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# Seismic Considerations for Steel Storage Racks Located in Areas Accessible to the Public

FEMA 460 – September 2005



**FEMA**





*of the National Institute of Building Sciences*

# **SEISMIC CONSIDERATIONS FOR STEEL STORAGE RACKS LOCATED IN AREAS ACCESSIBLE TO THE PUBLIC**

## **FEMA 460 — September 2005**

**Prepared by the  
Building Seismic Safety Council  
for the  
Federal Emergency Management Agency**

National Institute of Building Sciences  
Washington, D.C.  
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Building Seismic Safety Council activities and products are described at the end of this report. For further information, see the Council's website ([www.bssconline.org](http://www.bssconline.org)) or contact the Building Seismic Safety Council, 1090 Vermont, Avenue, N.W., Suite 700, Washington, D.C. 20005; phone 202-289-7800; fax 202-289-1092; e-mail [bssc@nibs.org](mailto:bssc@nibs.org).

## PREFACE

The Federal Emergency Management Agency (FEMA), which is part of the Department of Homeland Security, works to reduce the ever-increasing cost that disasters inflict on the nation. Preventing losses before they occur by designing and constructing buildings and their components to withstand anticipated forces from various hazards is one of the key components of mitigation and is the only truly effective way of reducing the cost of future disasters.

As part of its responsibilities under the National Earthquake Hazards Reduction Program (NEHRP), FEMA supports activities necessary to improve technical quality in the field of earthquake engineering. Among these activities are investigations of seismic and multihazard technical issues and the development, publication, and dissemination of technical design and construction guidance documents. One key document is the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. The *NEHRP Recommended Provisions* and its accompanying *Commentary* volume present minimum criteria for the earthquake-resistant design and construction of buildings and building components. The *NEHRP Recommended Provisions* is applicable anywhere in the nation at risk from earthquakes and is a voluntary resource document widely used by practicing design professionals and building officials. It serves as the basis for the seismic requirements of the nation's model building codes and standards.

During development of the 2003 edition of the *NEHRP Recommended Provisions*, the issue of the seismic safety of steel pallet storage racks located in areas accessible to the public was raised. This issue has taken on added significance given the increasing proliferation of "big-box stores" and the associated additional risk to the public in an earthquake, and was deemed important enough to justify FEMA's funding of the project resulting in this report. FEMA's interest and concern regarding the performance of steel storage racks accessible to the public actually predates this project. In 1998, FEMA funded the Collaborative for Disaster Mitigation (CDM) through the California Office of Emergency Services using Hazard Mitigation Grant Program funding. One of the projects funded by the CDM was a paper by Chan and Yee at San Jose State University entitled "Structural Behavior of Storage Racks Under Seismic Ground Motion."

This report is intended to serve as a stand-alone resource document as well as input for the 2008 update of the *NEHRP Recommended Provisions*. While a lack of available, nonproprietary research data was a constraint during development of this report, FEMA believes that the Building Seismic Safety Council's Storage Rack Project Task Group did an excellent job of assembling and assessing data that were available and preparing this report. The guidance contained in this document represents the best retail industry practices observed as well as the recommendations of a panel of distinguished experts in the storage rack and earthquake engineering fields. It is our hope that this guidance will be strongly considered by retail stores and their operators as it will serve to increase the safety of their customers as well as reduce their potential liability. FEMA wishes to express its gratitude to the task group as well as to the Rack Manufacturers Institute (RMI), the Retail Industry Leaders Association, and the other individuals and organizations contributing to this effort.

*Federal Emergency Management Agency*

## **ACKNOWLEDGEMENTS**

The Building Seismic Safety Council is grateful to all those who participated in this effort. Special thanks go to Rack Manufacturers Institute (RMI) and the Retail Industry Leaders Association for their cooperation and to the Storage Project Task Group for their outstanding contribution of time and expertise.

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## EXECUTIVE SUMMARY

During the past few decades, the number of large public warehouse stores (often referred to as big-box stores) across the nation has grown significantly, changing both consumer buying habits and the public's risk of injury during earthquakes. During an earthquake, occupant safety in a big-box store depends on both the structural performance of the building and on the performance of the storage racks and their contents. Earthquake ground motions can cause storage racks to collapse or overturn if they are not properly designed, installed, maintained, and loaded. In addition, goods stored on the racks may spill or topple off. Both occurrences pose a life-safety risk to the exposed shopping public.

The immediate stimulus for the project that resulted in this report was a 2003 request from the State of Washington to the Federal Emergency Management Agency (FEMA) for guidance concerning the life-safety risk posed by the storage racks in publicly accessible areas of retail stores, especially the risk of rack collapse or loss of stored goods during an earthquake. FEMA asked the Building Seismic Safety Council (BSSC) to develop the requested guidance. To do so, the BSSC established a Rack Project Task Group composed of practicing engineers, storage rack designers, researchers, representatives of the Rack Manufacturers Institute (RMI) and the Retail Industry Leaders Association, and members of applicable technical subcommittees responsible for updating the *NEHRP Recommended Provisions*.

In developing this guidance document, the Task Group focused primarily on steel single selective pallet storage racks. It reviewed available information on storage rack performance during earthquakes and the background on the development of standards and code requirements for storage racks; assessed seismic requirements for storage racks and current practices with respect to rack design, maintenance and operations, quality assurance, and post-earthquake inspections; and examined available research and testing data.

Based on its study, the Task Group developed short-term recommendations to improve current practice and formulated long-term recommendations to serve as the basis for improved standards documents such as the *NEHRP Recommended Provisions*, ASCE 7, and the RMI-developed storage rack specification (ANSI MH 16.1-04 *Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks*).

Over the near term, the Task Group recommends that the 2003 *NEHRP Recommended Provisions* requirements for steel single selective pallet storage rack design be followed and that connections be checked in accordance with a procedure to be developed by RMI. The Task Group also recommends that additional guidance presented in this report be voluntarily adopted by store owners and operators. Further, given the fact that maintenance and use of storage racks is a key element to their acceptable performance during earthquakes, store owners and operators should adopt an appropriate quality assurance plan; as a minimum, the best self-imposed practices of store owners and operators should be maintained.

The Task Group's primary long-term recommendation is that the RMI specification be brought into conformance with the 2003 *NEHRP Recommended Provisions*, which is the basis for seismic requirements found in current seismic design standards and model building codes. The Task Group also recommends that optional performance-based and limit state procedures and component cyclic testing procedures be incorporated into the RMI-developed specification. Compliance with these procedures will demonstrate that the storage racks have the capacity to resist maximum considered earthquake ground motions without collapse. Once the procedures are appropriately integrated into ANSI MH 16.1-04, it is recommended that the 50 percent increase factor and minimum force level included in the *NEHRP Recommended Provisions* requirements and building codes and standards be waived for racks designed in accordance with these procedures. It also is recommended that regulatory bodies periodically review the quality assurance programs of stores and implement any regulations needed to satisfy life-safety concerns that relate to the securing of rack contents and rack maintenance and use.

Additional recommendations appear in Chapter 13 and the reader is encouraged to consider that chapter as well as this summary.



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# Chapter 1

## INTRODUCTION

### 1.1 BACKGROUND

During the past few decades, the number of large public warehouse stores (often referred to as big-box stores) across the nation has grown significantly, changing both consumer buying habits and the public's risk of injury during earthquakes. Whereas traditional retailers typically store goods and products outside the retail space in limited access storage rooms and warehouse facilities, big-box stores keep goods in close proximity to the consumer at all times. Typically, shoppers in these stores browse in aisles between steel storage racks, 14 to 18 feet in height, that hold pallets of inventory, some of which can be very heavy.

During an earthquake, occupant safety in a big-box store depends on both the structural performance of the building and on the performance of the storage racks and their contents. Earthquake ground motions can cause storage racks to collapse or overturn if they are not properly designed, installed, maintained, and loaded. In addition, goods stored on the racks may spill or topple off. Both occurrences pose a life-safety risk to the exposed shopping public.

The seismic design for new warehouse stores, including both the buildings and storage racks, is governed by the building code in force in the jurisdiction where a store is built. The seismic requirements in building codes currently being enforced in most U.S. jurisdictions are based on the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, which is developed by the Building Seismic Safety Council (BSSC) of the National Institute of Building Sciences with National Earthquake Hazards Reduction Program (NEHRP) funding provided by the Federal Emergency Management Agency (FEMA).

The seismic requirements for new stores and storage racks, however, do not stipulate how goods are to be stored on the racks. For California, however, the state has enacted a California Labor Law that requires contents placed on storage racks 12 feet or higher above ground level to be secured. While this law includes a few suggestions for securing goods, it does not include detailed procedures. Thus, in virtually all jurisdictions, requirements for securing storage rack contents are self-imposed by store owners and operators. The situation is further complicated by the fact that these types of storage racks can be easily reconfigured (i.e., shelf level changed) to meet changing merchandising needs. The reconfiguration work, which can be performed many times over the life of the structure, generally is done by store employees who may not always understand required procedures. Further, fork lifts are used to load goods on the racks and the racks can easily be damaged in the process. Finally, heavy merchandise stored on the floor near storage racks can topple during an earthquake and damage rack columns and braces, perhaps even initiating rack collapse.

The immediate stimulus for the project that resulted in this report was a 2003 request from the State of Washington to FEMA for guidance concerning the life-safety risk posed by the storage racks in publicly accessible areas of retail stores, especially the risk of rack collapse or loss of stored goods during an earthquake. The goal was the possible development of state regulations in response to a fatality resulting from a broken pallet on a storage rack in a commercial retail hardware facility. It should be noted that the accident was not associated with an earthquake. In light of the 2001 Nisqually earthquake, however, the State of Washington recognized that a significant life-safety hazard could be associated with storage racks in earthquakes.

FEMA was aware that current storage rack requirements were somewhat out of date. For example, the latest edition of the storage rack design specification developed by the Rack Manufacturers Institute (RMI) references the obsolete 1994 *NEHRP Recommended Provisions* even though the *Provisions* document has been updated three times since then (i.e., in 1997, 2000, and 2003). Furthermore, during the 1994 Northridge earthquake (magnitude = 6.7), serious storage rack collapses occurred in several warehouse retail stores that would likely have resulted in injuries and possibly deaths if the earthquake had occurred during a time when the stores had significant public presence rather than at 4:30 a.m. on a federal holiday. Many existing racks have since been voluntarily strengthened or replaced and stricter quality assurance programs for rack loading and reconfiguration have been implemented by some owners with the hope of preventing a reoccurrence of the Northridge problems. In addition, the 1994 *NEHRP Recommended Provisions* included a 50 percent increase of seismic loads for storage racks in areas accessible to the general public. FEMA also recognized that consideration must be given in the design process to much larger earthquakes than those recently experienced in order to avert collapse in areas of highest seismicity. Thus, when asked for assistance by the State of Washington in developing guidance for storage racks in warehouse retail stores, FEMA concluded that such a project was well justified.

