MS}	Maximum considered earthquake spectral response accelerations for short period
S_s	Mapped spectral accelerations for short periods as determined per USGS
T	Fundamental period of the rack structure in each direction under consideration
V	Seismic base shear
w_i or w_x	Portion of the total gravity load of the rack, located or assigned to the bottom shelf level, level i or x
WL	Wind Load

$\mathbf{W_s}$	Loads on the structure that are used to compute the horizontal base shear. $\left(0.67xPL_{RF}xPL\right)+DL+0.25xLL$
α	Second-order load amplification factor used in the column check
α_s	Second-order amplification factor from FEMA 460 calculated using Ws as the vertical load.
$\boldsymbol{\theta_{Max}}$	Maximum rotation sustained by the beam to column connection over at least 2 cycles during testing
$\boldsymbol{\theta_D}$	Rotational seismic demand of the beam to column connection
σ_{ex}, σ_{ey}, σ_t	Compressive stresses calculated per AISI
ϕ	Angle between horizontal and diagonal braces
ϕ_c	Resistance factor for concentrically loaded compression member
$\phi_c P_n$	Design strength
$\boldsymbol{\Omega}$	Factor of safety for ASD

NOMENCLATURE

Note: Terms designated with † are common with AISI-AISC terms that are coordinated between the standards developers.

Automated Storage and Retrieval Systems (ASRS) - A rack structure in which loading and unloading of the racks is accomplished by a stacker crane, or similar vehicle, without the aid of an on-board operator.

Allowable strength† - Nominal strength divided by the safety factor

Allowable stress. - Allowable strength divided by the appropriate section property, such as section modulus or cross-section area.

Applicable code† - Code (enforced by the local building department)under which the structure is designed.

ASD (Allowable Strength Design)† - Method of proportioning structural components such that the allowable strength equals or exceeds the required strength of the component under the action of the ASD load combinations.

ASD load combination† - Load combination in the applicable building code intended for allowable strength design (allowable stress design).

Beam – Typically, a horizontal structural member that has the primary function of resisting bending moments.

Beam locking device - A pin, bolt, or other mechanism that resists disengagement of the beam connector from the column.

Braced frame† - An essentially vertical truss system that provides resistance to lateral forces and provides stability for the structural system.

Buckling - Limit state of sudden change in the geometry of a structure or any of its elements under a critical loading condition.

Buckling strength - Nominal strength for buckling or instability limit states.

Cantilever rack - A rack structure comprised primarily of vertical columns, extended bases, horizontal arms projecting from the face of the columns, and down-aisle bracing between columns. There can be shelf beams between arms depending on the product being stored. Cantilever columns may be free-standing or overhead tied.

Cantilever test - A test designed and conducted to determine the connection moment-resisting capacity and the rotational rigidity, F, of a beam-to-column connection. The test set-up employs one column segment and one beam segment connected to one another with a beam-to-column connector, with a load applied downwardly in the plane of the frame at the cantilever end of the beam segment.

Case flow rack - A specialized pallet rack structure in which either the horizontal shelf beams support case-flow lanes or case-flow shelf assemblies are supported by the upright frames. The case-flow lanes or shelves are installed at a slight pitch permitting multiple-depth case or box storage with loading from one service aisle and unloading or picking from another service aisle.

Cladding - Exterior covering of structure.

Cold-formed steel structural member† - Shape manufactured by press-braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold- or hot-rolled coils or sheets; both forming operations being performed at ambient room temperature; that is, without manifest addition of heat such as would be required for hot forming.

Column - Structural member that has the primary function of resisting axial force.

Concrete crushing - Limit state of compressive failure in concrete having reached the ultimate strain.

Concurrent forces - Two or more forces acting in conjunction with one another at a single location.

Connection† - Combination of structural elements and joints used to transmit forces between two or more members.

Cyclic tests - A test designed and conducted to determine the connection moment-resisting capacity and rotational rigidity, along with energy-dissipation properties, of beam-to-column connections when those connections are subjected to cyclic loading conditions. The test set-up employs one column segment and two beam segments connected to one another, using two beam-to-column connectors, as a double cantilever. Two parallel loads are applied, in opposing reversing cyclic fashion, in the plane of the frame at the ends of, and normal to, the cantilevered beam elements.

Design load† - Applied load determined in accordance with either LRFD load combinations or ASD load combinations, whichever is applicable.

Design strength† - Resistance factor multiplied by the nominal strength, [F R_n].

Design stress - Design strength divided by the appropriate section property, such as section modulus or cross section area.

Diagonal bracing - Inclined structural member carrying primarily axial force in a braced frame.

Distortional buckling. - A mode of buckling involving change in cross-sectional shape, excluding local buckling.

Double-stacking - When a shelf is loaded with loads stacked one on top of another in a pallet position.

Drive-in rack - A rack structure comprised primarily of vertical upright frames, horizontal support arms, and horizontal load rails typically used for one-wide by multiple-depth storage. This structure includes an 'anchor section' with horizontal beams supporting the load rails. Loading and unloading within a bay must be done from the same aisle. A two-way drive-in rack is a special case where back-to-back rows of drive-in racks are combined into a single entity with a common rear post.

Drive-through rack - A rack structure comprised primarily of vertical upright frames, horizontal support arms, and horizontal load rails typically used for one-wide by multiple-depth storage. This structure lacks the 'anchor section' found in drive-in racks; therefore, loading and unloading can be accomplished from both ends of a bay.

Effective length - Length of an otherwise identical column with the same strength when analyzed with pinned end conditions.

Effective length factor - Ratio between the effective length and the unbraced length of the member.

Effective section modulus - Section modulus reduced to account for buckling of slender compression elements.

Effective width - Reduced width of a plate or slab with an assumed uniform stress distribution which produces the same effect on the behavior of a structural member as the actual plate or slab width with its nonuniform stress distribution.

Factored load† - Product of a load factor and the nominal load.

Flexural buckling - Buckling mode in which a compression member deflects laterally without twist or change in cross-sectional shape.

Flexural-torsional buckling† - Buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

Force - Resultant of distribution of stress over a prescribed area.

Gravity load - Load such as that produced by dead and live loads, acting in the downward direction.

Kick-plate - A vertical plate (angle or barrier) that is installed at the edge of an elevated floor that is intended to prevent loose items from sliding off the edge of the floor. (Section 8.4.3.3).

Load factor† - Factor that accounts for deviations of the nominal load from the actual load, for uncertainties in the analysis that transforms the load into a load effect and for the probability that more than one extreme load will occur simultaneously.

Local buckling - Limit state of buckling of a compression element within a cross section.

LRFD (Load and Resistance Factor Design)† - Method of proportioning structural components such that the design strength equals or exceeds the required strength of the component under the action of the LRFD load combinations.

LRFD load combination† - Load combination in the applicable building code intended for strength design (load and resistance factor design).

Movable-shelf rack - A rack structure comprised primarily of vertical upright frames and horizontal shelf beams and typically used for one-deep pallet or hand-stack storage. Typically, the locations of a couple of shelf levels are 'fixed' with the location of the in-fill shelves being flexible.

Net area - Gross area reduced to account for removed material.

Nominal strength† - Strength of a structure or component (without the resistance factor or safety factor applied) to resist load effects, as determined in accordance with this Specification.

Out-of-plumb ratio - Maximum horizontal distance (in.) from the centerline of the column at the floor to a plumb line that extends downward from the centerline of the column at the top shelf elevation divided by the vertical distance (ft.) from the floor to the top shelf elevation.

Out-of-straight ratio – Maximum horizontal distance (in.) from the centerline at any point on the column to a plumb line from any other point on the column divided by the vertical distance (ft.) between the two points.

Overturning moment - An applied force that causes a structure to turn over.

Pallet beam - The front and back shelf members that bear the weight of the load and transfer the load to the upright frames.

Pallet flow rack - A specialized pallet rack structure in which the horizontal shelf beams support pallet-flow lanes. The pallet-flow lanes are typically installed on a slight pitch permitting multiple-depth pallet storage with loading from one service aisle and unloading from another service aisle.

Pallet load support member - Any load bearing member with the long axis on the horizontal plane and intended for use as support of unit loads in direct contact. (pallet and shelf supports and beams, not bracing).

Pallet rack - A rack structure comprised primarily of vertical upright frames and horizontal shelf beams and typically used for one and two-deep pallet storage.

Pick modules - A rack structure comprised primarily of vertical frames and horizontal beams typically having one or more platform levels of selective, case-flow, or pallet-flow bays feeding into a central pick aisle(s) [work platform(s)] supported by the rack structure.

Plaque – Signage permanently and prominently displayed depicting the permissible loading of the rack.

Portable rack (stacking frames) - An assembly, typically with four corner columns, that permits stacking of one assembly on top of another without applying any additional load to the product being stored on each assembly.

Portal test - A test designed and conducted to determine the connection moment-resisting capacity and the rotational rigidity, F, of a beam-to-column connection. The test set-up employs two column segments and one beam segment connected to one another using two beam-to-column connectors forming a portal frame, with the load applied laterally in the plane of, and to the corner of, the portal frame in the direction parallel to the beam segment.

Product load - The weight of the item(s) placed on the rack.

Push-back rack - A specialized pallet rack structure in which the horizontal shelf beams support push-back lanes comprised of tracks and carts. The push-back lanes are installed on a slight pitch permitting multiple-depth pallet storage. Loading and unloading are done from the same service aisle by pushing the pallets back.

Rack supported platforms - A decked working surface supported by a rack structure.

Rack supported structure - A rack structure similar to other rack structures; however, this structure also includes wall girts and roof purlins or equivalent components used to support wall and roof cladding. This structure is designed to carry, wind, snow, and rain loads in addition to the normal storage rack loads.

Resistance factor† - Factor that accounts for unavoidable deviations of the nominal strength from the actual strength and for the manner and consequences of failure.

Safety factor† - Factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual load from the nominal load, uncertainties in the analysis that transforms the load into a load effect, and for the manner and consequences of failure.
The nominal load divided by the safety factor results in the allowable load for an Allowable Strength Design.

Safety flooring - A surface that is provided in areas where order picking personnel may need to step off the normal walking area or pick module walkway to dislodge loads that may not have properly flowed to their correct position.

Seismic response modification coefficient - Factor that reduces seismic load effects to strength level.

Sidesway buckling - Limit state of lateral buckling of the tension flange opposite the location of a concentrated compression force.

Simple lip – Single plate elements used to stiffen a compression flange.

Site class definition - A classification assigned to a location based on the types of soils present.

Stability - Condition reached in the loading of a structural component, frame or structure in which a slight disturbance in the loads or geometry does not produce large displacements.

Stacking rack – See Portable rack.

Stacker rack - A rack structure similar to one of the other rack structures; that is serviced by an automated storage and retrieval machine.

Stiffness - Resistance to deformation of a member or structure, measured by the ratio of the applied force (or moment) to the corresponding displacement (or rotation).

Stress - Force per unit area caused by axial force, moment, shear or torsion.

Structural system - An assemblage of load-carrying components that are joined together to provide interaction or interdependence.

Stub column test – Concentric compression testing of members not affected by column buckling used to determine the column effectiveness.

Torsional buckling - Buckling mode in which a compression member twists about its shear center axis.

Torsional-flexural buckling. - Buckling mode in which compression members bend and twist simultaneously without change in cross section shape.

Trussed-braced upright frame – Upright frames having two columns similar to the chords of a truss and diagonal and horizontal bracing attached to and located between the columns. The diagonals and horizontals become the web members of the truss. (It is referred to as a vertical truss.).

Unbraced length - Distance between braced points of a member, measured between the centers of gravity of the bracing members.

Unit load - The total weight expected to be positioned in the rack consisting of the product load and pallet weight.

Upright frame – The main members that carry the vertical and horizontal loads to the floor. They are usually made up of two columns and bracing members between the columns. The beams of the rack are attached to the columns of the frames and carry the loads to the columns.

Vertical impact load - Additional downward force added to the beams produced during loading of the rack.

Yield point† - First stress in a material at which an increase in strain occurs without an increase in stress as defined by ASTM.

Yield strength† - Stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain as defined by ASTM.

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

1. GENERAL

1.1 SCOPE

This Specification and companion Commentary (hereinafter referred to as the Specification) applies to industrial pallet racks, movable shelf racks, rack supported systems and stacker racks made of cold-formed or hot-rolled steel structural members. Such rack types also include push back rack, pallet flow rack, case flow rack pick modules and rack supported platforms. This Specification is intended to be applied to the design of the storage rack portion of any rack structure that acts as support for the exterior walls and roof, except as noted. It does not apply to other types of racks, such as drive-in or drive-through racks, cantilever racks, portable racks, or to racks made of material other than steel.

1.2 MATERIALS

This Specification 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) North American Specification for the Design of Cold-Formed Steel Structural Members [1][1], and the American Institute of Steel Construction (AISC) Specification for Structural Steel 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 Specification, the AISI (2001) [1] and the AISC (2005) [2], as respectively applicable, are used in the determination of the available strength 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.

[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.

(7) Ensure that the racks are not modified or rearranged in a manner not within the original design configurations per 1.4.5.

1.4.2 Plaque

The owner is responsible for displaying in one or more conspicuous locations a permanent plaque(s) Each plaque shall have an area of not less than 50 square inches. Plaques shall show in clear, legible print (a) the maximum permissible unit load and/or maximum uniformly distributed load per level, (b) the average unit load ($PL_{Average}$, see Section 2.6.2) if applicable and (c) maximum total load per bay. The unit load is usually a single pallet or container and its contents mechanically transported. Storage levels having multiple stacking of unit loads shall be so identified. It is the responsibility of the owner to ensure 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 Specification shall be so identified by a plaque having the same characteristics as specified in Section 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 rack manufacturer representative for use by an inspecting body.

1.4.5 Multiple Configurations

If a pallet rack or stacker rack system is permitted in more than one shelf configuration or profile, the drawings (Section 1.4.4) are to include either (a) all the permissible configurations or (b) 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 is best furnished in table form on the drawings. A notice is to be included in conspicuous text on the drawings stating 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 (Section 1.4.4). For specific movable-shelf rack installations in which the overall rack height it is a controlling element, a conspicuous warning is to be placed in the owners' utilization instruction manual stating any restrictions to shelf placement or shelf removal. Such restrictions also are to be permanently posted in locations clearly visible to forklift operators.

1.4.7 Column Base Plates and Anchors

The bottom of all columns shall be furnished with column base plates, as specified in Section 7.2. All rack columns shall be anchored to the floor with anchor bolts 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 (3.65 m) 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) and having no multiple stacking on top shelf, the provisions given in Sections 1.4.4 and 1.4.5 may be waived.

1.4.9 Rack Damage

Preventing damage to rack is beyond the scope of this specification. See the 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 until the damaged portion is repaired or replaced.

1.4.10 Racks Connected to the Building Structure

If the racks are connected to the building structure, then the location and magnitude of the maximum possible horizontal and vertical forces (per Sections 2.1 and 2.2 of this Specification) that are imposed by the rack to the building are to be given to the owner of the building for his review.

1.4.11 Out-of-Plumb and Out-of-Straight Limits

1.4.11.1 Out-of-Plumb Limit

The maximum top to bottom out-of-plumb ratio for a loaded rack column is 1/240 (for example 1/2” per 10 feet (12.5 mm per 3 m) of height). Columns whose out-of-plumb ratio exceeds this limit should be unloaded and re-plumbed. Any damaged parts must be repaired or replaced.

Top to bottom out-of-plumb ratio – maximum horizontal distance (in.) from the centerline of the column at the floor to a plumb line that extends downward from the centerline of the column at the top shelf elevation divided by the vertical distance (ft.) from the floor to the top shelf elevation.

1.4.11.2 Out-of-Straight Limit

The maximum out-of-straight ratio for a loaded rack column is 1/240 (0.05” per foot or 1/2” per 10 feet (12.5 mm per 3 m) of height). Columns whose out-of-straight ratio exceeds this limit should be unloaded and re-plumbed. Any damaged parts must be repaired or replaced.

Out-of-straight ratio – maximum horizontal distance (in.) from the centerline at any point on the column to a plumb line from any other point on the column divided by the vertical distance (ft.) between the two points.

2. LOADING

Rack structures shall be designed using the provisions for Load and Resistance Factor Design (LRFD), or the provisions for Allowable Strength Design (ASD). Both methods are equally acceptable although they may not produce identical designs.

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 ASCE 7 [5] as modified below for racks.

For all rack members	Critical Limit State
1. DL	Dead Load Critical
2. DL + PL + LL + (Lr or SL or RL)	Gravity Load Critical
3. $0.6DL + 0.6PL_{app} - WL$	Wind Uplift Critical
$(0.6 - 0.11S_{ds})DL + (0.6 - 0.14S_{ds})PL_{app} - EL$	Seismic Uplift Critical
4. DL + PL + LL + (Lr or SL or RL) + WL	Gravity Plus Wind/Seismic Critical
$(1 + 0.11S_{DS})DL + (1 + 0.14S_{DS})PL + LL + (Lr$ or SL or RL) +EL	Gravity – Seismic Critical

For load support beams and their connections only:

5. DL + LL+ 0.5(SL or RL) + 0.88PL+ IL	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 rack supported platforms)
Lr = Roof Live Load
SL = Snow Load
RL = Load from rain including ponding
WL = Wind Load
EL = Seismic Load
IL = Impact loading on a shelf (Section 2.3)
PL = Maximum Load from pallets or products stored on the racks.
PL_{app} = When checking for seismic uplift, the 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 PL_{app}. See Commentary.
When checking for uplift due to wind, PL_{app} is equal to the minimum weight of the loads that must be present to develop the calculated lateral wind force. See Commentary

All loads in Cases 3 and 4 except the Dead Load may be multiplied by 0.75. In addition to the 0.75 multiplier, for Cases 3 and 4 the seismic force (EL) determined in accordance with Section 2.6 or another limit-states based code may be multiplied by 0.67 because the limit states based codes give higher seismic forces.

