slenderness ratio would reduce the carrying capacity of the columns as compared to the original configuration. Alternately, racks can be modified by installation of additional components; e.g., greater number of shelf beams at smaller vertical spacing with the original upright frames. This would reduce the slenderness ratios of the individual column segments and increase their load capacities. However, the additional loads, which can now be placed on the greater number of shelves, could increase the load on the column by an amount greater than the increased capacity resulting from the reduction of the unbraced length. These are just two examples of changed configurations which could make an originally adequate rack unsafe.

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

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

### 1.4.6 MOVABLE SHELF RACK STABILITY.

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

### 1.4.7 BEARING PLATES AND ANCHORS.

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

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

Anchors serve several distinct functions:

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

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

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

### 1.4.8 SMALL INSTALLATIONS.

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

### 1.4.9 RESISTANCE TO MINOR IMPACT.

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

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

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

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

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

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

### 1.4.10 RACKS CONNECTED TO THE BUILDING STRUCTURE.

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

### 1.4.11 PLUMBNESS.

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

# 2 LOADING

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

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

## 2.1 LOAD COMBINATIONS FOR THE ASD DESIGN METHOD

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

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

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

## 2.2 LOAD FACTORS AND COMBINATIONS FOR THE LRFD DESIGN METHOD

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

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

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

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

Load combination #7 in the LRFD and load combination #5 in the ASD have been added to give a more realistic treatment of impact loading for shelves. This combination will 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 ($\phi_b$) 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 as required in some codes and specifications. This is covered in the notes within section 2.2. of the RMI Specification. Also, when the method used to derive the seismic lateral forces is limit states based (such as section 2.7 of this specification) the load factor for EL in combinations #5 and #6 may be reduced to 1.0. This is consistent with other codes.

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

## 2.3 DESIGN LOADS.

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

## 2.4 VERTICAL IMPACT LOADS.

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

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

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

## 2.5 HORIZONTAL FORCES

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

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

Other specifications NEHRP [5], UBC [6] specify a drift limit for storage racks of 0.0125 $h_x$ and 0.0036 $h_x$ respectively. These specifications do not require 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 quantity of members necessary to balance out the horizontal forces. The usual configuration is a back-to-back rack row with two frames attached with back-to-back ties. Additionally, fork truck impact will have a greater effect in the cross-aisle direction. In the cross-aisle direction the frame bracing can generally accommodate a force of 1.5% of the frame vertical load. Similarly, in the cross-aisle direction, the connections of the bracing to the columns should also be checked.

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

## 2.6 WIND LOADS

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

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

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

## 2.7 EARTHQUAKE FORCES

### 2.7.1 GENERAL

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

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

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

### 2.7.2 MINIMUM SEISMIC FORCES

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

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

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

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

### 2.7.3 CALCULATION OF SEISMIC RESPONSE COEFFICIENT.

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

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

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

where:

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

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

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

T = the fundamental period of vibration.

$\Delta_i$ = 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:

$\Delta_p$ = primary story drift not including P$\Delta$ effects.

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

L = column span length.

$\Delta_{i-1}$ = Primary deflection just below the level being evaluated.

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

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

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

where:

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

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

For the total drift at level i.