## 1.2 CONDUCT OF THE PROJECT

To develop the requested guidance, a Rack Project Task Group was established composed of practicing engineers, storage rack designers, researchers, representatives of the Rack Manufacturers Institute (RMI) and the Retail Industry Leaders Association, and members of relevant technical subcommittees responsible for updating the *NEHRP Recommended Provisions*. Also included were representatives of various government entities with an interest in and/or responsibility for public safety. (See page v for a list of Task Group members and Appendix E for a list of other participants in the project.) To provide for broader input and review of an initial draft of this guidance document, the Task Group conducted a workshop for storage rack design engineers and manufacturers, retailers, building officials and other regulators, and structural engineers involved in the seismic code development process. In addition, the Task Group chair, FEMA, and NIBS staff met with representatives of the large retailers who use storage racks in public warehouse stores to discuss the draft guidance document and obtain additional feedback.

The *NEHRP Recommended Provisions for Seismic Regulations for New Building and Other Structures*, developed by a Provisions Update Committee (PUC) and a series of PUC Technical

Subcommittees, serves as a primary source document for the seismic code requirements found in current building codes in the United States. The information and recommendations in this report on storage racks will be submitted to the application PUC Technical Subcommittee for consideration during development of the 2008 *Provisions*. In addition, this report will be submitted to the applicable RMI technical committees for consideration during development of the next edition of the RMI design specification.

### 1.3 SCOPE OF THIS REPORT

This report highlights issues for consideration in the seismic design, installation, ongoing inspection, maintenance, and use of steel single selective pallet storage racks located in areas of retail warehouse stores and other facilities accessible to the general public. These considerations apply only to single selective steel pallet storage racks (Figure 1-1) with contents elevated 8 feet or more above the ground floor; they do not apply to shelving, racks less than 8 feet in height, merchandise displays, or cantilever racks of any height.



*Figure 1-1 Typical steel single selective pallet storage rack.*

The structural elements in a typical pallet storage rack are shown in Figure 1-2. Storage racks are composed of specially designed steel elements that permit easy installation and reconfiguration consistent with the merchandising needs of a warehouse retail store. Except where adjacent to walls, storage racks normally are configured as two rows of racks that are interconnected. Pallets typically can have plan areas of between 14 and 16 square feet and can

have a maximum loaded weight of approximately 3,500 pounds. Storage rack bays are typically 42 to 44 inches deep and 96 inches wide and can accommodate two pallets. The overall height of pallet rack structural frames found in retail warehouse stores varies between 14 and 18 feet. Column extensions (posts that extend above the top level to provide lateral support for netting, cables, etc.) provided on some racks typically range from 3 to 6 feet in height. The rack industry calls the longitudinal direction the down-aisle direction and the transverse direction, the cross-aisle direction. Proprietary moment connections are typically used as the structural system in the down-aisle direction and braced frames are typically used as the structural system in the cross-aisle direction. Photographs of typical down-aisle moment frame connections, cross-aisle braced frame connections, and column base plate connections are presented in Figures 1-3, 1-4, and 1-5.

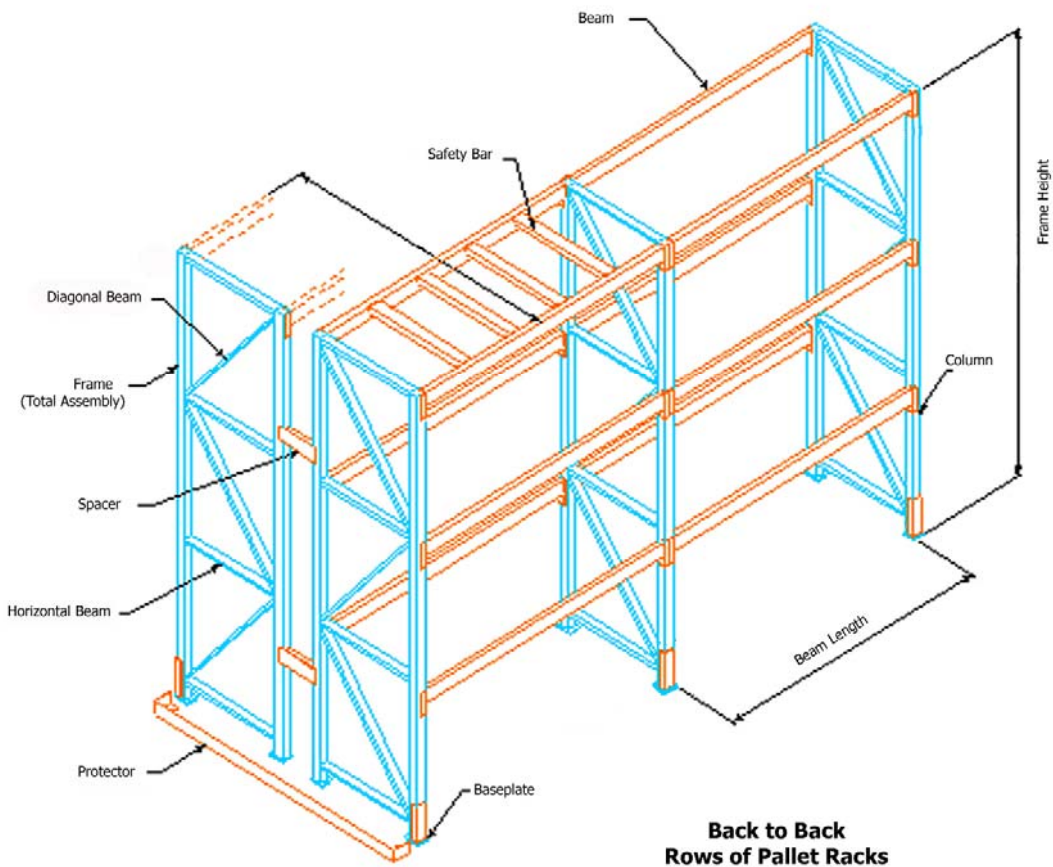


Figure 1-2 Typical storage rack configuration.



*Figure 1-3 Typical proprietary moment connection.*



*Figure 1-4 Typical rack bracing members and connections.*





*Figure 1-5 Typical column base plate connection.*

#### **1.4 CONTENTS OF THIS REPORT**

Included in this report are: a review of the performance of storage racks in past earthquakes, a history of the development of codes and standards used for storage rack design and current storage rack design practices; guidance on recommended performance goals and design requirements for storage racks; guidelines for implementation/responsibilities associated with the specification, procurement, and installation of pallet storage racks; suggested guidance for securing contents; recommendations for operations and use; suggested guidance for quality assurance programs; a discussion of current and past storage rack research and testing, suggestions for post-earthquake inspections; and proposed modifications to seismic design provisions and standards for racks.

Most of this report is intended for all readers who have an interest in the seismic protection of steel single selective pallet storage racks and their contents; however, Chapters 4 through 6 and Appendices A through D, are very technical in nature and will be of interest primarily to rack design engineers, seismic code and standards writers (including RMI and the *NEHRP Recommended Provisions* Technical Subcommittee 8, seismic policy makers, and building officials. Appendix B is intended specifically for the RMI Specification Advisory Committee whereas Appendix C is intended for TS 8.

## Chapter 2

# DAMAGE TO STEEL PALLET STORAGE RACKS AND CONTENT SPILLAGE IN RECENT EARTHQUAKES

### 2.1 BACKGROUND

To determine whether the seismic requirements in codes and standards are adequate, engineers and code officials carefully investigate the performance of structures subjected to earthquake ground motions. The use of steel single selective pallet storage racks in areas accessible to the public is a relatively recent development. Since first being introduced on a significant scale about 25 years ago, the number of big box stores utilizing storage racks in public areas has increased dramatically – especially during the past 15 years. Although relatively few damaging earthquakes have occurred during the period of time, it is valuable to document steel storage rack damage and contents spillage during several earthquakes in California (i.e., the 1987 Whittier, 1989 Loma Prieta, 1992 Landers, 1994 Northridge, and 2003 San Simeon earthquakes) and one in Washington (i.e., the 2001 Nisqually earthquake).

To assess storage rack earthquake performance, it is important to determine the ground motions that occurred at the site, the design capacity of the rack, the actual loading of the rack, rack design details, and any other extenuating conditions that would affect rack performance. When an earthquake occurs, it generates ground motions, but the actual ground motions that occur at a given site are the function of many factors. These factors include the magnitude of the earthquake, the depth of the earthquake below the ground's surface, the distance the site is from the earthquake fault that generated the earthquake, and the soil conditions at the site. Of particular interest are sites where ground motions were equal to or exceeded the design level ground motions on which seismic code provisions are based. Because of the nature of storage racks, of equal importance is the actual pallet loading at the time of the earthquake relative to the rated capacity of the storage rack. Thus, in a given earthquake, damage to a particular rack might not occur because the rack was lightly loaded or because the ground motion at the site was not severe. It is difficult to gauge the performance of racks that have not really been subjected to their design limit.

The observations reported in this chapter represent only a very small fraction of the facilities that have experienced earthquakes. Less than 1 percent of the total reports and inspections made over almost 20 years in these stores describe unsatisfactory performance, failure or life safety hazards. The other 99 percent have performed without incident. The two major failures described in this chapter appear to be the result of overloading beyond any approved design limit. However, a full design-level earthquake has not yet occurred in the area where many of these stores are located. The current database includes only one event -- the 1992 Landers, California, earthquake -- of relevance and the racks in the warehouse store there performed adequately. The reader should be aware, however, that given the limited data available, it cannot be assumed that past performance is indicative of what to expect in a design level earthquake in a heavily populated area.

The photographs and most of the seismic performance observation data presented below were provided by the office of Peter S. Higgins and Associates. The Task Group is grateful to Mr. Higgins for sharing this information and allowing it to be published in this document. Information on the San Simeon earthquake was provided by FEMA Project Officer Michael Mahoney and the FEMA Post-Earthquake Investigation Team, which included Task Group members Jeffery Lusk of FEMA Region X and Joshua Marrow of Simpson Gumpertz and Heger, Inc.