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 ASCE 7 [5] except as modified below for racks:

For all rack members: Critical Limit State

1.	$1.4DL + 1.2PL$	Dead load
2.	$1.2DL + 1.4PL + 1.6LL + 0.5(Lr \text{ or } SL \text{ or } RL)$	Live/Product load
3.	$1.2DL + 0.85PL + (0.5LL \text{ or } 0.8WL) + 1.6(Lr \text{ or } SL \text{ or } RL)$	Snow/Rain
4.	$1.2DL + 0.85PL + 0.5LL + 1.6WL + 0.5(Lr \text{ or } SL \text{ or } RL)$	Wind load
5.	$(1.2 + 0.2S_{DS})DL + (0.85 + 0.2S_{DS})PL + 0.5LL + 1.5EL + 0.2SL$	Seismic load
6.	$0.9DL + 0.9PL_{app} – 1.6WL$	Wind Uplift
	$(0.9 – 0.2S_{DS})DL+ (0.9 – 0.2S_{DS})PL_{app} – 1.5EL$	Seismic Uplift

For load support beams and their connections only:

7.	$1.2DL + 1.6LL+ 0.5(SL \text{ or } RL) + 1.4PL + 1.4 * IL$	Product/Live/Impact (for shelves and connections)

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

Note: The load factor for EL in load cases 5 and 6 must be 1.5 unless the seismic loading is determined in accordance with Section 2.6 or another limit-states based code. If Section 2.6 or a Limit States based code is used to determine the seismic forces the load factor for EL may be 1.0 for load cases 5 and 6.

For load case 6a (wind uplift), only pallet loads that must be present to develop the lateral wind forces shall be considered in PL_{app}. PL_{app} will be zero for an unloaded rack that supports exterior cladding.

All resistance factors are to be as stated in the AISI (2001) [1] or AISC (2005) [2]. 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 =0.40

2.3 VERTICAL IMPACT LOADS

Load-supporting beams and arms and connector components used to attach them to the columns are to be designed for an additional vertical impact load equal to 25 per cent of one unit load. This impact load is to be placed in the most unfavorable position when determining maximum load on each component. For beams or arms whose design capacity is determined by testing (Section 9.3), due allowance must be made for the additional impact load. This impact load need not be applied when checking beam deflections (Sections 5.3 and 9.3) or designing upright frames, columns, and other vertical components.

2.4 HORIZONTAL FORCES

2.4.1 Beam-to-column connections, frame bracing members, 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-to-column connections and frame bracing members and frame bracing connections must be designed for the most critical of:

1. Earthquake Loads (Section 2.6).

2. Wind Forces (Section 2.5)

3. For Allowable Strength Design: 1.5%DL plus 1.5%PL at all connections based on maximum loading.

   For Load and Resistance Factor Design: 1.5% factored DL plus 1.5% factored PL based on the maximum loading.

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

   The horizontal forces are to be applied simultaneously with the full vertical live load, product load and dead load. Bending loads at the beam-to-column connection 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).

2.4.2 Stacker racks or racks fully or partially supporting moving equipment shall meet the requirements of Sections 2.4.2.1, 2.4.2.2 and 2.6.

2.4.2.1 The moving equipment manufacturer is responsible for supplying to the rack manufacturer the magnitude, location, and direction of all loads (static and dynamic) transmitted from the moving equipment to the rack structure.

2.4.2.2 Forces described in Section 2.4.2.1 need not be applied concurrently with the loads described in Sections 2.5 and 2.6.

2.5 WIND LOADS

Wind forces shall be determined in accordance with ASCE 7 [5].

Racks directly exposed to the wind shall be designed for the wind loads acting both on the rack structure and 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 Section 2.4.1, except for that portion of horizontal loading resulting from an out-of-plumb installation, and Section 2.6 need not be assumed to act concurrently with wind loads. The forces described in Section 2.4.2 shall not be assumed to act concurrently with wind forces.

2.6 EARTHQUAKE LOADS

2.6.1 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 that are more than 8 ft (2.44 m) in height to the top load shelf and 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 between the storage rack and the building or other structures to avoid damaging contact during an earthquake.

Unless used to store hazardous material, storage racks are to be deemed Occupancy Category II structures.

2.6.2 Minimum Seismic Forces

The storage rack shall be designed for the total minimum lateral force as determined using the following considerations or, alternatively the seismic design evaluation may be performed using a displacement-based method, such as the method described in Section 6.5.1 of FEMA 460 [4].

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 2.6.3.

I_p = system importance factor:
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;

For storage rack in areas open to the public, (e.g., in warehouse retail stores), I_p = 1.5. If a displacement based evaluation of the rack structure is performed in either of the two principle directions of the rack, I_p may be taken as 1.0 in that direction.

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

where:

PL_{RF} = Product Load Reduction Factor

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

$PL_{Average}$ For warehouse retail stores, open to the general public, $PL_{Average}$ shall be taken as $PL_{Maximum}$.

For all other types of warehousing $PL_{Average}$ is the maximum total weight of product expected on all the beam levels in any row divided by the number of beam levels in that row.

$PL_{Maximum}$ Maximum weight of product that will be placed on any one beam level 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 nonbuilding structures supported by other structures, including the force and displacement effects caused by amplifications of upper-story motions.

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

2.6.3 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:

$$C_s = \frac{S_{D1}}{TR}$$

where:

S_{D1} = Design earthquake spectral response acceleration at a 1 second period, as described in Section 2.6.3.1.

R = Response modification factor 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 = Fundamental period of the rack structure in each direction under consideration established using the structural properties and deformation characteristics of the resisting elements in a properly substantiated analysis. For the unbraced direction (moment frame), the period shall be determined using a connection stiffness, F not less than the value from Section 9.4.2.3 or Section 9.6.

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

$$C_s = \frac{S_{DS}}{R}$$

where:

R is as above

S_{DS} = Design earthquake spectral response acceleration at short periods, as described in Section 2.6.3.1.

The seismic response coefficient, C_S, shall not be taken as less than $0.044S_{DS}$

Additionally, in locations for which the 1-second spectral response, S_1, is equal to or greater than 0.6, the value of the seismic response coefficient, C_s shall not be taken as less than:

$$C_s = \frac{0.5S_1}{R}$$

2.6.3.1 Design Spectral Response Acceleration Parameters

Five-percent damped design spectral response acceleration at short periods, S_{DS}, and at a 1-second period, S_{D1}, shall be determined from the following equations:

$$S_{DS} = (2/3)\ S_{MS}$$

$$S_{D1} = (2/3)\ S_{M1}$$

where:

S_{MS} = The maximum considered earthquake spectral response accelerations for short period as determined in Section 2.6.3.2.

S_{M1} = The maximum considered earthquake spectral response accelerations for 1 sec period as determined in Section 2.6.3.2.

2.6.3.2 Site Coefficients and Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameters

The maximum considered earthquake spectral response acceleration for short periods, S_{MS}, and at 1-second period, S_{M1}, adjusted for site class effects, shall be determined from the following equations:

$$S_{MS} = F_a S_S$$

$$S_{M1} = F_v\ S_1$$

where:

F_a = Site coefficient defined in Table 2.6.3.2 (2). If site class is unknown, use site class D

F_v = Site coefficient defined in Table 2.6.3.2 (3). If site class is unknown, use site class D

S_s = The mapped spectral accelerations for short periods as determined per USGS Open-File Report 01-437 “Earthquake Spectral Response Acceleration Maps” Version 3.10 values based on zip codes or longitude and latitude of site or Figure 1, 3, 5, or 7. Where zip codes are used to determine spectral accelerations, the largest value of any location within the zip code shall be used.

S_1 = The mapped spectral accelerations for a 1-second period as determined per USGS Open-File Report 01-437 “Earthquake Spectral Response Acceleration Maps” Version 3.10 values based on zip codes or longitude and latitude of site or Figure 2, 4, 6, or 7. Where zip codes are used to determine spectral accelerations, the largest value of any location within the zip code shall be used.

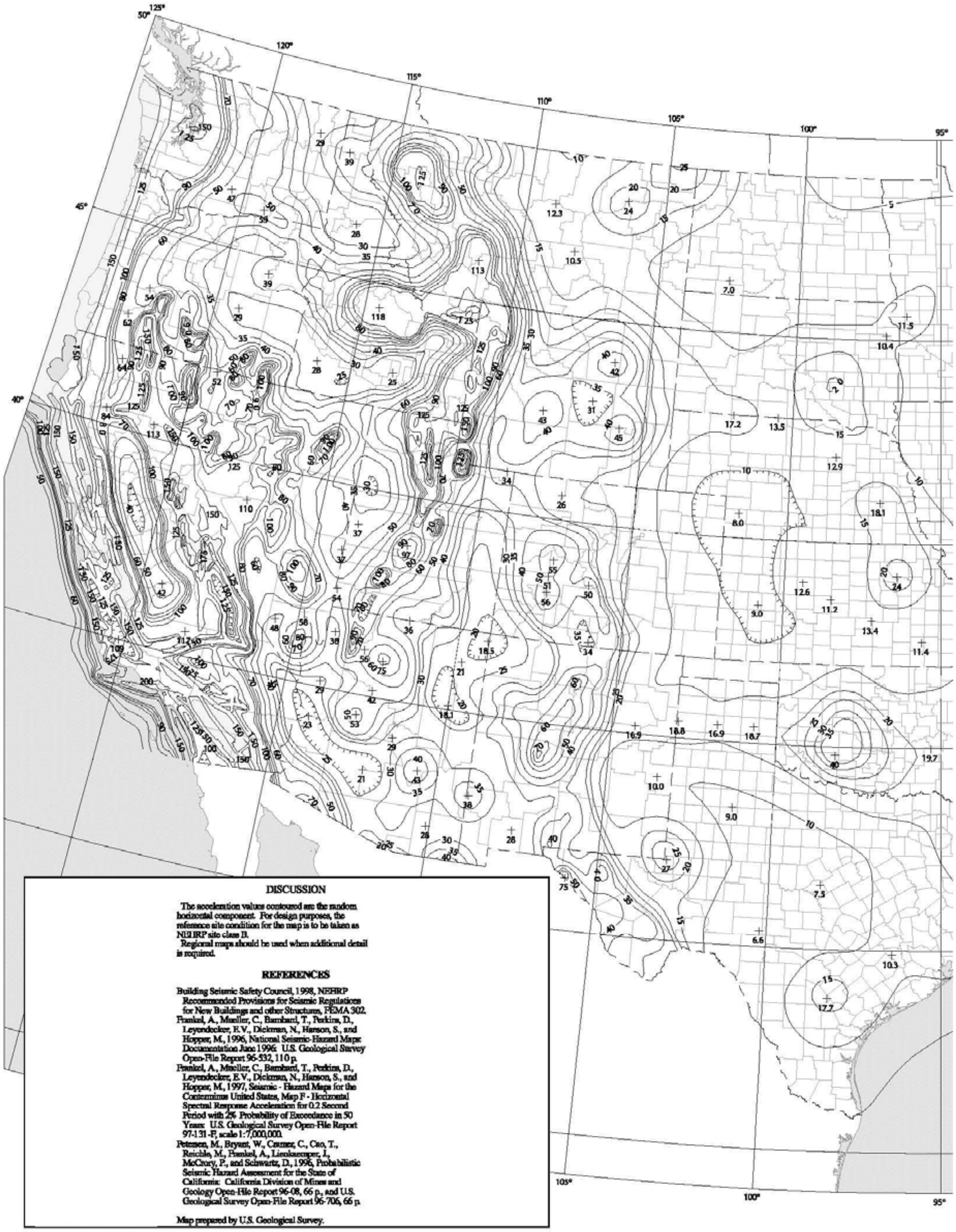

Figure 2.6.3-1 Maximum Considered Earthquake Ground Motion for Conterminous United States of 0.2 sec Spectral Response Acceleration (5% of Critical Damping), Site Class B

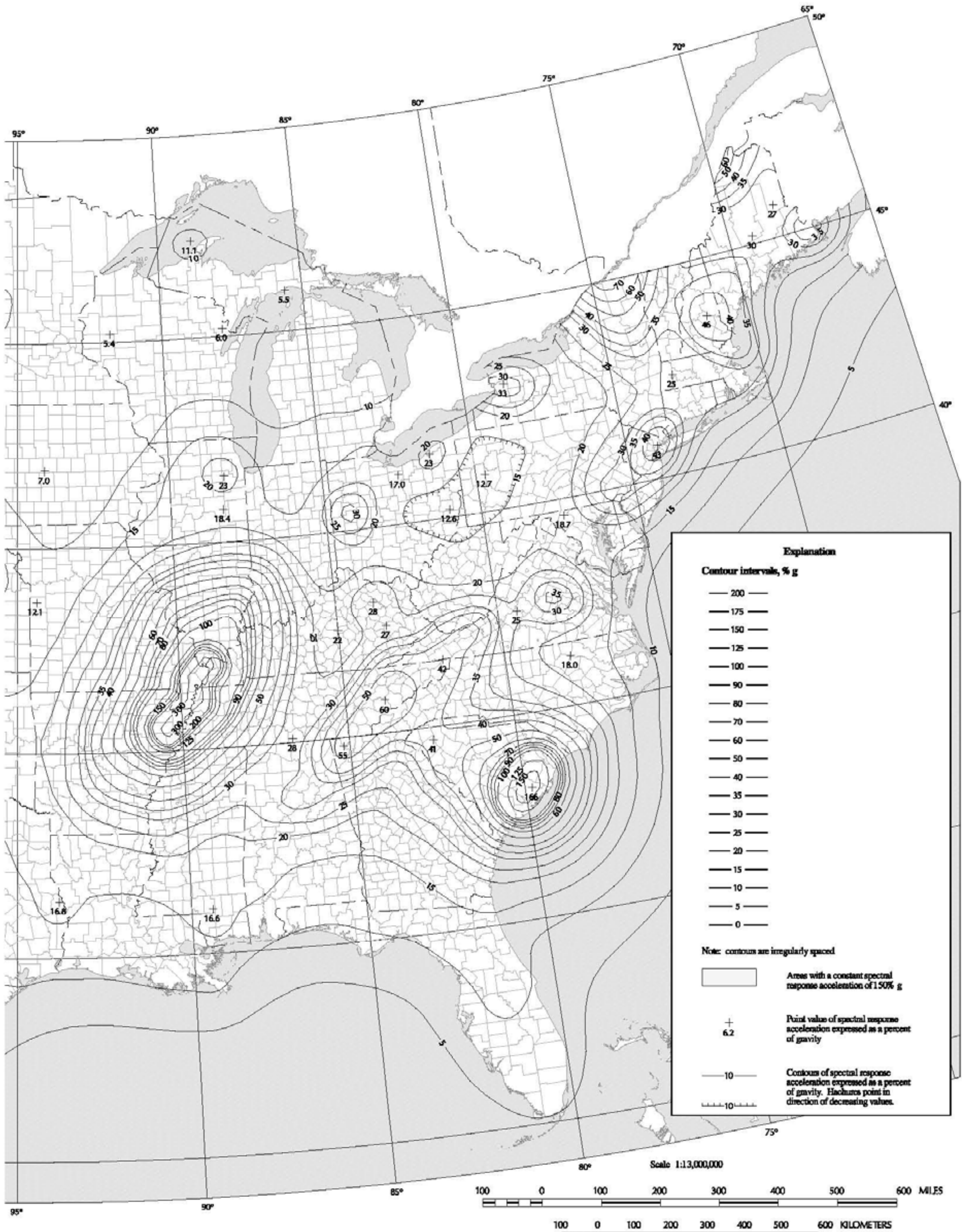

Figure 1 (Continued)

