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Design Guide - Seismic design of high level storage racking systems with public access

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DESIGN GUIDE

Seismic Design of High Level Storage Racking Systems with Public Access

G J Beattie and B L Deam

The work reported here was jointly funded by the Earthquake Commission Research Foundation, the Department of Building and Housing, and the Building Research Levy. Assistance in kind was provided by Hamilton Perry Ltd and Capital Racking Ltd.

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Preface

This Design Guide is intended for use by engineers and manufacturers designing high level publicly accessible storage racking systems for earthquake actions.

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DESIGN GUIDE – SEISMIC DESIGN OF HIGH LEVEL STORAGE RACKING SYSTEMS WITH PUBLIC ACCESS

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1. INTRODUCTION

The creation of this Design Guide came about because of the concern being voiced by the public about the safety of high level storage racking systems being used in places with public access, such as supermarkets, discount traders and home handyman stores. There was also confusion in the design fraternity about the need to satisfy the requirements of the New Zealand Building Code (NZBC).

1.1 Regulatory requirements

Because racking systems used in areas with public access are an integral part of the business operation, and as such they are a permanent structure in these stores, the Department of Building and Housing (DBH) – formerly the Building Industry Authority – has determined that these systems are subject to the New Zealand Building Code (NZBC) and therefore require a building consent.

1.2 Description of high level storage systems in public access warehouse stores

The scope of application of this document is to racking systems that extend higher than 2 m above the floor and that have palletised material stored above this level. A typical arrangement is shown in Figure 1.

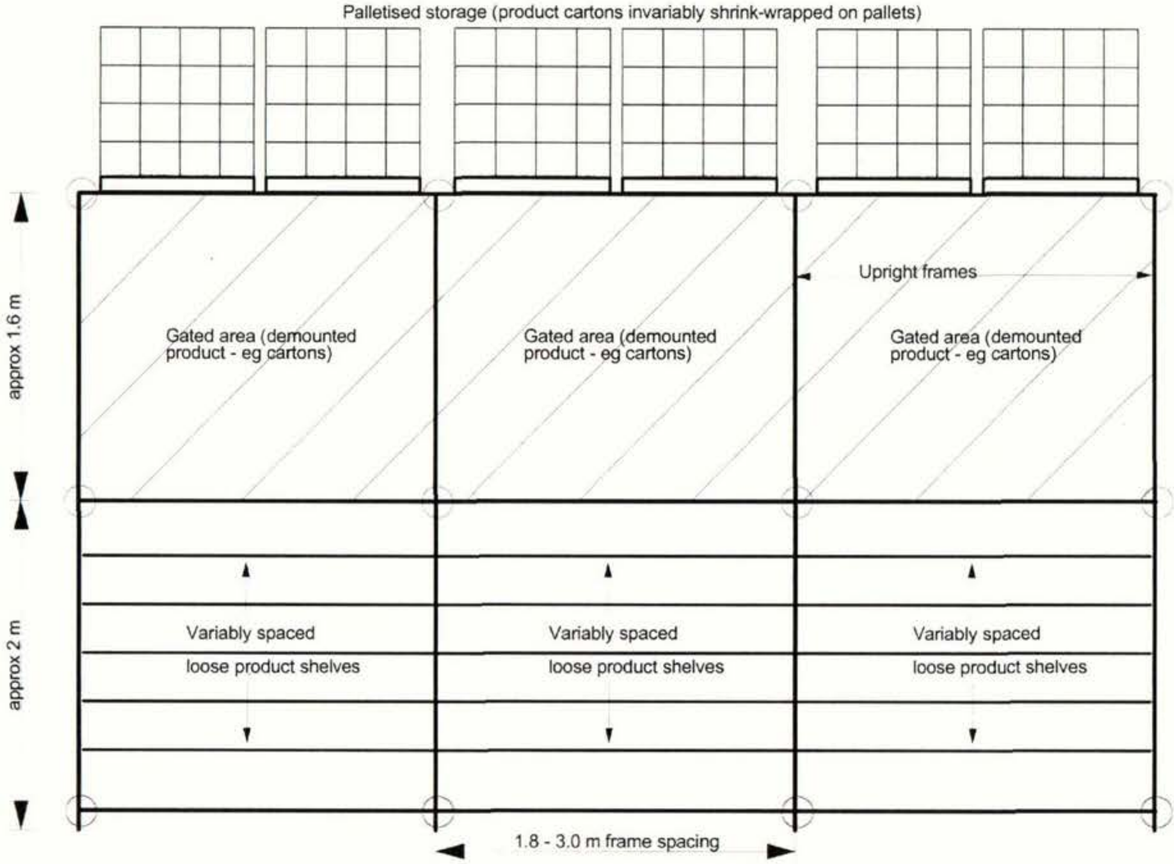
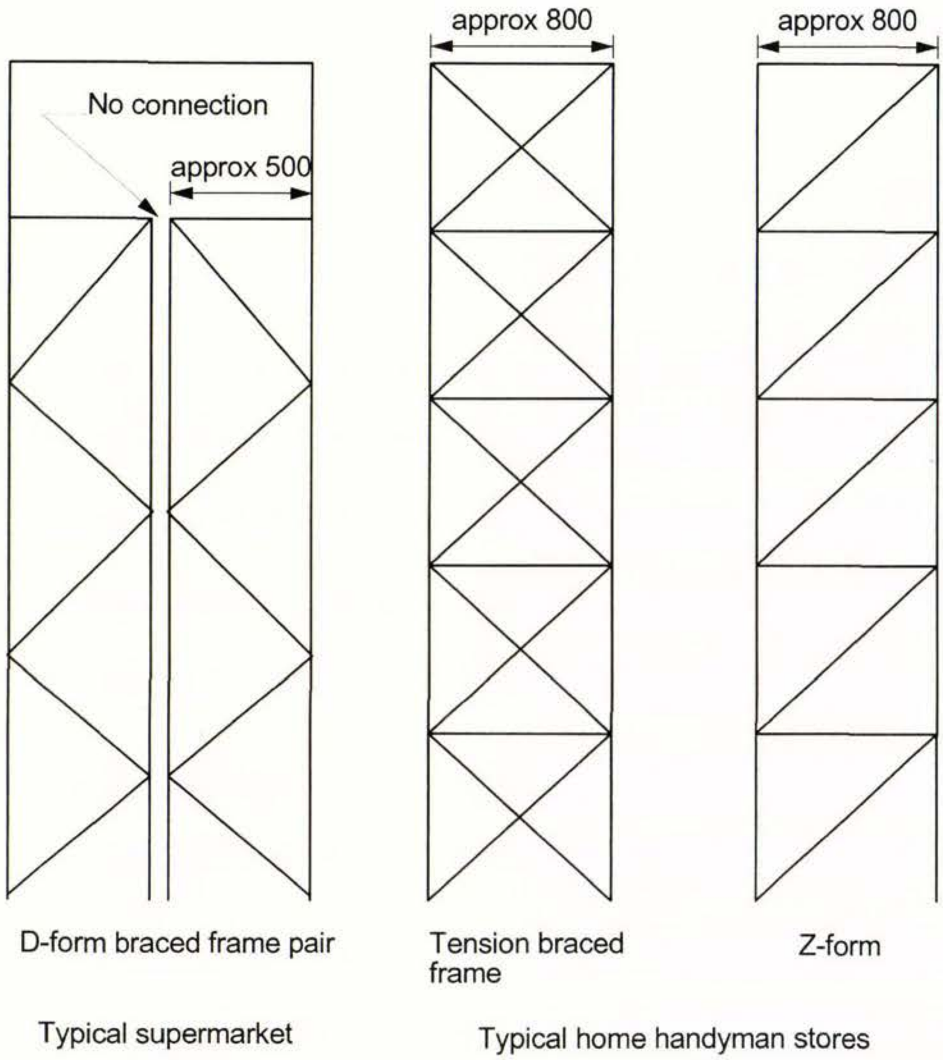


Figure 1: Elevation of a typical supermarket racking set-up (note that only circled joints provide moment restraint)

The racking systems are predominantly constructed using cold rolled steel sections for the columns, beams and braces. The columns and beams are fitted together using a variety of jointing systems which comprise either hooks or locating pins which are either pressed from a joint plate or are swaged into the plate and located in pre-formed holes in the column elements. Pairs of columns are placed along the length of the rack (usually about 1.8 m spacing for supermarkets but may be up to 3 m in home handyman stores). These frames are usually referred to as "upright frames". The column pairs are connected with diagonal brace elements either bolted or welded to the columns. The spacing between the columns varies, but tends to be around 500 mm for supermarkets and approximately 800 mm in home handyman stores (Figure 2).



(Comment: Note that FEM 10.2.08 Draft 04.1 does not allow the use of the typical supermarket D-form frames in seismically active regions)

Figure 2: Typical upright frame configurations

Because the beams can be easily fitted into the columns, and the columns are pre-punched to accept the beams at many levels, it is a simple process to reconfigure the system consistent with the needs of the store. Shelves of either particleboard or expanded metal mesh are placed between the beams to support the contents of the racks. The exception is the upper-most level where the pallets are supported either directly on the beams or on transverse steel members that are in turn supported on the longitudinal beams.

It is common for two rows of racking to be installed back-to-back, particularly in supermarkets and sometimes in home handyman stores. There may be frame spacers installed between the columns of the two upright frames. Otherwise, the connection between the two upright frames is usually only provided at the top level.

A significant difference between supermarkets and home handyman stores can occur at the top shelf (pallet) level. In supermarkets, because the depth of the customer picking shelves is short (approximately 500 mm) and there are usually two frames installed back-to-back, the pallets are supported on beams at only the outer faces of the system. In home handyman stores, the shelf depths are generally greater, either to accommodate sheet products for example, or are divided off to allow picking from both sides. The pallets are therefore generally supported directly on longitudinal beams at the front and back faces of individual racks.

Generally, the lower 2 m consists of shelves from which customers select their goods. In this area the products are fully demounted for easy selection, although some supermarket stores continue to use the packaging (e.g. cut-down cartons) to keep the stock tidy and to eliminate the cost of staff to unpack the products. Above 2 m in supermarkets, the products are invariably stored in unopened cartons on a single picking shelf, ready for night-fillers to re-stock the shelves below. In supermarkets the unopened cartons are generally stored behind some sort of grillage or door. In home handyman stores and bulk stores the size and degree of breakdown of the products is more variable. Typically, the top level of the racking system is at about 3.6 m above the floor and is reserved for palletised products, which are normally shrink-wrapped.

Restraint against lateral loading is provided in the down-aisle direction by portal frame action and in the cross-aisle direction by braced frame action. Commercial non-public access warehouse stores with predominantly pallet storage may utilise a down-aisle tension bracing system in the vertical plane at the back of the rack. In these instances, the design of the racks must take account of the different lateral restraint properties at the front and the rear of the rack. These cases are not included in this Design Guide because they are not typical in public access stores.

Reference has been made to overseas standards and guides in the compilation of this Design Guide. These have included:

- **Seismic Considerations for Steel Storage Racks Located in Areas Accessible to the Public – FEMA 460 – September 2005**, prepared for the Federal Emergency Management Agency (FEMA) by the Rack Project Task Group of the Building Seismic Safety Council [BSSC 2005]
- **FEM 10.2.02 The Design of Static Steel Pallet Racking – Racking Design Code** April 2001 Version 1.02, published by the Federation Europeenne de la Manutention – Section X – Equipement et Proceedes de Stockage [FEM 2001], and
- **FEM 10.2.08 The Seismic Design of Static Steel Pallet Racks – Draft-04.1**, April 2003 [FEM 2003].

It should be noted that the above reference documents are for racking systems where generally the racks are evenly spaced vertically and all contain palletised products. It appears from discussions with colleagues from the USA and Canada that the arrangement of shelves with pallet storage at the top, demounted products (e.g. cartons) on the mid-height shelf, and individual products which the customers to pick on lower shelves, is a relatively new development internationally.

1.3 Format of this Design Guide

First, the performance expectations for the racking systems are stated, followed by detailed seismic design considerations and then stocking recommendations and a suggested operational guide.

2. PERFORMANCE EXPECTATIONS

2.1 Introduction

The advent of the bulk retail stores has increased the exposure of the public to the associated seismic risk caused by either racking collapse or by impact from falling stock from above head level while shopping in such stores.

2.2 Seismic performance objectives

A seismic performance objective consists of one or more performance goals, each consisting of a target performance level coupled with an earthquake hazard. A clear, unambiguous definition of seismic performance objectives is necessary to provide appropriate tools for seismic risk decision-making. This Guide includes the development of performance objectives centred on the current NZBC and New Zealand building standards.