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

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

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

### 2.7.4 VERTICAL DISTRIBUTION OF SEISMIC FORCES

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

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

### 2.7.5 HORIZONTAL SHEAR DISTRIBUTION

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

### 2.7.6 OVERTURNING

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

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

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

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

# 3 DESIGN PROCEDURES.

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

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

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

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

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

# 4 DESIGN OF STEEL ELEMENTS AND MEMBERS.

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

## 4.2 COLD-FORMED STEEL MEMBERS

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

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

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

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

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

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

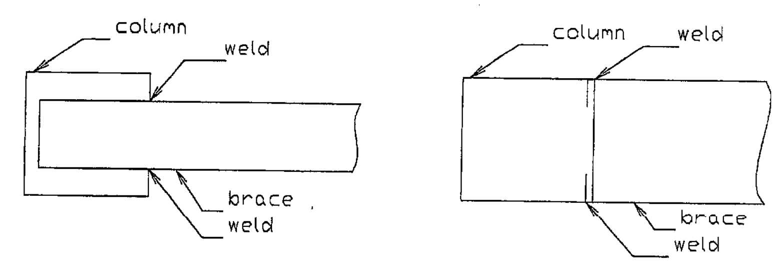
In the calculation of $M_e$, $\sigma_{ex}$, $\sigma_{ey}$, and $\sigma_t$ the section properties are to be based on full unreduced gross section considering round corners except for J, j, $r_o$ and $C_w$ which shall be based on the full unreduced gross section using sharp corners because the calculation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

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

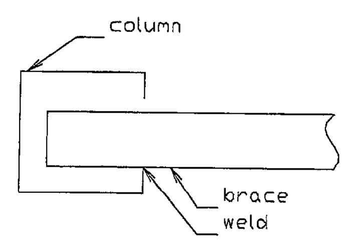
### 4.2.3 CONCENTRICALLY LOADED COMPRESSION MEMBERS. {The AISI Specification [4] Section 4}.

Compression members can buckle in either of two ways: purely flexurally, i.e., by simple bending about one of the principle axes without twist; or torsional-flexurally, i.e., by bending accompanied by twisting of the member. Some types of members which buckle purely flexurally are: all closed box-type members, sections whose shear center and centroid coincide, which is true for doubly-symmetrical members [e.g., I-sections], equal flange Z-sections, and others. Many other open thin walled shapes can be subject to torsional flexural buckling, such as singly symmetrical channel-, C-, hat-, and plain or lipped angle-sections, and others. In all these shapes, centroid and shear center do not coincide. However, whether such members actually will buckle torsional-flexurally or just flexurally in the direction of the axis of symmetry depends not only on the type of cross section but also on its relative dimensions. Thus, channels with wide flanges tend to buckle torsional-flexurally, while narrow-flanged channels generally buckle only flexurally. For some of the more common shapes, Part VII of the AISI Cold-Formed Steel Design Manual [14] contains curves which permit one to decide whether or not a member of given dimensions will buckle torsional-flexurally. Another way is to compare elastic torsional flexural buckling stress Eq C4.2-1 of Reference 4 with the elastic flexural buckling stress Eq C4.1-1 of Reference 4.

In designing columns for flexural buckling without torsion, the effective length factors K shall be taken as specified in Section 6.3 of this specification. For singly symmetrical shapes these methods are quite straightforward, provided that the effective length is the same for bending about the axis of symmetry (x-axis) and for twisting. This is generally the case for building-type frames, but need not be so for rack structures. For instance, for a pallet rack with channel or C-columns placed so that the x-axis is in the plane of the upright frame, the unbraced length $L_x$ for buckling about the x-axis is the length from the floor to the center line of the bottom beam, or between successive beam center lines, as the case may be. (This is the unbraced length $L_x$, not the effective length $K_xL_x$.) However, for torsion it can be assumed that even light members, such as the diagonal or horizontal struts of upright frames, will prevent twisting at the point where they are connected to the columns, provided the connection itself does not permit twist. Typical connection details between the columns and the bracing which are expected to inhibit twist and those that are not are shown in Fig. 4.2.3. For those racks with proper connection details, the unbraced length $L_t$ for torsion will be the free length between adjacent connections to any members which counteract torsion. For instance, if a diagonal of an upright frame meets the column somewhere between the floor and the lowest beam, then the longer of the two lengths, from the diagonal connection to either the floor or the beam, represents the unbraced length for torsion, $L_t$.




a. Joint detail that would restrain column twist




b. Joint detail that would not restrain column twist

Fig. 4.2.3-1 Joint details

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

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

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

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

# 5 BEAMS

## 5.2 CROSS SECTION

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

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

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

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

where:

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

E = the modulus of elasticity

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

$I_b$ = the beam moment of inertia about the bending axis

L = the span of the beam

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

where:

