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

Rack Manufacturers Institute

RACK MANUFACTURERS INSTITUTE

Commentary on the Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks - 2008 Edition

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MATERIAL HANDLING INDUSTRY OF AMERICA®
The Alliance of Material Handling Equipment, Systems, and Service Providers

PREFACE

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

It is important to bear in mind that the 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

2008 EDITION

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.

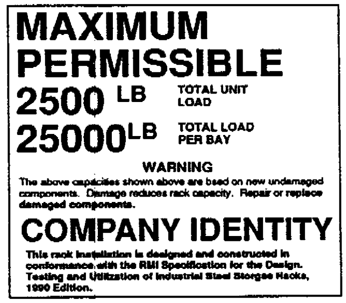


Figure 1.4.2a Example of Load Capacity and Compliance Plaque.
(1134 kg Total Unit Load)
(11340 kg Total Load per Bay)

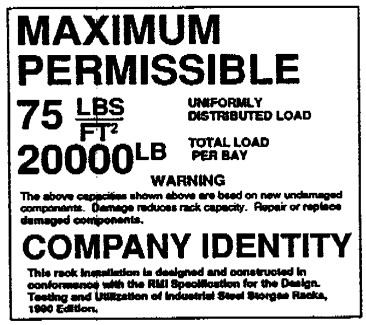


Figure 1.4.2b Example of Load Capacity and Compliance Plaque.
(366 kg/m^2 Uniformly Distributed Load)
(9072 kg Total Load per Bay)

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.

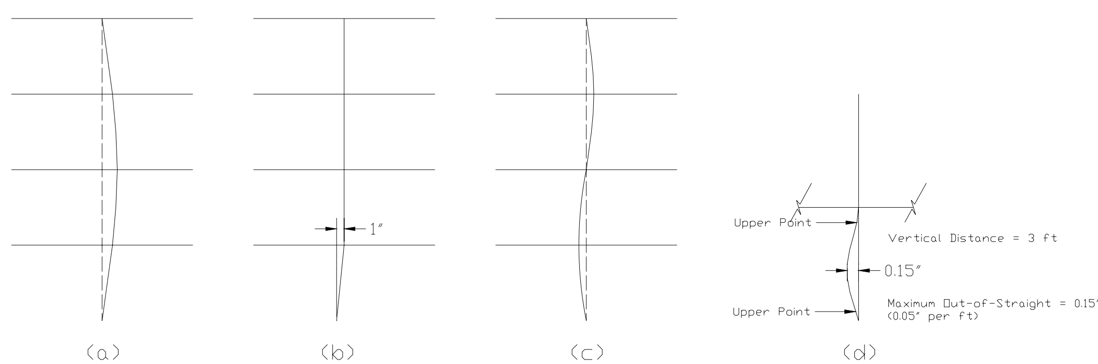


Figure 1.4.11-1

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 $\phi = 0.95$

$$1.587 / 0.95 = 1.67$$

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 PΔ 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-Δ 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 PΔ 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 PΔ 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

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

Equation:

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 $P\Delta$ 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:

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

where:

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

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

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 on the CD-ROM that is included with this specification and it may also be accessed at the USGS web site www.eqhazmaps.usgs.gov or through the RMI website at www.MHIA.org/RMI.

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 C_d. 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:

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

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 $1 + \alpha$ 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.

$$W_{pi} = (PL_{RF} x PL) + DL + 0.25 x LL$$

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:

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

M_e = the beam end moment

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

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

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

where:

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

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

5.3 DEFLECTIONS.

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

6 UPRIGHT FRAME DESIGN

6.1 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.