## **2.2 DAMAGE REPORTED IN THE 1987 WHITTIER EARTHQUAKE**

The Whittier, California, earthquake with a moment magnitude ( $M_w$ ) of 5.9 occurred on October 1, 1987, at 7:42 a.m. PDT at a depth of 5-3/4 miles. The epicenter was located in the Whittier Narrows region of southern California approximately 13 miles southeast of downtown Los Angeles and approximately 30 miles from Canoga Park. Several Home Depot and Home Club stores were inspected but no seismic damage was found except for damage associated with impact or reconfiguration. Of particular interest (for reasons that will be discussed later) were the Home Depot and Home Club stores in Canoga Park, both of which were operating at the time with racks in their originally installed reconfiguration. The racks in these stores were not damaged, which is not surprising since the peak ground motions in the Canoga Park area were reported to be in the range of 0.05g, far below the design basis earthquake ground motion levels.

## **2.3 DAMAGE REPORTED IN THE 1989 LOMA PRIETA EARTHQUAKE**

The Loma Prieta, California, earthquake with a moment magnitude ( $M_w$ ) of 6.9 occurred on October 17, 1989, at 5:04 p.m. PDT at a depth of 11-1/2 miles. The epicenter was located in the Santa Cruz Mountains about 60 miles south of San Francisco. Many seismically vulnerable structures in the Bay Area were damaged. Because of the distance to the epicenter, measured ground motions in the Bay Area were actually significantly below design level ground motions. After the earthquake, 28 Bay Area Price Club stores were inspected and no damage was observed except for one line of racks in the bakery area of one store. These racks were heavily damaged but did not collapse, and it was later determined that store personnel had removed the bottom beam level of these racks so they could put bread carts within the pallet racks to get them "out of the way." With the bottom beam level removed, the racks had less than a third of their rated capacity, which appears to be the primary reason for the damage.

## **2.4 DAMAGE REPORTED IN THE 1992 LANDERS EARTHQUAKE**

The Landers, California, earthquake with a moment magnitude ( $M_w$ ) of 7.3 occurred on June 28, 1992, at 4:57 a.m. PDT at a depth of 10-3/4 miles. The epicenter was located near the town of Landers in the high desert area of California about 120 miles east of downtown Los Angeles. Because of the remote location, there was only one store with storage racks accessible to the public located in the area of severe ground motion. A relatively new Builders Emporium (opened two to three years earlier) was located very close to the epicenter and less than 100 yards from the fault rupture zone. The building was heavily damaged. Glulam girders had broken away from their anchorages in the wall pilasters and, in some places, had battered their

way through the walls. The ledgers were almost completely destroyed. Unanchored gondola shelving units were not seriously damaged and were placed back into service but a great deal of small merchandise fell from the shelves and, in some cases, the units “walked” over 12 inches from their initial positions. Many small items also fell off storage racks, but items more than 24 inches in plan dimension (e.g., roof ventilators and HVAC pipes) stayed on the racks regardless of the height at which they were stored.

## **2.5 DAMAGE REPORTED IN THE 1994 NORTHRIDGE EARTHQUAKE**

The Northridge, California, earthquake with a moment magnitude ( $M_w$ ) of 6.7 occurred on January 17, 1994, at 4:31 a.m. PST at a depth of 11 miles. The epicenter was located in Reseda in the San Fernando Valley. Because of the fault orientation and rupture direction, ground motions were at or above design levels in the north end of the valley and were just about at code design levels in Santa Clarita, about 10 miles north of the valley.

**2.5.1 Price Club, Northridge, California.** No damage was observed except for the “soda wall” racks (storage racks fully loaded with cases of soft drinks), which had failed longitudinally. It is believed that this failure resulted when floor-stacked merchandise toppled over and hit the racks, inflicting heavy damage because identical racks with the same or heavier loads standing nearby were undamaged.

**2.5.2 Home Depot, Santa Clarita, California.** This store had only been open for a few days when the earthquake occurred. Several bays of racks failed completely in the cross-aisle direction, and there was also considerable loss of contents (see Figures 2-1 through 2-7).

A subsequent investigation revealed that the racks that failed had been overloaded by more than 50 percent of their rated capacity (i.e., the ceramic tile racks were loaded three pallets wide rather than the assumed two pallets wide and building materials pallets, which weighed 3,500 pounds each at this store, had been improperly placed on general merchandise racks rated to carry only 1,500 pounds per pallet position). Although no post-earthquake evaluation of the failed racks is publicly available, it was noted that properly loaded racks immediately adjacent to the overloaded bays were not damaged.

One row of building materials storage racks had a pronounced lean after the earthquake. This was apparently caused by material on the floor toppling onto the uprights and bracing during the earthquake. Once this material was removed, the racks self-centered, stood plumb again, and remained in service.

As a result of the failures at this new store, over 160 Home Depot stores were retrofitted with heavier, higher capacity racks in their ceramic tile areas. The cost of the retrofit is estimated to be approximately \$15 million (in 1993 dollars).



*Figure 2-1 Cross-aisle collapse in the building materials section of the Santa Clarita store.*



*Figure 2-2 Cross-aisle collapse in the tile section of the Santa Clarita store.*



*Figure 2-3 Cross-aisle collapse at Santa Clarita store showing good performance of shrink wrapped merchandise.*

*Figure 2-4 Contents spillage in the door/window area of a Santa Clarita store.*



*Figure 2-5 Contents spillage in the wood trim area of a Santa Clarita store.*



*Figure 2-6 Contents spillage in the paint area of a Santa Clarita store.*



*Figure 2-7 Contents spillage in a small-items section of a Santa Clarita store.*



**2.5.3 Home Depot, Canoga Park, California.** Home Depot replaced 32 percent of their pallet positions in this store due to failed or seriously damaged racks. There is no question that serious injury and perhaps loss of life might have occurred had the store been open to the public at the time of the earthquake. It is believed that the cause of the failures was essentially the same as in Santa Clarita – significant overloading of racks.

Home Depots and Home Clubs initially did not sell large amounts of building materials, and their storage racks were designed for general merchandise rated at 1,500 pounds per pallet position. As the stores grew, the merchandise mix changed and more building materials were kept in stock. Building materials storage racks at this store are rated to carry up to 3,700 pounds per pallet position or more than twice that of the general merchandise racks. Further, the building materials storage racks usually were one level taller than the general merchandise racks in the older stores. The change in merchandise mix occurred in 1991 and, as noted above, the racks in these stores had been undamaged in the earlier 1987 Whittier earthquake.

This older store had been scheduled for a retrofit to heavier rack designs but it had a low ceiling that could not accommodate the taller rack designs. Thus, it appears that the retrofit was cancelled because of the low ceiling problem but the switch to heavier loads was made inadvertently without the concomitant stronger racks required. This oversight was largely responsible for the failures. The racks in this store subsequently were replaced with heavier, higher capacity 12-foot high designs.

**2.5.4 Home Depot, Glendale, California.** Only minor damage was observed in this store. There were small amounts of fallen merchandise. One pallet did fall out of a storage rack, but the rack upright had been seriously damaged before the earthquake and was scheduled to be replaced the day the earthquake occurred.

**2.5.5 Home Club, Canoga Park, California.** This store was just down the street from the Home Depot and also suffered extensive damage, in some cases more severe than that in the Home Depot. As was the case with the Home Depot, storage racks loaded beyond their rated capacity were deemed to cause the problems. The reasons for this overloading are discussed in more detail above in Section 2.5.2. The store was later fitted with heavier racks but subsequently closed.

There was one interesting observation at this store. A properly designed set of racks, fully loaded with heavy building materials, survived the earthquake without damage or incident. However, the racks were bootlegged (e.g., not permitted) and were sitting on asphalt with no anchorage.

## **2.6 DAMAGE REPORTED IN THE 2001 NISQUALLY EARTHQUAKE**

The Nisqually, Washington, earthquake with a moment magnitude ( $M_w$ ) of 6.8 occurred on February 28, 2001, at 10.24 a.m. PST at a depth of 31 miles. The epicenter was located in the Puget Sound area. Because of the great depth, the surface ground motions were considerably less than those of the Northridge earthquake even though the magnitude of the Nisqually earthquake was greater. The measured ground motions were significantly less than code level

design ground motions at all locations in the area. Stores in Seattle were investigated including one on reclaimed land, one adjacent to the fallen highway overpasses, and one south of Olympia.

**2.6.1 Home Depot, Olympia, Washington.** Of the stores examined, only the Olympia store located in the area nearest to the epicenter had storage rack damage and that consisted of element buckling of some diagonal braces in the building materials racks. Very little merchandise fell out of the racks and almost all of what did fall consisted of very small items that posed little apparent life safety hazards. The water heaters stored on racks had recently been fitted with restraints and many were observed to be leaning on these restraints indicating that they probably would have fallen had there been no restraints.

**2.6.2 Costco, Various Washington Locations.** No Costco store was observed to have any damage but this is not surprising considering the level of ground motion was far less than code design levels.

## **2.7 DAMAGE REPORTED IN THE 2003 SAN SIMEON EARTHQUAKE**

The San Simeon, California, earthquake with a moment magnitude ( $M_w$ ) of 6.5 occurred on December 22, 2003 at 11:16 a.m. PST at depth of 4-1/2 miles. The epicenter was located 6 miles north of San Simeon and about 30 miles north of Atascadero where the area's largest big-box store with pallet storage racks was located. Measured ground motions were far less than code level ground motions and no storage rack damage was observed at the store; however, a small lawn tractor on a steel pallet slid off the top of one rack and cans of roofing tar fell off another rack from a significant height, breaking when they hit the ground and splattering tar high enough to reach the roof of the store. The fall of the lawn tractor on the steel pallets raises issues regarding the increasing use of pallets with a lower coefficient of friction (e.g., steel and plastic) than the traditional wood pallets.



## Chapter 3

### CODES AND STANDARDS FOR STORAGE RACKS

#### 3.1 RMI STANDARD

Pallet storage racks were created to optimize warehouse and distribution center operations. As individual storage rack manufacturers developed new and competing products, the need for design and utilization standards and their implementation by the user and producer industries became obvious.

The Rack Manufacturers Institute (RMI) was established and incorporated in 1958 to deal with industry-wide issues. Among its initial activities was development of the first edition of an RMI standard, *Minimum Engineering Standards for Industrial Steel Storage Racks*, which was issued in 1964. This document was a short, simple, direct exposition of what had been developed and what was known by the members of the industry at that time. It represented the first step in developing specifications and other products designed to suit the needs of users, manufacturers, and the engineering and code-enforcement communities.

In the late 1960s, RMI engaged Professor George Winter of Cornell University to undertake analysis and testing needed to provide a sound basis for the development of a more rigorous standard for the industry. Professor Winter was chosen because of his national and international reputation and his demonstrated expertise in the structural behavior of cold-formed light-gage steel structures. The RMI membership and several other organizations provided financial and engineering support for the research effort that included analysis and testing related to the expanding range of products made by the growing number of industry members. The results of the work conducted by Professor Winter and his graduate students provided the basis for a new RMI standard, *Interim Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks*, which was issued in 1972 and which required earthquake loads to be considered in a manner mimicking the approach to building structures as stated in the *Uniform Building Code (UBC)* promulgated by the International Conference of Building Officials (ICBO).