2.3 Performance expectations for buildings

The quoted performance expectations of the NZBC are that:

"B1.3.1 Buildings, building elements and sitework shall have a low probability of rupturing, becoming unstable, losing equilibrium, or collapsing during construction or alteration and throughout their lives.

B1.3.2 Buildings, building elements and sitework shall have a low probability of causing loss of amenity through undue deformation, vibratory response, degradation or other physical characteristics throughout their lives, or during construction or alteration when the building is in use.

B1.3.3 Account shall be taken of all physical conditions likely to affect the stability of buildings, building elements and sitework including:

- *(a) Self-weight*
- *(b) Imposed gravity loads arising from use*
- *(c) Earthquake*
- *(d) Fire*
- *(e) Impact*
- *(f) Influence of equipment, services, non-structural elements and contents, and*
- *(g) Removal of support*

B1.3.4 Due allowance shall be made for:

- *(a) The consequences of failure*
- *(b) The intended use of the building*
- *(c) Variation in the properties of materials and the characteristics of the site, and*
- *(d) Accuracy limitations inherent in the methods used to predict the stability of buildings"*

There are several other conditions contained in B1.3.3 and B1.3.4 that are not applicable to racking systems.

2.4 Performance expectations for high level storage racks

As previously mentioned, the storage racks are considered to be 'buildings' and therefore are covered by the NZBC. They must therefore satisfy the provisions of Clause B1 of the NZBC.

The seismic performance of the storage racks is dependent on two parameters: the seismic performance of the rack itself, and the response of the stored contents. Racks can pose safety hazards if they collapse, partially collapse or overturn. Contents can pose falling hazards if they become dislodged and fall to accessible areas.

Life safety performance will be achieved if the following conditions are met:

- failure of components that could result in rack collapse or contents shedding is prevented
- rack overturning is prevented, and
- there is no loss of stored items from rack shelves 2 m or more above the floor.

Collapse prevention performance is achieved if the following conditions are met:

- rack collapse is prevented, and
- rack overturning is prevented.

While preventing shedding of contents is not explicitly stated as a goal for collapse prevention performance, measures undertaken to protect contents for life safety performance will help limit contents shedding anticipated under more extreme seismic excitations. However, some loss of life associated with contents shedding is expected at the collapse prevention performance level.

Comment: The State of California enacted a law in 2001 requiring all pallet loads on rack shelves more than 12 feet (3.6 m) above the floor to have their loads plastic wrapped or encapsulated to the pallet [ref BSSC 2005]. BSSC further recommend that shelves be constructed to prevent 'fall-through', particularly shelves higher than 8 feet (2.4 m).

2.5 Health and Safety in Employment Act 1992

Sections 6 and 16 of the Health and Safety in Employment Act place requirements on the "persons who control places of work" to "take all practical steps to ensure that no hazard that is or arises in the place harms" their employees and those who "are there to undertake activities that include buying or inspecting goods from whose sale the person derives or would derive (directly or indirectly) any gain or reward".

Thus, this person is responsible for ensuring that the racking system in his/her store is designed and operated in a manner that does not present a hazard to his/her employees and customers.

2.6 Design considerations for storage racks

A major difference between the design of buildings and the design of racks is that limited post-yield capacity exists in some rack elements such as base plate fixings under post-uplift and "soft" lip channel brace/column connections. But otherwise there is little post-yield capacity with rolled elements and the loss of a single rack element can cause failure of the entire rack unit. As a consequence, there is little difference between life safety performance and collapse prevention performance.

2.7 Contents securing considerations

Strategies considered to be good practice for the securing of rack contents include:

- complete encapsulation of pallet loads (e.g. stretch-wrap, shrink-wrap, banding), with the wrapping extending below the merchandise and around the pallet
- mechanical restraints on the racks such as steel mesh sliding gates and hinged doors, and,
- the use of flexible nets.

The above performance objectives are seen as the minimum requirements, but store operators may choose to impose more stringent objectives.

3. RACK DESIGN (*INCLUDING COMMENTARY*)

These design recommendations are limited to racks with a maximum height of 5 m that utilise portal frame action in the down-aisle direction and braced frame action in the cross-aisle direction.

3.1 Applicable design standards

Applicable standards may be sub-divided into loading standards and resistance (material) standards.

3.1.1 Loading standards

In the consideration of loadings on racking systems, the primary reference standards for New Zealand are:

- AS/NZS 1170.0 Structural Design Actions – General principles
- AS/NZS 1170.1 Structural Design Actions – Permanent, imposed and other actions
- NZS 1170.5 Structural Design Actions – Earthquake actions – New Zealand Standard.

3.1.2 Load combinations

From AS/NZS 1170.0, the applicable loading combinations are:

Under **ultimate limit state** loads:

$1.2G + 1.5Q$, and

$G + \psi_c Q + E_u$,

and under **service limit state** loads:

$G + \psi_l Q + E_s$ (long-term service live load)

where

G	=	the dead weight of the rack structure
Q	=	the superimposed live load (the contents of the rack)
E_u	=	the ultimate limit state earthquake load
E_s	=	the serviceability limit state earthquake load
Ψ_c	=	the load combination factor
Ψ_l	=	the long-term load factor.

3.1.3 Gravity load from shelf contents

Consideration must be given to the storage capacity of the racking, the extent to which the imposed loads will be present (i.e. the percent of full load capacity) and the effects of simultaneous vertical acceleration from the earthquake. Vertical accelerations can potentially cause a loss of friction between the shelf and the contents, which may be sufficient to cause sliding of the contents. A high friction coefficient between the shelf and the contents may also lead to toppling of the shelf contents, particularly for either tall individual items (e.g. hot water cylinders) or single stacks of objects such as cartons.

Table 4.1 of AS/NZS 1170.0 provides ψ factors for buildings, depending on the character of the imposed action. For floors in retail occupancies, $\psi_l = \psi_c = 0.4$ and for storage occupancies, $\psi_l = \psi_c = 0.6$. For the cross-aisle direction, racking systems must be designed using a ψ_c of 1.0 and for the down-aisle direction, a figure of 0.6 may be used.

Comment:

Studies of typical loading levels in two Christchurch supermarkets and a discount trader (discount warehouse) have been made [Berry 2003]. From these it was determined that the imposed load from the palletised stock on the top level of the rack ranged from 0.7 kPa to 10.6 kPa. Consideration must be given to the expectations of the store owner about the pallet density on the top shelf. The stock stored on the picking shelf is invariably stored in an ad-hoc fashion with isolated towers of cartons, which are susceptible to toppling in an earthquake. Berry found that in the stores surveyed by him, the picking shelves were on average 61% full, while at the stock shelf levels they were 73% full.

A less detailed survey of supermarkets, discount warehouses and home Handyman stores has also been undertaken in Lower Hutt, as part of this study. General observations were that:

- *Supermarket stock levels at the customer-accessible shelves were mostly at the capacity of the shelves.*
- *Supermarket storage at the picking shelf level was extremely varied, with boxes often stored in tall towers with significant gaps between these.*
- *Supermarket storage of pallets above the public access areas was limited to products with low density (e.g. toilet paper) or the height of product on the pallet was relatively low.*
- *Several shelves were used at the discount warehouse for storage of demounted products, not expected to be accessed directly by the public, above 2 m from the floor, and up to 3 m. These shelves were densely packed with few gaps between items. No restraints were provided to prevent loss of this stock from the shelves in an earthquake.*
- *The upper-most shelf (approximately 3 m above the floor) at the discount warehouse contained demounted stock (no pallets stored at this level) with the restraint being nets fitted between two taught horizontal wires. The stock at this level tended not to be as densely packed as on the lower shelves.*
- *The stock placement at the home handyman stores varied greatly. At one store, the smaller items, stored at levels above the customers' heads, were contained behind solid hinged doors with mechanical latches. However, at the same store, larger items, such as oil column heaters, were densely stored on open shelves higher than 2.5 m from the floor with no restraints. Pallets were stored on the top shelf, at about 4 m above the floor, with no mechanical restraint provided.*
- *At another home handyman store, no palletised products were stored on the top shelf, which was about 3 m above the floor. Instead, items such as pails of paint were stored loosely on all shelves from the ground to the top of the rack. Cartons were at times*

stored up to three high on the upper-most level. The stock density generally reduced with height up the rack.

FEM and RMI have quite different factors to be used in the determination of the gravity load from the stock on the rack. The RMI uses a factor of 0.85 whereas the FEM uses a 0.9 multiplier in combination with a partial safety factor of 1.5 (or 1.4 for unit pallet loads), yielding an equivalent ψ_c of 1.35 (or 1.26 for unit pallet loads).

3.1.4 Seismic weight

Determination of the seismic weight is not an easy process due to the high variability in the weights and distributions of stored product in stores with public access, as confirmed in the above-mentioned surveys. Once the shelf capacity has been agreed between the store owner and the designer for all shelves, the system can be designed around this upper limit.

For loading in the cross-aisle direction, it will be appropriate to design the individual upright frames for a full condition so that in the event of an earthquake attack in the cross-aisle direction there is no potential for progressive collapse to occur.

For loading in the down-aisle direction, it will be possible to use a proportion of the total capacity of the shelf system. This recognises that the whole length of the shelves will not all be full at the same time and also that the loss of friction between the contents and the shelves from the vertical motion will reduce the inertial effects.

The seismic weight at level i shall be calculated from:

$$W_i = G_i + \psi_E \psi_M Q_i$$

where

G_i = contributing dead weight of the rack at level i, including added components for securing contents (e.g. screens)

ψ_E = the area reduction factor
= 1.0 for cross-aisle direction
= 0.8 for down-aisle direction

ψ_M = the rigid mass factor
= 0.67

Q_i = *the maximum design stock load on the rack at level i (for one bay for the cross-aisle calculation and the full length of the rack for the down-aisle calculation) couldn't indent here or throughout the document where this occurs*

The total seismic weight, W_t, is the sum of the seismic weights at all levels more than 300 mm above the base of the rack.

Comment: Reference has been made to recognised overseas standards and guides for guidance on appropriate seismic weights for racking systems.

The Canadian Draft Standard, A344.2 [CSA 2004], notes that where the damping of the load is minimal and the storage racks are fully occupied at the maximum design load, the seismic weight, W, shall be 80% of the maximum design load. If the effects of damping of the load and reductions of the load due to normal reduced occupancy are considered, then the seismic weight must not be reduced to less than 60% of the maximum design load.

The RMI [RMI 1997] states that the seismic design forces shall not be less than that required by the following equation:

$V = C_s I_p W_s$

where

C_s	=	*seismic response coefficient*
I_p	=	*system importance factor*
	=	*1.5 for storage racks in areas open to the general public*
W_s	=	*seismic weight*
	=	$(0.67 \times \psi_c \times Q_s) + G + 0.25 \times Q$
		(note that terms have been converted to New Zealand equivalents)
Q_s	=	*maximum load from pallets and products stored on the rack*
Q	=	*live load other than products (e.g. work platforms) – generally 0 in New Zealand*
G	=	*dead load of the rack*
ψ_c	=	*product load reduction factor (= New Zealand live load combination factor)*
	=	*1.0 for the cross-aisle direction*
	=	*PLaverage/PLmaximum for the down-aisle direction*
$PL_{average}$	=	*the maximum total weight of product expected on the shelves in any row divided by the number of shelves in that row*
$PL_{maximum}$	=	*the maximum weight of product that will be placed on any one shelf in that row*

FEM 10.2.08 states that the design seismic pallet weight shall be determined from:

$W_s = R_F \times E_D \times Q_s$

where

R_F	=	*rack filling grade reduction factor*
	=	*1.0 in the cross-aisle direction*
	≥	*0.8 in the down-aisle direction*
E_D	=	*reduction factor due to energy dissipation by product/pallets moving*
	=	*0.67 (as in RMI method above)*
Q_s	=	*design pallet load (scope is provided for replacement of the design pallet load with the average pallet load if this figure is reliably available and formal agreement is reached between the specifier/user and the designer)*
		(note that terms have been converted to New Zealand equivalents where possible)

Note that FEM 10.2.02 states that if the total self-weight of the structure is less than 5% of the total applied vertical load, the self-weight of the structure may be neglected. However, because of the variable stocking of such systems in the New Zealand context, this lack of consideration is not recommended.

3.1.5 Material standards

The relevant material standards are:

- AS/NZS 4600:1996 *Cold formed steel structures*
- NZS 3404:1997 *Steel structures standard (to a lesser extent)*
- AS 4084:1993 *Steel storage racking.*

Generally, the cold formed steel structures standard will apply for the rack member design because the metal thickness on most members is less than 3 mm. This standard is based on the 1996 edition of the American Iron and Steel Institute Load and Resistance Factor Design Specification for Cold Formed Steel Structural Members, and is in a limit states format. Note that AS 4084 *Steel storage racking* was last issued in 1993. It therefore makes reference to the Cold Formed Steel Structures Code, AS1538:1988, which is the predecessor of AS/NZS 4600. AS 4084 provides rules for the treatment of perforated members by alteration of the member properties in AS1538 (e.g. the section elastic modulus and the cross-sectional area). AS/NZS 4600 appears to take account of sections with patterns of circular holes, as will be found in some racking systems. For the calculation of the capacities of other perforated sections, there are still features in AS 4084 that can be applied.