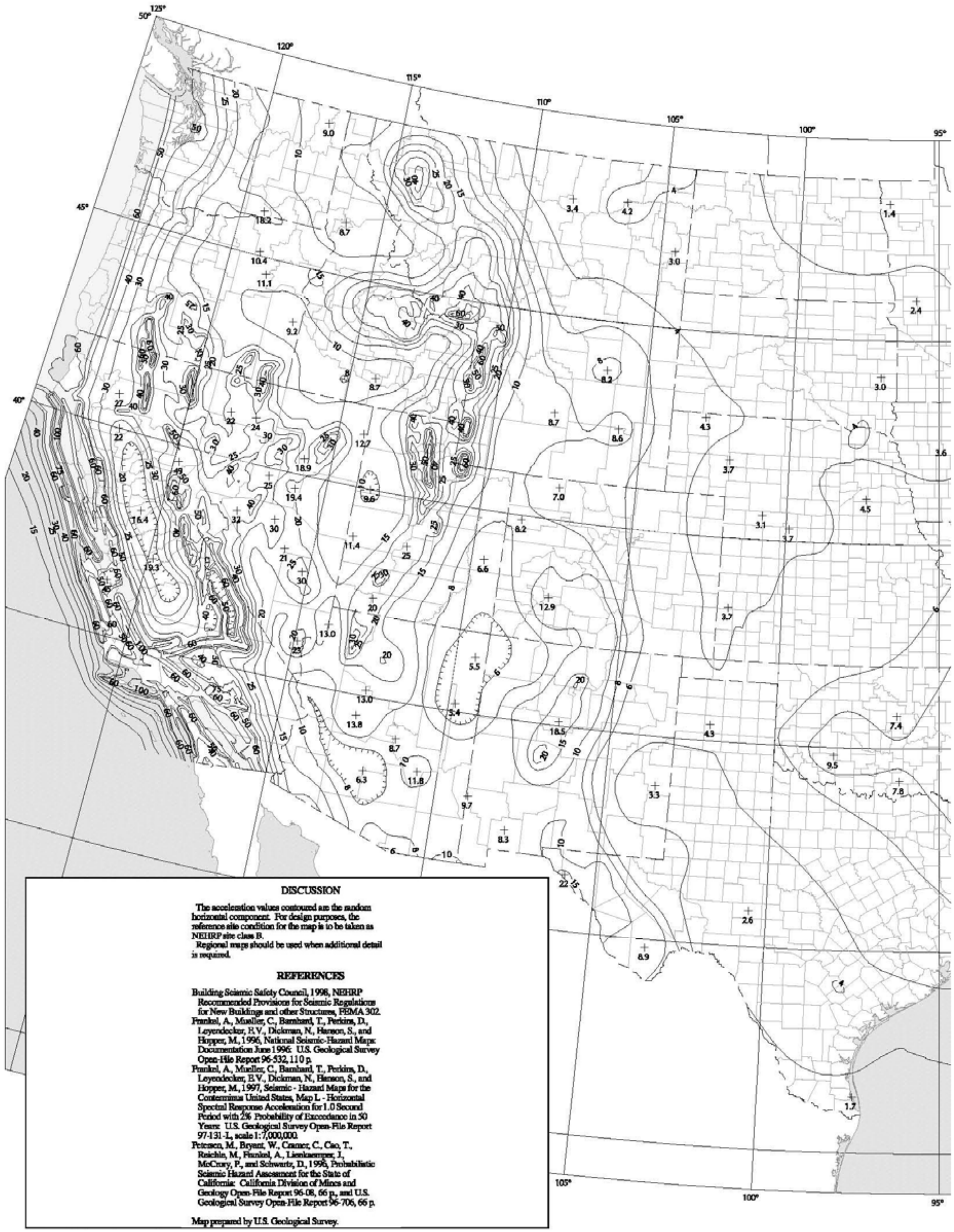

Figure 2.6.3-2 Maximum Considered Earthquake Ground Motion for Conterminous United States of 1.0 sec Spectral Response Acceleration (5% of Critical Damping), Site Class B

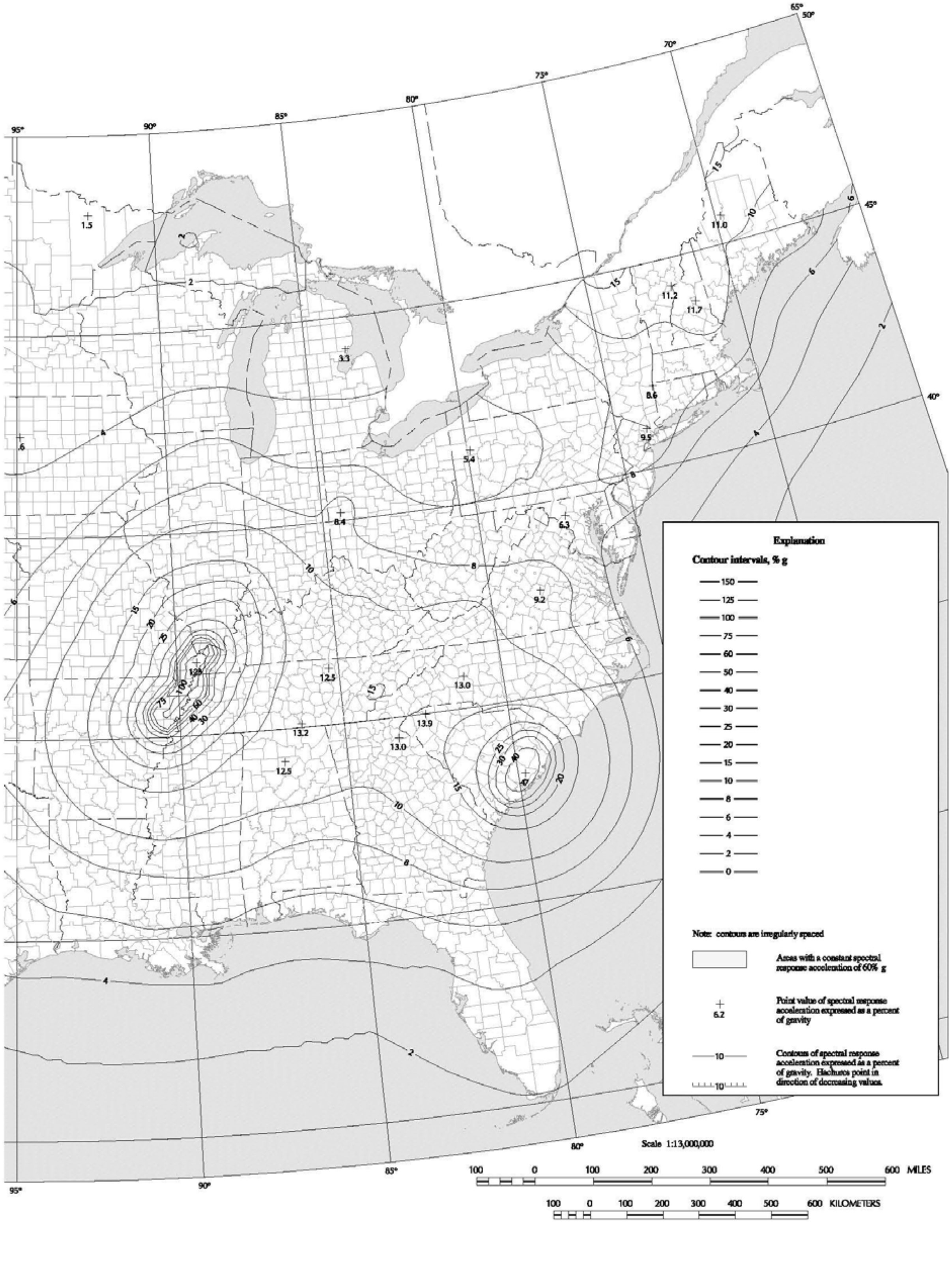

Figure 2 (Continued)

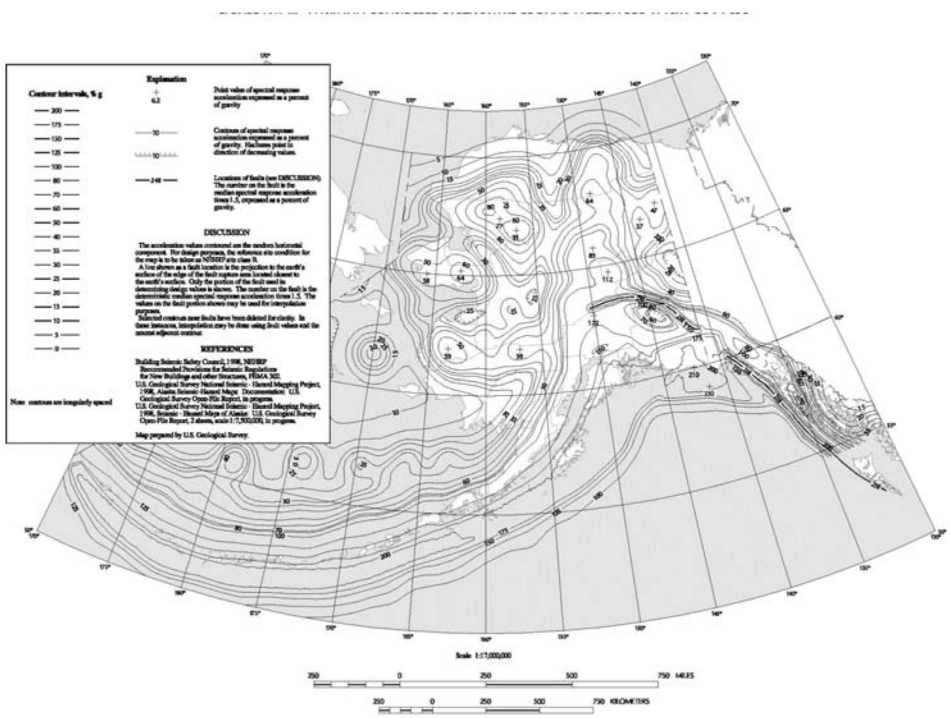

Figure 2.6.3-3 Maximum Considered Earthquake Ground Motion for Alaska of 0.2 sec Spectral Response Acceleration (5% of Critical Damping), Site Class B

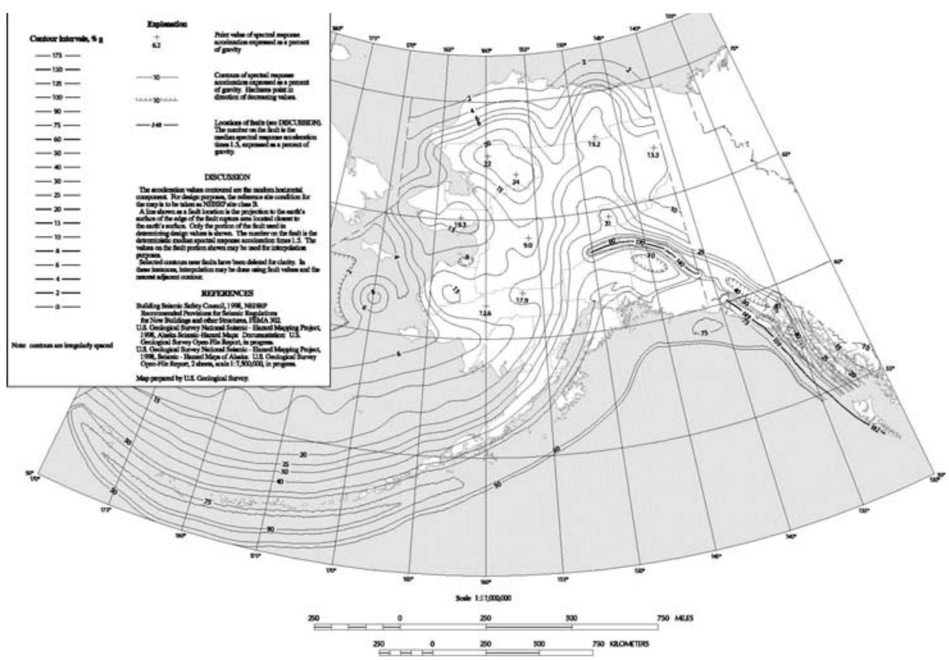

Figure 2.6.3-4 Maximum Considered Earthquake Ground Motion for Alaska of 1.0 sec Spectral Response Acceleration (5% of Critical Damping), Site Class B

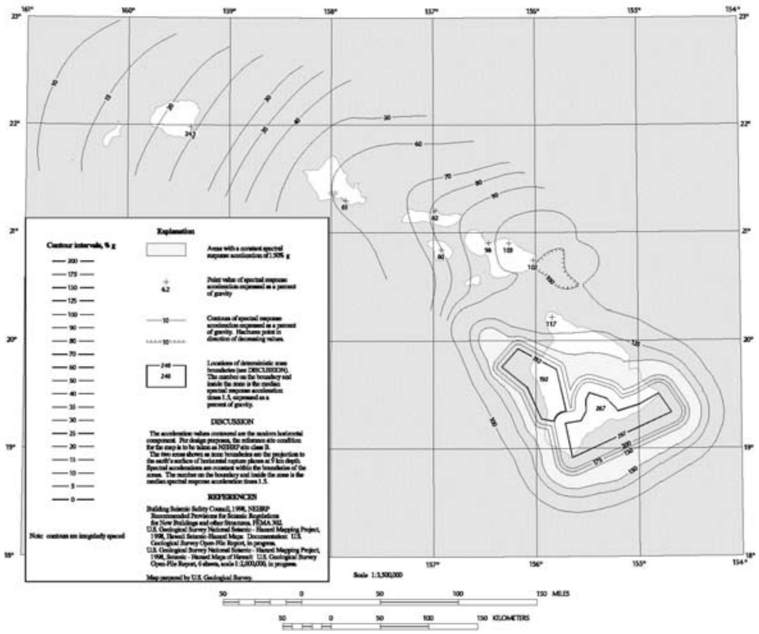

Figure 2.6.3-5 Maximum Considered Earthquake Ground Motion for Hawaii of 0.2 sec Spectral Response Acceleration (5% of Critical Damping), Site Class B

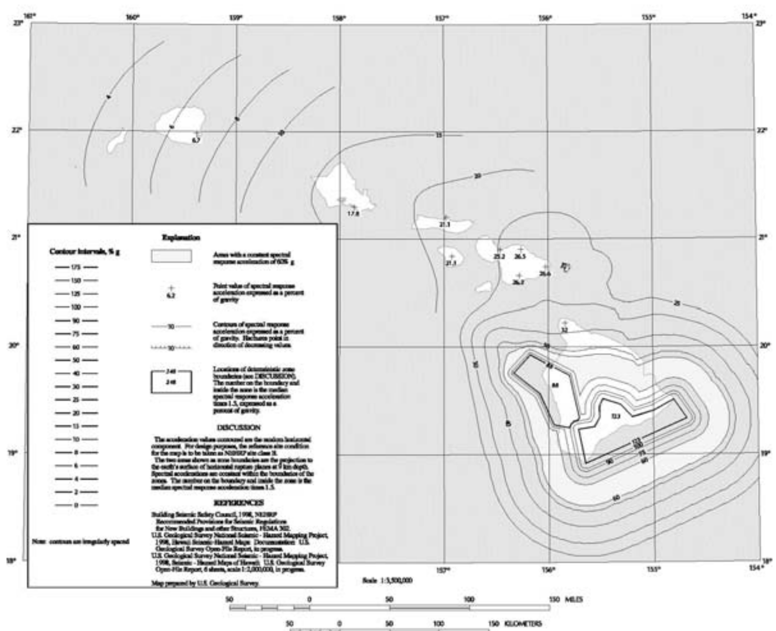

Figure 2.6.3-6 Maximum Considered Earthquake Ground Motion for Hawaii of 1.0 sec Spectral Response Acceleration (5% of Critical Damping), Site Class B

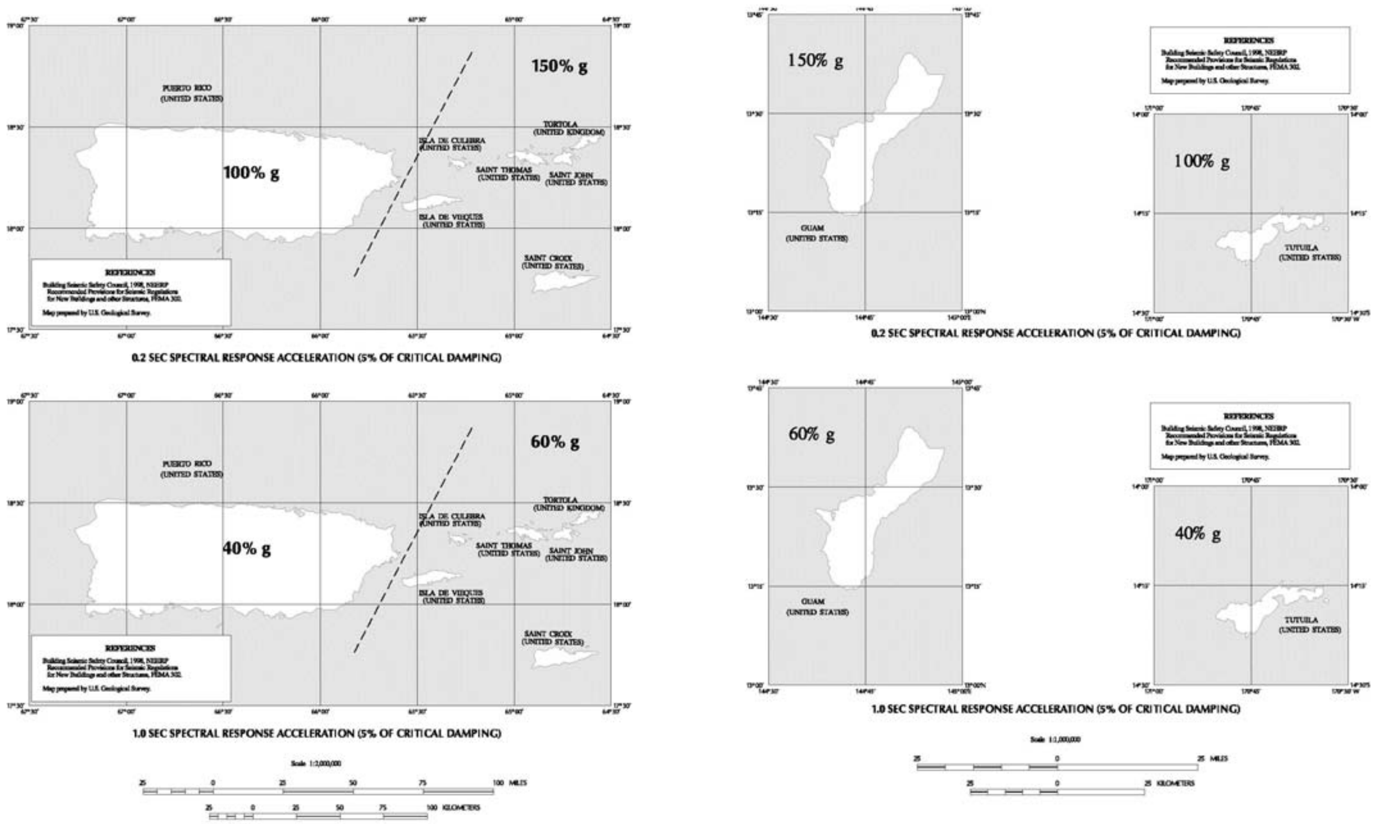

Figure 2.6.3-7 Maximum Considered Earthquake Ground Motion for Puerto Rico, Culebra, Vieques, St Thomas, St John, St Croix, Guam and Tutuilla of 0.2 and 1.0 sec Spectral Response Acceleration (5% of Critical Damping), Site Class B

TABLE 2.6.3.2 (1)
SITE CLASS DEFINITIONS

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 feet, AS PER SECTION 1615.1.5		
		Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \bar{v}_s \leq 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \leq 2{,}500$	$\bar{N} > 50$	$\bar{s}_u \geq 2{,}000$
D	Stiff soil profile	$600 \leq \bar{v}_s \leq 1{,}200$	$15 \leq \bar{N} \leq 50$	$1{,}000 \leq \bar{s}_u \leq 2{,}000$
E	Soft soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1{,}000$
E	—	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 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 ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet)		

For SI: foot = 304.8 mm, 1 square foot = 0.0929 m^2, 1 pound per square foot = 0.0479 kPa. N/A = Not applicable

TABLE 2.6.3.2 (2)

VALUES OF SITE COEFFICIENT F_a AS A FUNCTION OF SITE CLASS AND MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS (S_s)[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_s.
b. Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine appropriate values.