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

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

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

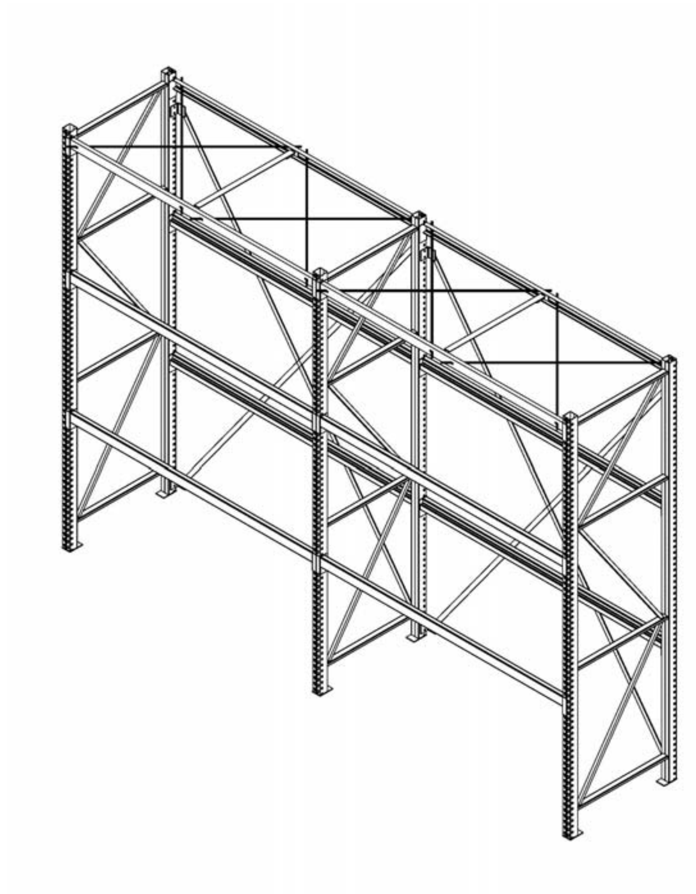


Figure 6.3.1.2 (a)

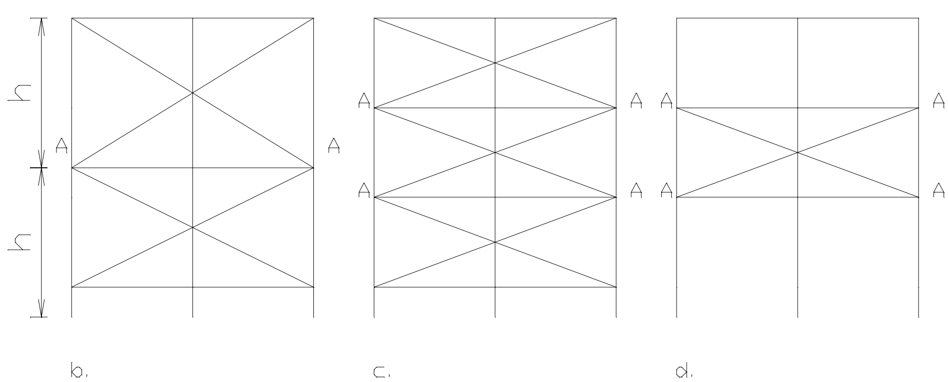
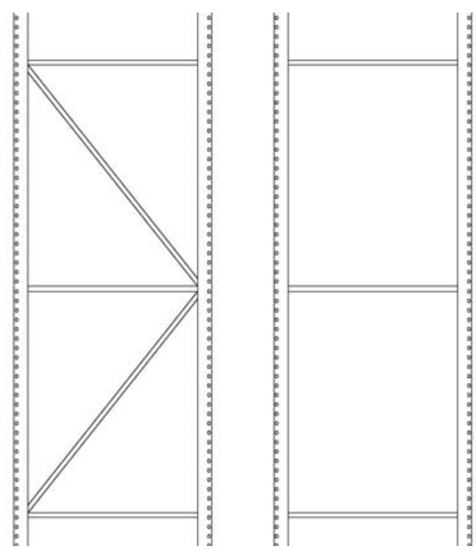