In the *UBC*, design seismic forces for different types of building structures were based on  $K$  factors. The  $K$  factors for ordinary moment frame building structures braced framed structures were 1.0 and 1.33, respectively. These were the factors used to define the seismic forces in 1972 edition of the RMI standard in the down-aisle and cross-aisle directions, respectively.

### 3.2 MODEL BUILDING CODE REQUIREMENTS

As the use of storage rack structures increased across the nation, they began to be subjected to diverse loading conditions and received more scrutiny by the engineering and building code communities. Storage rack structures were mentioned for the first time in the 1973 edition of the *UBC* in the form of a footnote to a list of structures. The 1976 *UBC* referenced Standard 27-11, an ICBO-developed standard that addressed storage racks specifically and included design seismic forces for storage racks. The Building Officials Code Administrators International (BOCAI) and the Southern Building Code Congress International (SBCCI) soon added seismic provisions for storage rack structures to their codes -- the *National Building Code* and *Standard Building Code*, respectively.

### 3.3 NEHRP RECOMMENDED PROVISIONS REQUIREMENTS

The first edition of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* was issued in 1985. The *NEHRP Recommended Provisions* was intended to serve as a resource document for use by organizations in developing seismic requirements for inclusion in codes and standards and to provide a venue for ongoing improvement of these requirements. A Provisions Update Committee (PUC) composed of structural engineers, industry and standards representatives, and building officials guides the triennial *Provisions* update process. RMI became a BSSC member organization in 1987 and, since that time, RMI representatives have served on the PUC Technical Subcommittees responsible for requirements for steel structures; for architectural, mechanical, and electrical components and systems; and for nonbuilding structures.

The 1991 edition of the *NEHRP Recommended Provisions* introduced design values for storage racks in the chapter on architectural, mechanical, and electrical components. The design seismic forces were independent of period and the lateral force coefficient, based on allowable stress design (ASD), was 0.40 for areas of highest seismicity. In the 1994 *Provisions*, design values for storage racks were significantly revised to be more consistent with the RMI seismic design criteria. *R* values of 6 and 4 were assigned for storage racks in the down-aisle direction and the cross-aisle direction, respectively. Further, an importance factor of 1.5 was assigned for racks in areas accessible to the public. The *R* factor values of 6 and 4 were basically a translation of the *UBC K* values from the early 1970s.

Note that the first several editions of the *NEHRP Recommended Provisions* focused almost entirely on building structures; however, by the mid-1990s formal recognition was given to the fact that storage rack structures (and some other structures) are neither building structures nor architectural, mechanical, or electrical elements or components. Starting in the 1997 *NEHRP Recommended Provisions*, nonbuilding structures including storage racks as well as cooling and storage towers, which had been treated in the chapter on architectural, mechanical, and electrical components were now covered in a separate nonbuilding structures chapter.

### 3.4 ASCE 7 REQUIREMENTS

As the *NEHRP Recommended Provisions* became more influential with the various model code organizations, the American Society of Civil Engineers (ASCE) chose to become a stronger

participant in the efforts to establish the basis for the specification of minimum loads on structures and developed the first of a series of updates to the ASCE 7 standard, *Minimum Design Loads for Buildings and Other Structures*. The 1993 edition of ASCE 7 adopted the 1991 *NEHRP Recommended Provisions* as its seismic provisions and the 1995 edition of ASCE 7 reflected the 1994 edition of the *NEHRP Recommended Provisions* and covered storage rack structures under the category of architectural, mechanical, and electrical components and systems.

### **3.5 CHANGING LANDSCAPE OF CODES AND STANDARDS**

Over time, the responsibilities for seismic standards development have evolved. Currently, the *NEHRP Recommended Provisions* still serves as the basic resource for the ASCE 7 Seismic Task Committee, which translates the *Provisions* into mandatory language and can be adopted by reference in codes and standards. The model building code landscape also has changed. The three earlier model building code organizations (ICBO, BOCAI, and SBCCI) have merged into one entity, the International Code Council (ICC), and the ICC's *International Building Code (IBC)* has replaced the three earlier codes. In the first edition of the *IBC* issued in 2000, seismic provisions were included directly in the code and were based primarily on 1997 *NEHRP Recommended Provisions*. Some of these requirements were replaced by reference to ASCE 7 in the 2003 *IBC* and most of the remaining detailed seismic requirements are expected to be replaced by reference to ASCE 7 in the 2006 edition. During the late 1990s, the National Fire Protection Association (NFPA) decided that it, too, would develop and promulgate a model building code; this decision led to the issuance in 2003 of *NFPA 5000: Building Construction and Safety Code*, which largely references ASCE 7 for its seismic requirements.

During this time of change, RMI participated in and contributed to *NEHRP Recommended Provisions* update, ASCE 7, and model code efforts in order to attempt to keep its standard up to date and coordinated with the ongoing codes and standards efforts and in the hope that its standard would eventually become a stand-alone industry standard. Because of the changing codes and standards landscape in the mid-1990s, RMI decided not to update its standard until things settled down and, as a consequence, the RMI standard is currently considerably out of date since it is based on the 1994 *NEHRP Recommended Provisions*.

### **3.6 CURRENT SEISMIC REQUIREMENTS FOR STORAGE RACKS**

The 2003 edition of the *NEHRP Recommended Provisions* (FEMA 450) treats storage rack structures as nonbuilding structures and references the seismic provisions of the RMI standard but also requires that the mapped ground motions from the *NEHRP Recommended Provisions* be used for design and that certain limitations be placed on minimum base shear and seismic displacements. These requirements are “intended to assure comparable results from the use of the RMI standard, the *NEHRP Recommended Provisions*, and the *IBC* approaches to rack structural design.” In addition, procedures are provided to “distinguish between the methods employed to design storage racks supported at grade (treated as nonbuilding structures) from those supported above grade (treated as architectural, mechanical, and electrical components).” The treatment within the *NEHRP Recommended Provisions* “helps to clarify and coordinate the multiple references to rack structures in the *NEHRP Recommended Provisions* and the different means by which rack structures are analyzed and designed.” Of particular interest to rack designers is the

minimum base shear coefficient of about 10 percent of the weight of the rack in areas of high seismicity. This minimum was originally imposed in the 2000 *IBC* to account for uncertainties of rack moment connection capacity to accommodate maximum considered earthquake (MCE) demands.

Based on the approach in the 2003 *NEHRP Recommended Provisions*, the 2002 and 2005 editions of ASCE 7 allow use of the RMI standard subject to the *Provisions* requirements on ground motions, limitations on minimum base shear and seismic displacement or drift default values, again distinguishing between the approaches for storage racks supported at the base (which are to be designed as nonbuilding structures) and those supported above the base (which are to be designed as architectural, mechanical, and electrical components and systems).

The 2003 edition of the *IBC* references the 2002 edition of ASCE 7 for its seismic requirements, thereby invoking the use of the RMI standard subject to the requirements and limitations imposed by ASCE 7. The 2006 edition of the *IBC* is expected to reference the 2005 edition of ASCE 7.

The 2003 edition of *NFPA 5000*, like the *IBC*, states that the design, testing, and utilization of industrial steel storage racks shall be in accordance with the RMI standard subject to the requirements and limitations imposed by the Section 9 (Earthquake Loads) of the 2002 edition of ASCE 7.

### **3.7 THE EVOLVING RMI STANDARD**

As noted previously, the 1972 edition of the RMI standard introduced seismic requirements for storage rack structures; it was updated and reissued in 1979, 1985, 1990, and 1997 with each new edition being an expanded version of the previous one and each representing an effort to reflect seismic provisions that were being articulated in the most current editions of the *NEHRP Recommended Provisions*, ASCE 7, and the model codes. In a desire to provide higher levels of safety in locations open to the general public, the 1997 edition of the RMI standard, which was based on the 1994 *NEHRP Recommended Provisions*, introduced a higher importance factor for storage rack installations in such places as retail warehouse stores that increased the magnitude of the design seismic base shear, the basis for much of the structural design detail for specific installations.

Also as noted above, the 2002 edition of the RMI standard remains largely based on the 1994 edition of the *NEHRP Recommended Provisions* and the 1997 *UBC*, both of which are considered to be out of date. In producing the 2002 edition, RMI followed the ANSI canvassing process and the document is designated ANSI Standard MH16.1-2004, *Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks*. Even while the canvassing process was under way, the RMI membership was contemplating some additional changes to the standard that would reflect the most current editions of the *NEHRP Recommended Provisions*, ASCE 7, *IBC*, and *NFPA 5000* – particularly regarding formatting of the seismic data and use of information obtained from the 2003 *NEHRP Recommended Provisions* CD-ROM rather than the dated paper earthquake ground motion maps issued with earlier versions of the *Provisions*.

The evolution of the RMI standard is continuing with additional analytical and experimental research focused on improving the understanding of rack structural behavior under seismic conditions. The results of this work will provide a basis for input to the BSSC PUC and Technical Subcommittees, the ASCE 7 Seismic Task Committee, and the relevant ICC and NFPA committees as changes to these groups' documents are proposed and deliberated. Further, new and better information will allow future new editions of the RMI standard to more closely represent the behavior of rack structures during seismic events which, in turn, will permit the determination of more realistic and rigorous requirements related to displacements, base shear, beam-to-column connector properties, and component design and their role in the prediction of the period, drift, and overall rack structural behavior and performance during seismic events.

### **3.8 OTHER SIGNIFICANT RESEARCH EFFORTS**

Since the early 1970s, RMI has sponsored many analytical and experimental storage rack research projects conducted by Cornell University faculty members and graduate students. These studies have included full-scale, component, and element tests focusing on, among other things, hot-rolled and cold-formed structural elements, beams, columns, perforations, beam-to-column connectors and connections, base plates, flexural and torsional-flexural buckling, and testing and loading protocols. Some recent testing at Cornell is discussed in more detail in Chapter 11.

During the late 1970s and early 1980s, major research projects were undertaken, including sub-assembly tests at Stanford University and full-scale shake-table testing at the University of California/Berkeley using El Centro 1940 records, by URS/Blume (see John A Blume and Associates, 1973; and Chen, Scholl, and Blume, 1980a, 1980b, and 1981), with funding from the RMI membership and a large grant from the National Science Foundation (NSF). The results of that testing, along with analytical studies, provided important baseline information about storage rack seismic performance, helped identify avenues for further research, and articulated issues needing further study. The results of these projects are discussed in more detail in Chapter 11.