TABLE 2.6.3.2 (3)

VALUES OF SITE COEFFICIENT F_V AS A FUNCTION OF SITE CLASS AND MAPPED SPECTRAL RESPONSE ACCELERATION AT 1-SECOND PERIOD (S_1)[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1.
b. Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine appropriate values.

2.6.4 Connection Rotational Capacity

The rotational capacity Θ_{Max} of the beam to column connection shall be demonstrated, by tests from Section 9.6, to be greater than the rotational demand, $\Theta_{D.}$

$$\Theta_D = \frac{C_d(1+\alpha_s)\Delta_s}{h_{total}}$$

where:

C_d is the deflection amplification factor. From FEMA 460 C_d =5.5

h_{total} is the height of the top shelf level

a_s is the second order amplification factor from FEMA 460 calculated using the same W_s as the vertical load.

D_s is the seismic displacement from Section 2.6.5

Alternately, for racks assigned to Seismic Design Category A, B, or C, the rotational connection capacity check need not be made if the seismic response coefficient is taken as

$$C_s = \frac{S_{DS}}{R}$$

2.6.5 Seismic Displacement

The displacement from the seismic load at the top shelf level is D_s . The displacement shall be determined using the same structural system stiffness as used to determine the period for the base shear calculation in Section 2.6.3 and using the base shear from Section 2.6.2, including the I_p factor.

2.6.6 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:

$$F_1 = C_s I_p w_1$$ For the first shelf level

and

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

If the centerline of the first shelf level is greater than 12” (30.5 cm) above the floor:

$$\mathrm{F_x} = \frac{\mathrm{V w_x h_x}}{\sum_{\mathrm{i=1}}^{\mathrm{n}} \mathrm{w_i h_i}}$$ 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 on the racks, including live load, dead load and product load, times the product load reduction factor, (Section 2.6.2) that are located or assigned to the designated shelf level, level i or x

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

2.6.7 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 at the level(s) under consideration based on the relative lateral stiffnesses of those elements.

2.6.8 Overturning

Safety against overturning moment shall be designed on the basis of the following conditions of Product Load PL:

1. Weight of rack plus every storage level loaded to 67 percent of its rated load capacity
2. Weight of the rack plus the highest storage level only loaded to 100 percent of its rated capacity

The design shall consider the actual height of the center of mass of each storage load component.

2.6.9 Concurrent Forces

Forces described in Sections 2.4.1 and 2.5 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 AISI (2001) [1] for cold-formed steel components and structural systems and the AISC (2005) [2] 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 (2001) [1].

[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.

No slenderness limitations shall be imposed on tension members that are not required to resist compression forces under the various load combinations specified in Section 2.1 or 2.2.

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 (2001) [1] and the AISC (2005) [2] as described below.

4.1 COLD-FORMED STEEL MEMBERS {The AISI (2001) [1] Section C}

4.1.1 Properties of Sections {The AISI (2001) [1] Section C1}

Exceptions to the provisions of the AISI (2001) [1] for computing the section properties are given in Sections 4.1.2 and 4.1.3. Except as noted all cross-sectional properties shall be based on full unreduced and unperforated sections considering round corners.

4.1.2 Flexural Members {The AISI (2001) [1] Section C3}

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

S_c = Elastic section modulus of the net section for the extreme compression fiber times

$$1-\frac{(1-Q)}{2}\left(\frac{F_c}{F_y}\right)^Q$$

The value of Q shall be determined according to Section 9.2.2. Section properties j, r_o, and C_w shall be permitted to be computed assuming sharp corners.

Inelastic reserve capacity provisions of the AISI (2001) [1] Section C3.1.1 (b) shall not be considered for perforated members.

4.1.3 Concentrically Loaded Compression Members {The AISI (2001) [1] Section C4}

4.1.3.1 Effective Area

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.

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. Torsional warping constant C_W may be calculated based on sharp corners.

4.1.3.2 Distortional Buckling

Open sections except those with unstiffened elements or only simple lip edge stiffeners shall be checked for the effects of distortional buckling by testing or rational analysis.

4.2 HOT-ROLLED STEEL COLUMS (AISC (2005) [2] Chapter E)

All hot-rolled steel columns shall be designed according to Section E7, of the AISC (2005) [2] except as noted below.

The nominal compressive strength P_n shall be calculated as follows:

$$P_n = A_e F_{cr}$$

A_e is defined in Section 4.1.3.1 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 (2001) [1] or the AISC (2005) [2].

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 deflection 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 LENGHTS

Effective lengths for columns are those specified in Sections 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. One column stability design method should be used consistently throughout one structure.

6.3.1 Flexural Buckling in the Direction Perpendicular to the Upright Frames

L_x 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 Section 6.3.1.1, 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} to 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 axes 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 Diagonals and Horizontals

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

The analysis and design of the upright frame joints (or connections) shall include a consideration of the transfer of the member forces into and through those joints along with their connections and the deformation of the member legs, lips, and stiffening elements that make up the cross section of the various members coming into each joint.

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 (2001) [1] or the AISC (2005) [2] 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. BEAM CONNECTIONS AND COLUMN BASE PLATES

7.1 BEAM-TO-COLUMN 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 resisting 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 resisting 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 COLUMN BASE PLATES

7.2.1 Bearing on Concrete

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

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

$$\phi_c = 0.60$$

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 (2.1×10^6 kg/m^2).

7.2.2 Base Plate Design

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 column base plates of each type of upright frame in the installation.

When rational analysis is used to determine base plate thickness, the base plate shall be designed for the following loading conditions, where applicable:

7.2.2.1 Downward Vertical Force

The effective area of the base plate is defined as the minimum area needed to satisfy the concrete bearing requirements or the minimum bearing area required by the concrete slab designer. This area may be the area bounded by the perimeter of the rack column, the full area of the base plate or some area in between these two values. The resulting area is defined as the effective base plate area. The base plate thickness shall be calculated assuming that the bearing pressure is uniformly distributed over the effective base plate area and the plate shall be analyzed as a rigid member.

7.2.2.2 Uplift/tension Force

When the base plate configuration uses a single anchor bolt and a net uplift force exists, the minimum base plate thickness shall be determined based on a design bending moment in the plate equal to the uplift force times 1/2 the distance from the centerline of the anchor to the nearest edge of the rack column. When the base plate configuration consists of two anchor bolts located on either side of the column and a net uplift force exists, the minimum base plate thickness shall be determined based on a design bending moment in the plate equal to the uplift force on one anchor times 1/2 the distance from the centerline of the anchor to the nearest edge of the rack column.

7.2.2.3 Axial Load Plus Bending (Down-aisle Seismic or Wind)

When downward axial loads and bending moments due to lateral loads exist, the base plate thickness and anchor forces shall be determined as follows:

When $e = M/P <= N/6$, where N = effective length of the base plate in the down-aisle direction, no uplift of the base plate will occur. Therefore, no tension force will be present in the anchors and the anchors shall be designed for the maximum calculated shear force. The base plate thickness shall be determined as in Section 7.2.2.1.

When $e > N/6$, where N = effective length of the base plate in the down-aisle direction, then a tension force may occur in the anchor(s). The tension force can be calculated directly once the compressive stress block underneath the plate has been established. In order to calculate the anchor tension, the designer must assume either the peak magnitude of the stress distribution or the length of the stress block. Once the concrete stress block distribution and magnitude has been established, the anchor tension can be calculated directly through the equations of equilibrium. The base plate thickness shall be determined as in Section 7.2.2.1.

Anchors shall be designed to resist the tensile and shear forces based on the anchor manufacturer's stated capacities and the resistance factors contained in Section 2 of this Specification.

7.2.3 Maximum Considered Earthquake Rotation

The base connection shall have a rotational capacity not less than the rotational demand of the beam-to-column connection, Θ_D as calculated in Section 2.6.4. Otherwise, the base connection shall be considered pinned for the computation of the period and seismic displacement.

7.2.4 Shims

Shims may be used under the base plate to maintain the plumbness of the storage rack. The shims shall be made of a material that meets or exceeds the design bearing strength (LRFD) or allowable bearing strength (ASD) of the floor. The shim size and location under the base plate shall be equal to or greater than the required base plate size and location

In no case shall the total thickness of any set of shims under a base plate exceed six times the diameter of the largest anchor bolt used in that base.

Shims that are a total thickness of less than or equal to six times the anchor bolt diameter under bases with less than two anchor bolts shall be interlocked or welded together in a fashion that is capable of transferring all the shear forces at the base.

Shims that are a total thickness of less than or equal to two times the anchor bolt diameter need not be interlocked or welded together.

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 (Sections 2.5 and 2.6).

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 measured to the topmost beam position, unless the rack is anchored or braced externally to resist all forces.

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 2.4, 2.5 or 2.6.

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 Section 1.4.10).

8.3 INTERACTION WITH BUILDINGS

Storage rack located at levels above the ground level (as described in Section 2.6.2), 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.

8.4 PICK MODULES AND RACK SUPPORTED PLATFORMS

Pick modules and rack supported platforms that are used by authorized or trained personnel and not open to the general public shall be designed in accordance with this section.

8.4.1 Posting of Design Loads

The design loads for the floor areas of the rack supported platforms and pick module walkways shall be shown on the rack configuration and load application drawings. These design loads shall also be displayed in one or more conspicuous locations within the structure such as at the top of the access stairway.

8.4.2 Design Requirements

Rack supported platforms and pick module walkways shall be designed for the maximum concentrated loads and the maximum uniformly distributed loads that are to be imposed on the rack supported platform floor. The owner shall advise the designer of the rack supported platform or pick module of all loads that are expected on the structure for its present or future use.

The design load for the foot traffic on pick module walkways shall be at least 60 psf (290kg/m^2) live load superimposed over the entire area of the foot traffic walkway. Where applicable the pick module floor shall also be designed for conveyor leg loading, pallet staging, shelving, mobile-handling equipment or any other items that could cause additional load on the pick module walkway. In some cases conditions may require a higher design load. The user should advise the designer of all such conditions. The pick module walkway shall also be designed for other items such as lights or sprinkler pipes that may be hung from the pick module walkway floor or floor supports.

If the project specifications dictate that a pick module walkway live load greater than or equal to 100 psf (488 kg/m^2) is required and there are two or more floor levels, the live load may be reduced by 20 percent for the design of the column framing system which includes the support columns, the frame bracing, the frame bracing connections and the base plates. This reduction does not apply to the beams that support the floor or their connections.

The maximum live load deflection for beams that support rack supported platforms shall not be greater than L/240. The total load deflection shall not be more than L/180

The clear width of a pick module walkway shall be at least 30” (76 cm).

8.4.3 Rack supported platform and Pick Module Walkway

Guardrails - Members that are installed on an elevated rack supported platform or pick module walkway whose purpose is to provide fall protection for the occupants of the structure. Guardrails consist of a top rail, an intermediate rail and posts.

Safety Flooring – A surface that is provided in areas where order picking personnel may need to step off the normal walking area or pick module walkway to dislodge loads that may not have properly flowed to their correct discharging positions.

Kick-plate – Kick-plates are vertical plates that extend upward at the edge of a floor surface to prevent loose items from sliding off the edge of the floor.

8.4.3.1 Guardrail Requirements

The horizontal top rail of the guardrail shall be 42” (407cm) above the walking surface. Guardrails shall have a top rail and intermediate members such that a 21” (53 cm) sphere can not pass through below the top rail level. The ends of the rails shall not extend beyond the post except where extending the rails will not create a hazard. Where there is a discontinuity of the guardrail that exceeds 6” (15 cm) such as between vertical members or between stairs and a vertical member, filler guardrail is required to provide fall protection for this space. The top and intermediate guard rails or any other part of the guardrail assembly must be designed to resist the following loads applied separately (not simultaneously):

1. Concentrated live load of 200 pounds (90.7 kg) applied at any location along the top rail assembly in any direction.
2. Distributed live load of 20 plf (292 N/m) applied in any direction along the length of any member that is part of the assembly.

Guardrails are not required to be in place where they would interfere with product being loaded into or removed from the pick module system. Guardrail must be provided to close any other openings through which an order picker may fall. Where guardrails are omitted for pallet drops, safety gates, removable guardrail sections or removable chains must be used for fall protection. These devices must meet the same strength and configuration requirements as the permanently installed guardrail.

8.4.3.2 Safety Flooring Requirements

Safety flooring shall be designed for a 300# (136 kg) concentrated load (to support the picker) and a distributed live load of 60 psf (290 kg/m²) acting separately. The pickers shall not walk out onto the safety flooring without observing the correct safety procedures that are required for the pick module use. The pickers shall stay at least 4 feet (1.22 m) away from the open end of the safety flooring.

8.4.3.3 Kick-plate Requirements

Kick-plates shall extend at least 4” (10 cm) above the floor surface. Kick-plates are not required at picking locations but are required at pallet drop locations. The user shall specify to the designer any additional areas where kick-plate may be needed for safety due to the configuration of the pick module.

8.4.3.4 Special Conditions

Floor openings under the conveyor path that are used for the discharge of trash need not have guardrail or kick plates as this would interfere with the efficient discharge of trash.

Where conveyor inclines rise through an opening in the floor, guardrail is generally required on all sides except the side where use of such guardrail would interfere with the conveyor or product.

Kick-plates are not required where rack frame bracing or other structural components such as shelf decking or safety flooring are next to the edge of the floor.

Guardrails are not required at locations where other structural members such as rack frame horizontal members are provided that meet the strength and configuration requirements of the guardrail.

8.4.4 Stairways

Fixed stairways shall be provided for access to elevated rack supported platforms or pick modules by authorized or trained personnel. Fixed stairways shall be designed and constructed to carry a load of 300 psf (1460 kg/m²) but shall not be of less strength than to carry a concentrated live load of 1000 lbs (4880 kg/m²) at any point along the stairway.[other requirements may need to be considered]

Fixed stairways shall have a minimum tread width of 30" (76 cm). A vertical clearance of seven feet shall be maintained between the stairway and any overhead obstruction measured from the leading edge of the tread. Stairways shall be installed at angles to the horizontal of between 30 and 50 degrees. The sum of the rise and the run of a single step should be approximately 17.5 inches (45 cm) with the minimum rise of 6.5 in (16.5 cm) and a maximum rise of 9.5 in (24 cm).

Rise height and tread length shall be uniform throughout any flight of stairs including any foundation structure used as one or more of the treads of the stairs. Open risers are allowed.

Stairway landings shall be no less than the width of the stairway and a minimum of 30" (76cm) in length measured in the direction of travel. Intermediate landings are required if vertical rise exceeds 12 ft (3.66m).

Handrails shall be provided on both sides of all stairways. If the total rise of the stairway is less than 44" (112 cm) stair handrails are not required.

Stair Handrail – Smooth, continuous railing that runs up a stair rise assembly to provide added balance and safety for the occupants as they walk up or down the stair rise assembly.

Stair handrail shall be 30" to 34" (76 cm to 86cm) in height when measured from the top of each tread at the face of the tread. Stair handrail brackets or posts supports shall be spaced at no more than 8 ft. centers (2.44 m) and the rail shall be mounted so a clearance of at least 3" (7.6 cm) exists horizontally between the rail and any obstruction. Stair handrails shall be designed for the same forces as guardrails.

Stair handrail extensions are not required in pick module or rack supported platform stair assemblies.

8.4.5 Product Fall Protection

The owner should specify to the designer any locations where operations may require horizontal or vertical safety barriers. These barriers shall prevent product from falling into those areas.

8.5 AUTOMATED STORAGE AND RETRIEVAL SYSTEMS (STACKER RACKS)

Stacker racks may be "Load Arm and Rail Type" or "Beam Column Type" and can be used in "Rack Supported Systems".

Shown in parenthesis in the heading are the numbers corresponding to parts of this Specification.

8.5.1 Tolerances (1.4.11)

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

8.5.2 Vertical Impact Loads (2.3)

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.

8.5.3 Horizontal Loads (2.4)

Horizontal loads specified in Section 2.4.1 and 2.4.2 of the Specification shall be used in the design of racks.

8.5.4 Wind (2.5) 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 2.4.1, 2.4.2 and 2.6 need not be assumed to act concurrently with wind loads.

8.5.5 Deflections (5.3)

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

8.5.6 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.

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 channels or 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 (2001) [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 $\pm$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 (2001) [1], if cold work effects are to be considered.

$A_{net\ min}$ = 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 test load shall be applied to the test assembly to ensure 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 in (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 in (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 (2001) [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.

9.3.2.4 Number of Tests Required

The number of tests for determining design loads shall be as specified in Section F of the AISI (2001) [1].