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

$M_e$ =the beam end moment

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

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

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

where:

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

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

### 5.3 DEFLECTIONS.

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

## 6 UPRIGHT FRAME DESIGN

### 6.3 EFFECTIVE LENGTHS.

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

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

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

The behavior of rack structures and hence the effective length factor depends on the unique design of racks such as rigidity of the connection between columns and beams. Due to the wide variety of details and cross sectional dimensions in rack structures, the effective length factors vary within a very broad range. For example, a simple portal frame with pinned column bases, the effective length factor approaches infinity as the connection between the beam and the columns approaches a pinned condition due to the connection details.

The values of the effective length factors given in this specification are by no means maximum values. They are average values assuming the racks to be designed according to good engineering practice and judgment. In all cases rational analysis would indicate whether the stipulated values are too conservative or too unconservative for the particular rack. Possible rational analysis procedures are presented later in this commentary.

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

The buckling considered here is parallel to the aisle. In general, racks have singly symmetric sections for columns and also in general the axis of symmetry is perpendicular to the aisle. The buckling of such sections parallel to the aisle, namely, about the axis of symmetry takes the form of torsional-flexural buckling. For such cases, the effective length factor is intended to be used in computing $\sigma_{ex}$ in Section 4.2.2; $\sigma_{ex}$ is in turn used in computing the torsional-flexural buckling load.

### 6.3.1.1 Racks Not Braced Against Sidesway.

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

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

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

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

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

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

where:

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

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

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

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

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

where

$I_b$ = the actual moment of inertia of the pallet beams

$L_b$ = the actual span of the pallet beams

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

E = the modulus of elasticity

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

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

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

where

$I_c$ the column moment of inertia

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

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

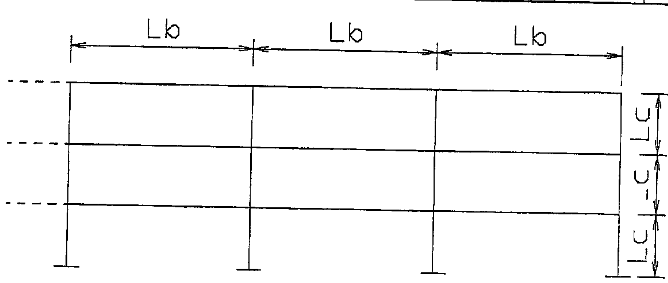
The effective length factor is then found directly from references 16 and 17 on the basis of $G_a$ and $G_b$.

The expression used above for $I_f/L_f$ is based on References 20 and 21. The expression given in these references are modified to reflect the situation for rack columns which in general have thin base plates. This expression is a crude representation of the base fixity. The base fixity depends among other parameters, on the ratio of the base moment to the axial load, namely the eccentricity of the axial load. A general formulation would be quite complex. Though direct test data is not available it seems reasonable to expect that the above equation would estimate the fixity rather closely for eccentricities corresponding to design load and 1.5% lateral loads. This reference using the above procedure reaches reasonably satisfactory correlation between the computed and the observed test results. It must be noted, however, that the base fixity is just one of many properties of the rack that affect the structural behavior.

The expression for $I_f/L_f$ given above assumes that the floor is concrete. The joint rigidity F is to be determined by a portal test. As the frame sidesways as the type of buckling under consideration implies, the beams of the frame will have different joint rigidities at each end. This is due to the fact that at one end the rotation is increased while the rotation is decreased at the other end. The portal method yields an intermediate value between the values of the rigidities of the two ends.

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

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




Assumed configuration

Columns 3"x3"

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

### 6.3.1.2 Racks Braced Against Sidesway.

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

Horizontal movement, or translation, of the front column relative to the rear column of rack with bracing in the rear vertical plane can, in some cases, be prevented by the presence of pallets on the load beams. To prevent translation of the front column, the frictional forces between the pallets and the load beams must be capable of resisting horizontal force perpendicular to the plane of the upright. The magnitude of this force at a bracing point should be at least 1.5% of the column load immediately below the beam acting as the horizontal brace. Whether or not sufficient force exists to prevent translation must be determined by rational analysis giving full consideration to factors such as, but not limited to, lighter than normal loads and the absence of any or all loads.