Some storage racks are typically constructed of cold-formed steel members; therefore, their design depends on the thorough understanding and application of the American Iron and Steel Institute's (AISI) *Specification for the Design of Cold-Formed Steel Structural Members*. Other storage rack members are manufactured and fabricated from hot-rolled steel structural sections using the applicable seismic provisions of the American Institute of Steel Construction's (AISC) *Specification for Structural Steel Buildings, Allowable Stress Design*, and *Load and Resistance Factor Design Specification for Structural Steel Buildings*. AISI is currently developing a standard for the seismic design of structures using cold-formed steel members, and this standard may prove valuable in providing detailing requirements for the design of storage racks for seismic loads.

The Federation Europeenne de la Manutention (FEM) or European Federation of Maintenance, in cooperation with RMI, has conducted research standards development activities for the European Union (EU). A 2004 FEM seismic design standard, FEM10.2.08, *The Seismic Design of Static Steel Pallet Racks*, has been one result. Current FEM work includes analytical research and element, static and dynamic, and shake-table testing. Stub-column tests and beam-to-column connection tests for moment-rotation characteristics and properties also have been conducted.



Using test facilities at the University of Trento in Italy and at the National Technical University in Athens, full-scale steel pallet racks have been tested at various ground accelerations up to failure. This research indicates that movement of merchandise within packaged unit loads, movement of unit loads or packages on a pallet, and movement of pallets on pallet beams within the rack occur even at relatively low ground accelerations. Specific sliding tests have been developed to improve the understanding of these phenomena and their relationship to damping, period, and overall structural behavior.

The Canadian Standards Association has developed two storage racks standards: A344.1 *User Guide for Steel Storage Racks*, and A344.2, *Design and Construction of Steel Storage Racks*. RMI has supported these efforts.

### **3.9 FUTURE DIRECTION**

Among the most important ongoing RMI initiatives is the current testing program to determine the moment-rotation characteristics of the beam-to-column connectors of RMI members' products. The testing protocol will shed light on the role of connector properties in the seismic performance of rack structures including information on damping, drift, base shear, and period. The protocol is designed to mimic accepted testing provisions for building connections.<sup>1</sup> This testing program, being conducted for RMI by an independent testing laboratory, covers cold-formed and hot-rolled members as well as the linear elastic, nonlinear elastic, and inelastic behavior of the connector elements. The results of this beam-to-column connection testing program will help inform future proposals for change in the RMI standard, the *NEHRP Recommended Provisions*, ASCE 7, the *IBC*, and *NFPA 5000* and should contribute to a convergence of the seismic requirements in those documents. In turn, the application of those documents will help create pallet storage rack structures, including those specifically designed for use in areas open to the general public, that will perform well during earthquake events.

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<sup>1</sup> See *Seismic Provisions for Structural Steel Buildings* (AISC 2000); *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, FEMA 350 (FEMA 2000), *Structural Response Modification Factors*, ATC 19 (Applied Technology Council 1995); *Guidelines for Cyclic Seismic Testing of Components of Steel Structures*, ATC 24 (ATC 1992); and the *SEAOC Blue Book, Recommended Lateral Force Requirements and Commentary* (1999) for information on the seismic testing and evaluation of connections.

## Chapter 4

# CURRENT STORAGE RACK SEISMIC DESIGN PRACTICES

### 4.1 OVERVIEW

Single selective steel pallet storage racks are typically designed for seismic forces using the equivalent lateral force procedures found in model building codes and in the RMI standard (ANSI 16.1-04). Storage rack structural systems generally are moment frames in the down-aisle (longitudinal direction) and braced frames in the cross-aisle direction (transverse direction). The figures in Chapter 1 illustrate typical structural configuration. Storage racks placed in the middle of a floor area usually are attached back to back (Figures 1-1 and 1-2) whereas single rack configurations are used near building walls. Storage racks in store areas accessible to the public typically are loaded with pallets; however, in some merchandising situations, merchandise is stored directly on the shelves. Intermediate shelf heights vary depending on merchandising needs. Storage racks typically are subjected to the greatest loads when pallets are fully loaded and all racks in a given rack configuration contain fully loaded pallets. Pallets are designed to carry the maximum pallet design load. Criteria for determining effective seismic weights of pallets used in rack seismic design procedures vary between the model building codes and the RMI standard.

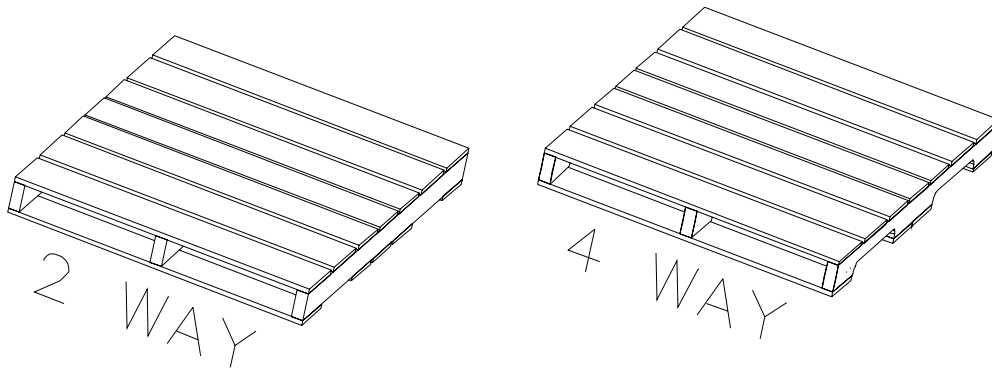
Because storage racks normally do not have horizontal bracing at the shelf levels, they typically are treated as structures with flexible diaphragms and are evaluated analytically as two-dimensional structural systems. For seismic loads, equivalent lateral forces are applied to the structural model, and the member forces are determined. These forces then are added to other loads using a series of load combinations to obtain design member and connection forces. The design member loads then are checked against member capacities and/or allowable stresses. The capacities and allowable stresses take into account that the members generally are cold form steel members and that the columns are likely perforated. Moment connections typically are checked for the computed moments by comparing against tested capacities (usually based on monotonic tested values with a factor of safety). These capacities typically are provided by the rack manufacturer. In the cross-aisle direction, base plates and anchor bolts are checked for the computed uplift forces. Other connections are part of a manufacturer's standardized components and are rarely included as part of the permit application process. Current storage rack design practice does not include increasing any of the connection forces or moments by the  $\Omega_o$  factor although that is inherent in current building code procedures for building structural systems. Also, there currently are no ductility type prescriptive requirements for connection designs. Currently, P-delta effects are typically considered by the moment magnifier for member design.

Since current storage rack seismic design practice revolves primarily around the level of design seismic loads, rack designers are interested primarily in the determination of those loads. The procedures currently used to compute rack seismic loads vary depending upon whether the prevailing requirements are from the model building codes, the *NEHRP Recommended Provisions* or ASCE 7, or the RMI standard. In some cases, there is more than one acceptable method of calculating seismic loads.

This chapter describes the factors used for determining seismic loads, each of the currently used procedures and design factors, and how members and connections are designed.

## 4.2 PALLET LOADS AND EFFECTIVE SEISMIC WEIGHTS

The pallet is the supporting structure for the basic unit that is stored on the typical single selective steel storage rack. There are many pallet sizes, shapes, materials, and types of construction; however, the vast majority of pallets used in the United States are what is termed the “GMA pallet.” The GMA pallet may be a two-way pallet or a four-way pallet, depending upon whether the handling equipment can lift the pallet from two or four sides (Figure 4-1).



*Figure 4.1 Typical pallets used in retail operations.*

The GMA pallet is 40 inches wide by 48 inches long and is constructed of hard wood with three stringers running front-to-back and many regularly spaced deck boards oriented side to side on both the top and the bottom of the pallet.

The unit of the load to be stored on the rack includes both the pallet and the product on the pallet and is the unit that is individually handled by the fork-lifts or other handling equipment. The weight, size, and pallet type is specified by the operator of the rack system.

The unit load weight ( $PL$ ) that the rack components are designed for is determined by the specifier of the rack system, who usually the end user.

The primary design weight is usually the maximum weight unit load ( $PL_{Maximum}$ ) that will be present in the warehouse. If there is more than one weight unit load and the weights differ by a significant amount, then the second weight will also be specified and the locations of the storage of each weight load will be designated.

$PL_{Maximum}$  is used for the design of the load beams bending and deflection, the upright column axial capacity, and the cross-aisle frame bracing requirements (for vertical loads). The maximum pallet weight also is used to resist uplift when combined with cross-aisle seismic forces.

The average weight of the unit load ( $PL_{Average}$ ) is the maximum total load expected in any individual rack row divided by the total number of storage positions in that row. This maximum total weight includes less-than-full-weight pallets and accounts for the number of storage positions normally expected to be empty. This is used for determining the down-aisle seismic force.

In addition to the product load ( $PL$ ), the rack structure is designed for the effects of the structure dead load ( $DL$ ) and any live load ( $LL$ ) that might be present. The dead load includes the frame weight, the shelf beam weight, and the weight of any accessory items on the rack such as pallet supports or wire mesh decks. The dead load is typically much less than 5 percent of the weight of the product stored on the rack structure. The live load is typically 60 psf or less.

The product load includes the pallet weight and the weight of the product on the pallet. The storage rack system is designed based on the full, maximum indicated product load. For a warehouse store, it is assumed that all rack system positions will be filled with full weight pallets.

For most storage racks, the effective seismic weight ( $W_s$ ) is a function of the ratio of the average pallet weight to the maximum pallet weight and is determined in accordance with Sec. 2.7.2 of ANSI MH 16.1-04, the RMI standard, as:

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

where  $PL_{RF}$  = product load reduction factor.

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

$PL_{Average}$  is the maximum total weight of product expected on the shelves in any row divided by the number of shelves in that row.

$PL_{Maximum}$  is the maximum weight of product that will be placed on any one shelf in that row.

The 0.67 modification of the pallet weight comes not from the average load but from a determination of the amount of load that participates in developing the dynamic seismic force. Experience has shown that the full mass of the merchandise stored on the storage rack system does not participate in the inertia generated from the ground motion. There is some friction inducing energy dissipating relative movement between the storage rack, the pallets stored on the rack, and the product on the pallets during the seismic motions. This 0.67 factor represents the fraction of the load on the fully loaded system contributing to the effective horizontal seismic weight. This effective seismic weight factor does not apply to vertical gravity loads.

It should be noted that the effective seismic weight ratios provided in the *UBC* are different from those above (see Sec. 4.4).