Figure 6.3.1.2-1 Racks Braced Against Sidesway

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)**

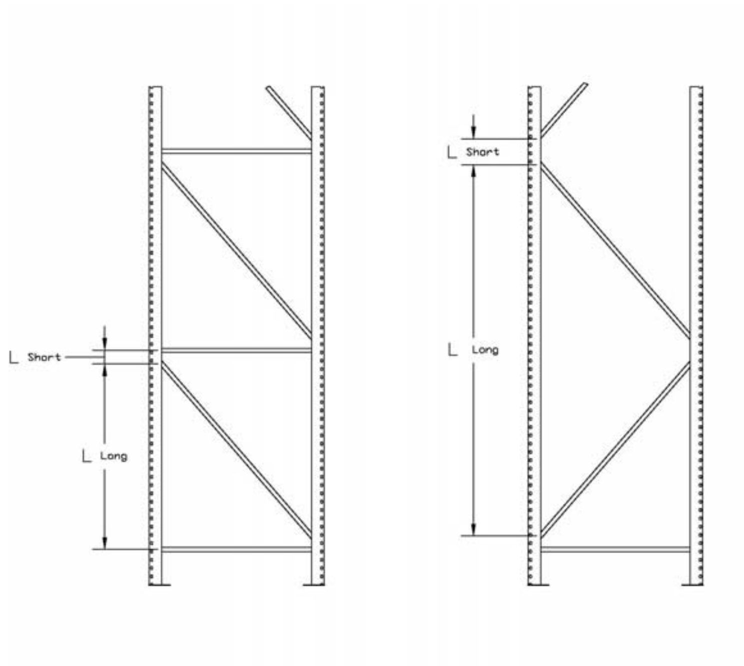
Figure 6.3.2-1
Braced and Unbraced Frames

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.



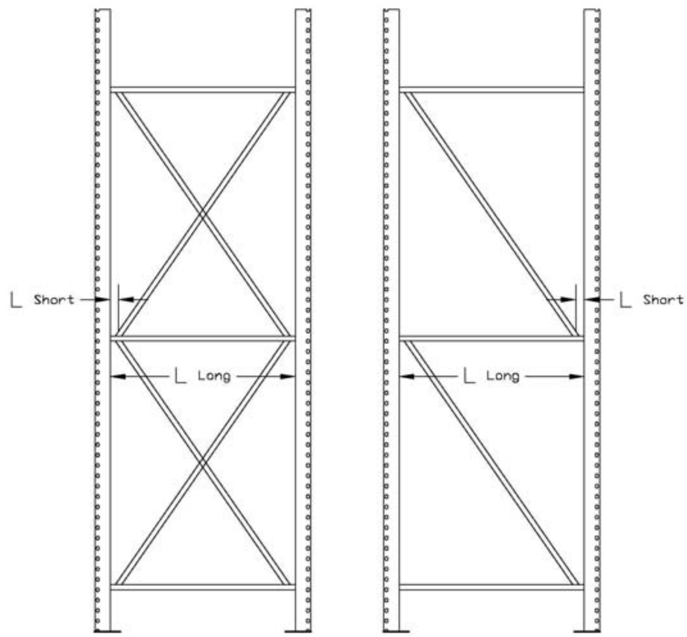
(a) **(b)**

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.



(a) **(b)**

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.

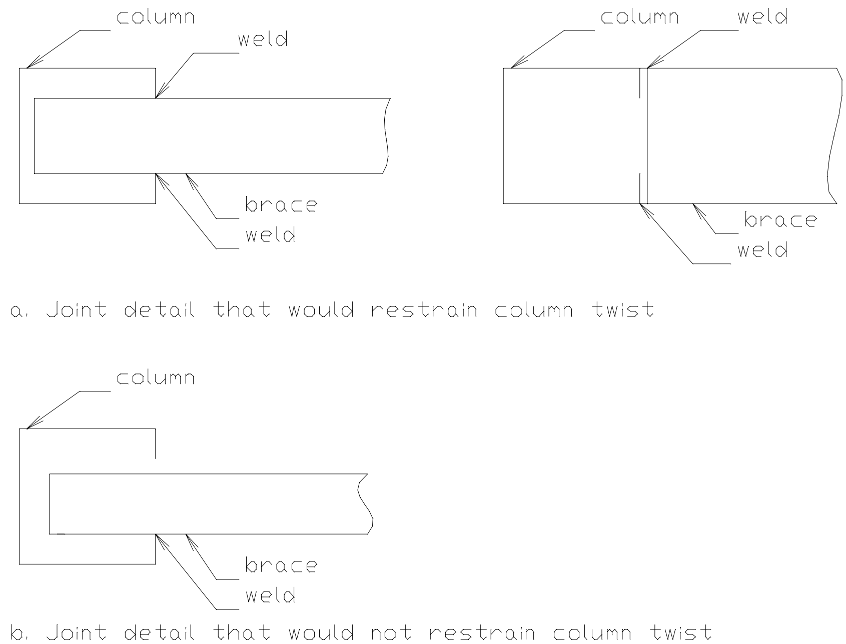


Figure 6.3.3-1 Joint details

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 Reference23 with the exception of the value of K. The expressions given in the reference were for members that have constant axial force throughout their entire length. The effective length factor K is intended to modify these expressions for the case of non-uniform distribution of axial forces. The provisions of this section are more likely to govern for high rise racks.