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

In order to cut down the unsupported lengths of the columns, the diagonal bracing should divide the brace plane as shown in Fig. 6.3.1.2[b] and [c]. At the same time rigid or braced fixed shelves are to be provided at levels AA in order to have unsupported lengths of h as shown in the figures. If such shelves are not provided at levels AA, then the column will be designed in accordance with Section 6.3.1.1.

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

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

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

and

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

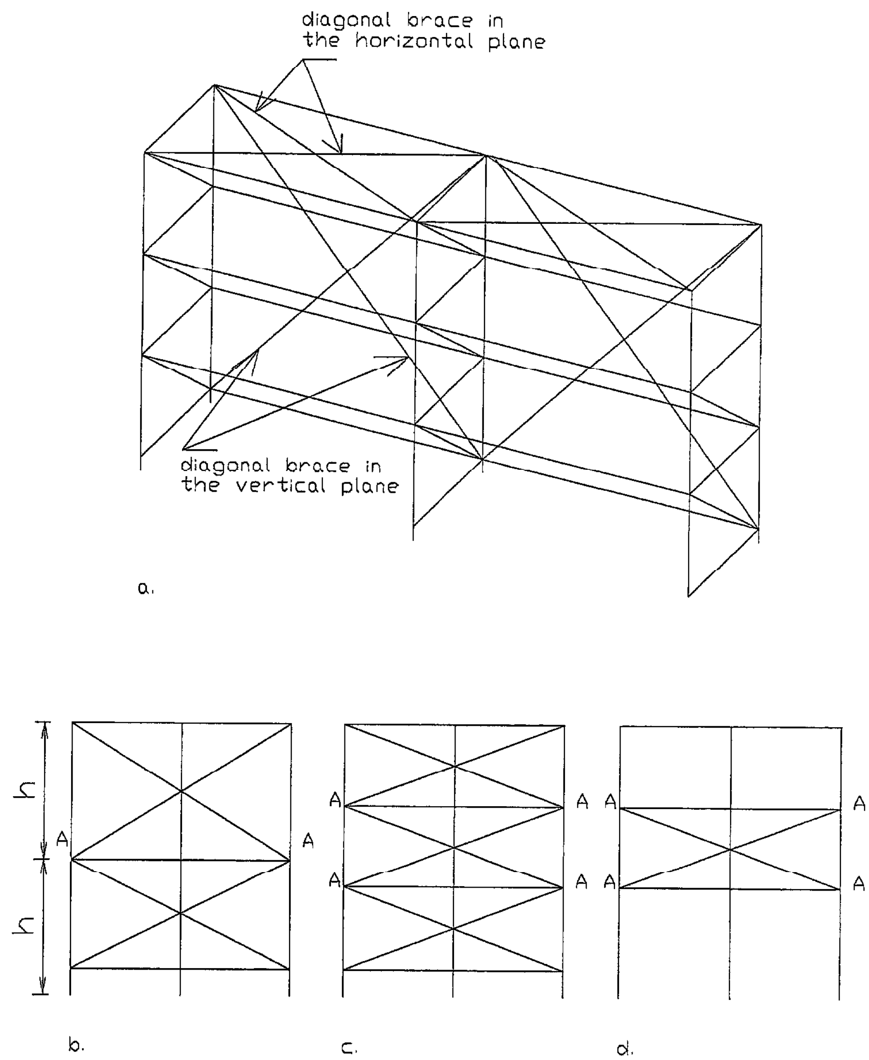



Figure 6.3.1.2-1 Racks Braced Against Sidesway

## 6.3.2 FLEXURAL BUCKLING IN THE PLANE OF THE UPRIGHT FRAME.

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

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

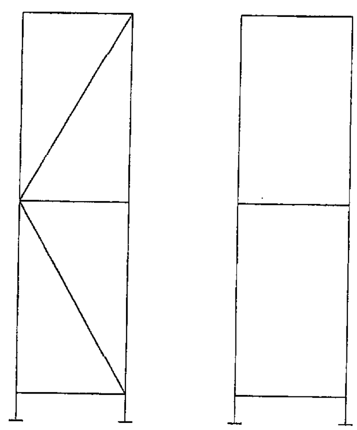



Figure 6.3.2-1 Braced and Unbraced Frames

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

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

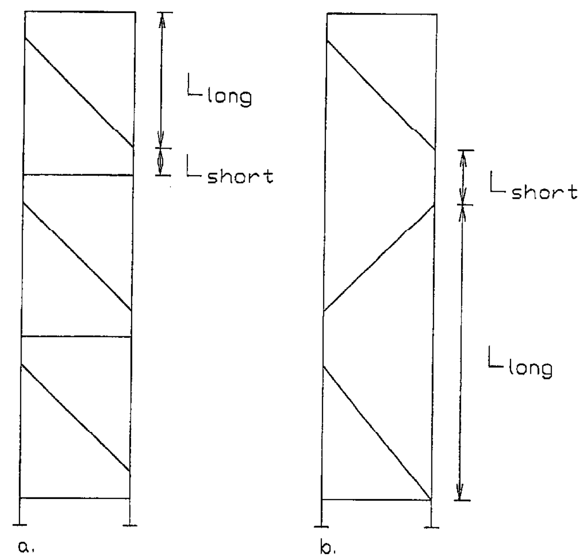
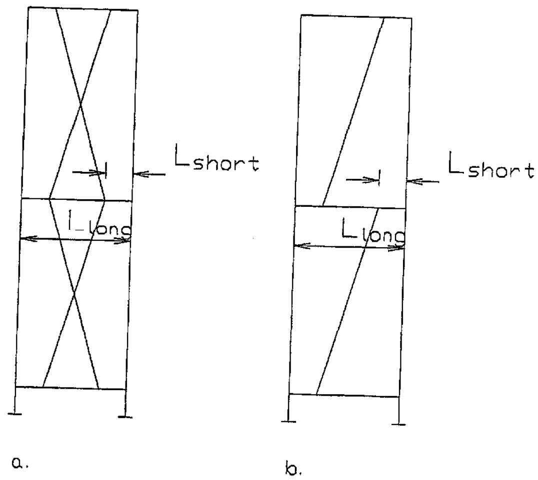



Fig. 6.3.2-2 Upright Frames with Diagonal Braces that intersect the Columns

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

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




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

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

### 6.3.3 TORSIONAL BUCKLING.

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

## 6.4 STABILITY OF TRUSSED-BRACED UPRIGHT FRAMES.

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

# 7 CONNECTIONS AND BEARING PLATES

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

## 7.1 CONNECTIONS

### 7.1.1 GENERAL

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

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

### 7.1.2 BEAM LOCKING DEVICE

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

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

### 7.1.3 MOVABLE SHELF RACKS

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

## 7.2 BEARING PLATES

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

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

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

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

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

# 8 SPECIAL RACK DESIGN PROVISIONS

## 8.1 OVERTURNING

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

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

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

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

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

## 8.2 CONNECTIONS TO BUILDINGS

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

### 8.3 INTERACTION WITH BUILDINGS

This section recognizes that building structures and rack structures are likely to have different structural characteristics. During an earthquake, this could have a magnifying effect for structures that are interconnected but which have differing periods of vibration. Thus, the connections must be designed to ensure that neither structure causes damage to the other during a seismic event.

# 9 TEST METHODS

## 9.1 GENERAL

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

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

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

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

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

Because of the interplay of three influences which affect a cold-formed perforated compression member, [i.e., local buckling, perforations, and cold-work of forming] recourse must be taken to determination by tests. This is done by stub column tests, [i.e., by careful concentric compression testing of pieces of the member short enough so as not to be affected by column buckling]. The details of such testing are spelled out in Part VIII of the AISI Cold-Formed Steel Design Manual [14].