9.3.2.5 Deflection Test

Once the design load has been determined as specified in Sections 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 in (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 2.4.1 as well as the effects of semi-rigid connections. This procedure is also applicable to Section 2.5 and 2.6 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 2.4.1.
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 (2001) [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 Section 9.5.1.1, unsymmetrical loading in Section 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 Section 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 2.4.1.

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 Section 9.5.1.1.3.

9.6 CYCLIC TESTING OF BEAM-TO-COLUMN CONNECTIONS

9.6.1 General

This protocol includes requirements for qualifying cyclic tests of beam-to-column moment connections in steel storage rack beam-to-column connectors. The purpose of the testing described in this document is to provide evidence that a beam-to-column connection satisfies the requirements for Strength and Drift Angle comparable to those stated in the AISC Provisions. Alternate testing requirements are permitted when approved by the Engineer of Record and the Authority Having Jurisdiction. It is also the purpose of this series of tests to determine the moment-rotation characteristics, or "dynamic spring relationship" of the beam-to-column connections of the various designs and manufacturers.

9.6.2 Definitions

Definitions of the elements, components, variables, parameters, and dimensional characteristics of the physical test set-up will be specified as representative of this testing protocol.

9.6.3 Test Subassemblage Requirements

The Test Subassemblage shall replicate as closely as is practicable the conditions that will occur in the Prototype during earthquake loading. The Test Subassemblage shall include a column element and two cantilever beam elements with integral attached beam-to-column connectors.

9.6.4 Essential Test Variables

The Test Specimen shall replicate as closely as is practicable the pertinent design, detailing, and construction features, and the material properties of the Prototype.

9.6.4.1 Sources of Inelastic Rotation

Inelastic Rotation shall be developed in the Test Specimen by inelastic action in the same members and connection elements as anticipated in the prototype, i.e., in the beam, in the column, in the panel zone, or within the connection elements.

9.6.4.2 Size of Members

The size of the beams and columns used in the Test Specimen shall be representative of typical full-size storage rack beams and columns.

9.6.4.3 Connection Details

The beam-to-column connectors and the connection details used in the Test Specimen shall represent the Prototype connection details as closely as possible.

9.6.4.4 Material Strength

Each member of the connection element that contributes to the Inelastic Rotation at yielding will be tested to determine its yield stress and yield strength.

9.6.4.5 Welds

Welds on the Test Specimen shall satisfy and be performed in strict conformance with the requirements of *Welding Procedure Specifications* (WPS) as required.

9.6.4.6 Bolts

The bolted portions of the Test Specimen shall replicate the bolted portions of the Prototype connection as closely as possible.

9.6.5 Testing Procedure

The testing program should include tests of at least two specimens of each combination of beam and column and connector size. The results of the tests should be capable of predicting the median value of drift angle capacity for the performance states of Strength Degradation and Ultimate Drift Angle Capacity.

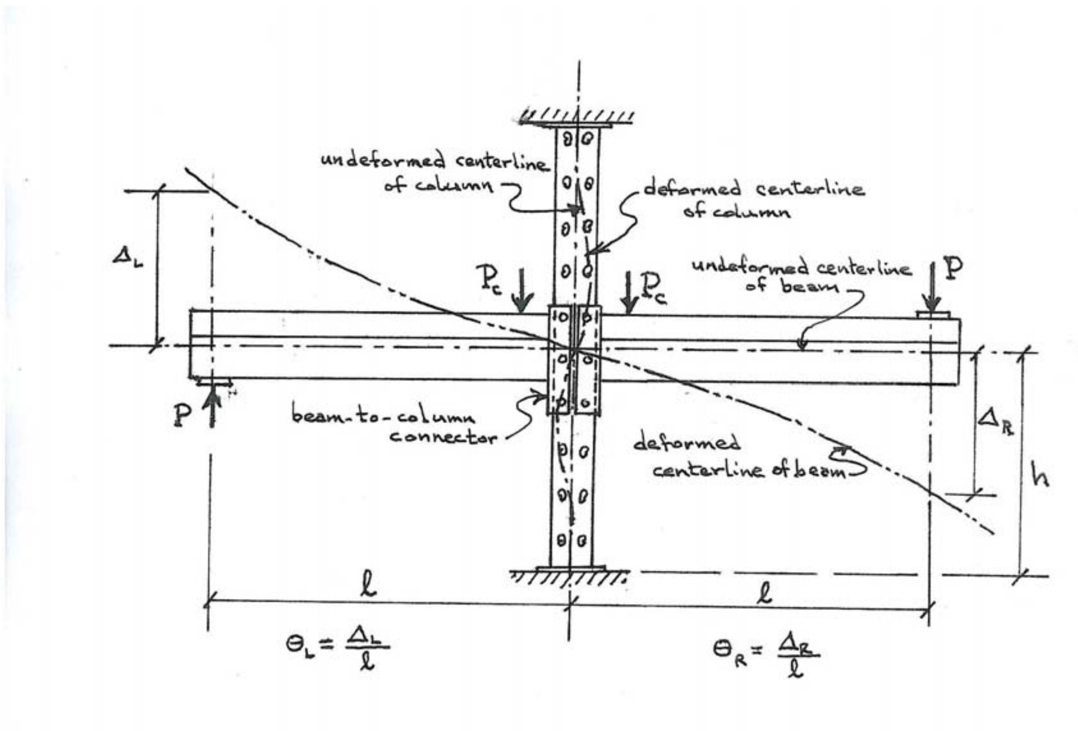

Figure 9.6.5-1 Test Set Up

9.6.6 Loading History-General Requirements

Prior to the application of any cyclic loading, a constant downward load, P_c, of one kip shall be applied to each beam segment adjacent to each connector on both sides of the beam-to-column connection simulating the design downward-acting gravity pallet loads that serve to fully engage the beams and their connectors into the columns receiving them.

The Test Specimen shall be subjected to cyclic loads according to the requirements prescribed for beam-to-column moment connections in Moment Frames. Other loading sequences may be used when they are demonstrated to be of equivalent or greater severity. Qualifying tests shall be conducted by controlling the Peak Drift Angle imposed on the Test Specimen.

Loading will proceed with the application of equal displacements at each end of each beam, and the measurement of the force corresponding to each such displacement.

9.6.7 Instrumentation

Sufficient instrumentation shall be provided on the Test Specimen to permit measurement or calculation of the quantities required to produce meaningful, reproducible results for this testing protocol.

9.6.8 Material Testing Requirements

9.6.8.1 Tension Testing Requirements

Tension testing shall be conducted on samples of steel taken from the material adjacent to each Test Specimen. Tension-test results from certified mill test reports shall be reported but are not permitted to be used in place of specimen testing for the purposes of this Section.

9.6.8.2 Methods of Tension Testing

Tension-test results shall be based upon testing that is conducted in accordance with the appropriate ASTM testing protocols for the particular materials being used.

9.6.9 Test Reporting Requirements

For each Test Specimen, a written test report meeting the requirements of the Authority Having Jurisdiction and the requirements of this Section shall be prepared. The report shall thoroughly document all key features and results of the test. Additional drawings, data, photographs, and discussion of the Test Specimen or test results are permitted to be included in the report.

9.6.10 Acceptance Criteria

The Test Specimen must satisfy the Strength and Drift Angle requirements of this protocol for the connection, as applicable. The Test Specimen must sustain the required Drift Angle for at least one complete loading cycle. The test results will also include the beam-to-column moment-rotation characteristics and “dynamic spring relationship” for each of the combinations tested.

10. REFERENCES TO THE TEXT

1. American Iron and Steel Institute (AISI) (2001), *North American Specification for the Design of Cold-Formed Steel Structural Members*, Washington, DC.

2. American Institute of Steel Construction (AISC) (2005), *Specification for Structural Steel Buildings,* ANSI/AISC 360-05, Chicago, IL

3. American Iron and Steel Institute (AISI) (1996), *Cold-Formed Steel Design Manual,* Washington, DC.

4. Federal Emergency Management Agency (FEMA) (2005), *Seismic Considerations for Steel Storage Racks Located in Areas Accessible to the Public*, FEMA 460, FEMA, Washington, DC

5. American Society of Civil Engineers (ASCE) (2005), *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-05, ASCE, Reston, VA.

Commentary to MH16.1 – 2008
(a revision of MH16.1 – 2004)

Commentary to MH16.1 – 2008 Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks

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PREFACE TO THE COMMENTARY

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 RMI 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.

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

1 GENERAL

1.1 SCOPE

The scope limits the applicability of the Specification to pallet racks, movable shelf racks, rack supported structures and stacker racks made of hot-rolled or cold-formed steel. Although only these 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, but are not limited to, 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 structures.

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" [ref] regarding safety practices in the use of storage racks for further information. The user may also refer to FEMA 460 [5] for additional guidance on proper operation and maintenance for racks installed in areas accessible to the public

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 Figures 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.

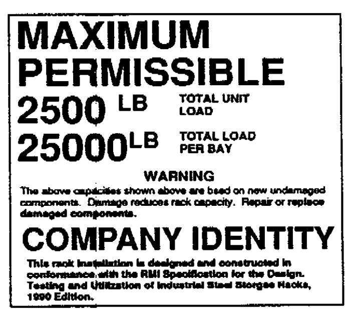


Figure 1.4.2a Example of Load Capacity and Compliance Plaque.
(1134 kg Total Unit Load)
(11340 kg Total Load per Bay)

Figure 1.4.2b Example of Load Capacity and Compliance Plaque.
(366 kg/m^2 Uniformly Distributed Load)
(9072 kg Total Load per Bay)

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 according to this Specification. To this end, Section 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 Section 1.4.4 provides that 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 Section 1.4.5 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 Column Base Plates and Anchors

It is the function of column base 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 column base plates on concrete floors are given in Section 7.2. Adequate connection of the column to the bearing plate is required to properly transfer the forces.

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

Anchors serve several distinct functions:

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

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.

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 Sections 1.4.4 and 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 Rack Damage

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 (0.61 m to 1.22 m) 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 base plates 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. The force transfer between any two structures is dependent on their relative movement and stiffness. Absent detailed knowledge of the other structure, it is not generally possible to compute the rack force being transferred. In such cases, the rack designer may provide forces assuming that the adjacent structure is infinitely stiff. The rack designer should also consider the alternative: the adjacent structure may transfer load to the rack.

1.4.11 Out-of-plumb and Out-of-straight Limits

The purpose of these provisions is to keep the axial load eccentricity to a minimum. An out-of-plumb or out-of-straight condition will cause axial load eccentricity that will reduce the load carrying capacity of a rack column. The reduction can be significant. A rack that is out-of-plumb from top to bottom or a rack column that is not straight is likely to become further out-of-plumb or out-of-straight when it is loaded. The limits on out-of-plumb and out-of-straight that are given in Sections 1.4.11.1 and 1.4.11.2 are for loaded racks. They are provided so the user may know when his racks may need to be re-plumbed and possibly repaired. If an empty rack exceeds these limits, it should be corrected prior to loading. Some installations may require tighter limits, for example, a structure that is loaded and unloaded by an automatic (unmanned) vehicle.

1.4.11.1 Out-of-plumb Limit

The limit given for top to bottom out-of-plumb in Section 1.4.11.1 is for a loaded rack and is not intended to be an installation tolerance. The installer should obtain the installation tolerances from the rack supplier prior to the start of an installation. These tolerances should be such that the loading of the racks will not cause the racks to exceed the out-of-plumb limit given in Section 1.4.11.1. This limit is intended to prevent the use of racks that have a down aisle or a cross aisle lean.

1.4.11.2 Out-of-straight Limit

The out-of-straight limit is new in this edition of the specification and is given to prevent excessive “bows” or “dogleg” conditions that may exist in a rack column. A column could be plumb from top to bottom but have an unacceptable bow at mid-height, see Figure 1.4.11(a), or, a 20 ft. high column could be out 1” from top to bottom, which would be acceptable using a simple top-to-bottom out-of-plumb measurement, but the entire out-of-plumb could be between the floor and the 5 ft. level, see Figure 1.4.11(b). This dogleg condition would be very harmful. This condition could be caused by fork truck impact. The column could have a sine wave shape and be out-of-straight as shown in

Figure 1.4.11(c). The column could also be locally bent and exceed this limit, see Figure 1.4.11(d). As rewritten, the specification now prevents these situations from being acceptable if they exceed the 0.05” per foot out-of straight limit.

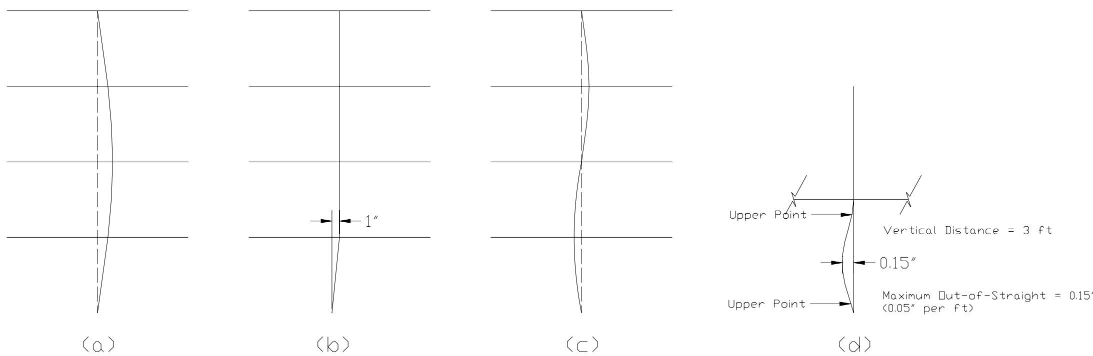


Figure 1.4.11-1

2 LOADING

The purpose of this section is to clarify the design methods used in the AISI [1] and the AISC [3] Specifications as they apply to storage racks and to show how the ASCE-7 [6] load combinations should be applied 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 2.4.2.

The load combinations have been written to agree with the load combinations from ASCE-7 [6] as they apply to storage racks with the addition of the product load (PL) added to each combination. Roof live load (L_r) for rack supported structures has been added since the last edition of the RMI Specification. The vertical component of seismic load (S_{ds} coefficient) has also been added. This is a new load effect that has been added to the ASCE-7 [6]. This term is added as a factor to the dead load and the product load.

Since the last edition of the RMI Specification LRFD design has become much more commonplace for cold-formed and structural steel. The AISI [1] and the AISC [3] have each combined LRFD and ASD in their respective specifications. The two methods of the analysis should give results that are similar but they will not be the same. The RMI allows the designer to use either method. The designer may see some benefit to the LRFD method due to the product load factor that has been incorporated in the load combinations.

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 loads that could reasonably act together but also not to combine loads that are unlikely to 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

2.1 LOAD COMBINATIONS FOR THE ASD DESIGN METHOD

The ASD design method uses mostly unfactored applied loads and then compares them with nominal strengths divided by factors of safety. 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. Other load factors that appear in the ASD method are due to changes in the ASCE-7 [6] combinations. All loads resulting from these combinations must be checked against nominal strengths from the AISC [3] or AISI [1] divided by the appropriate Ω (safety factors) given therein.

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 structures 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.

In combinations #3 and #4 all loads except the dead loads may be multiplied by 0.75. This is a change from the previous edition where all loads were multiplied by 0.75. This change is made to reflect the same change in ASCE-7 [6]. The coefficient of S_{ds} in load combinations 3 and 4 is 0.11 rather than 0.14 because the vertical seismic effect is a seismic load so the 0.14 coefficient has been multiplied by 0.75 to result in a coefficient of 0.11. This is an adjustment that is made because the DL term is no longer permitted to be multiplied by 0.75. Since the dead load of a rack structure is usually a small percentage of the total load the use of the 0.75 factor is essentially 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 load combinations have been written to agree with the load combinations from ASCE-7 [6] as they apply to storage racks with the addition of the product load (PL) added to each combination. 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 – Specification [3] or the AISI Specification [1]. The load factors and combinations have been updated to reflect similar changes made in ASCE-7. Roof live load (L_r) for rack supported structures has been added since the last edition of the RMI Specification. The vertical component of seismic load (S_{ds} coefficient) has also been added. This is a new load effect that has been added to the ASCE-7. This term is added as a factor to the dead load and the product load.

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 for cold-formed structural steel and 0.9 for hot-rolled structural steel.

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 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:

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

For ϕ = 0.95

$$1.587 / 0.95 = 1.67$$

This corresponds to the traditional 1.67 factor of safety. A resistance factor (ϕ_b) of 0.9 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 (488 kg/m^2) 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 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 2.3 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 2.3. 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.

2.4 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 2.5, Wind Loads and Section 2.6 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 the 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∆ 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. It is important to remember that design of a beam-column member requires the inclusion of P-∆ effects.

Other specifications, NEHRP [7], and UBC [10], specify a drift limit for storage racks of 0.0125 h_x and 0.0036 h_x respectively. These specifications do not require P∆ 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∆ 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 quantities 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.

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.

2.5 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.

2.6 EARTHQUAKE FORCES

2.6.1 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 International Building Code [8] promulgated by the International Code Council and American Society of Civil Engineers, ASCE-7 [6].

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.

2.6.2 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. The I_p factor of 1.5 for warehouse retail stores is equivalent to having the racks being designed for maximum considered event performance, which is consistent with the stated performance goals of FEMA 460.

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. It does not apply to vertical load under any load combination nor to the fraction of vertical load used for restoring moment in the evaluation of seismic stability. 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.

2.6.3 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 spectral response acceleration S_S and S_1 are to be taken from the accompanying contour maps or USGS Open-File Report 01-437 “Earthquake Spectral Response Acceleration Maps” Version 3.10 as specified by the building code authority.

Period computations must employ rational methods. The empirical equations for buildings are not applicable to storage racks, and cannot be used. There is no restriction on the period thus computed (ASCE 7 15.4.4). In the down-aisle direction storage racks, typically, have much higher drifts than buildings, resulting in much longer periods than a building.