3.2 Earthquake design process

Most low rise structures are currently designed using equivalent lateral force procedures, and collapse prevention at the ground motions associated with an ultimate limit state earthquake event is not explicitly demonstrated but only inferred based on past experience. Storage racks are expected to be designed in the same manner. Because the inelastic behaviour of rack structural system members and connections are significantly different from building structural systems (although the systems appear physically similar), it is recommended that in addition to equivalent lateral force design, a demonstration of collapse prevention be explicitly made using displacement-based design principles and cyclic connection testing.

There are two basic types of structural systems that are used in the design of high public access steel storage racks. In the cross-aisle direction (transverse direction), steel braced frames are typically used in seismic applications. In the down-aisle direction (longitudinal direction), steel moment frames are typically used with special connections between the columns and beams. The typical steel storage rack configuration used in general public applications is illustrated in Figure 1. It should be noted that these racks do not usually have horizontal diaphragms or cross bracing and therefore basically behave as structures with flexible diaphragms.

Comment: Note FEM 10.2.02 requires that the uprights be designed for the combination of both cross-aisle and down-aisle direction loads. This would generally only be critical at the end of an aisle.

The steel braced frames used in the transverse direction of racks are very similar in appearance to structural steel concentrically braced frames defined in NZS 3404:1997. While it is expected that the distribution of forces to braced steel racks will be very similar to that of hot rolled structural steel frames, the inelastic behaviour of the members and connections may be significantly different. The horizontal struts and bracing members are typically light gauge open sections that are either welded or bolted directly to open section columns. There is no use of gusset plates for transferring loads between braces and beam and columns. The inelastic response is somewhat dependent on the behaviour of the bracing connection which tends to greatly reduce the stiffness of the rack in the cross-aisle direction. In addition, in areas of high seismicity some shelf contents may tend to slide at high levels of ground motion, which will introduce damping, and contents are expected to be lost from the lower public access shelves, both reducing the effective seismic weight. Member connection design details are to be based on AS/NZS 4600. It is also recommended that the displacement-based evaluation procedure of Section 3.4.2 of this document be used.

The moment frames used in the down-aisle direction of steel storage racks, while appearing very similar to steel moment frames, behave inelastically very differently from structural steel moment frames in buildings. While the moment connections of structural steel moment frames are designed to cause inelastic member deformations in the beams away from the connections, connections used in racks have their inelastic behaviour occur directly in the rack beam to upright connections. It has been suggested [BSSC 2005] that for many connections there is a significant difference in behaviour between positive and negative moments because of the unsymmetrical geometry of the connections, although this was not found to be the case when testing two typical connection types available in New Zealand. Furthermore, because each beam end has pairs of joints rotating in a counter direction to each other under lateral motion, the characteristics for the two directions can be combined. Also, although the system exhibits highly non-linear behaviour up to a very large relative rotation, the system remains essentially elastic in the sense that the behaviour does not cause permanent deformation and the racks can recover their initial properties if they are pushed back to their original position after these apparent non-linear displacements. The inelastic rotation capacity of these connections is significant, and for some connections can exceed 0.20 radians as compared to building connections which are in the range of 0.04 radians. However, the rotation demands on rack moment connections are in the order of four times greater than rotation demands on building moment frame connections, because of their relatively short height for comparable fundamental periods, so this rotation capacity is necessary to withstand strong earthquake ground motions. It is recommended that both the adequacy of the moment connection system be demonstrated by the optional cyclic testing and a displacement-based evaluation approach that is discussed in Section 3.4.2 of this document.

Because the detailed behaviour of rack connections can be so fundamentally different from those used in buildings, it is recommended that typical building system type detailing approaches not be applied to racks. Instead, the approaches should consider the non-linear behaviour of the racks and the necessary detailing that will assure that the seismic response will be acceptable.

3.2.1 Drift limitations

Drift limits must be imposed to prevent impact of the deflecting rack on the surrounding building and also to control P-Δ effects.

For completeness, it is recommended that for concentrically braced frames operating in the cross-aisle direction, the drift limits be checked although they will rarely govern. For the moment frame systems, if the recommended optional displacement-based approach discussed in Section 3.4.2 is used to demonstrate structural stability, the drift limits need not be checked.

For the purposes of separation of the rack system from the surrounding structure, the assumed total relative displacement for storage racks shall not be less than 5% of the height above the base unless a smaller value is justified by test data or a properly substantiated analysis.

It is expected that a value much less than 5% will be computed in the cross-aisle direction by a simple frame analysis using the equivalent static force method. The computed displacement shall be that determined by elastic analysis multiplied by the ductility assumed in the determination of the seismic load on the system and a drift modification factor of 1.2. Such computed displacements properly done are deemed to be properly substantiated analyses. In the down-aisle direction, a 5% drift might possibly be calculated and therefore other analysis procedures may be required unless 5% is assumed.

3.3 Seismic analysis

Storage racks should be designed using the equivalent static force method for the ground motions defined in NZS 1170.5:2004, as if the rack was a building structure. Invariably, the rack systems are installed at ground level and there will therefore be no amplification effects from the supporting structure. However, it is becoming more common for stores to have parking beneath the floors on which the racking is placed. If the parking is below the ground, it is likely that there will be no horizontal amplification effects because the basement structure will move with the ground. However, it will be necessary to ensure that the supporting structure has been designed to appropriately resist the hold-down forces from the base of the rack system.

For the down-aisle direction, it will be necessary to have available an experimentally established moment-curvature relationship for the connections to be used in the design.

Steel storage racks generally have either drop-in particleboard or steel mesh shelves which do not perform effectively as horizontal diaphragms. Therefore they can be analysed as two dimensional systems, using loading assumptions associated with flexible diaphragms, for each braced frame and moment frame line of resistance.

Consideration should be given to the placement of products on the shelves in the cross-aisle direction. For example, product stacked for display near the front of shelves will place greater down-aisle demand on the front frame than the back frame.

Braced frame systems: Steel rack braced frames may be treated as statically determinate structures and hand analyses may be used to determine member design forces resulting from applied base shear forces. Computerised frame analyses may also be used to determine design forces. For such analyses, the racks are treated as linearly elastic with the bases modeled to represent the semi-rigid connection of the base plates to the concrete slab. For this modelling, it will be necessary to have available empirical stiffness data for:
(a) lattice members and
(b) base plate uplift.

Moment frame systems: It is typical in rack structures to use the portal method to determine forces in rack members resulting from applied base shear forces. Computer frame analyses may also be used to determine member forces resulting from applied base shear forces. For such analyses, the rack members are treated as linearly elastic members, the connection stiffnesses may be assumed to be equal to k_c (see Section 6.1.1), and bases modelled to represent the semi-rigid connection of the base plates to the concrete slab.

For regular racks with evenly spaced levels, it is recommended that the fundamental period be determined using the equation provided in Section 3.4.2.1. In such analyses, it is acceptable to assume that the upright base plate connection to the concrete floor has a moment rotation stiffness that is the same as a beam column connection (i.e $k_b = k_c$).

P-Delta effects: Steel Rack Braced Frame Systems typically are relatively stiff structures (prior to brace buckling) and P-Delta effects can be typically ignored in design. Flexible base plates will contribute to the total lateral deflection of the frame. In these instances, consideration should be given to the effect of this action on the P-Delta performance.

Steel Rack Moment Frame Systems are very flexible structures. Therefore P-Delta effects are likely to be significant and should be included.

It should be noted that if the moment-rotation connection properties have been determined using a portal test (see Section 6.1.2), the P-Delta effects have been partially included in the testing and therefore inclusion of P-Delta effects requires careful consideration in the structural analyses modelling and procedures. It is therefore recommended that the connection properties be determined using the cantilever test.

3.3.1 Elastic site spectra

Determine the elastic site hazard spectrum, $C(T_1)$, using the following equation from NZS 1170.5:

$$C(T_1) = C_h(T_1)ZRN(T_1, D)$$

The spectral shape factor, $C_h(T_1)$, shall be determined for the site subsoil class and rack first mode natural (fundamental) period, T_1, for the direction being considered at the site where the rack is to be constructed. If the site soil conditions are not known, site subsoil class D should be assumed.

Determine the hazard factor, Z, for the site and the return period factor, R_u or R_s, depending on the limit state condition being considered. Note that for 'standard design' racking systems, it may be more useful to design for the worst hazard area so that the system can be used in all areas of New Zealand.

It could be argued that the consequence for loss of human life in a supermarket is high in the event of a design earthquake, and that there would be considerable social and environmental consequences (i.e. inability to access food stocks after an earthquake), yielding an importance level of between 2 and 3. For discount traders and home handyman stores, the importance level is more likely to be 2.

The design life for a racking system is more likely to be 25 years than the 50 year minimum design life requirement for a building. Hence, the annual probability of exceedance for ultimate limit states should be between 1/250 and 1/500 for supermarket racks and 1/250 for discount traders and home handyman stores (refer Table 3.3 in AS/NZS 1170.0). Derived return period factors are presented in Table 1.

It is important to recognise the regulatory requirements associated with the choice of design life. A choice of a 25 year design life means that once the 25 years has elapsed, the Territorial Authority can require that the rack be altered to continue to comply with the applicable provisions of the Building Code to at least the same extent as before the alteration.

Table 1: Choice of return period factor for earthquake design

Design working life (yrs)	Importance level	Annual probability of exceedance for ULS	R_u (ULS)	Annual probability of exceedance for SLS II	R_s (SLS)
25	2	1/250	0.75	1/25	0.25
25	3	1/500	1.0	1/25	0.25

If the first mode period of the rack is calculated to be less than 1.5 s, there will be no need to include a factor for near-fault effects in centres affected by the presence of earthquake faults. Dynamic laboratory testing undertaken at BRANZ during the preparation of this Design Guide indicated that in the down-aisle direction, the rack period was less than 1.5 s (see calculation of fundamental period in Section 3.3.1.1). Therefore, near-fault effects do not need to be included.

3.3.1.1 Fundamental period of the rack

Cross-aisle direction period

The fundamental period in the cross-aisle direction should be determined using either modal analysis procedures or equivalent approximate procedures that are based on the assumption that members and connections are linearly elastic. To account for the significant flexibility of the bracing connections, the axial stiffness of the braces may be adjusted. Chen et al [1980] suggest that this adjustment be between 1/7 and 1/12 times the axial stiffness, EA/l, of the braces.

The seismic weight used in the period calculations should be the effective seismic weight, W_t, specified in Section 3.1.4.

In lieu of doing a period calculation, the period may be taken as equal to or less than the ratio of the short period spectral shape factor divided by the spectral shape factor at a period of 1 s.

Down-aisle direction period

The computed period of steel rack moment frame systems can vary appreciably because the rotational stiffnesses of the moment connections vary significantly with applied lateral load (and displacement). The fundamental period used in the base shear calculation should be consistent with computed base shear. In other words, it should be demonstrated in the design calculations that when the design base shear is applied to a down-aisle model of the storage rack, the moment determined for the beam-column connections is consistent with the connection stiffness assumed in the analysis model. The determination of the fundamental period may therefore require an iterative procedure. Alternatively, the period may be determined utilising the connection rotational stiffness kc determined in accordance with Section 6.1.1 of this Design Guide. Other rational procedures may be used to compute the period, but it should be demonstrated that the assumed connection stiffness does not result in computed moments from the base shear calculation having a corresponding connection stiffness less than that assumed.

3.3.2 Structural ductility factor

Because the nature of the construction of racking systems is significantly different from structural steel frames, the available ductility is often limited. The following values are prescribed:

For the cross-aisle direction, where lateral load resistance is provided by braced frames and where the most common failure mode is sudden buckling of the web members, a maximum structural ductility, μ, of 1.25 may be used. There is potential for some energy dissipation to occur as the column base plates yield, but the buckling web members may override this mechanism. Hence the setting of the upper limit on μ to 1.25.

Comment: AS/NZS 4600:1996 requires the maximum structural ductility to be no more than 1.25 unless a special study allows an increase up to a maximum of 4.