### 9.2.2 EVALUATION OF TEST RESULTS.

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

The basic definition of Q is:

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

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

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

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

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

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

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

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

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

It is acceptable to linearly interpolate the Q-values for a series of shapes with identical cross-section and perforation dimensions, but with a variety of thicknesses. For this purpose $Q_{max}$ and $Q_{min}$ should be determined from stub column tests on specimens made with the maximum and minimum thicknesses of coil from which stub column was made.

This correction is necessary in order to avoid unsafe design in case the virgin yield stress [before forming] of the specimens was significantly higher than the specified minimum.

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

## 9.3 PALLET BEAM TESTS.

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

The loading in these tests is applied by means of a test machine or jacks. This loading may restrain the torsional distortions and hence, may lead to unconservative results for members subject to such distortions.

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

### 9.3.1 SIMPLY-SUPPORTED PALLET BEAM TESTS.

This test can also be used in the design of beams, in general, when the end restraint is deemed not to lead to significant increase in the load carrying capacity.

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

#### 9.3.1.1 Test Setup.

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

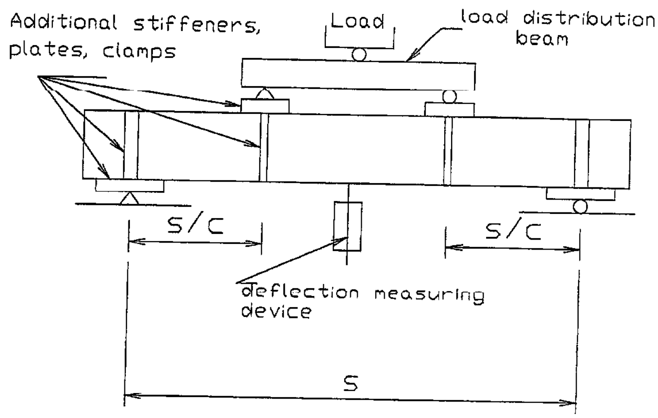



Fig. 9.3.1-1 Simply-Supported Pallet Beam Tests.

The value of C shown in the figure above shall be between 2.5 and 3 and has been chosen to avoid shear failure and to ensure a sufficiently long portion with constant moment.

For most pallet beams, the end connection detail is such that the beam can be placed directly on the supporting surface and have simply supported end conditions. In this case, the clamps, diaphragms of stiffeners at the supports most likely not be needed.

### 9.3.1.2 Test Procedure

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

## 9.3.2 PALLET BEAM IN UPRIGHT FRAMES ASSEMBLY TESTS.

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

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

### 9.3.2.1 Test Setup.

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

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

### 9.3.2.3 Evaluation of Test Results.

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

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

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

## 9.4 PALLET BEAM-TO-COLUMN CONNECTION TESTS.

The tests specified in this section have two objectives. One is to determine the moment capacity of the connection, the other is the determination of the joint spring constant F described below for use with the rational analysis approach.

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

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

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

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

### 9.4.1 THE CANTILEVER TEST.

The Cantilever Test provides a simple means of determining the connection moment capacity and rigidity. However, it has the disadvantage that the ratio of shear force [that is the vertical reaction] to moment at the joint is not well represented. For typical rack connections this ratio is probably higher than it is in the cantilever test as spelled out in the Specification.

In general a higher ratio would probably lead to a more rigid connection. However, bending moment and shear force would interact and lower the ultimate load of the connection. This effect should be studied by reducing the length of the cantilever to the distance between the end of the beam and the expected location of the inflection point.

This test is suitable for determining F for computing stresses due to vertical loads. A somewhat more tedious but more accurate determination of F can be achieved by tests according to Sec. 9.3.2.