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 2.6: 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 2.6.6..

g = acceleration due to gravity (386.4 in/sec^2) (9.81 m/s^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 shown here:

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

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 of many methods 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:

where:

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

K_x = Effective length factor for story buckling in the down aisle direction as determined from Section 6.3.1.1.

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.

An alternate acceptable method of computing the period is provided in FEMA 460 [5] using the rotational stiffness F from Section 9.4.2.3.

Minimum seismic response coefficient

The International Building Code references ASCE-7 [6] which requires that racks designed with the provisions of the RMI Specification have a minimum base shear coefficient of 0.14 S_{DS} This minimum was imposed pending tests of the connections for rotational capacity.

The testing has been performed, and rack connections are more than adequate to resist the rotational demands made upon them. Indeed, many have similar rotational capacities to meet the drift demand of what was once called a “ductile” moment frame in buildings.

The minimum seismic base shear equation is

$$0.044S_{DS} \leq C_S$$

2.6.3.1 Site Coefficients and Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameters

The 2008 edition of the RMI Specification utilizes spectral response seismic design maps that reflect seismic hazards on the basis of contours. These maps were developed by the United States Geological Survey (USGS) and were updated in 2005. The USGS also developed a companion software program that calculates spectral values for a specific site based on a site's longitude, latitude and site soil classification. The software program is the preferred method for establishing spectral values for design because the maps in Section 2.6.3.2 are too large a scale to provide accurate spectral values for most sites. The software program is available through the RMI website and the USGS website as shown below:

RMI Specifications and Technical Links page
http://www.mhia.org/industrygroups/rmi/specifications

USGS Seismic Design Values for Buildings
http://earthquake.usgs.gov/research/hazmaps/design/

2.6.4 Connection Rotational Capacity

This section resulted from the report done in FEMA 460 [5]. C_d is the deflection amplification factor for a moment resisting frame and is obtained from Table 15.4-1 in ASCE 7 [6].

The connection rotational capacity must exceed the maximum rotational demand. The demand may be computed directly using known earthquake records scaled in accordance with ASCE 7, 16.1.4, as is done for buildings. This reduces the uncertainties in establishing α and Cd. Where available, as with buildings, such computations may be in lieu of the section 2.6.4 requirements. At present, such analyses are not currently practical for everyday design office use.

As a simplification, the demand equation in this section is an upper bound based on the assumption that the column and beam deformations are very small relative to the deflections due to connector rotation. The basic connector rotational demand may then be taken as the maximum earth displacement divided by the height of the rack (the top level is assumed stationary).

While perhaps convenient, this formulation may obscure the origin of the displacement demand. It arises from expected maximum displacement of the ground, and is not any function of the structure itself. While not obvious, this formula is derived from ASCE 7 equation 17.5-3 (which was used in developing the FEMA 460 Appendix A equations). For example, at the Design Earthquake, the displacement demand would be:

$$C_d\Delta_s \equiv D = \frac{gS_{D1}T}{4\pi^2 B}$$ (the B values are identical in FEMA 460)

Where T is the effective period of the rack determined using the effective stiffness of the rack at displacement D that been appropriately modified to account for P-Delta effects.

Engineers may wish to employ this alternate formulation to the complex FEMA 460 calculations.

In the equation for the connection rotational demand the 1 + α term is to estimate the effects of P-Delta. Based on FEMA 460 the P-Delta amplification may be estimated as:

$$\alpha = \frac{\sum_{i=1}^{N_L} W_{pi} h_{pi} \left(\frac{k_c + k_{be}}{k_c k_{be}} \right)}{\left(N_c + N_b \left(\frac{k_b k_{ce}}{k_c k_{be}} \right) \left(\frac{k_c + k_{be}}{k_b + k_{ce}} \right) \right)}$$

where:

W_{pi} =The mass used in the calculation of the seismic force scaled by the effective horizontal seismic weight factor, 0.67.

W_{pi} = $(PL_{RF} xPL) + DL + 0.25xLL$

k_c = Rotational stiffness of each beam-to-upright connection from testing in Section 9.6

k_b =Rotational stiffness of each base plate connection (which may be assumed to be equal to k_c for installations where there is at least one anchor bolt on opposite sides of the column in the down-aisle direction)

N_c =Number of beam-to-upright connections

N_b =Number of base plate connections

k_{be} =beam end rotational stiffness assumed to be given by:

$$k_{be} = \frac{6EI_b}{L}$$

k_{ce} =bottom column end rotational stiffness assumed to be given by:

$$k_{ce} = \frac{4EIc}{H}$$

2.6.5 Seismic Displacement

The connection stiffness used for the design of the components, upright and beams, should be the connection secant stiffness from testing consistent with the base shear applied loads and resulting displacements. This will be a connection stiffness in the lower moment range. A possible starting stiffness could be the connection stiffness F from Section 9.4.

2.6.6 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 based on the recommendations of FEMA 460 [5].

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 (30.5 cm) 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 (30.5 cm) 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.

2.6.7 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.

2.6.8 Overturning

The overturning checks are intended for only anchor uplift and floor reactions. This specification requires two separate overturning checks. One is for the case of all storage positions loaded to 67% of the full rated capacity and the other for 100% in the top load position only.

The overturning checks must be done considering the lateral forces acting at the elevation of the center of mass of the loads.

2.6.9 Concurrent Forces

Considering the probabilities, it is not reasonable to expect that the effects of out-of-plumbness, impact, wind forces, and seismic events will 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 [1] and AISC [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 Section F1 of the AISI Specification [1].

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 [1] is quite specific about this in Section F1. It should be noted that confirmatory tests have a different nature and are covered in the AISI Specification [1] Section F2.

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

4 DESIGN OF STEEL ELEMENTS AND MEMBERS

Neither the AISI [1] nor the AISC [3] Specifications make provisions for perforated members 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 [1] bases the effective section properties on the unperforated section. Further information on the development of the AISI Specification [1] can be found in Reference 12.

4.1 COLD-FORMED STEEL MEMBERS

4.1.1 Properties of Sections

4.1.2 Flexural Members {The AISI (2001) [1] Section C3}

The RMI Specification approach involves the replacement of the section properties used in the AISI Specification [1] by the effective net section properties. The effective net section is the effective section determined based on 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 F_c. 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:

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

In the calculation of F_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, x_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.

4.1.3 Concentrically Loaded Compression Members {The AISI (2001) [1] Section C4}

4.1.3.1 Effective Area

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.

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 Figure 4.2.3-1. 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.

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 [1] 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 Reference12:

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

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

4.1.3.2 Distortional Buckling

Singly symmetric compression members may be subject to distortional buckling effects. Methods given in the AISI for unperforated sections may be used for sections with perforations. Other methods such as but not limited to finite element methods, structural testing are also acceptable.

4.2 HOT-ROLLED STEEL COLUMNS

5 BEAMS

5.1 CALCULATIONS

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 Section 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:

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

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

6.1 DEFINITION

6.2 GENERAL

6.3 EFFECTIVE LENGTHS

The AISI [1] and the AISC [3] Specifications 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 Reference [22]. 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 [1] and the AISC [3] Specifications 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.

This 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 3 and 22. The procedures described in this reference needs 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 to be reduced by $(I_b/L_b)_{red}$ due to the semi-rigid nature of the joints.

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

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 AISC [3].

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

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

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.

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 Figure 6.3.1.2 (a).) 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 designed 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 Figure 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 Figure 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 Section 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

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

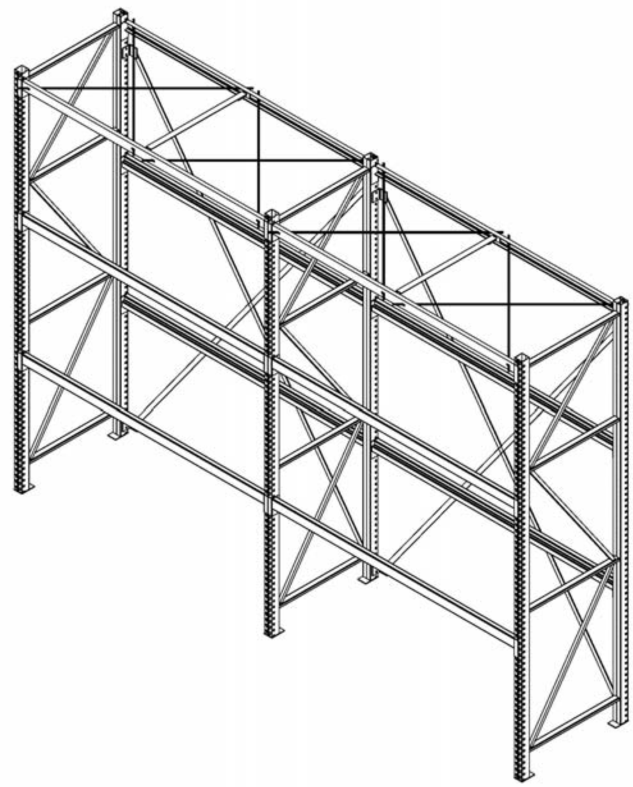


Figure 6.3.1.2 (a)

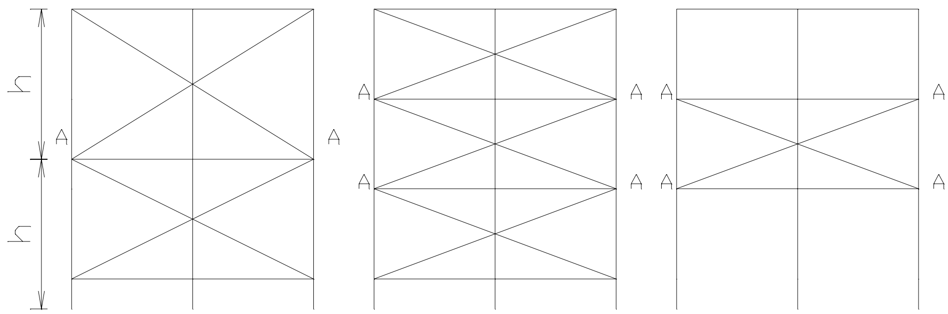
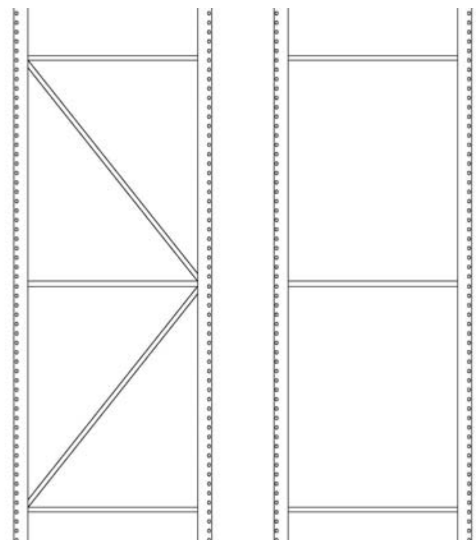


Figure 6.3.1.2-1 Racks Braced Against Sidesway

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 Figure 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 Figure 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 Figure 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 Figure 6.3.2-1 (b) is in general greater than one and can be found by rational analysis.



(a) (b)

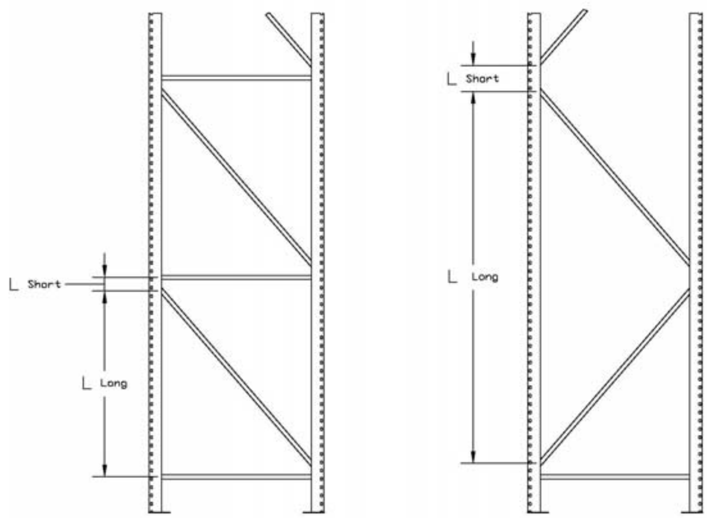
Figure 6.3.2-1
Braced and Unbraced Frames

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 Figures 6.3.2-1 (a) and 6.3.2-1 (b). The following three subsections treat various bracing configuration possibilities.

6.3.2.1

6.3.2.2 Frame Bracing Location

Upright Frames with Diagonal Braces or a Combination of Diagonal and Horizontal Braces that intersect the Columns are illustrated in Figures 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 Figure 6.3.2-2(b) and hence, the effective length factor can be greater than one.



(a) **(b)**

Figure 6.3.2-2
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 Figures 6.3.2-3 (a) and (b). As the ratio L_{short}/L_{long} increases, the basic behavior of the frame approaches that of Figure 6.3.2-3 (b) and hence the effective length factor can be greater than one.

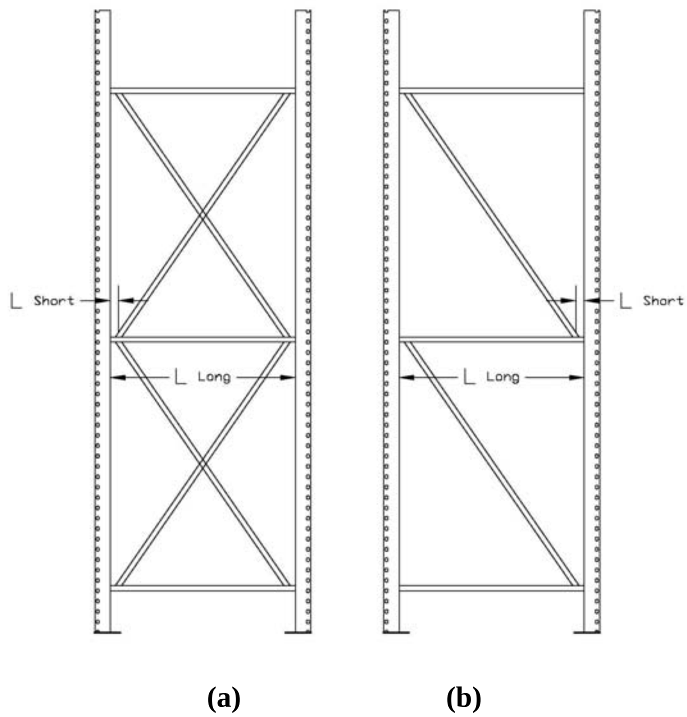


Figure 6.3.2-3
Upright Frames with Diagonal Braces that intersect the Horizontal Braces

6.3.2.4

For uprights having bracing patterns such as the configuration shown in Figure 6.3.21 (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 6.3.3 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 Figure 6.3.3-1.

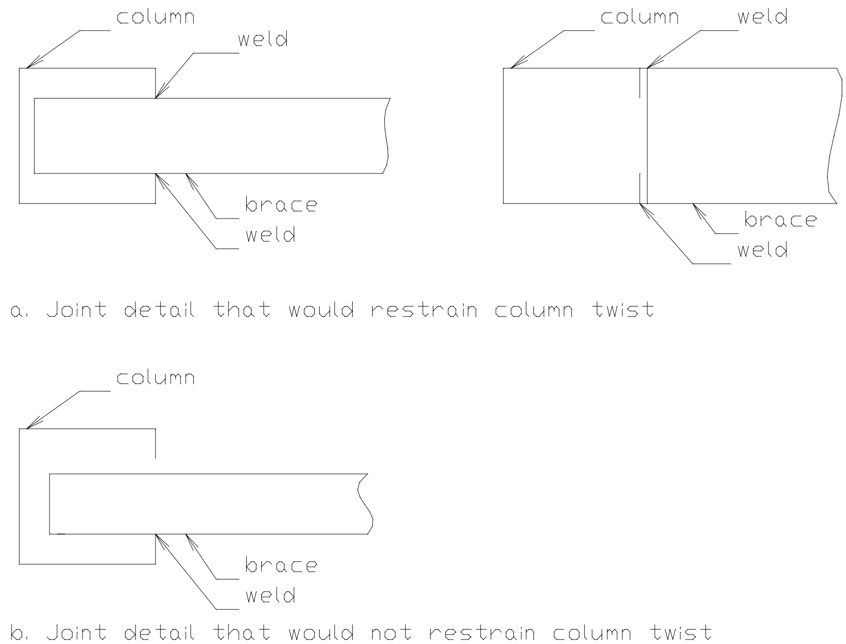


Figure 6.3.3-1 Joint details

6.3.4 Diagonals and Horizontals

The design procedures for upright frames in the cross-aisle, or transverse, direction should include the design detailing of the structural connections in those frames. Typically, the diagonal and horizontal framing members, often arranged in a truss-like configuration, frame into the front and rear columns of the frame, as well as into or onto one another. The framing members are members, of closed or open cross section, which are inserted into the open sections of the front and rear open column cross sections. The column channel sections may be some variation of C-sections, with and without stiffening legs, which may have, in turn, additional return stiffening elements to stiffen the reinforcing legs. There exist a large variety of combinations of horizontal and diagonal member cross sections, framing into and onto one another, and their various internal framing arrangements, framing into or onto column sections, and welded or bolted in a variety of patterns. Because of the large number of proprietary combinations, each manufacturer has a responsibility to provide the documentation of the adequacy of their connection designs to the Authority Having Jurisdiction. This documentation may take the form of a detailed analytical procedure demonstrating the adequacy of the joints within the context of Section 6.5.2 of FEMA 460[4]. Alternatively, the results of a testing protocol for the frames subjected to forces in the plane of the frames in the cross-aisle or transverse direction may be undertaken.