For the down-aisle direction, where lateral load resisting performance is reliant only on the strength of the connections between columns and beams, the ductility should be determined with reference to the performance of the connectors determined in experimental investigations of the joint moment-rotation behaviour. When the results of such investigations are not available, the maximum assumed ductility shall be 1.25. In this situation, the beams and columns are assumed to have sufficient over-strength on the joints. Investigative testing of a range of joint sizes conducted at BRANZ in the development of this Design Guide indicate that it is reasonable to assume a down-aisle ductility of 3.0, but this should be confirmed by each manufacturer for their system components. On no occasion should the assumed ductility in the down-aisle direction exceed 3.0 without a detailed study being undertaken.

3.3.3 Structural performance factor, S_p

The chosen structural performance factor will be dependent on the system ductility in the ultimate limit state. For systems with μ = 1.0, an S_p = 1.0 shall be used. S_p may be linearly reduced as μ increases to a maximum of 2.0, as follows:

$S_p = 1.3 - 0.3\mu$.

For μ greater than 2.0, the S_p factor is 0.7.

For the serviceability limit state, use S_p = 1.0.

Comment: Note that this figure is different to that specified in NZS 1170.5 which allows an S_p = 0.7 for the serviceability limit state, in recognition of the enhanced stiffness and strength of real buildings when compared to structural models which generally only take account of primary structural elements. However, racking systems do not have an added benefit of strength contributions from non-structural elements.

3.3.4 Horizontal design action coefficients

The horizontal design action coefficient for the ultimate limit state shall be determined from:

$$C_d(T_1) = \frac{C(T_1)S_p}{k_\mu}$$

where

$C(T_1)$ = the ordinate of the elastic site hazard spectrum, determined in Section 3.3.1 using the first mode natural (fundamental) period of the rack, T_1

S_p = the structural performance factor determined in Section 3.3.3

k_μ = μ (the system ductility factor – see Section 3.3.2).

The horizontal design action coefficient for the serviceability limit state shall also be determined from the above equation, except that $C(T_1)$ shall be the ordinate of the elastic site spectrum for the largest translational period of vibration determined using R_s (see Section 3.3.1).

3.3.5 Horizontal base shear

The horizontal base shear force is given by the following equation:

$V = C_d(T_1)W_t$

where

$C_d(T_1)$ is given in Section 3.3.4 and W_t is calculated in Section 3.1.4.

Based on the recommendations of the Canadian Draft Standard, A344.2 [2], the base shear is distributed over the various levels of the rack as if each was a storey of a multi-storey building, except that there shall be no extra force applied to the top level. That is, the distribution of forces shall be in accordance with the following equation:

$$F_i = V\left(\frac{W_i h_i}{\sum_{i=1}^{n} W_i h_i}\right)$$

where

W_i and h_i are the seismic weights and heights respectively of the shelf levels above the floor. Note that any shelf that is less than 300 mm above the floor is not required to have its mass included in the determination of the base shear, but this mass should be included for the determination of overturning resistance (and will assist the restoring force).

3.4 Rack design details

Rack member demands are to be determined using the basic load combinations of Section 3.1.2, and compared with member capacities determined in accordance with an appropriate standard. AS/NZS 4600 is the relevant standard for Australia and New Zealand for cold formed steel structures, but reference will be required to AS 4084 and AS 1538 to determine the capacities of perforated sections.

Comment: AS 1538 is the predecessor of AS/NZS 4600 and was written in working stress design format. AS 4084 provides modifications to the design of cold formed members in compression to account for the inclusion of perforations.

Alternatively, test results should be used to demonstrate that the rack structural system will maintain its structural stability when subjected to the ultimate limit state ground motion. An alternate displacement-based procedure that may be used to evaluate and verify the adequacy of moment connections and cross-aisle frames is given in Section 3.4.2.

3.4.1 Design approach for base plates, base plate connections and anchor bolts

3.4.1.1 Base plates

Base plate analyses should be in accordance with rational methods which consider the loads imparted to the plate and their delivery to the slab surface. The design forces should be based on the basic load combinations without over-strength. The literature has extensive guidance in this area. Generally, the base plate thickness is such that flexural yielding of the plate will occur as the rack sways in the down-aisle direction, effectively creating a pinned joint.

Similarly, in the cross-aisle direction, base plate yielding will improve the ductility in the system. To account for the increased ductility brought about by this action, base plate component tests should be undertaken to determine the uplift force-displacement relationship.

Comment: FEM 10.2.02 notes that it is conservative to assume a pinned joint. Alternatively, the moment-rotation properties may be determined by test.

3.4.1.2 Base plate connections to columns

It is recommended that the welded or bolted connection between the base plate and the column be designed for the load combinations with over-strength or for the nominal yield capacity of the base plate, whichever is least.

3.4.1.3 Anchor bolts

It is recommended that proprietary anchors that are installed after the floor system is cast, and that connect the rack base plate to the slab on grade, be designed for the load combinations with over-strength. However, the anchor bolt design forces need not exceed the nominal yield capacity of the base plate. The anchor bolt capacities should have been determined by appropriate test evaluations.

3.4.1.4 Floor slab capacity evaluation

It is recommended that the capacity of the floor slab to resist rack loading resulting from earthquakes be based on rational procedures from accepted good engineering practice. Such procedures must take account of the potential for punching shear failure of the slab under increased compression forces caused by the earthquake, and also the potential for the slab to uplift under overturning forces. Such actions will be more likely in the cross-aisle direction than the down-aisle direction.

3.4.2 Procedure for the displacement-based ultimate limit state evaluation of steel storage rack systems

The implementation of this procedure is expected to satisfy the performance objective that the rack system will not collapse when subjected to the ultimate limit state ground motions. To demonstrate that storage racks will likely not collapse in these events, displacement-based evaluation procedures are provided for both the down-aisle (moment frame) and cross-aisle (braced frame) directions. These procedures have been recently developed by the Rack Project Task Group of the Building Seismic Safety Council in the USA.

3.4.2.1 Displacement-based procedure for evaluating collapse prevention in the down-aisle direction

To evaluate whether the storage rack will likely not collapse in the moment frame direction, the optional displacement-based evaluation procedure provided below is recommended. The procedure is based in part on the Equal Displacement-based Design Procedure found in Appendix I-Part B "Tentative Guidelines for Performance-based Seismic Engineering" of the 1999 SEAOC Recommended Lateral Force Requirements and Commentary. The fundamental technical assumptions of the procedure are that:

- Similar moment connections are used throughout the moment frame system of the racks *(Comment: For supermarket racking this may not always be the case, and in this situation the individual moments of the joints at the rotation corresponding to the rotation at the maximum moment of the stiffest connection may be summed. The shelves at the customer level in a supermarket are often supported on secondary beams which, while providing some lateral stability to the upright frame columns, their end joints offer no moment resistance).*
- All moment connections of the racks simultaneously experience very similar rotations at all times.
- The vast majority of inelastic behaviour occurs at the moment connections.
- The overall seismic response can be reasonably modelled as a single degree of freedom system.

- That connection moment versus connection rotation curves have been developed based on third cycle peaks of the cyclic testing in accordance with Section 6.1.

Comment: Well-known procedures exist for the determination of the moment-rotation properties of the beam-column connections. These are commonly known as the portal test (see Clause 8.4.2 of AS 4084) or the cantilever test (see Clause 8.4.1 of AS 4084). These have been adapted for New Zealand so that an envelope curve is derived, passing through the third cycle peaks recorded in the rotation cycles of joints. The portal test has the benefit of the inclusion of a gravity force to ensure that the connections are correctly locked in place during cyclic loading. However, in service the gravity component is expected to be present and this can be simply accommodated if cantilever tests are undertaken by locally propping the connection in its fully locked down position. Locking pins, when part of the system, serve to prevent the beam component from disconnecting from the column, but these are loose connections and cannot be relied on in the test to keep the components fully locked together. While there is potential for "slackness" to develop in the connection more so in this test, the resulting moment-curvature relationship will err conservatively. The accidental omission of the pins in service is not expected to alter the performance of the rack system as long as the rack beams are loaded with stock. There is the potential for an unloaded and unpinned beam to lift when the rack is subjected to lateral down-aisle loading, resulting in possible disconnection. Hence, it is important that the pins are always in place in service.

Note that when connections with different properties are present in a particular rack, k_{cj} shall be the stiffness corresponding to the rotation at the maximum moment of the stiffest connection.

Comment: Testing undertaken at BRANZ as part of this study showed that the actual stiffness of the assembled rack in the down-aisle direction was very similar to the stiffness predicted by assuming that the individual stiffnesses of the joints in the assembled rack, determined from the cantilever test, could be summed. A plot showing the correlation is given in Figure 3.

STEP 1.

Determine the fundamental period of the rack, $T_{1,}$ based on the following equation:

$$T_1 = 2\pi \sqrt{\frac{\sum_{i=1}^{N_L} W_i h_i^2}{g\left(\sum_{j=1}^{N_t} N_{cj}\left(\frac{k_{cj}k_{be}}{k_{cj}+k_{be}}\right) + N_b\left(\frac{k_b k_{ce}}{k_b + k_{ce}}\right)\right)}}$$

Where

W_i = the effective horizontal seismic weight of the i[th] shelf level in the storage rack (see Section 3.1.4)

h_i = elevation of the approximate centre of gravity of the i[th] shelf level with respect to the base of the storage rack

g = acceleration of gravity

N_L = number of loaded levels

k_{cj} = rotational stiffness of each type of beam-to-column connection (note that this stiffness is for one side of the beam-column joint only – when a beam attaches to the other side of the column the total stiffness of the joint will doubled)

k_b = rotational stiffness of each base plate connection (k_b may be assumed to equal to k_{cj}, where j is the stiffest beam-to-column joint type, for installations where there is at least one bolt on opposite sides of the post in the down-aisle direction. When a single bolt is present at the neutral axis of the column, assume $k_b = 0$)

N_{cj} = the number of beam-to-column connections of a particular type

N_t = the number of beam-to-column joint types

N_b = the number of base plate connections

and where k_{be} is the beam end and k_{ce} is the base column end rotational stiffness assumed to be given by:

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

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

and where

E = Young's modulus of the beams and columns

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

L = clear span of the beams

I_c = moment of inertia of each base upright

H = clear height of the upright.

STEP 2.

Compute the maximum displacement D without P-Delta effects at the effective height of the single degree of freedom (SDOF) system of the rack based on the period T_1, for the ULS ground motion using the following equation.

$$D = \frac{gC(1)ZT_1}{4\pi^2 B}$$

where

$C(1)$ = the elastic site hazard coefficient at a period of 1 second for the site location and subsoil class (from NZS 1170.5 Figure 3.1).

Z = the hazard factor for the site (from NZS 1170.5 Table 3.3 or Figure 3.3 or 3.4)

B = Damping Coefficient as a function of the effective damping from the following table (taken from Table 13.3-1 of the 2003 NEHRP Provisions).

Damping	B factor
5%	1.0
10%	1.2
20%	1.5
30%	1.7

The above values of equivalent damping are based on experimental shake table testing performed by Chen et al [1980] and on extrapolation of recent testing performed by Filiatrault and Wanitkorku [2004].

STEP 3.

Adjust the displacement demand, D, to account for P-Delta effects by multiplying by the factor $(1+\alpha)$.

$D_{\max} = (1+\alpha)D$

where

$$\alpha = \frac{\sum_{i=1}^{N_L} \frac{W_i}{0.6} h_i \left(\frac{k_c + k_{be}}{k_c k_{be}} \right)}{\left(N_{cj} + N_b \left(\frac{k_b k_{ce}}{k_c k_{be}} \right) \left(\frac{k_c + k_{be}}{k_b + k_{ce}} \right) \right)}$$

where

k_c = the rotational stiffness of the stiffest beam-column connection
N_L = the number of loaded levels and all other terms are as defined earlier.

STEP 4.

Determine maximum demand rotation, θ_{demand} (drift angle), as follows:

$\theta_{\text{demand}} = D_{\max}/0.72h_{tot}$

where

h_{tot} = height of centre of gravity of the pallets on the top level
$0.72h_{tot}$ = height of centre of gravity of equivalent SDOF rack.

STEP 5.

Check whether maximum demand rotation, θ_{demand}, is less the maximum rotation capacity, $\theta_{\max}$. If the demand rotation is less than maximum rotation capacity, the connection design is acceptable and the designer can move to the next step. If not, the connection design is not acceptable, and it will be necessary to provide an alternate connection design and repeat the process from the determination of the first mode period of the rack (Step 1).

STEP 6.

If the connections are acceptable, assume maximum column moment, M_c, and axial load, P_c, from seismic loads (does not include gravity loads) are as follows:

$$M_c = 1.2\ M_{max}$$

$$P_c = 1.2\,N_L\ M_{max}/0.5L \quad \text{(end columns)}$$

$$P_c = 0 \quad \text{(central columns)}$$

where M_{max} has been determined for the joints in Section 6.1

and evaluate columns in accordance with the load combinations specified in Section 3.1.2 and design capacities determined using AS/NZS 4600.