### 4.3 RACK CONFIGURATION

The number of pallet loads wide on each shelf between uprights generally is specified by the operator of the warehouse. In a typical storage rack system, there usually two pallet loads wide between uprights. If there are cross-aisle tunnels in the system to allow movement between aisles, the bay will have three or four pallets wide but not have storage in the bottom two or three levels. At the tunnel bay, the column will have shelves attached on one side in the bottom region and on both sides above the tunnel. Infrequently, the tunnel may be located at the end of a rack row and the last frame in the row will not have shelf beam support in the bottom region, in which case a special design for that frame is required.

The shelf elevations are determined by the operator of the warehouse based on the height of the loads and the shelf beam size and clearances required for storing and removing the load. The shelves may be spaced regularly for the full height of the rack system if all the loads are the same height or the spacing may be varied to accommodate different height loads with a minimal amount of lost space.

The overall storage height is based on the lifting height limitation of the handling equipment and the clear building height. Handling equipment characteristics also may necessitate a low bottom shelf beam at or near the floor. This shelf beam is typically 6 inches from the floor to the bottom of the beam and, considering a 4-inch beam, 8 inches to the centerline of the beam. When distributing the seismic forces vertically, this bottom beam does not participate in the determination of seismic forces to levels above as would be indicated by the traditional vertical distribution. The seismic force including this beam will only govern the design on the horizontal force on the anchors. This situation is addressed in Sec. 2.7.4 of ANSI MH 16.1-04, the RMI standard.

### 4.4 SEISMIC LOADS

The seismic loading requirements are determined by the local building authority having jurisdiction at the location of the facility where the racks are to be installed. The *International Building Code (IBC)* currently is used by most building authorities.<sup>1</sup> There is, however, one notable exception, the State of California. The *California Building Code (CBC)* is still based on the 1997 edition of the *Uniform Building Code* and, although it appears that the state ultimately will adopt the *IBC*, this will be several years away. While no longer supported by any code development organization, the *UBC* will continue to serve as the basis for the *CBC* until the state adopts a new model building code; therefore, it will be discussed in this document.

A description of the various procedures for determining seismic loads is presented below. Also provided for purposes of comparison are tabulations adjusted for allowable stress design (ASD) of the seismic force coefficients in the down-aisle direction for each of the applicable codes and standards (see Sec. 4.4.7).

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<sup>1</sup> Although a 2003 edition of the *IBC* is available, most jurisdictions are still using the 2000 edition of the *IBC*.

**4.4.1 The 2000 Edition of the *International Building Code*.** The primary reference for storage racks in the 2000 *IBC* is Sec. 1622.3.4 which presents two procedures for determining the seismic forces. The first procedure requires design in accordance with Sec. 1622.3.4.1 through 1622.3.4.4. The second procedure involves use of methods from Sec. 2.7 of ANSI MH 16.1-04, the RMI standard, with two added requirements. If the RMI standard is used, the seismic response coefficient ( $C_s$ ) is determined using  $C_a$  equal to  $S_{DS}/2.5$  and  $C_v$  equal to  $S_{DI}$ , but  $C_s$  shall not be less than  $0.14S_{DS}$ .

The determination of the seismic coefficients using the first procedure (i.e., *IBC* Sec. 1622.3.4.1 through 1622.3.4.4) is presented in Figure 4-2 for the down-aisle direction and for five different representative sites and periods ranging between 0.10 and 3.0 seconds. Near-field sites are not included in the represented sites.

The determination of the seismic coefficients using the second procedure (i.e., basically using the RMI standard with added requirements) is presented in Figure 4-3 for the down-aisle direction and for five different representative sites and periods ranging between 0.10 and 3.0 seconds. Near-field sites are not included in the represented sites.

**IBC 2000 Seismic Lateral Force  
Down-aisle**

|              |      |      |      |      |
|--------------|------|------|------|------|
| $S_s = 1.50$ | 1.00 | 0.75 | 0.50 | 0.25 |
| $S_1 = 0.75$ | 0.63 | 0.50 | 0.30 | 0.15 |

Site Class D

|              |      |      |      |      |
|--------------|------|------|------|------|
| $F_a = 1.00$ | 1.10 | 1.20 | 1.40 | 1.60 |
| $F_v = 1.50$ | 1.50 | 1.50 | 1.80 | 2.20 |

|                 |      |      |      |      |
|-----------------|------|------|------|------|
| $S_{DS} = 1.00$ | 0.73 | 0.60 | 0.47 | 0.27 |
| $S_{D1} = 0.75$ | 0.63 | 0.50 | 0.36 | 0.22 |
| $R = 4.0$       | 4.0  | 4.0  | 4.0  | 4.0  |
| $I_E = 1.5$     | 1.5  | 1.5  | 1.5  | 1.5  |

| T    | 0.7 V/PL |       |       |       |       |
|------|----------|-------|-------|-------|-------|
| 0.10 | 0.189    | 0.139 | 0.113 | 0.088 | 0.050 |
| 0.20 | 0.189    | 0.139 | 0.113 | 0.088 | 0.050 |
| 0.30 | 0.189    | 0.139 | 0.113 | 0.088 | 0.050 |
| 0.40 | 0.189    | 0.139 | 0.113 | 0.088 | 0.050 |
| 0.50 | 0.189    | 0.139 | 0.113 | 0.088 | 0.050 |
| 0.60 | 0.189    | 0.139 | 0.113 | 0.088 | 0.050 |
| 0.70 | 0.189    | 0.139 | 0.113 | 0.088 | 0.050 |
| 0.80 | 0.177    | 0.139 | 0.113 | 0.085 | 0.050 |
| 0.90 | 0.158    | 0.131 | 0.105 | 0.076 | 0.046 |
| 1.00 | 0.142    | 0.118 | 0.095 | 0.068 | 0.042 |
| 1.10 | 0.129    | 0.107 | 0.086 | 0.062 | 0.038 |
| 1.20 | 0.118    | 0.098 | 0.079 | 0.057 | 0.035 |
| 1.30 | 0.109    | 0.091 | 0.073 | 0.052 | 0.032 |
| 1.40 | 0.101    | 0.084 | 0.068 | 0.049 | 0.030 |
| 1.50 | 0.095    | 0.079 | 0.063 | 0.045 | 0.028 |
| 1.60 | 0.089    | 0.074 | 0.059 | 0.043 | 0.026 |
| 1.70 | 0.083    | 0.069 | 0.056 | 0.040 | 0.024 |
| 1.80 | 0.079    | 0.066 | 0.053 | 0.038 | 0.023 |
| 1.90 | 0.075    | 0.062 | 0.050 | 0.036 | 0.022 |
| 2.00 | 0.071    | 0.059 | 0.047 | 0.034 | 0.021 |
| 2.10 | 0.071    | 0.059 | 0.045 | 0.032 | 0.020 |
| 2.20 | 0.071    | 0.059 | 0.043 | 0.031 | 0.019 |
| 2.30 | 0.071    | 0.059 | 0.041 | 0.030 | 0.018 |
| 2.40 | 0.071    | 0.059 | 0.039 | 0.028 | 0.017 |
| 2.50 | 0.071    | 0.059 | 0.038 | 0.027 | 0.017 |
| 2.60 | 0.071    | 0.059 | 0.036 | 0.026 | 0.016 |
| 2.70 | 0.071    | 0.059 | 0.035 | 0.025 | 0.015 |
| 2.80 | 0.071    | 0.059 | 0.034 | 0.024 | 0.015 |
| 2.90 | 0.071    | 0.059 | 0.033 | 0.023 | 0.014 |
| 3.00 | 0.071    | 0.059 | 0.032 | 0.023 | 0.014 |

$$V = C_s W$$

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

$C_s$  need not exceed:

$$C_s = \frac{S_{D1}}{(R/I_E)T}$$

But not less than:

$$C_s = 0.044 S_{DS} I_E$$

And for Design Category E and  $S_1 \geq 0.6$  not less than:

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

if the Dead Load is assumed to be 5% of the Pallet Load

$$W = (0.05 PL) + (0.67 PL)$$

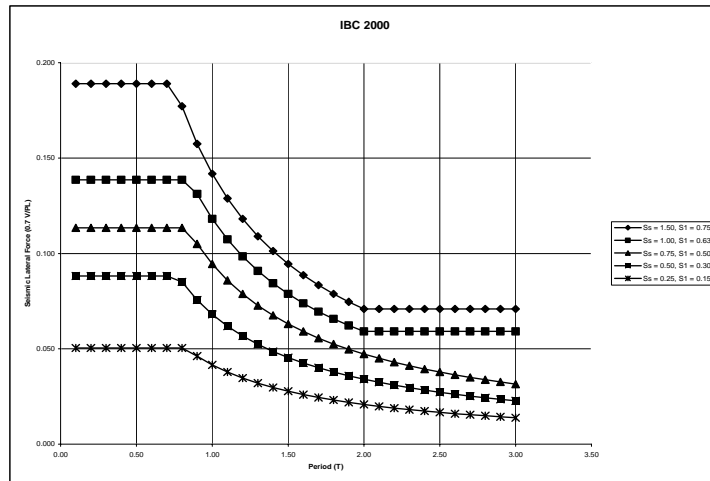


Figure 4-2 Determining seismic coefficients using IBC 2000 Sec. 1622.3.4.1 through 1622.3.4.4.