The analysis and design of the upright frame joints (or connections) shall include a consideration of the transfer of the member forces into and through those joints along with their connections and the deformation of the member legs, lips, and stiffening elements that make up the cross section of the various members coming into each joint. It is recognized that under large forces caused by seismic loads, these joints will behave in a manner that allows inelastic deformation of the members as well as their joints and distortion of their cross sections. Inelastic deformations that result from seismic demand contribute to the overall energy-absorbing and energy-dissipating structural behavior of the overall rack system, a mechanism that helps the rack systems to survive while continuing to carry their product loads.

The detailed analysis of the members, because of the complex nature of those joints as described above, is often not amenable to rigorous analysis. Alternatively, a testing protocol discussed in Section 6.5.3 of FEMA 460 [4], based on the work of Krawinkler, may be undertaken to demonstrate the adequacy of the rack structural system, including all the members and their joints, subjected to transverse loadings. A report of the results of such tests shall provide the basis of the documentation of the adequacy, along with the stiffness and ductility of the connection joints. Joints of rack upright frames are complex, varied, often proprietary, and usually not amenable to rigorous stress analysis or structural analysis. Under static loading conditions, and particularly under dynamic or seismic loading conditions, the stiffness and ductility properties may enable structural performance into the nonlinear inelastic regions. These complex behaviors contribute to the energy-absorbing and energy-dissipating damping processes that allow rack structures to withstand the applied forces, dissipate energy without shedding their loads, and to survive the design-level earthquakes in order to carry their products safely for another day. The processes discussed here are the beginning of the development of performance-based design of such systems.

6.4 STABILITY OF TRUSSED-BRACED UPRIGHT FRAMES

The provisions of this section are based on Reference 23 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. (Reference Section 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 COLUMN BASE PLATES

7.2.1 Bearing on Concrete

Formulas for determining the maximum permissible bearing stress (ASD) or load (LRFD) on the concrete floor are given in the specification. These resultant values may be used to design the column base plates unless the concrete floor designer requires a larger bearing area.

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. For bearing surfaces other than concrete, special design is required.

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

7.2.2 Base Plate Design

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

Welds from the base plate to the column should be adequate to properly transfer all loads. When analysis indicates, the bearing plate and welds to the rack column shall be designed for uplift forces and/or bending moments.

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

This edition of the specification contains new detailed methods for calculating the required thickness of the column base plates. Three load cases are considered: 1) downward vertical load; 2) uplift; and 3) axial load plus bending.

The provision to determine base plate thickness by load test has been retained from the previous edition without any change.

7.2.3 Maximum Considered Earthquake Rotation

The base shall have the rotational capacity of not less than Θ_D of the beam-to-column connection. This is because the deflected column is modeled as a straight line in FEMA 460. This rotational stiffness can be determined by rational analysis or by testing. If no analysis or testing is done then the base shall be considered as a pinned connection.

7.2.4 Shims

Since shims are required to maintain plumbness of the columns, it is necessary that the shims remain in position. The shims may be either restrained by welding them together or by bolting them to the floor, or by using nestable shims. They must be fixed in position so that through normal usage they can not be dislodged by fork trucks or other equipment. There should be no coating on the shims because the coating may reduce the friction between the shim

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 2.5), earthquake (Section 2.6) 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 to 1 ratio is exceeded, the rack must be analyzed for overturning even in the absence of seismic and wind forces. A 350 pound (159kg) 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 to 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 require 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.

8.4 PICK MODULES AND RACK SUPPORTED PLATFORMS

Pick modules are found in warehouse and distribution centers and allow rapid throughput of product. They are customized multi-level racks that support one or more product storage bays having a fork truck aisle on one side and a pick aisle floor on the opposite side. Pallets or products are generally inserted into a product storage bay from fork trucks on the fork truck aisle side, and removed by workers from the pick aisle side. The pallets may either be stationary in the product storage bay or may flow toward the pick aisle floor.

Most pick modules are frame-beam style racks with integrated pick module walkways or platform levels that are used by authorized or trained order picking personnel for the loading and unloading of products. These structures are intended to be in an industrial distribution environment and are not open to the general public. Pick modules are free standing structures within a warehouse. The pick module walkways have flooring, guardrails, stairways, and often have conveyor systems that deliver and/or remove products. These structures should be designed using the provisions of this Specification. This section is intended to provide special provisions for these structures that are needed in addition to the requirements of the rest of the Specification.

Rack-supported platforms have elevated platforms like pick modules but the platforms may be more wide open and involve other activities in addition to order picking.

8.4.1 Posting of Design Loads

The design loads for a rack-supported platform or pick module walkway should be on the rack configuration and load application drawings. The design loads should also be posted on the structure and serve as a reminder to the users of the load limit for the pick module walkway or rack-supported platform.

8.4.2 Design Requirements

The minimum pick module walkway design live load of 60 psf (293kg/m²) is given to support the order picking personnel. The user should advise the designer if there are to be such activities or equipment on the pick module floor that would require a higher design load. Also the conveyor live and product loads, dead loads and any other equipment or fixtures that are on the platform floor should be considered such as lighting, sprinkler piping, etc.

When the project specifications require a design live load of more than 100 psf (488 kg/m²) and there are more than two elevated floor levels, the Specification allows the designer to reduce the live load by 20 percent for the design of the support framing. The support framing includes the columns, the frame bracing, the frame bracing connections and the base plates. It does not include the platform support beams and their connections. It would be excessively conservative to require the columns (and support structure members) to be sized for all of the floor levels having all of the live loads present at the same time. This reduction only applies to the floor live load for the walkway areas. It does not apply for and other loads such as dead loads or product loads.

A tighter limit on live load beam deflection is required for floor supporting beams because the L/180 limit used for rack beams may result in too much deflection and could cause the floor to “bounce”. For this reason a rack manufacturer’s beam tables should not be used to select beams for platforms without proper consideration of deflection. The deflection from the total load may not exceed L/180. The designer may conservatively limit the total deflection to L/240 or check the deflection separately for both cases.

A 30” (76cm) minimum clear pick aisle walkway width is recommended to allow the order pickers the clearance to safely navigate the walkway and perform the picking operations.

8.4.3 Rack-Supported Platform and Pick Module Guards

Since pick modules and rack-supported platforms involve order picking personnel on elevated platforms or walkways adequate safety systems that provide fall protection for the workers must be in place and properly designed. The purpose of this section is to provide the requirements for the pick module guardrail and handrail systems and also the safety decking system if required. These are the most common systems used to provide fall protection on pick module structures. These systems are not intended to serve as a substitute for proper training and proper conduct of the workers who use these structures. Rather, they are intended to provide reasonable protection for workers who are working in accordance with the safety procedures to which they have been trained.

8.4.3.1 Guardrail Requirements

Because these are specialized structures that are not open to the general public and intended to be used by authorized or trained personnel, guardrails may be used instead of handrail systems for fall protection. On the stair assembly itself, however, handrail systems are to be provided. On stairways, the top guardrail may serve as a handrail if it meets all of the design requirements of a handrail. Handrails are not required on stair landings but guardrails must be used to provide 42” (107 cm) high fall protection on the stair landing. Intermediate landings that are provided in a straight continuous stairway may use handrail or guardrail. Kick-plates are required where the guardrails are used. They may also be required at additional places as required and specified by the owner such as under the charge side of floor-level carton flow shelves that are raised off the floor to create pitch. Often kick-plates are not required at edges because there may be an adjacent deck or structural element that would prevent product from sliding off the edge of the floor.

Many modules are designed to have static pallet drop-off locations on the elevated floor levels of the module. Where these are used the floor must be properly designed for the load weights and a gate, removable section of guardrail or removable chains must be used. These gates or removable guardrails (or chains) must be secured at all times except when a load is being picked up or deposited at the pallet drop location. Proper safety precautions must be adhered to at all times when opening and closing the guardrail section, gate or chains at the pallet drop-off location and when removing or depositing the loads. When removable chains are used the chains may not have excessive slack if they are to provide safe fall protection. For this reason a limit has been placed on the sag of chains. An intermediate chain must also be used as is required for guardrail systems. Kickplates are required where removable handrails or chains are used for the purpose of providing a load drop-off point where the loads are being placed into the module.

Because of the nature and use of these structures some exceptions to normal practice for guardrail and kick plate are needed. These exceptions are provided to avoid situations where guardrail or kick plates, etc. could actually create obstacles to the use of the structure, which could prove to be hazards rather than safety enhancements. However, care must be taken in the design to ensure that the occupants of the structure are safe when they are properly using the pick module or rack-supported platform.

8.4.3.2 Safety Flooring Requirements

Pick modules often contain product flow lanes. Because loads can sometimes hang up or not flow freely, safety flooring is recommended or required. Safety flooring is designed by the flooring manufacturer with the following specifications:

300# (136 kg) concentrated load (to support the picker),

Dynamic distributed load of 60 psf (293 kg/m²) acting separately, and

Any other issues necessary to protect both the picker and pick module.

Order pickers should have proper training and should follow the safety procedures that are established for stepping onto this safety flooring. An example of this procedure may be that the pickers should not walk on the outermost safety flooring load positions where they could fall from the module. The Specification limits this distance to 4 feet (1.22 m). These procedures will vary depending on the configuration of the structure and the working environment. It is not the purpose of this specification to establish the exact procedures, as they may vary, but rather to stress the importance of having safety procedures that are strictly followed. Under no circumstances should a picker climb or walk into the rack when safety flooring has not been provided for that purpose.

8.4.4 Stairways

The requirements for stairs in this section are intended to match the stairway requirements common to stairways that are required for an industrial environment. Building codes will often have requirements for stairways that are more stringent than those outlined in this section because such requirements are intended for stairways that are open to the general public.

Handrail systems are required for stairways. The handrail system may be guardrail if the top rail of the guardrail system meets the same as requirements as a stairway handrail.

Stair handrail extensions are not needed on module structures and can actually be obstacles to swift orderly egress during an emergency situation. This section recommends that stair handrail extensions not be used.

8.4.5 Product Fall Protection

There also may need to be systems in place to protect areas within or around the structure from products that could accidentally fall. These locations may be areas where people could be or areas where falling product could cause other types of property damage or safety hazards. These areas should be identified by the owner and brought to the attention of the designer and the proper barriers, if required, should be supplied and installed. These requirements will vary depending on the products, the operation and configuration of the structure.

8.5 AUTOMATED STORAGE AND RETRIEVAL SYSTEMS

9 TEST METHODS

9.1 GENERAL

Many factors affecting the design of rack are difficult to account for analytically. Section 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 [1] and AISC [3] Specifications 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 [2].

9.2.1 Test Specimen and Procedure

9.2.2 Evaluation of Test Results

Q is a factor used in Sections 4.1.2 and 4.1.3. 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 [1] 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

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

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 Specification [1] Section C4 the following needs attention: In calculating column strength according to AISI Specification [1] Section 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 [1] 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 Specification [1] Section C4.

For a series of columns having different thicknesses, the thickest and the thinnest may be tested. For any intermediate thickness, the Q so determined should be used in column strength calculations according to the AISI Specification [1] Section 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 Specification [1] that it can only be equal to or smaller than, but not larger than 1.0. Correspondingly, the Specifications provide 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 [1].

9.3.1.1 Test Setup

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

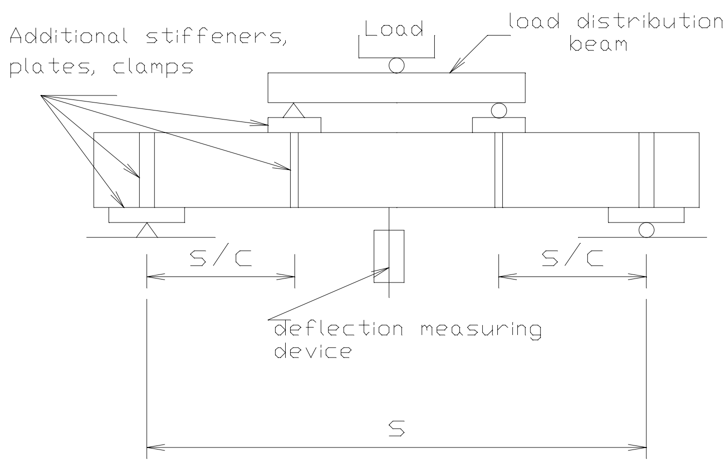


Figure 9.3.1-1 Simply-Supported Pallet Beam Tests.

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 Section 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 Section 9.4. For vertical loads this test may reflect the actual behavior of the connections more accurately than the test described in Section 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.

9.3.2.2 Test Procedure

9.3.2.3 Evaluation of Test Results

General guidelines given in Section 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.

9.3.2.4 Number of Tests Required

9.3.2.5 Deflection Test

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

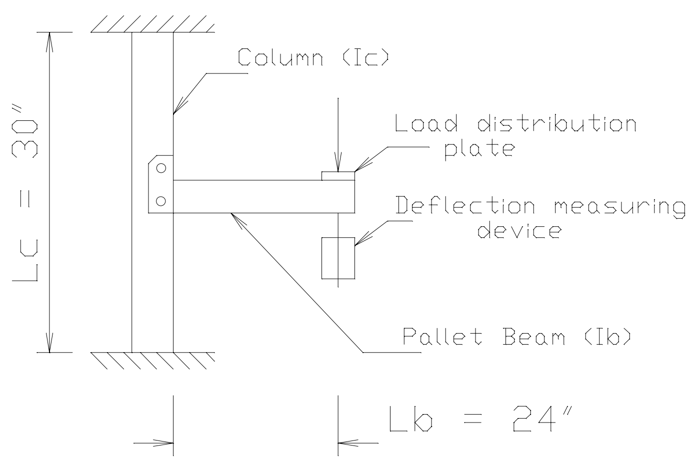
The Cantilever Test provides a simple means of determining the connection moment capacity and rigidity. However, it has the disadvantage that the ratio of shear force (that is the vertical reaction) to moment at the joint is not well represented. For typical rack connections this ratio is probably higher 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 Section 9.3.2.

9.4.1.1 Test Setup

This test setup illustrated in Figure 9.4.1.1-1



(Lc = 76 cm) (Lb = 61 cm)

Figure 9.4.1.1-1 Cantilever Test

9.4.1.2 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.

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

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 (61 cm).

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.

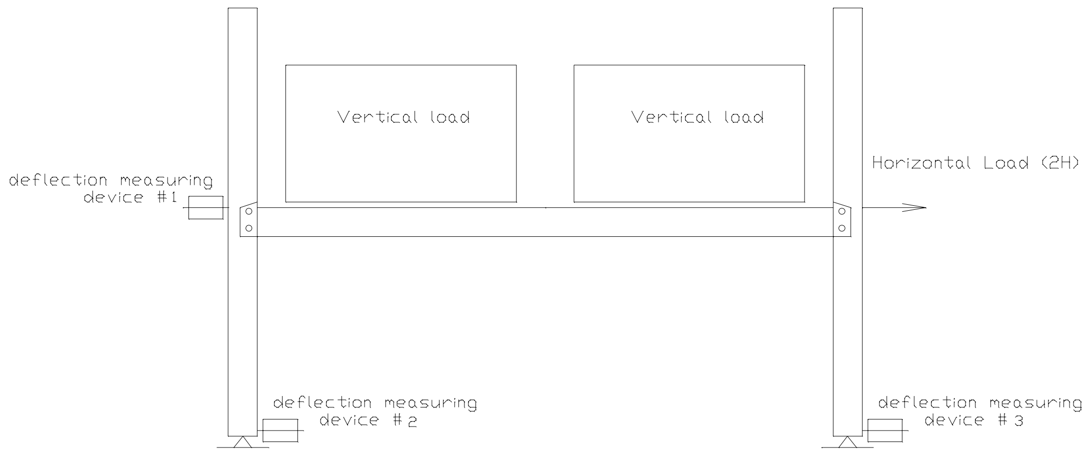


Figure 9.4.2.1-1 Portal test

9.4.2.2 Test Procedure

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:

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

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 Section 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.

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

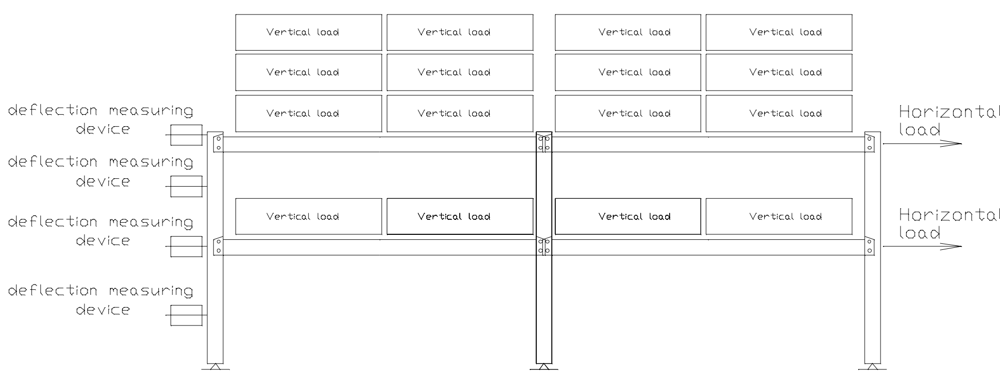


Figure 9.5.1.1.1 Test Setup

9.6 CYCLIC TESTING OF BEAM-TO-COLUMN CONNECTIONS

9.6.1 General

There has been much concern written or otherwise expressed by some members of the structural engineering community in the last several decades about rack structural behavior. However, the rack industry, through the Rack Manufacturers Institute, has worked long and hard with and through the various model code organizations, with the BSSC, with the ASCE, with the ICC and with the NFPA to have its products be covered rigorously but fairly by existing and evolving design provisions as applied to building-like

Nonbuilding Structures. It is known that rack structural systems that have been designed, permitted through a code-enforcement process, manufactured, installed, and utilized in accordance with applicable RMI provisions, have performed well in recent seismic events.