(Note that the 1.2 factor is the assumed over-strength factor for the connection.)

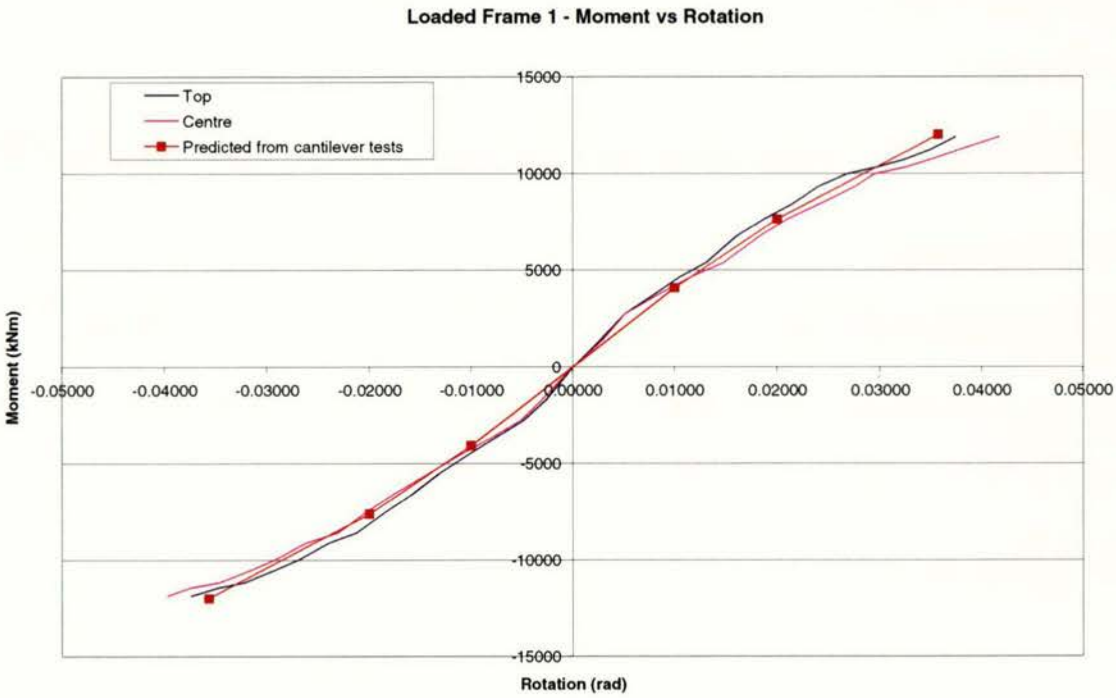


Figure 3: Correlation between stiffness of a single bay rack predicted from cantilever tests and actual behaviour

3.4.2.2 Example calculation for displacement-based procedure – down-aisle direction

Consider a single bay supermarket frame similar to the frame subjected to static and dynamic testing in the BRANZ Structures Laboratory. Details of the frame are given in Figure 4.

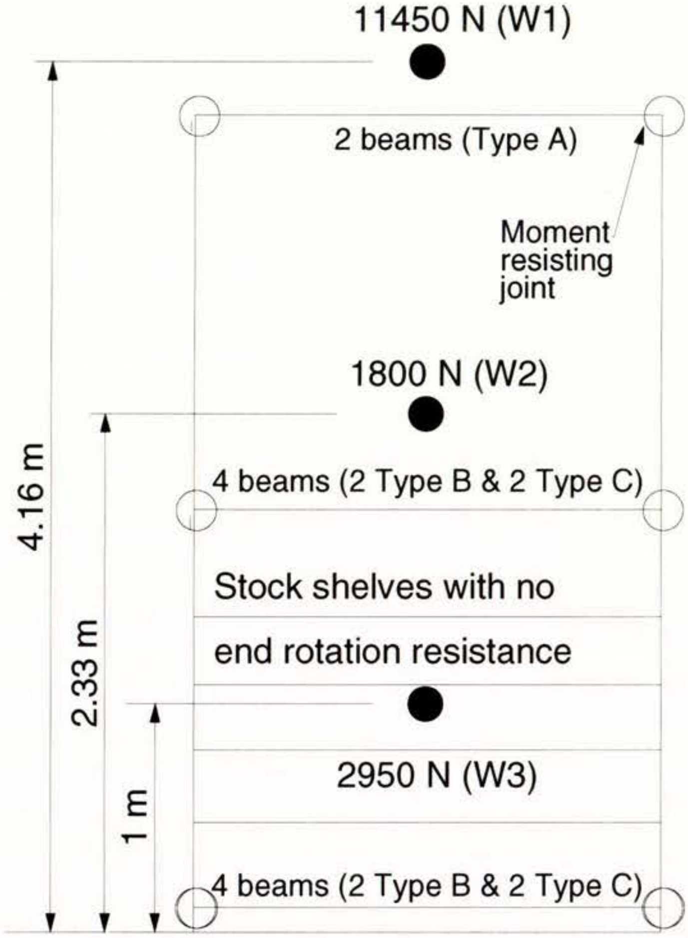


Figure 4: Details of example one bay frame

The properties of the frame elements are given in Table 2 and the weight distribution is given in Table 3.

Table 2: Frame element properties

Col 1	Col 2	Col 3	Col 4	Col 5	Col 6
Member	**Connection, k_{cj} (Nm/rad)**	**No. of connections, N_{cj}**	**k_{be} (Nm/rad)**	$\frac{k_{cj} \cdot k_{be}}{k_{cj} + k_{be}}$	**Col 3xCol 5**
Type A	18200	4	558338	17625	70502
Type B	7880	8	98414	7071	56569
Type C	8300	8	79106	7218	57745
	Sum N_c	20		Sum	184816
Column	--	--	k_{ce} = 168844	$\frac{k_b \cdot k_{ce}}{(k_b + k_{ce})}$	
Baseplate		4	k_b = 18200	16429	131433
		$\frac{k_c + k_{be}}{k_c \cdot k_{be}}$	$\frac{k_b \cdot k_{ce}}{(k_c \cdot k_{be})}$	$\frac{k_c + k_{be}}{(k_b + k_{ce})}$	
		5.67E-5	0.3024	3.082	

Table 3: Weight distribution

Col 1	Col 2	Col 3	Col 4	Col 5
Level	W_i **(N)**	h_i **(m)**	$W_i.(h_i)$	$W_i.(h_i)^2$
1	11450	4.16	47632	198149
2	1800	2.33	4194	9772
3	2950	1.00	2950	2950
		Sum	54776	210871

1. Calculate the fundamental period, T_1.

$$T_1 = 2\pi\sqrt{\frac{210871}{9.81x(184816+131433)}} = 1.6s \text{ (cf 1.2s in laboratory test)}$$

2. Calculate the displacement, without P-Δ effects, at the effective height of the single degree of freedom system.

$$D = \frac{gC(1)Z(T_1)}{4\pi^2 B} = \frac{9.81x2x0.4x(1.6)}{4\pi^2 x1.2} = 0.265m \text{ (10\% damping assumed)}$$

3. Adjust the displacement demand to account for P-Δ effects

$$\alpha = \frac{54776x5.67E-5}{20+(4x0.3024x3.082)} = 0.113$$

$$D_{\max} = (1+\alpha)D = (1+0.113)x0.265 = 0.295m$$

4. Calculate the maximum demand rotation

$$\theta_{demand} = D_{\max}/0.72h_{tot} = 0.295/0.72x4.160 = 0.098rad$$

5. Compare the maximum demand rotation with the maximum rotation capacity, θ_{max} (= 0.066). In this case, the capacity is exceeded. Therefore it will be necessary to either increase the stiffnesses of the joints or to reduce the load capacity of the system. For this example, if the capacity of the top pallet shelf is reduced to 5000N, $\theta_{demand} = 0.064$, which is satisfactory. Alternatively, the stiffnesses of the joints must be increased.

3.4.2.3 Evaluation and design procedures for cross-aisle direction

To evaluate the stability of the storage rack in the cross-aisle direction, the optional displacement-based procedure provided below in Section 3.4.2.4 is suggested for sites in areas of low seismicity.

It is assumed in this procedure that representative rack braced frames, including base plates and hold-down bolts, have been cyclically tested in-plane (i.e. in the cross-aisle direction) to determine the load-displacement characteristics of the rack to a point beyond the peak lateral load resisting capacity of the frame. It is assumed that the rack will have had axial load applied to simulate the load carrying capacity of the shelves at all levels during the lateral cyclic loading process. A force-displacement curve will have been developed from the tests which provides the cross-aisle displacement at the top level of the rack as a function of totally applied lateral force.

The procedure assumes that the rack can be modeled as a single degree of freedom (SDOF) system in the cross-aisle direction. Alternatively, the force displacement curve up to the maximum lateral force capacity of the rack can be obtained analytically where a curve is 'synthesised' from a step-linear analysis with releases inserted locally for yielding elements. The particular non-linear characteristics of the bracing systems and base plates must be accurately represented in such analyses. The axial and rotational capacity of the connections and members must be able to resist without fracture the forces induced at the maximum displacements.

It should be noted that the displacement-based evaluation procedure does not explicitly account for sliding of pallets and shifting of contents. For sites where the ultimate limit state short period ground acceleration levels are greater than 0.25 g, the procedure provided in Section 3.4.2.4 is likely to be very conservative because significant sliding of pallets and shifting of contents are expected to occur which should result in greatly reduced force demands on the columns and bracing.

In lieu of the displacement-based procedure in Section 3.4.2.4, an alternate limit state design approach is suggested in Section 3.4.2.6 for such situations. This approach directly considers that the forces in the columns are limited by pallet sliding. Such sliding has not been possible to accurately recreate in testing at New Zealand research facilities with the available equipment.

Comment: The BSSC Rack Project Task Group [BSSC 2005] acknowledges that both of the approaches that have been reproduced here in Sections 3.4.2.4 and 3.4.2.6 still require significant development, testing and verification before they gain general acceptance as a design procedure.

3.4.2.4 Displacement-based cross-aisle evaluation procedure

STEP 1.

From the experimentally established force displacement curve over third cycle peaks, determine the maximum lateral force, F_{max}, applied at the top level that the cross-aisle frame resists laterally. Also determine the lateral displacement, D_{max}, at the top level corresponding to the maximum lateral force. It is assumed that the displacement of the top level and the displacement at the centre of gravity of the pallets on the top level are the same and the displacements increase linearly with height. Therefore determine the displacement at the equivalent single degree of freedom height, D_{equiv}, by scaling by 0.72 where 0.72 is the factor which converts h_{tot} to the height of the single degree of freedom system.

$$D_{equiv} = 0.72\, D_{max}$$

STEP 2.

Determine the stiffness of the equivalent system as:

$$K = F_{max} / D_{equiv}$$

STEP 3.

Determine the equivalent period as:

$$T = 2\pi\left(\frac{W_s}{gK}\right)^{1/2}$$

where

W_s = the effective seismic weight contribution to one cross-aisle frame (see Section 3.1.4).

g = acceleration of gravity.

STEP 4.

Determine the spectral shape factor for the subsoil class, $C_h(T)$, at period T.

STEP 5.

Compute displacement demand, D_{demand}, of SDOF equivalent system as follows:

$$D_{demand} = \frac{C_h(T)ZW_s}{BK}$$

where

B = Damping Coefficient as a function of the effective damping taken from the following table (taken from Table 13.3-1 of the 2003 NEHRP Recommended Provisions).

Damping	*B* factor
5%	1.0
10%	1.2
20%	1.5
30%	1.7

STEP 6.

Check whether the displacement demand, D_{demand}, is less than the maximum displacement capacity, D_{equiv}. If the displacement demand is less than the displacement capacity, the cross-aisle direction performance is expected to be acceptable. If not, the cross-aisle direction performance is unacceptable and it will be necessary to provide an alternate cross-aisle frame design and repeat the above steps.

Comment: FEM 10.2.02 (Clause 4.3.3.1) notes that if the design value of the vertical load on the cross-aisle frame is less than 10% of the elastic critical value of the vertical load for failure in a sway mode, then the frame is sufficiently stiff to be able to neglect P-Δ effects. If the ratio is between 0.1 and 0.3, then the P-Δ effects are treated indirectly.

FEM 10.2.08 offers an alternative check. An inter-storey drift sensitivity coefficient, θ, is calculated based on:

$$\theta = {(P_{tot} d_r)}/{(V_{tot} h)} \leq 0.10)$$

where

P_{tot} = *total gravity load at and above the level considered, in the seismic design situation*

d_r = *design inter-level drift, evaluated as the difference of the average lateral displacements at the top and bottom of the level under consideration and calculated according to Section 2.4.5 of FEM 10.2.08*

V_{tot} = *total seismic level shear*

h = *inter-shelf height*

when 0.1 ≤ θ ≤ 2.0 the P-Δ effects can be approximately taken account of by increasing the relevant seismic action by a factor equal to 1/(1-θ). The value of θ shall not exceed 0.3.