7 CONNECTIONS AND BEARING PLATES

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

7.1 CONNECTIONS

7.1.1 General

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

The effects of eccentricity of the connection and the effect of rotation of an attachment to the edge of an unstiffened flange must be evaluated. The influence of these connections on the overall behavior is significant. (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 shims..

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 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 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 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” 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" 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# concentrated load (to support the picker),

Dynamic distributed load of 60 psf 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. 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

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

In regard to the yield strength F_y to be used by determining Q by test, and the value F_y for calculating the strength of columns according to AISI 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.

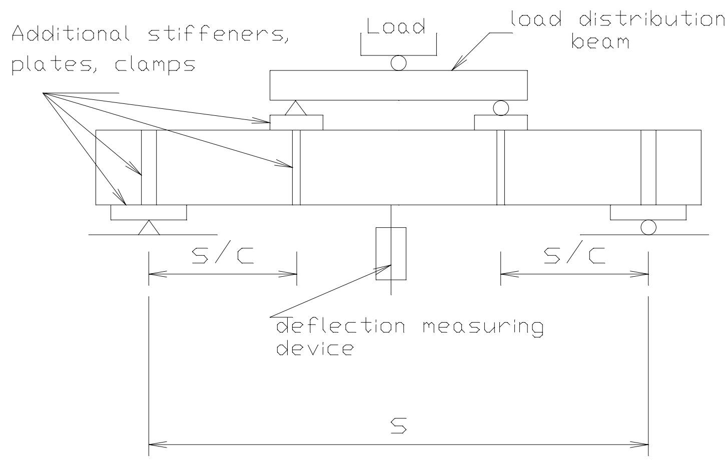


Figure 9.3.1-1 Simply-Supported Pallet Beam Tests.

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.

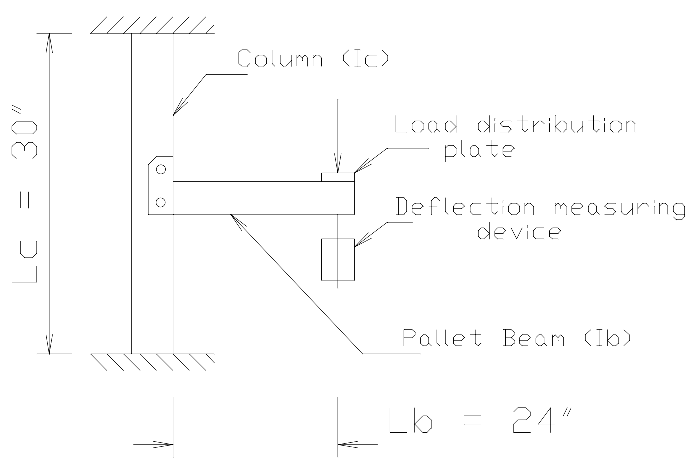
The Cantilever Test provides a simple means of determining the connection moment capacity and rigidity. However, it has the disadvantage that the ratio of shear force (that is the vertical reaction) to moment at the joint is not well represented. For typical rack connections this ratio is probably higher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

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

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

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

9.4.2 The Portal Test.

The portal test is desirable when the value of F obtained is to be used in a sidesway analysis either for lateral deflections or stability. Under vertical loads the connections in general "tighten up". Subsequently, under sidesway, the connection at one end of the beam "tightens up" while the connection at the other end "loosens." The portal test gives an approximate average value of the spring constants involved in the process. Thus it is more desirable to use the portal test for evaluating sidesway behavior, namely, the effective lengths and horizontal deflections.