**IBC 2000 & RMI 2002 Seismic Lateral Force  
Down-aisle**

|              |      |      |      |      |
|--------------|------|------|------|------|
| $S_s = 1.50$ | 1.00 | 0.75 | 0.50 | 0.25 |
| $S_1 = 0.75$ | 0.63 | 0.50 | 0.30 | 0.15 |

Site Class D

|              |      |      |      |      |
|--------------|------|------|------|------|
| $F_a = 1.00$ | 1.10 | 1.20 | 1.40 | 1.60 |
| $F_v = 1.50$ | 1.50 | 1.50 | 1.80 | 2.20 |

|                 |      |      |      |      |
|-----------------|------|------|------|------|
| $S_{DS} = 1.00$ | 0.73 | 0.60 | 0.47 | 0.27 |
| $S_{D1} = 0.75$ | 0.63 | 0.50 | 0.36 | 0.22 |
| $R = 6.0$       | 6.0  | 6.0  | 6.0  | 6.0  |
| $I_p = 1.5$     | 1.5  | 1.5  | 1.5  | 1.5  |

$$V = C_s I_p W_s$$

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

Where  $C_v = S_{D1}$

But need not be greater than:

$$C_s = \frac{2.5C_a}{R}$$

Where  $C_a = S_{DS}/2.5$

and not less than:

$$C_s = 0.14S_{DS}$$

if the Dead Load is assumed to be 5% of the Pallet Load

$$W = (0.05 PL) + (0.67 PL)$$

| T    | 0.67 V/PL |       |       |       |       |
|------|-----------|-------|-------|-------|-------|
| 0.10 | 0.121     | 0.088 | 0.072 | 0.056 | 0.032 |
| 0.20 | 0.121     | 0.088 | 0.072 | 0.056 | 0.032 |
| 0.30 | 0.121     | 0.088 | 0.072 | 0.056 | 0.032 |
| 0.40 | 0.121     | 0.088 | 0.072 | 0.056 | 0.032 |
| 0.50 | 0.121     | 0.088 | 0.072 | 0.056 | 0.032 |
| 0.60 | 0.121     | 0.088 | 0.072 | 0.056 | 0.032 |
| 0.70 | 0.121     | 0.088 | 0.072 | 0.056 | 0.032 |
| 0.80 | 0.121     | 0.088 | 0.072 | 0.056 | 0.032 |
| 0.90 | 0.116     | 0.088 | 0.072 | 0.056 | 0.032 |
| 1.00 | 0.109     | 0.088 | 0.072 | 0.052 | 0.032 |
| 1.10 | 0.102     | 0.085 | 0.068 | 0.049 | 0.030 |
| 1.20 | 0.101     | 0.080 | 0.064 | 0.047 | 0.028 |
| 1.30 | 0.101     | 0.076 | 0.061 | 0.047 | 0.027 |
| 1.40 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 1.50 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 1.60 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 1.70 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 1.80 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 1.90 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 2.00 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 2.10 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 2.20 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 2.30 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 2.40 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 2.50 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 2.60 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 2.70 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 2.80 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 2.90 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |
| 3.00 | 0.101     | 0.074 | 0.061 | 0.047 | 0.027 |

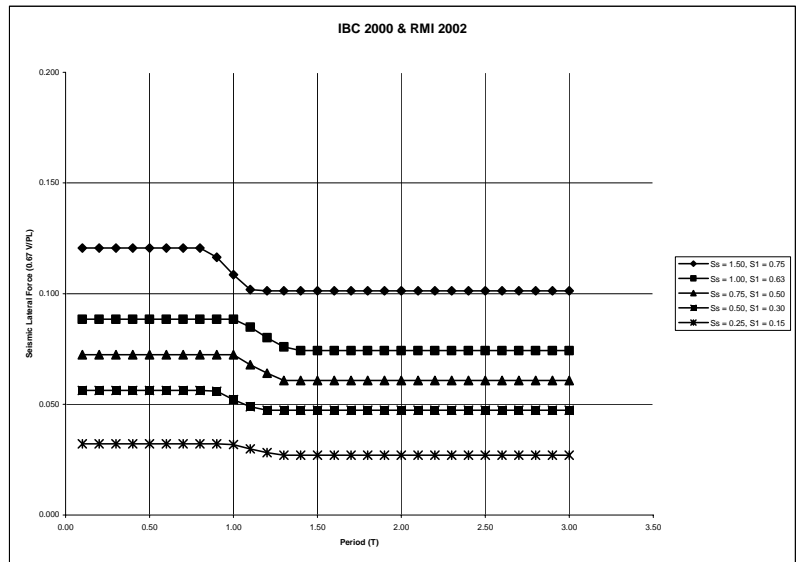


Figure 4-3 Determining seismic coefficients using IBC 2000 Sec. 1622.3.4 (the RMI standard with additional requirements).



**4.4.2 The 2003 Edition of the *International Building Code*.** The reference for storage racks in the 2003 *IBC* is Sec. 2208:

SECTION 2208

STEEL STORAGE RACKS

2208.1 Storage racks. The design, testing and utilization of industrial steel storage racks shall be in accordance with the *RMI Specification* for the Design, Testing and Utilization of Industrial Steel Storage Racks. Racks in the scope of this specification include industrial pallet racks, movable shelf racks and stacker racks, and does not apply to other types of racks, such as drive-in and drive-through racks, cantilever racks, portable racks or rack buildings. Where required, the seismic design of storage racks shall be in accordance with the provisions of Section 9.6.2.9 of ASCE 7.

This section requires that all listed types of steel storage racks be design in accordance with the 1997 edition of the RMI standard but further states that the seismic design of storage racks shall be in accordance with Sec. 9.6.2.9 of ASCE 7-02, which is discussed below.

**4.4.3 American Society of Civil Engineers, *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02.** ASCE 7 Sec. 9.6.2.9 requires that storage racks be designed to meet the force requirements of ASCE 7 Sec. 9.14, which pertains to nonbuilding structures. Sec. 9.14.6.3 directly addresses storage racks and presents two procedures.

The first procedure requires that the design satisfy Sec. 9.14.6.3.1, which, in turn, requires the design to be in accordance with the general provisions of Sec. 9.5 and Sec. 9.14.5 and the specific storage rack provisions of Sec. 9.14.6.3.1 through 9.14.6.4.3. These requirements for seismic force mandate the use of the requirements of Sec. 9.5 and an  $R$  of 4. The importance factor ( $I_p$ ) is to be determined in accordance with Sec. 9.6.1.5 and, for warehouse retail stores open to the public, is 1.5. The minimum base shear coefficient is  $0.14S_{DS}I$ . In addition, where  $S_I$  is equal to or greater than  $0.75g$ , the minimum base shear shall not be less than  $0.8S_I/R$ . The storage racks must be designed considering two loading conditions: (a) 67 percent of all positions fully loaded and (b) the top storage position loaded to 100 percent only. The vertical distribution of the base shear force as specified in Sec. 9.5.5.4 with the factor  $k$  taken as 1.0.

**4.4.4 Rack Manufactures Institute, *Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks*, 2002 Edition, with ASCE 7-02 Required Changes.** The second design procedure for storage racks is to use the RMI standard with the following additional changes:

- $C_a$  equal to  $S_{DS}/2.5$  and  $C_v$  equal to  $S_{DI}$  but  $C_s$  shall not be less than  $0.14S_{DS}$ .
- $I_p$  from Section 9.6.1.5 where for warehouse retail stores open to the public  $I_p$  is 1.5.

The seismic design requirements are in presented in Sec. 2.7 of the RMI standard along with the required changes of SEI/ASCE 7-02. The required seismic force coefficient is determined using the equivalent static force method:

$$V = C_s I_p W_s$$

where:

$I_p$  = the system importance factor as defined below  
 1.5 for essential facilities or hazardous material storage  
 1.5 for storage racks in areas open to the public  
 1.0 for all other structures

$W_s$  = the effective seismic weight (see Sec. 4.2.7 of this document)

$C_s$  = the seismic response coefficient from Sec. 2.7.3 of the RMI standard:

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

where

$C_v = S_{DI}$  from ASCE 7 Sec. 9.14.6.3

$R = 4.0$  for the braced direction and  $6.0$  for the unbraced direction and

$T =$  the fundamental period of the rack in the direction under consideration

$C_s$  need not be greater than:

$$C_s = \frac{2.5C_a}{R}$$

where  $C_a$  is  $S_{DS}/2.5$  from ASCE 7 Sec. 9.14.6.

$C_s$  shall not be taken less  $0.14 S_{DS}$  (per ASCE 7 Sec. 9.14.6.3).

In determining the  $C_s$  factor, it is important that the period include all the characteristics of all the structural members that make up the load path for the forces. For example, in the down-aisle direction, the period should include the deformation of the beam-to-column connection; in the cross-aisle direction the eccentricities of the frame bracing connections should be included. The period should be the elastic period consistent with code applied forces. Current practice is to use rotational spring stiffness ( $F$ ) based on the test procedures of Sec. 9.4.1 of the RMI standard when determining the down-aisle fundamental period. Because of the nonlinear stiffness of the beam-to-column connection, the down-aisle period changes significantly with applied load. Shake-table testing for high levels of ground motions has shown that, for some racks, the measured period may be greater than 1.5 seconds. However, the building code limits the  $C_s$  factor to  $0.14S_{DS}$ , effectively putting an upper bound on the period of slightly less than 1.0 sec.

For the seismic loads, the  $S_{DS}$  and  $S_{DI}$  factors may be determined by looking up the values of  $S_S$  and  $S_I$  for the site location on the ASCE 7 seismic maps -- Figure 9.4.1.1(a) through (j) -- and adjusting for the site class effects with factors found in ASCE 7 Tables 9.4.1.2.4a and b. The resulting values of  $S_{MS}$  and  $S_{MI}$  are scaled by two-thirds to obtain  $S_{DS}$  and  $S_{DI}$ . However, today it is easier to obtain the seismic factors using the Seismic Design Parameters CD that accompanies the *NEHRP Recommended Provisions* and other codes and standards. The latitude and longitude

of the site are needed to use the CD and can be obtained from a variety of other software or internet resources. Because it has not yet been widely recognized that the site classification is as important a seismic design factor as “seismic zone,” it usually is not included as a part of the information that is transmitted to the rack designer. Consequently, Site Class D is usually used and required if the site class is unknown.

The RMI standard then distributes the base shear to each storage level based on the height and load but modifies the standard distribution for a low first shelf. Sec. 2.7.4 requires that:

If the centerline of the first shelf level is 12 inches (30.5 cm) above the floor or less:

$$F_1 = C_s I_p w_1 \text{ for the first shelf level}$$

and

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

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

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

where

$V$  = total design lateral force or shear at the base of the rack

$w_i$  or  $w_x$  = the portion of the total gravity load (including live load, dead load and product load times the product load reduction factor, see Sec. 2.7.2) of the rack, located or assigned to the bottom shelf level, level  $i$  or  $x$

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

$k$  = an exponent related to the structure's period

period  $\leq 0.5$   $k = 1$

period  $\geq 2.5$   $k = 2$

For racks having a period between 0.5 and 2.5 seconds,  $k$  shall be 2 or shall be determined by linear interpolation between 1 and 2. If the base shear is based on the default  $C_s$ , then the  $k$  shall be taken as 1.

An example of the RMI/ASCE7-02 seismic force calculation for a given site is presented below. Near-field sites have not been included.

Find the latitude and longitude of the warehouse location. Use the Seismic Design Parameters CD (see Sec. 4.5.4.2) to determine the parameters  $S_s$  and  $S_I$  for the site. The factors  $F_a$  and  $F_v$

also can be determined using the CD for a specific site class and most often rack design is based on the default Site Class D.