3.4.2.5 Example calculation for displacement-based procedure – cross-aisle direction

1. Consider a single cross braced frame with an experimentally determined F_{max} of 10 kN and D_{max} of 50 mm at the top level and an effective seismic weight, W_s, of 15 kN

   $D_{equiv} = 0.72\ D_{max} = 36$ mm.

2. The stiffness of the equivalent system is:

   $K = F_{max}/D_{equiv} = 10/0.036 = 278$ kN/m.

3. The equivalent period is:

   $T = 2\pi(W_s/gK)^{1/2} = 2\pi(15/9.81/278)^{1/2} = 0.46$ s.

4. The spectral shape factor for the subsoil class at period T is 3.0.

5. The displacement demand of the equivalent single degree of freedom system is:

   $$D_{demand} = \frac{C_h(T)ZW}{BK} = \frac{3x0.4x15}{1.2x278} = 0.054\text{m} = 54 \text{ mm} > D_{equiv}$$

   Therefore, the design will need to be revised and the process continued from Step 1.

3.4.2.6 Cross-aisle limit state design approach

The following procedure is recommended in areas of high seismicity:

Determine the lateral limit state design forces on the racks as the weight of the pallets/contents on the rack times a design coefficient of friction of the pallets/contents.

For determining these design forces, it is recommended that the full pallet/contents weight be used. The design coefficient of friction should be the upper limit static coefficient of friction based on testing between the rack and the pallet/contents times a factor such as 1.2. It is suggested that the static coefficient of friction not be taken less than 0.30 unless testing and detailed analytical investigations indicate that a lower value is warranted and can be safely used without unacceptable amounts of pallet slippage.

Verify that the ratio of the section moduli of the beam minor to major axes is at least equal to design coefficient of friction.

Design the uprights, bracing and floor connections to stay elastic for the design limit state forces.

Determine the displacement of the uprights and beams for the design limit state forces.

Estimate the sliding displacements of pallets contents to be sure they are within acceptable limits. When determining the sliding displacement demands, it recommended that consideration be given to reducing the demands based on the concurrent displacements of the uprights and beams.

4. STOCKING RECOMMENDATIONS

4.1 Delivered stock and handling

The usual handling process for merchandise is for the stock to arrive at the store on pallets with either shrink-wrapping, stretch-wrapping or banding to keep the individual product items bound. The stretch-wrapping, shrink-wrapping and banding may also secure the products to the pallet. On arrival at the store the pallets are unloaded and stored unchanged. Generally, outside of store's public operating hours, the pallets will be loaded onto the top shelves of the racking systems. Also outside of the public hours, the pallets will be lowered and broken down either into smaller loads on new pallets, re-wrapped or re-banded, or into cartons or boxes that are able to be handled by staff, and stacked on the picking shelves. Stock is then picked by store employees from the picking shelf and the public access shelves are filled. The net result is that the loads on the shelf systems are ever-varying, depending on the arrival of new stock at the store, the time of day at which the break-down of pallets occurs and throughput of shoppers.

The nature of the racking systems employed in supermarkets, discount stores and home handyman stores is such that the likelihood of there ever being physical restraints provided for the stock that is picked by the customer is very low because of the inconvenience that would be caused to shop customers. For this reason, the maximum height of shelves above floor level containing unsecured stock is to be limited to 1.8 m.

4.2 Supermarkets

4.2.1 Picking shelves (above 2 m)

Product that is being prepared for stocking the public access shelves is stored at a level approximately 2 m above floor level in supermarkets. Only store workers are allowed access to this level. The stock generally takes the form of individual cartons that are often stored as towers relating to product type. These towers are potentially unstable in an earthquake and must therefore be restrained from falling onto the customers below.

Suitable restraint systems include:

- wire mesh grillages that may be slid sideways in guide channels to allow access
- hinged wire mesh doors with mechanical latches
- hinged panel doors with mechanical latches
- cargo nets with appropriate perimeter fixings that will provide support for the net, but allow easy decoupling for access.

In all cases, the restraint system must be reinstated immediately after the stock items have been removed to fill the lower shelves.

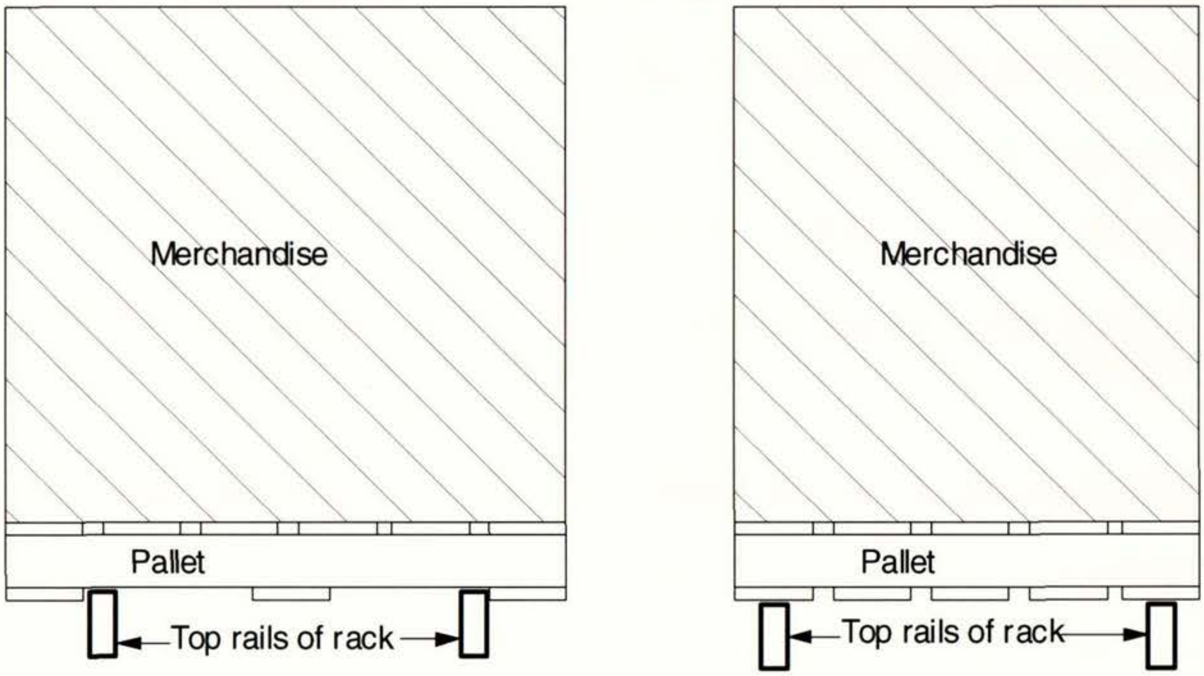
Such restraints must be capable of resisting a lateral force equivalent to 10% of the weight of the stored stock applied as a uniform distribution over the area of the restraint system. Experimental investigations [Sarabjit 2004] have shown that there is sufficient friction present between the lowest carton and the shelf to prevent sliding, and therefore the forces imposed on the restraint system are only due to toppling cartons of product.

4.2.2 Pallet shelves

Pallet shelves either take the form of longitudinal beams at the front and back of the system on which the pallets are directly supported (Figure 5) or, in the case of back to back shelf systems, secondary beams span between the longitudinal members and the pallets are supported on these (Figure 6). As can be seen in Figure 5(a), the pallet is mechanically restrained on the top beams, whereas in Figure 5(b) there is a potential for the pallet to slide off the beam under severe lateral motion. The pallet in Figure 6 can potentially slide on the transverse beams, but is restrained by the upstand at the outside edge of the beam.

When pallets are restrained from sliding, and if the restraint is provided at the base of the pallet, there is a potential for the pallet to rock and possibly overturn. Assuming a uniform density for the product stored on the pallet, and if the pallet is subjected to a 0.5g lateral acceleration and it has a height-to-width ratio of 2, then there is a potential for the pallet to rock and possibly overturn [Nigbor et al 1994]. However, the reversing action of the earthquake is likely to result in no more than rocking as long as the height-to-width ratio does not exceed 2.

Pallets must not be stored on racks if they are damaged. Damage includes pallets with missing or broken boards or stringers.



(a) (b)

Figure 5: Pallets supported directly on beams

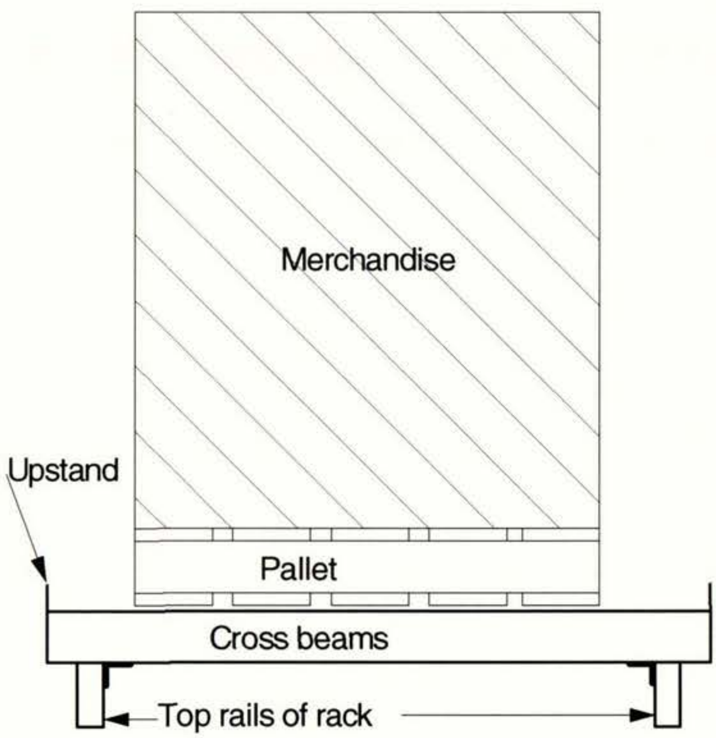


Figure 6: Pallets supported on secondary beams over longitudinal rack beams

4.2.3 Recommendations for supermarket product storage

1. All arriving pallets of product should be properly secured to the pallet. This may be achieved by either shrink-wrapping or stretch-wrapping that envelopes not only the product but the pallet itself, or banding of the product items to the pallet. See Section 6.2.1 for pallet tilt test procedure for determination of effectiveness.

2. Pallets which contain heavy items such as canned goods should be checked to ensure that the pallet weight is within the weight limits for the supporting shelves and, if not, the product should be demounted and re-stacked in smaller lots on new pallets. Re-wrapping or re-banding will be necessary.

3. Institute a store policy of only storing lightweight items such as toilet paper and packeted goods on pallet shelves in areas where the public has access. Generally, stores will have a storage area away from public access where heavy goods can be stored.

4. Ensure that the height-to-least-width ratio of the palletised products stored above public access areas does not exceed 2.

5. Ensure that pallets are prevented from sliding off the pallet shelf by either using pallets with missed bottom boards, so that the pallet stringers sit directly on the longitudinal beams and the outer boards form a mechanical lock to prevent slippage (Figure 5(a)), or ensuring that the ends of the transverse beams have solid upright tabs to secure the pallet (Figure 6).

6. Pallets are sometimes required to span between longitudinal beams on the rack. Ensure that the pallets being used to store products on the rack are not damaged to the extent that the damage may lead to premature collapse. Such damage includes cracked stringers and missing boards.

7. Demounted cartons and boxes stored on picking shelves above 2 m from the floor should be prevented from toppling on store customers in the event of an earthquake. Adequate restraint systems include sliding steel mesh doors, hinged mesh or solid doors with mechanical latches and perimeter-fixed cargo nets, all having the ability to resist a force equivalent to the weight of 10% of the stored product spread uniformly over the area of the restraint. Deformation of the restraint system is acceptable provided the products are retained.

8. It is recommended that products on shelves to which the public has access be stored so that the heavier items are stored at the lowest practical level and the lighter items are stored at the highest practical level. Wooden shelves provide a greater slip resistance for products than either flat steel or steel mesh shelving [Sarabjit 2004]. If possible, products such as jars, cans and bottles should be stored in single layers, thus maximising the slip resistance between the product and the shelf.

9. When possible, storage of individual items within 'cut-down' cartons will serve to provide some restraint to the products against toppling from the shelves.

4.3 Home handyman stores and discount warehouses

4.3.1 Picking shelves and public access shelves

There is not the same demarcation between goods being selected by the public and goods that are being temporarily stored before being moved to the public access shelves, as occurs in a supermarket. Because of this, there are not always gates or screens provided to prevent stock toppling in an earthquake.