### 9.4.1.1 Test Setup.

This test setup illustrated in Fig. 9.4.1.1-1

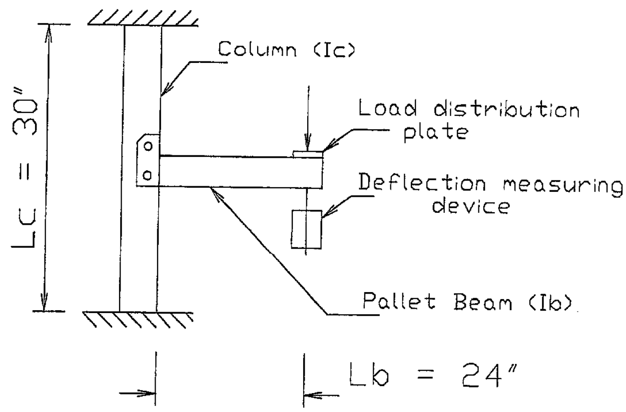



Figure 9.4.1.1-1 Cantilever Test

### 9.4.1.3 Evaluation of Test Results.

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

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

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

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

### 9.4.2 THE PORTAL TEST.

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

### 9.4.2.1 Test Setup.

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

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

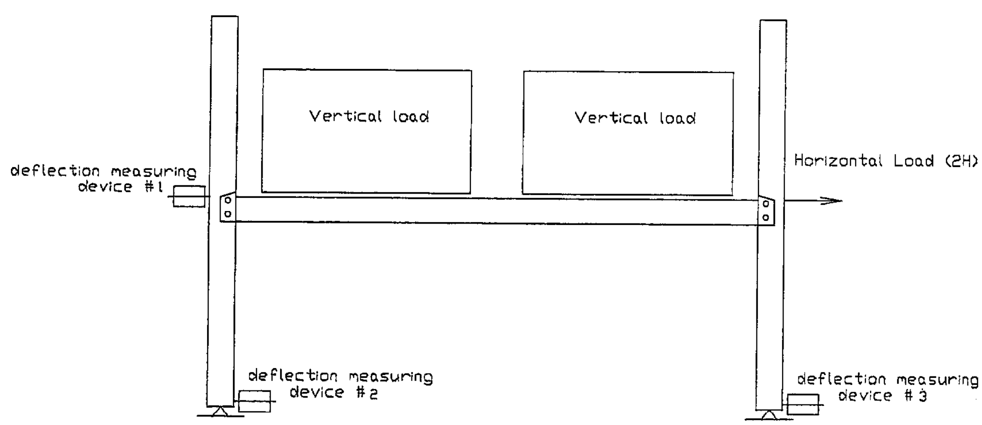



Figure 9.4.2.1-1 Portal test

### 9.4.2.3 Evaluation of Test Results.

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

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

Solving this equation for F, the following is obtained:

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

[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.

$\delta$ = Sway deflection corresponding to a lateral load of 2H.

Since the behavior at both the design load and the ultimate load is of interest, portal tests are to be conducted at both load levels. Multiple tests as recommended in the commentary on Sec. 9.4.1.3 are also recommended here.

## 9.5 UPRIGHT FRAME TEST.

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

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

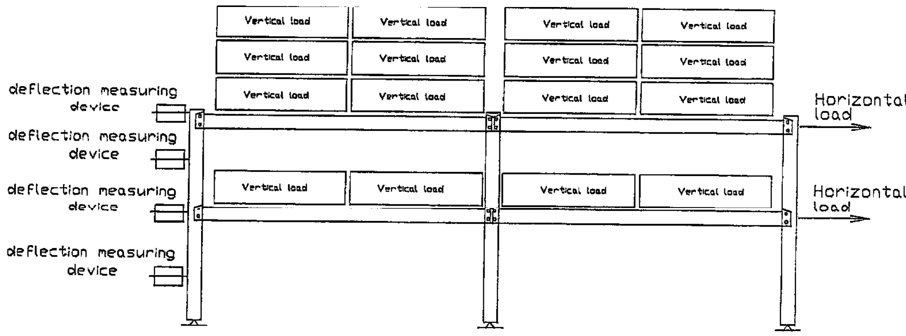



Figure 9.5.1.1.1 Test Setup

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