Storage rack structural systems are presently designed in accordance with the Rack Manufacturers Institute Specification for the Design, Testing, and Utilization of Industrial Steel Storage Rack, along with the added provisions of NEHRP [7] Section 14.3.5, ASCE 7 [6] Section 9.6.2.9, and IBC [8] Section 2208. The consequence of the added provisions as they appear in the NEHRP, ASCE, and IBC is to cause an upper limit or cap to be imposed on the period of rack structural behavior under seismic conditions. In turn, this causes artificially-large base shear forces to be predicted in the resulting structural analysis since the seismic behavior of racks during strong earthquake had not been rationally explained.

The imposition of inordinately large base shear forces has been the requirement since the early 1970’s, when the UBC first introduced provisions to be applied to seismic behavior of steel storage rack. The current cap that results from current provisions imposes an upper limit of 0.6 seconds on the period of the rack structural response where it is well known that typical storage rack may have periods of 2 to 4 seconds in the longitudinal direction. Further, it is well known that rack periods, rack damping, and overall rack structural behavior is very dependent on the beam-to-column connectors and connections and their moment-rotation characteristics that are the key and integral component of rack structures.

9.6.2 Definitions

The following definitions shall characterize the test set-up and the conduct of the test.

Complete Loading Cycle. A cycle of rotation taken from zero force to zero force, including one positive and one negative peak.

Drift Angle. Displacement divided by height, radians.

Inelastic Rotation. The permanent or plastic portion of the rotation angle between a beam and a column of the Test Specimen, measured in radians. The Inelastic Rotation shall be computed based on an analysis of the Test Specimen deformations. Sources of inelastic rotation include yielding of members, yielding of connection elements and connectors, and slip between members and connection elements. For beam-to-column moment connections in Moment Frames, inelastic rotation shall be computed based upon the assumption that inelastic action is concentrated at a single point located at the intersection of the centerline of the beam with the centerline of the column.

Prototype. The connections, member sizes, steel properties, and other design, detailing, and construction features to be used in the actual storage rack frames.

Test Specimen. A portion of a frame used for laboratory testing, intended to model the prototype.

Test Setup. The supporting fixtures, loading equipment, and lateral bracing used to support the load and Test Specimen.

Test Subassemblage. The combination of the Test Specimen and pertinent portions of the Test Setup.

9.6.3 Test Subassemblage Requirements

The Test Subassemblage shall replicate as closely as is practicable the conditions that will occur in the Prototype during earthquake loading. The Test Subassemblage shall include the following features:

(1) The Test Specimen shall consist of at least a single column element with beam segments attached to both sides of the column.

(2) Points of inflection in the test subassemblage shall coincide approximately with the anticipated points of inflection in the prototype under earthquake loading.

(3) Lateral bracing of the test subassemblage is permitted near load application or reaction points as needed to provide lateral stability of the Test Subassemblage. Additional lateral bracing of the Test Subassemblage is not permitted, unless it replicates bracing to be used in the Prototype.

9.6.4 Essential Test Variables

The Test Specimen shall replicate as closely as is practicable the pertinent design, detailing, and construction features, and the material properties of the Prototype. The following variables shall be replicated in the Test Specimen.

9.6.4.1 Sources of Inelastic Rotation

Inelastic Rotation shall be developed in the Test Specimen by inelastic action in the same members and connection elements as anticipated in the prototype, i.e., in the beam, in the column, in the panel zone, or within the connection elements. The fraction of the total Inelastic Rotation in the Test Specimen that is developed in each member or connection element shall be at least seventy-five percent of the anticipated fraction of the total Inelastic Rotation in the Prototype that is developed in the corresponding member or connection element.

9.6.4.2 Size of Members

The size of the beams used in the Test Specimen shall be representative of typical full-size storage rack beams.

The size of the columns used in the Test Specimen shall be representative of typical full-size storage rack columns, and shall properly represent the inelastic action in the column, as defined in Section 9.6.3 (1).

Extrapolation beyond the limitations stated in this section shall be permitted subject to qualified peer review and approval by the Authority Having Jurisdiction.

9.6.4.3 Connection Details

The beam-to-column connectors and the connection details used in the Test Specimen shall represent the Prototype connection details as closely as possible. The connection elements used in the Test Specimen shall be full-size typical connectors and connection elements used in the Prototype and in typical storage rack installations, for the member sizes being tested.

9.6.4.4 Material Strength

The following additional requirements shall be satisfied for each member of the connection element of the Test Specimen that contributes to Inelastic Rotation at yielding.

(a) The yield stress shall be determined by material tests on the actual materials used for the Test Specimen, as specified in the Section below on Materials Testing. Because of the amount of cold-working to which the connector is subjected in manufacture and testing, the yield stress for connectors will be determined from connectors taken from identical neighboring components in the manufacturing sequence. The use of yield stress values that are reported on certified mill test reports are not permitted to be used for purposes of this Section.

(b) The yield stress of the beam shall not be more than 15 percent below R_yF_y for the grade of steel to be used for the corresponding elements of the Prototype. Columns, connectors, and connector elements with a tested yield stress shall not be more than 15 percent above or below R_yF_y for the grade of steel to be used for the corresponding elements of the Prototype. R_yF_y shall be determined in accordance with Section 6.2 of AISC Seismic [23]. Here, F_y is the minimum specified yield strength; and R_y is the ratio of the expected yield strength to the minimum expected yield strength F_y.

9.6.4.5 Welds

Welds on the Test Specimen shall satisfy and be performed in strict conformance with the requirements of Welding Procedure Specifications (WPS) as required.

9.6.4.6 Bolts

The bolted portions of the Test Specimen shall replicate the bolted portions of the Prototype connection as closely as possible.

(a) The bolt grade used in the Test Specimen shall be the same as that used for the Prototype.

(b) The type and orientation of bolt holes used in the Test Specimen shall be the same as those to be used for the corresponding bolt holes in the Prototype.

(c) When inelastic rotation is to be developed either by yielding or by slip within a bolted portion of the connection, the method used to make the bolt holes in the

Test Specimen shall be the same as that to be used in the corresponding bolt holes in the prototype.

(d) Bolts in the Test Specimen shall have the same installation and faying surface preparation as that to be used for the corresponding bolts in the Prototype.

9.6.5 Testing Procedure

Section 9.4 of the RMI Specification presents a testing and evaluation protocol intended to evaluate the characteristics of typical rack beam-to-column connections. These tests are to be executed on behalf of each storage rack manufacturer in order to determine and evaluate the moment/rotation stiffnesses and their limiting values for their various beam-to-column connectors. This testing protocol is based on FEMA 350 Table 3-14 scaled up by a factor appropriate for rack beam-to-column connectors, These characteristics, when evaluated in a dependable and reproducible manner by an independent testing laboratory, will then become the basis for the removal or modification of the present cap on rack structural period. A more reasonable value of period will be used to calculate a more reasonable prediction of rack structural behavior, including drift, which is more representative of the response of real systems in the field under seismic conditions. Typical rack behavior will be tested with beam-to-column rotations of up to 0.1 radians, with up to five cycles, and will be representative of rack structures having displacements resulting in drift of h/50. Following the last cycle of the cyclic tests, the moment/rotation behavior will be recorded to failure where the rotation will be on the order of 0.3 radians.

Rack beam-to-column connectors normally exhibit a large degree of ductility in response to demand placed on such connections. For example, assuming a drift index of 0.02 (h/50) which is about the most ever seen on a shake table, the demand rotation would be 0.04 radians. This is because shake tables have not had the displacement capacity to test actual earthquake motions in the 2 to 4 second period range. However, rack connections can achieve failure rotations of 0.2 to 0.3 radians, some ten times the drift index. Comparing this to a building structure, the UBC requires that joints accommodate a drift of 0.0025 at 0.03 radians for a ductile frame, and around 0.015 for an “ordinary” moment frame, which is six times the building drift. Rack connections generally exhibit more ductility than any representative building connection. This capacity is needed since demand on rack connections is many times the demand on building structural connections, so it is quite possible for a rack connection with a capacity of 0.10 radians to be inadequate.

The FEMA/AISC testing protocol requires a large number of cycles leading finally up to approximately 0.03 radians, where but 1 to 3 cycles are needed. For a rack beam-to-column connection, such a large number of cycles could be excessive. Thus, to cut down on testing time, it is proposed that connections be cycled as shown in Commentary Section 9.6.6.1

The testing program should include tests of at least two specimens of for each combination of beam and column and connector size. The results of the tests should be capable of predicting the median value of drift angle capacity for the performance states described below. The drift angle limits θ for various performance levels shall be defined as indicated in the following figure.

Performance Level	Symbol	Drift Angle Capacity
Strength degradation	θ_{SD}	Taken as the value of θ, from the following Figure, at which either failure of the connection occurs or the strength of either connection degrades to less than the nominal capacity, whichever is less.
Ultimate	θ_U	Taken as the value of θ, from the following Figure, at which connection damage is so severe that continued ability to remain stable under gravity loading is uncertain.

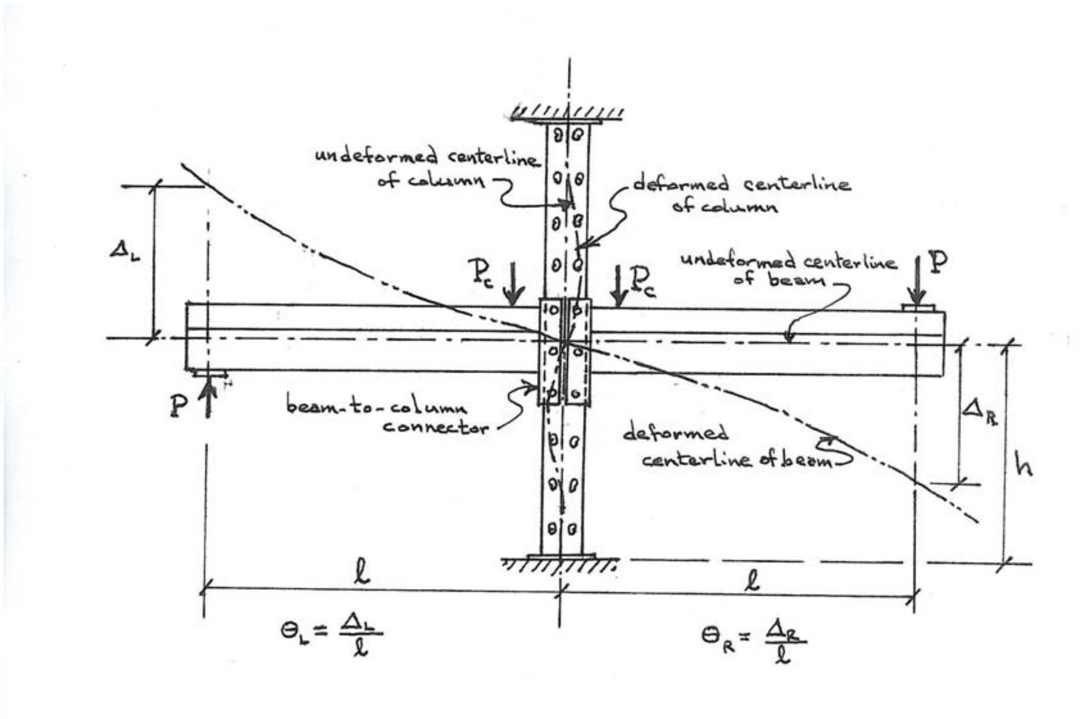


Figure 9.6.5-1 Test Setup

9.6.6 Loading History

9.6.6.1 General Requirements

Prior to the application of any cyclic loading, a constant downward load, P_c, of one kip shall be applied to each beam segment adjacent to each connector on both sides of the beam-to-column connection simulating the design downward-acting gravity pallet loads that serve to fully engage the beams and their connectors into the columns receiving them.

Loading will proceed with the application of equal displacements at each end of each beam, and the measurement of the force corresponding to each such displacement. Thus, the testing setup and apparatus requires the use of two independent actuators to measure the two different forces being developed at the two beam-ends where equal displacement are being applied.

The Test Specimen shall be subjected to cyclic loads according to the requirements prescribed for beam-to-column moment connections in Moment Frames. Loading sequences other than those specified here may be used when they are demonstrated to be of equivalent or greater severity.

(2) Loading Sequence for Storage-Rack Beam-to-Column Connections

Qualifying cyclic tests of storage-rack beam-to-column connections shall be conducted by controlling the peak Drift Angle, θ, imposed on the Test Specimen as follows:

Load Step #	
(1)	3 cycles at θ = 0.025 radians
(2)	3 cycles at θ = 0.050 radians
(3)	3 cycles at θ = 0.075 radians
(4)	3 cycles at θ = 0.100 radians
(5)	2 cycles at θ = 0.150 radians
(6)	2 cycles at θ = 0.200 radians

Continue loading at increments of θ = 0.050 radians, with two cycles of loading at each step.

9.6.7 Instrumentation

Sufficient instrumentation shall be provided on the Test Specimen to permit measurement or calculation of the quantities listed in the Section on Test Reporting Requirements that follows.

9.6.8 Material Testing Requirements

9.6.8.1 Tension Testing Requirements

Tension testing shall be conducted on samples of steel taken from the material adjacent to each Test Specimen. Tension-test results from certified mill test reports shall be reported but are not permitted to be used in place of specimen testing for the purposes of this Section

Tension-test results shall be based upon testing that is conducted in accordance with the Section on Methods of Tension Testing. Tension testing shall be conducted and reported for the following portions of the Test Specimen:

(a) Flange(s) and web(s) of beams and columns at standard locations.

(b) Any element of the connector that contributes to Inelastic Rotation by yielding.

9.6.8.2 Methods of Tension Testing

Tension testing shall be conducted in accordance with the appropriate ASTM testing protocols for the particular materials being used, with the following exceptions:

(a) The yield stress F_y that is reported from the test shall be based upon the yield strength definition in ASTM A370, using the offset method at 0.002 strain.

(b) The loading rate for the tension test shall replicate, as closely as practicable, the loading rate to be used in the Test Specimen.

9.6.9 Test Reporting Requirements

For each Test Specimen, a written test report meeting the requirements of the Authority Having Jurisdiction and the requirements of this Section shall be prepared. Some of these items may come from the manufacturer of the sample and others from the testing laboratory. The report shall thoroughly document all key features and results of the test. The report shall include the following information:

(1) A drawing or clear description of the Test Subassemblage, including key dimensions, boundary conditions at loading and reaction points, and location of any lateral braces.

(2) A drawing of the connector and connection details, showing member sizes, grades of steel, the sizes of all connector and connection elements, welding details including any filler metals, the sizes and locations of any slots or bolt holes, the size and grade of bolts, and all other pertinent details of the connection.

(3) A listing of all other Essential Variables for the Test Specimen, as listed in the Section on Essential Test Variables.

(4) A listing or plot showing the applied load and displacement history of the Test Specimen.

(5) A plot of the applied load versus the displacement of the Test Specimen. The displacement reported in this plot shall be measured at or near the point of load application. The locations on the Test Specimen where the loads and displacements were measured shall be clearly identified.

(6) A plot of Beam Moment versus Drift Angle for beam-to-column moment connections. For beam-to-column connections, the beam moment and the Drift Angle shall be computed with respect to the centerline of the column.

(7) The Drift Angle and the total Inelastic Rotation developed by the Test Specimen. The components of the Test Specimen contributing to the total Inelastic Rotation due to yielding or slip shall be identified. The portion of the total Inelastic Rotation contributed by each component of the Test Specimen shall be reported. The method used to compute Inelastic Rotations shall be clearly shown.

(8) A chronological listing of significant test observations, including observations of yielding, slip, instability, tearing, and fracture of any portion of the Test Specimen, as applicable.

(9) The controlling failure mode for the Test Specimen. If the test is terminated prior to failure, the reason for terminating the test shall be clearly indicated.

(10) The results of the material tests specified under Material Testing Requirements, above.

(11) The Welding Procedure Specifications (WPS) and welding inspection reports.

Additional drawings, data, photographs, and discussion of the Test Specimen or test results are permitted to be included in the report.

9.6.10 Acceptance Criteria

The Test Specimen must satisfy the Strength and Drift Angle requirements of this protocol for the connection, as applicable. The Test Specimen must sustain the required Drift Angle for at least one complete loading cycle. The test results will also include the beam-to-column moment-rotation characteristics and "dynamic spring relationship" for each of the combinations tested.

Thus, a process is presented herein by which the structural beam-to-column connections will be evaluated by series of tests conducted by an independent testing laboratory. While many of the rack manufacturers use cold-formed light-gauge structural sections for their rack structural systems; the procedure presented herein is equally applicable to systems employing hot-rolled sections. The intent of this proposal, in the absence of other provisions, is to apply and mimic the test procedures which have developed for connection behavior of hot-rolled structural sections as articulated in FEMA 350 [24], AISC Seismic Provisions [23], ATC 19 (1995) [25], ATC 24 [26], and the SEAOC Blue Book (1999).

10 REFERENCES

***Note**: The number within brackets [x] signifies the reference number in the MH16.1-2008 specification.*

[1] **1.** AISI (2001), North American Specification for the Design of Cold-Formed Steel Structural Members. American Iron and Steel Institute, Washington, DC, 2001 Edition.

[3] **2.** Cold-Formed Steel Design Manual, American Iron and Steel Institute, Washington, DC, 1996 Edition

[2] **3**. AISC (2005), Specification for Structural Steel Buildings, ANSI/AISC 360-05 An American National Standard, American Institute of Steel Construction, Chicago, Ill, March 9,2005.

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