Home handyman stores and discount warehouses contain a greater variety of stock sizes than a supermarket, and the items cover a much wider range of products (e.g. crockery, ornaments, footware, electrical and plumbing fittings, household appliances). The nature of the end use of the goods may mean that individual products are either heavy or bulky and the rack arrangements are adjusted to suit this.

Bulk discount stores use cargo net restraints, but inspection of a typical bulk discount store revealed that the nets were stretched between two horizontal wires on a shelf approximately 3 m above the floor. No restraints were provided on shelves below this level, and yet there were many heavy items such as crockery and tools stored on shelves above 2 m from the floor. In some cases these items were stacked two or more high on a shelf.

The situation in home handyman stores also varies between store chains. Of two companies inspected, one had racks up to approximately 4 m above the floor. Where smaller items such as tools, electrical components, etc, were stored, solid timber doors were used to restrain goods on shelves above approximately 2.5 m from the floor. However, when the items were large (e.g. oil column heaters) no restraint was provided, even though they were stored above 2 m. The second home handyman store had racking up to approximately 3 m. No restraints were provided on any of the shelves. Heavy items such as paint pails were stored as high up as the top shelf. On occasions, the items were positioned with their outer edges slightly proud of the shelf front edge, negating the retaining against sliding advantage of the generally small step up from the timber shelf onto the steel support beam.

4.3.2 Pallet storage

No pallets were observed on any shelves in the inspected discount store. Pallet storage in the home handyman stores generally is in the form shown in Figure 5 because the shelves are wider to accommodate the larger items and are therefore more amenable to supporting the pallets directly.

4.3.3 Recommendations for discount store product storage

1. Institute a store policy of only storing lightweight items such as towels and linen on the upper-most shelves. Generally, stores will have a storage area away from public access where heavy goods can be stored.
2. Ensure that the height-to-least-width ratio of the products stored above public access areas does not exceed 2 where the width is measured in the front to back direction of the shelf.
3. Demounted cartons and boxes stored on shelves above 2 m from the floor should be restrained against toppling on store customers in the event of an earthquake. The products on these shelves are not expected to be accessed by the public, but by store staff. Adequate restraint systems include sliding steel mesh doors, hinged mesh or solid doors with mechanical latches, bars, horizontal cables and perimeter-fixed cargo nets, all having the ability to resist a force equivalent to the weight of 10% of the stored product spread uniformly over the area of the restraint system. Deformation of the restraint system is acceptable provided the products are retained.
4. It is recommended that products on shelves to which the public has access be stored so that the heavier items are stored at the lowest practical level and the lighter items are stored at the highest practical level. Wooden shelves provide a greater slip resistance for products than either flat steel or steel mesh shelving [Sarabjit 2004]. If possible, products such as jars, cans and bottles should be stored in single layers, thus maximising the slip resistance between the product and the shelf.

5. When possible, storage of individual items within 'cut-down' cartons will serve to provide some restraint to the products against toppling from the shelves.

4.3.4 Recommendations for home handyman store product storage

1. All arriving pallets of product should be properly secured to the pallet. This may be achieved by either shrink-wrapping or stretch-wrapping that envelopes not only the product but the pallet itself, or banding of the product items to the pallet. See Section 6.2.1 for pallet tilt test procedure for determination of effectiveness.
2. Pallets which contain heavy items such as pails of paint should be checked to ensure that the pallet weight is within the weight limits for the supporting shelves and, if not, the product should be demounted and re-stacked in smaller lots on new pallets. Re-wrapping or re-banding will be necessary.
3. Institute a store policy of only storing lightweight items on pallet shelves in areas where the public has access. Generally, stores will have a storage area away from public access where heavy goods can be stored.
4. Ensure that the height to least width ratio of the palletised products stored above public access areas does not exceed 2.
5. Ensure that pallets are prevented from sliding off the pallet shelf by either using pallets with missed bottom boards, so that the pallet stringers sit directly on the longitudinal beams and the outer boards form a mechanical lock to prevent slippage (Figure 5(a)), or ensuring that the ends of the transverse beams have solid upright tabs to secure the pallet (Figure 6).
6. Pallets are generally required to span between longitudinal beams on the rack. Ensure that the pallets being used to store products on the rack are not damaged to the extent that the damage may lead to premature collapse. Such damage includes cracked stringers and missing boards.
7. Demounted cartons and boxes stored on picking shelves above 2 m from the floor should be prevented from toppling on store customers in the event of an earthquake. Adequate restraint systems include sliding steel mesh doors, hinged mesh or solid doors with mechanical latches, fixed horizontal cables and bars, all having the ability to resist a force equivalent to the weight of 10% of the stored product spread uniformly over the area of the restraint. Deformation of the restraint system is acceptable provided the products are retained.
8. It is recommended that products on shelves to which the public has access be stored so that the heavier items are stored at the lowest practical level and the lighter items are stored at the highest practical level. Wooden shelves provide a greater slip resistance for products than either flat steel or steel mesh shelving [Sarabjit 2004]. If possible, products such as jars, cans and bottles should be stored in single layers, thus maximising the slip resistance between the product and the shelf.
9. Products should be stored on the shelf and not allowed to overhang the front edge of the shelf.
10. When possible, storage of individual items within 'cut-down' cartons will serve to provide some restraint to the products against toppling from the shelves.

5. OPERATIONAL GUIDE (QA)

5.1 Statement of design information and assumptions

At the design stage, it is recommended that the designer clearly indicate to the store owner the minimum number of beam-column joints required to be present in the rack for the given rack gravity load capacity, to ensure that the earthquake load capacity in the down-aisle direction is maintained. This information should also include the maximum allowable unsupported length for the upright columns.

In the cross-aisle direction, a similar notification to the store owner is recommended, although the supplied beams will govern the spacing of the upright frames and a lengthening of the span could not occur without new beams being provided. However, the store owner may elect to move the complete rack at a later date, and therefore clear details of the required fixing strength of the cross-aisle frame bases to the floor are also required.

5.2 Public notification of shelf capacities

Currently, some companies provide a clear label on all shelves of the racking system giving the capacity of the shelf (Figure 7). It is recommended that this be a mandatory requirement of all public access racking systems, and possibly should be considered also for systems that do not have public access.

Figure 7: Example shelf capacity label

5.3 Alterations

In terms of the NZBC, any shift of a racking system within a store would not require a Building Consent because there would be no change of use. Similarly, alterations to an existing installation would not require a Building Consent, provided that the alteration was consistent with the original design. However, the involvement of either the original designer or a suitably qualified engineer is strongly recommended when an alteration is carried out.

Particularly in the down-aisle direction, the strength and stiffness of the rack system is highly dependent on the connections between the upright frames and the beams. These will have been designed by the rack designer to resist the owner-specified gravity loads and the earthquake design loads for the location. Removal of beams to accommodate different products, which may have a higher mass density, will be a direct removal of earthquake load resistance and possibly a removal of critical buckling support for the columns in the upright frames.

Once the alteration has been completed, an inspection should be carried out by an appropriately qualified person (e.g. the original rack designer) to confirm that the original design assumptions have not been violated in the alteration/reconfiguration. Of particular importance is to ensure that all connection locking devices have been re-installed.

5.4 Damage to racks

Inspections of current installations have suggested that the potential for damage to the racking system from the operation of forklifts is low in the public access situations covered by this Design Guide. Forklift access tends to be outside of normal public access hours, and because only the top shelf is required to be accessed with a forklift, the chances of impact damage from a forklift are far less than in a commercial bulk store. Nevertheless, all situations where any deformation of the rack has been caused by forklift impact must be immediately made known to the store manager who will be either able to decide on the severity of the damage or who be able to obtain expert advice. In such circumstances, the area should either be cordoned off until cleared by the manager or their adviser or repaired.

5.5 Routine inspections

Regular inspections of racking systems are recommended. The frequency of inspections is likely to vary, depending on the degree of use. Nevertheless, routine inspections should be undertaken at intervals of not less than one year.

Such inspections should cover at least the following:

- missing or damaged anchor bolts
- missing or damaged connection locking devices
- damaged, dented, buckled or bent rack members
- corrosion and other deterioration
- fractured welds (if present)
- rack frame misalignment (tolerance on verticality = height/350 (source FEM 10.2.02)
- beam spreading and improper beam installation
- overloading and other improper material storage
- inadequately restrained contents.

5.6 Post-earthquake inspection

After earthquakes that have caused racks to collapse, or to become seriously misaligned, the store should be immediately closed to the public and an assessment carried out by an appropriately qualified person on the ongoing safety of the damaged system for recovery of trapped people and clean-up of stock. Under no circumstances should personnel be allowed into the area without expert advice on the safety of the damaged racks. It is likely that temporary shoring will be required to provide adequate support to the damaged rack before it will be sufficiently safe to allow entry for rescue and clean-up operations.

Immediately after all earthquakes that have caused the loss of products from the shelves, but no immediately apparent damage to the racks, the store should be closed to the public and a thorough inspection of the racking system should be carried out. It will be particularly important to ascertain the status of all stored items with respect to their stability and to ascertain the condition of the rack itself.

Beam end connectors must be fitted with locks to prevent connectors from disengaging during accidental vertical uplift shear load, for example, during forklift operation. Unloaded beams could also disengage in earthquakes if not locked in position.

6. COMPONENT TESTS

6.1 Mandatory tests

6.1.1 Cantilever tests

Cantilever tests are required to establish the moment-curvature properties of the joints between the beams and the upright frame columns. For cold formed steel section racks, the stiffnesses of the beams and columns are significantly greater than the stiffness of the joint itself. Their contribution to the joint flexibility can therefore be conservatively ignored in the determination of the joint properties in the cantilever test.

Comment: The test is based on that detailed by the Federation Europeenne de la Manutention [FEM 2001], the Rack Manufacturers Institute [RMI 1997] and Australian Standard AS 4084 [SA 1993], except that for New Zealand the joint is subjected to reversed cyclic loading. For those systems with fitted lock mechanisms to prevent disconnection of the joint, it may be possible to cycle the joint without further modification. However, the expectation is that in service the shelves will be loaded, which will keep the joints firmly locked together, and so it is important that stops be added to the test specimen to simulate the loaded situation.

The cantilever test evaluation procedure is as follows:

The test set-up is as shown in Figure 8. The column section is held firmly between supports and a reversing load is applied to the cantilevering beam section. The joint is cycled through increasing levels of displacement, three cycles to each increment, until failure occurs. A typical displacement history for a joint is given in Figure 9. Failure is generally identified by shearing of the connection 'teeth' or pins.