9.4.2.1 Test Setup.

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

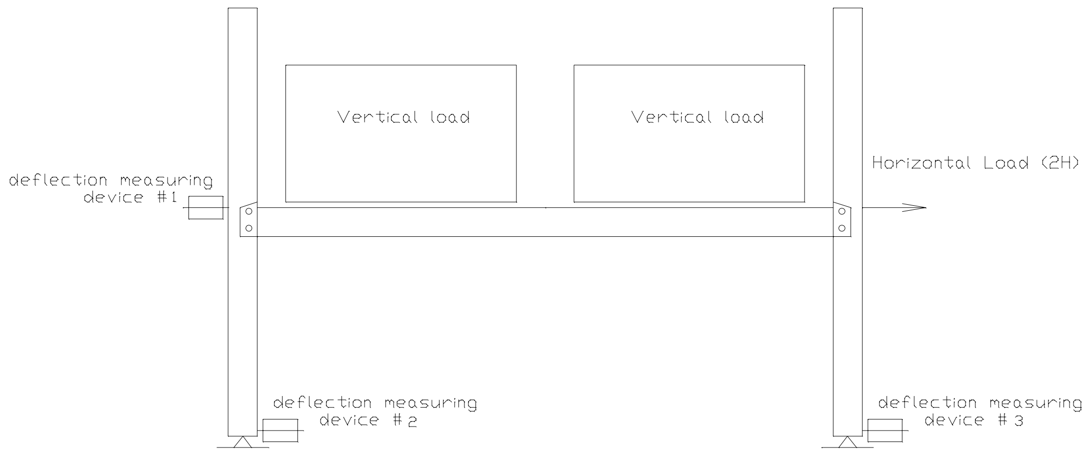


Figure 9.4.2.1-1 Portal test

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.

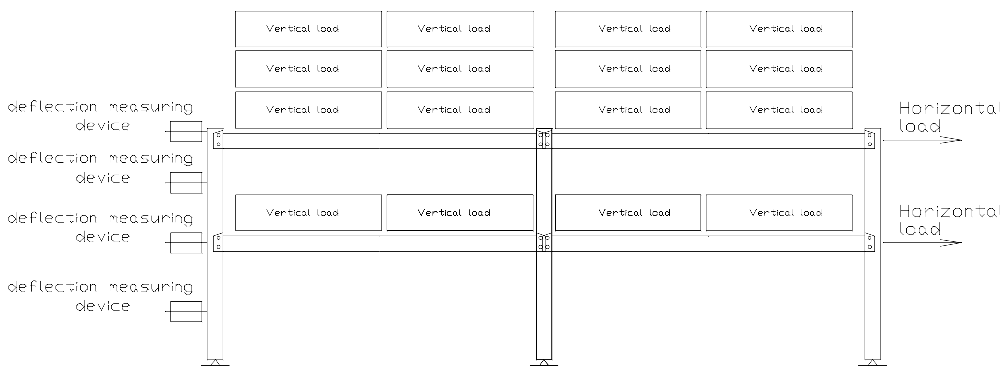


Figure 9.5.1.1.1 Test Setup

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.

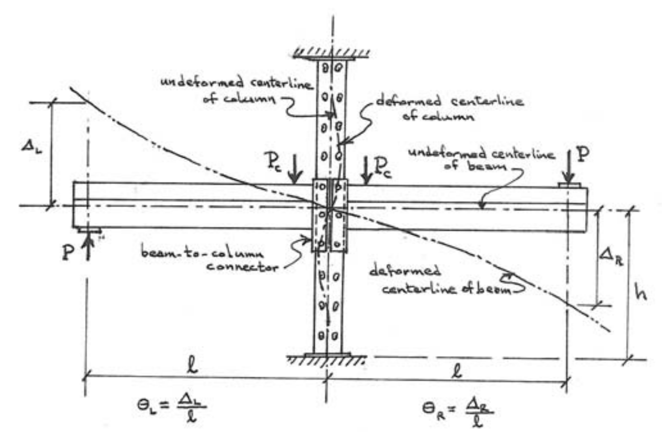


Figure 9.6.5-1 Test Setup

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 $\theta = 0.025$ radians
(2)	3 cycles at $\theta = 0.050$ radians
(3)	3 cycles at $\theta = 0.075$ radians
(4)	3 cycles at $\theta = 0.100$ radians
(5)	2 cycles at $\theta = 0.150$ radians
(6)	2 cycles at $\theta = 0.200$ radians

Continue loading at increments of $\theta = 0.050$ 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).

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