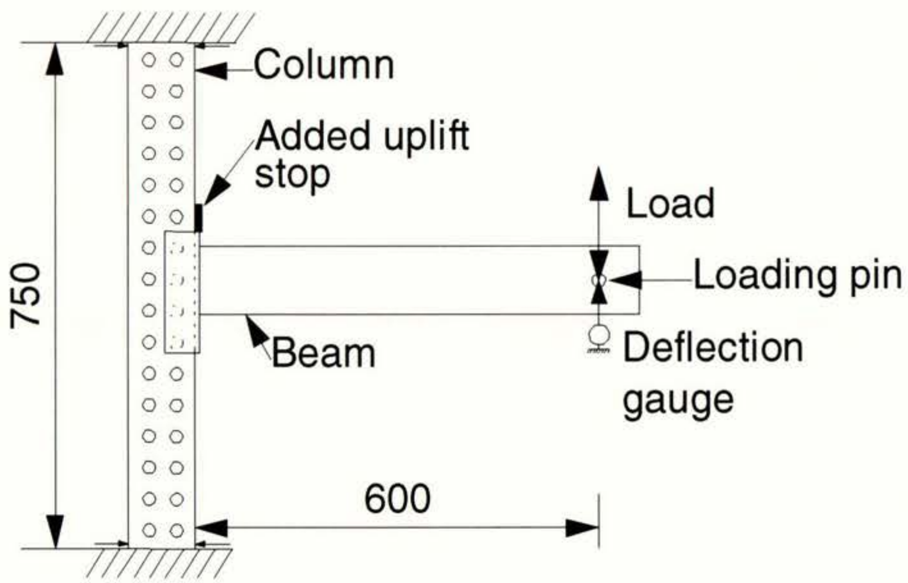


Figure 8: Cantilever test set-up

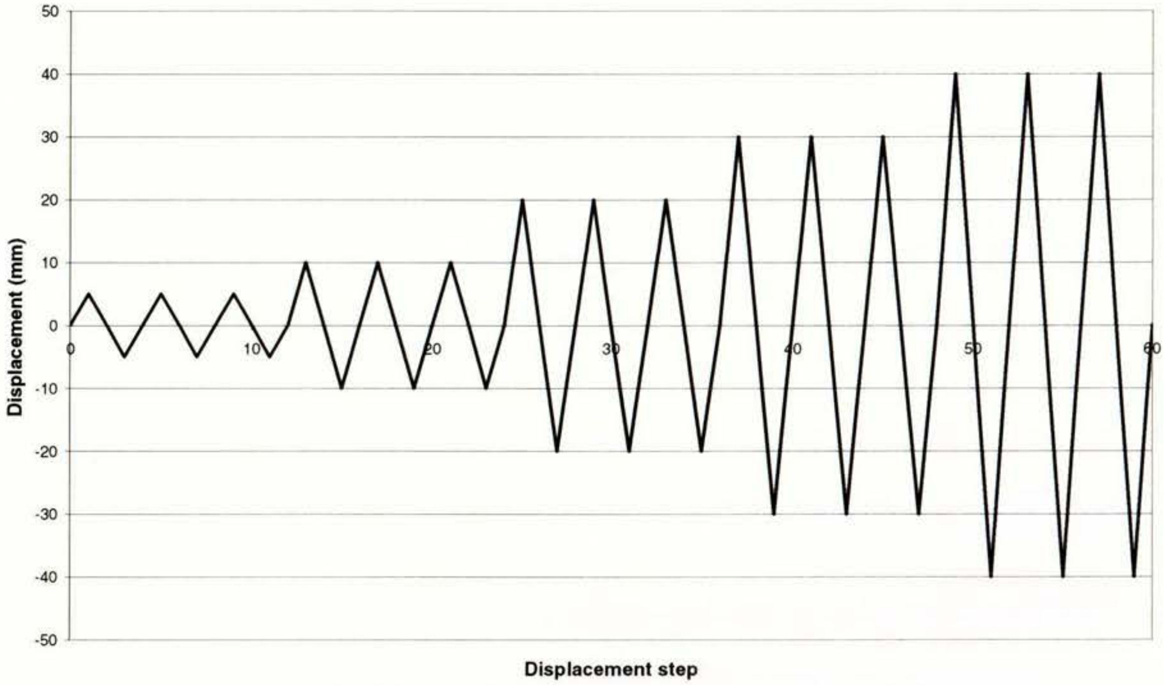


Figure 9: Typical displacement history for the loading point in a cantilever test

A typical moment-curvature plot for the joint is presented in Figure 10. A line is drawn through the maximum moments achieved in the third cycle in both directions and the mean of the two is determined. The maximum moment, M_{max}, is the peak moment on this curve.

From the average third cycle peak moment-rotation curve established for the connections for both directions of loading, determine the rotation, θ_{max}, at the maximum moment, M_{max}. Determine the maximum permitted connection stiffness, k_c, as follows:

$k_c = M_{max} / \theta_{max.}$

A minimum of three replicate tests must be undertaken and the characteristic failure moment and the characteristic stiffness of the joint shall be the lower 5%ile of the values obtained in the replicate tests, calculated as the mean of the individual results minus k_s times the standard deviation, where k_s is defined in Table 4.

Table 4: k_s factor

Number of replicates	k_s
3	3.15
4	2.68
5	2.46
6	2.33

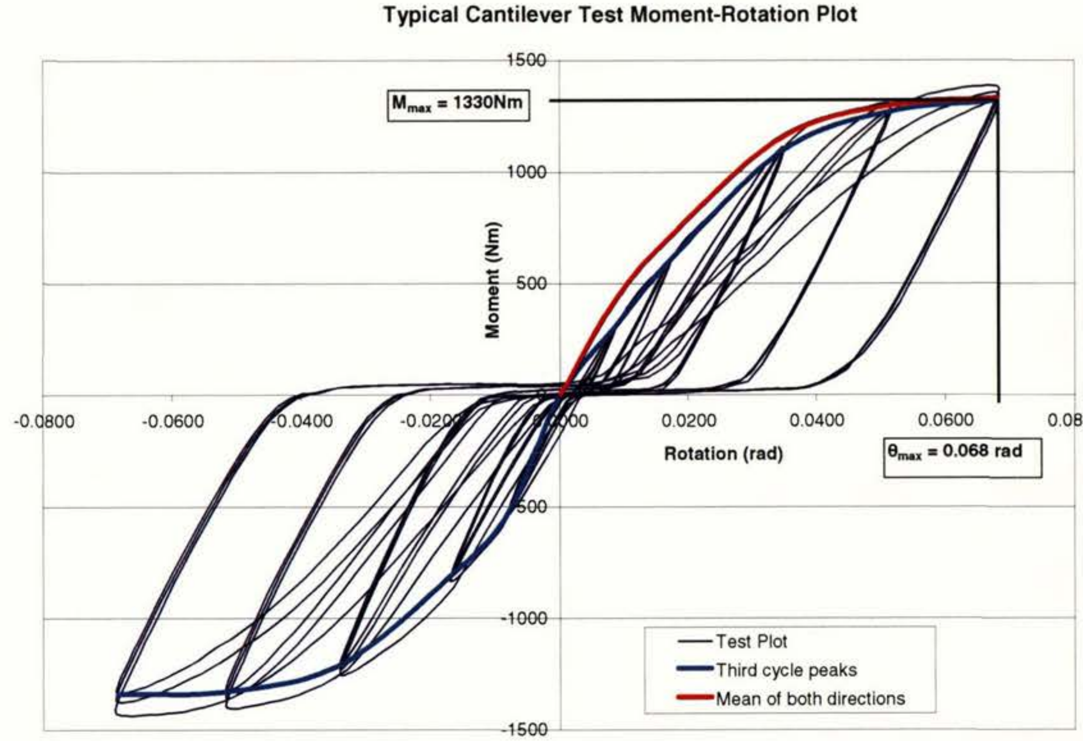


Figure 10: Determination of the joint stiffness and strength from the moment-rotation plot

Comment: Note that in cantilever tests undertaken at BRANZ on typical beam-column connections, it was determined that the moment-rotation performance in one direction was generally similar to the other. FEM 10.2.02 notes (Clause 4.2.2.1) that the reverse capacity of such connections should be limited to 50% of the capacity in the usual direction of loading if subjected to reverse bending under pattern loading. However, given the testing experience at BRANZ, this limitation is not considered necessary for cold formed steel racking systems available in New Zealand.

6.1.2 Portal tests (alternative to the cantilever test)

Portal tests may be undertaken instead of cantilever tests, but the set-up is significantly more expensive than for the cantilever test, involving more rack components and the inclusion of representative shelf loads.

Comment: The test is based on that detailed by both the Rack Manufacturers' Institute [RMI 1997] and FEM. [FEM 2001], except that for New Zealand the joint is subjected to reversed cyclic loading. The FEM provides a means of determining the maximum moment and the corresponding rotation and from these the connector stiffness. This is reproduced below.

Two upright frames are required, seated on pinned bases, with two beams placed at a point that is a minimum of 600 mm above the floor. A load equivalent to the design load of the beams, W, is placed on the beams and a horizontal force equal to the horizontal design load corresponding to the vertical load on the assembly is applied equally to the two columns at one end of the assembly, at the level of the top of the beams and in the direction of the beams (Figure 11). The specimen is cycled three times to each of a sequence of increasing displacements until the maximum achievable horizontal load is reached at the third cycle and the load is measured continuously. The horizontal deflection is also measured continuously at the top of the beams.

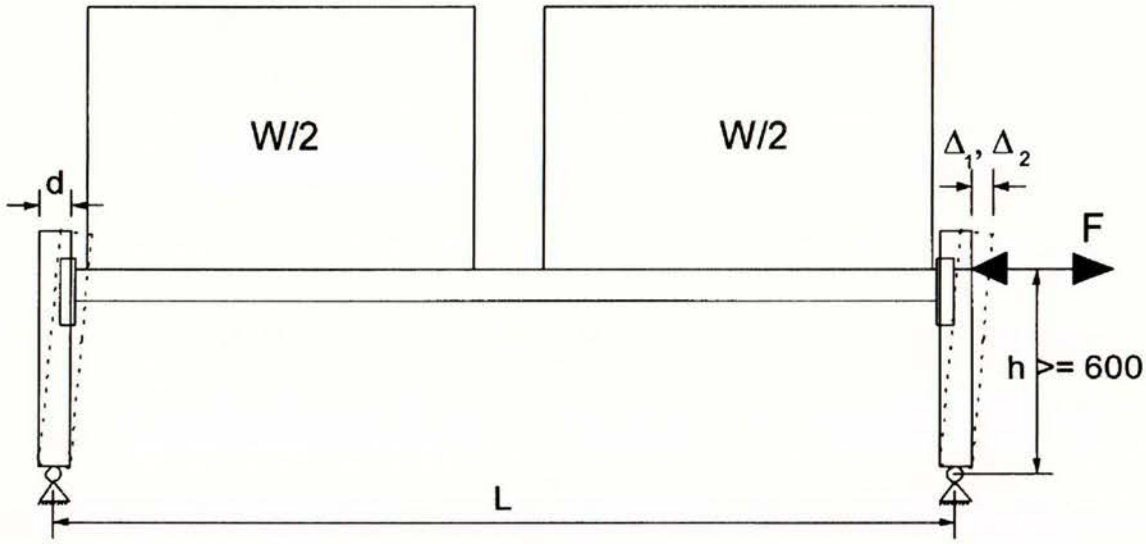


Figure 11: Portal test set-up

From the plot of load versus displacement, the maximum moment in the connector is calculated from:

$$M = \left[\frac{Fh}{4} + \frac{W\Delta}{2}\right]\left[1 - \frac{d}{L}\right]$$

where

F	=	maximum achievable third cycle load
h	=	height of the beam above the pivot point
L	=	length between the uprights
W	=	shelf load
d	=	width of the face of the upright
Δ	=	$(\Delta_1 + \Delta_2)/2$ and Δ_1 and Δ_2 are the deflections of the two portal frames.

The corresponding rotation is calculated from the deflection at the maximum achievable third cycle load, taking into account the flexibility of the beams and columns, as follows:

$$\theta = \frac{\Delta}{h} - F\left[\frac{h^2}{12EI_c} + \frac{hL}{24EI_b}\right]$$

where

θ	=	corrected rotation of the connection
EI_c	=	flexural rigidity of the upright (gross cross-section)
EI_b	=	flexural rigidity of the beam.

The characteristic failure moment and the characteristic stiffness may be determined using the same procedure as for the cantilever test (Section 6.1.1).

6.2 Non-mandatory tests

6.2.1 Pallet tilt tests

The pallet tilt test is one which can be used by store owners to establish the effectiveness of shrink-wrapping, stretch-wrapping and banding in securing merchandise to pallets. The angle of tilt is equivalent to the application of a 0.34g lateral force on the contents of the pallet.

The procedure is as follows:

1. The merchandise is bound to the pallet.
2. The pallet is lifted on one side mid-distance between the corners to produce a 20° angle between the ground and the bottom surface of the pallet (Figure 12).
3. If the merchandise remains restrained in place for a minimum of 5 minutes without appreciable movement, the load secured to the pallet is considered to have adequate confinement and passes the tilt test.
4. If the merchandise shifts appreciably, or the securing material breaks and the merchandise topples out, the merchandise must be re-secured by an alternative appropriate means and the test is repeated.

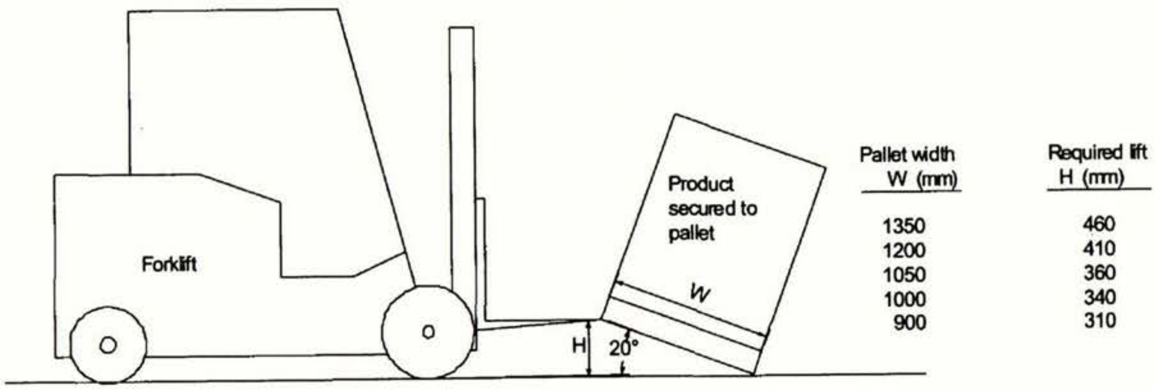


Figure 12: Pallet tilt to test to check stability of products secured to pallets

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