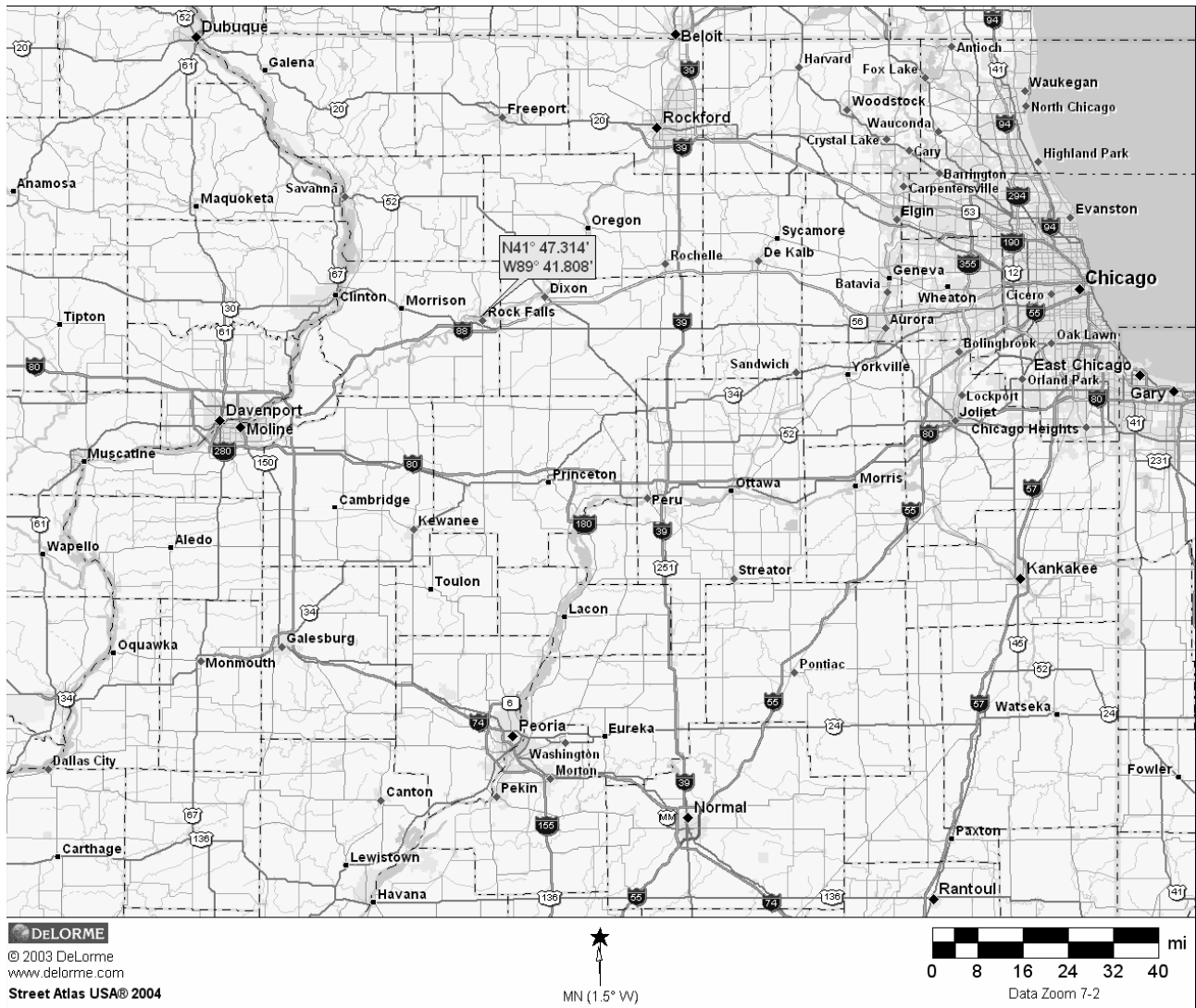


Figure 4-4 Map to determine latitude and longitude.

**MCE Ground Motion – Conterminous 48 States**

Latitude = 41.7891                      Longitude = -89.6922

| Period<br>(sec) | MCE $S_a$<br>(%g)                       |
|-----------------|---|
| 0.2             | 016.6 MCE Value of $S_a$ , Site Class B |
| 1               | 006.3 MCE Value of $S_I$ , Site Class B |

Spectral Parameters for Site Class D

|     |                          |              |
|-----|--------------------------|--------------|
| 0.2 | 026.6 $S_{MS} = F_a S_s$ | $F_a = 1.60$ |
| 1   | 015.1 $S_{MI} = F_v S_I$ | $F_v = 2.40$ |

**ASCE 7-02 and the 2002 RMI Standard**

$$S_{DS} = 2/3 S_{MS} = 0.177$$

$$S_{DI} = 2/3 S_{MI} = 0.101$$

$$C_a = S_{DS} / 2.5 = 0.071$$

$$C_v = S_{DI} = 0.101$$

where

$$T = 1.5 \text{ sec}$$

$$R = 4.0$$

$$I = 1.50$$

$$C_s = \frac{1.2C_v}{RT^{2/3}} = 0.023$$

or

$$C_s = \frac{1.2C_v}{R} = 0.044$$

but not less than

$$C_s = 0.14S_{DS} = 0.025 \leftarrow \text{USE THIS}$$

$$V = C_s I (0.67P) = 0.025PL$$

**4.4.5 The 2003 NEHRP Recommend Provisions.** The 2003 *NEHRP Recommended Provisions* serves as the basis for the ASCE 7-05 seismic requirements. The next editions of the two model

building codes (i.e., the 2006 edition of the *IBC* and the 2005 edition of *NFPA-5000*) will both reference ASCE 7-05 for their seismic requirements; therefore the seismic provisions found in these editions of the building codes will essentially be based on the 2003 *NEHRP Recommended Provisions*. The seismic provisions for storage racks found in *Provisions* Sec. 14.3.5 are essentially identical to those found in ASCE 7-02. The only difference is that for the first procedure, the minimum base shear coefficient of  $0.14S_{DS}I$  is replaced by  $0.01S_{DS}I$ . For the second procedure which references the ANSI MH 16.1-04, the minimum value of  $C_s$  is still  $0.14S_{DS}I$ .

**4.4.6 The 1997 Uniform Building Code.** As noted above, the *UBC* was originally developed and promulgated by the International Conference of Building Code Officials (ICBO). It has been superseded by the *IBC* and is generally considered to be obsolete; however, it will serve as the basis for the *California Building Code* until the state adopts a new code, which should occur in the relatively near future. *UBC* Chapter 22, Division X, “Design Standard for Steel Storage Racks,” directly addresses the design of storage racks for all situations including seismic and is based on the 1990 edition of the RMI standard. For seismic design, Division X, Sec. 2228.5.2 specifies minimum earthquake forces. This section prescribes the lateral forces, at strength design levels, be determined by the static force procedure in Sec 1630.2.1 with the  $R$  factor and  $W$  as defined in Division X. Further, this section modifies the distribution of the seismic force to not include a distinct top force.

The required seismic force coefficient is determined using the equivalent static force method of *UBC* Sec. 1630.2.1:

$$V = \frac{C_v I}{RT} W \quad (\text{UBC Eq. 30-4})$$

but need not exceed:

$$V = \frac{2.5C_a I}{R} W \quad (\text{UBC Eq. 30-5})$$

nor not be less than:

$$V = 0.11C_a I W \quad (\text{UBC Eq. 30-6})$$

In addition, in Seismic Zone 4,  $V$  shall not be less than:

$$V = \frac{0.8ZN_v I}{R} W \quad (\text{UBC Eq. 30-7})$$

The following terms are defined in Division X:

$R =$  4.4 for racks where lateral stability is dependent on diagonal or x-bracing.  
This is usually the factor used of the cross-aisle direction.

$R =$  5.6 for racks where lateral stability is wholly dependent on moment – resisting frame action.

$W =$  weight of racks plus contents

If four or more columns are connected together in both the cross-aisle and the down-aisle direction, 50 percent of the rack content load may be used. Further, in Seismic Zones 3 and 4 in wholesale and retail sales areas, the 50 percent may be used only when  $C_v/RT$  is 0.7  $C_a$  in Eq. 30-4 and with  $2.5/R$  is 0.7 in Eq. 30-5.

The other terms are the standard UBC definition of the terms:

$C_a$  from *UBC* Table 16-Q

$C_v$  from *UBC* Table 16-R

$I$  is the seismic importance factor from *UBC* Table 16-K

$Z$  is the seismic zone factor from *UBC* Table 16-I

$N_v$  from *UBC* Table 16-T

$T$  is the fundamental period of vibration of the rack.

It should be noted that *UBC* Sec. 1630.2.2 Method A is only applicable to building structures and is not applicable to rack structures. The building approximation, accounting for the damping of nonstructural elements and contents is not valid for rack structures. Method B, without its limitation to Method A, is what is recommended by RMI.

The total base shear is distributed over the height of the rack triangularly proportioned based on the loading, without a special concentrated force at the top:

$$F_i = \frac{VW_i h_i}{\sum_{i=1}^n W_i h_i}$$

An example of the 1997 *UBC* seismic force calculation for five representative sites (Seismic Zones 1, 2A, 2B, 3 and 4) is presented in Figure 4-5. Near-field sites have not been included.

UBC 1997 Division X & Section 1630.2.1

9/16/2004

Down-aisle

|      |      |      |      |      |       |
|------|------|------|------|------|-------|
| Zone | 4    | 3    | 2B   | 2A   | 1     |
| Z=   | 0.40 | 0.30 | 0.20 | 0.15 | 0.075 |

Site Class D

|      |      |      |      |      |      |
|------|------|------|------|------|------|
| Ca = | 0.44 | 0.36 | 0.28 | 0.22 | 0.12 |
| Cv = | 0.64 | 0.54 | 0.40 | 0.32 | 0.18 |
| Nv = | 1.00 |      |      |      |      |

|     |     |     |     |     |     |
|-----|-----|-----|-----|-----|-----|
| R = | 5.6 | 5.6 | 5.6 | 5.6 | 5.6 |
| I = | 1   | 1   | 1   | 1   | 1   |

W = DL + 0.5PL  
not in Zones 3 & 4

W = DL + 0.5PL  
in Zone 3 & 4 if  
 $C_v / RT = 0.70 C_a$  in (30-4)  
 $2.5 / R = 0.70$  in (30-5)

$$V = \frac{C_v I}{RT} W \quad (30-4) \quad \text{or}$$

$$V = 0.70 C_a I W$$

But need not exceed:

$$V = \frac{2.5 C_a I}{R} W \quad (30-5) \quad \text{or}$$

$$V = 0.70 C_a I W$$

And not less than:

$$V = 0.11 C_a I W$$

In addition, in Zone 4, not less than:

$$V = \frac{0.8 Z N_v I}{R} W$$

In Seismic Zones 3 & 4, wholesale and retail areas, the 50% may only be used when

if the Dead Load is assumed to be 5% of the Pallet Load

$$W = (0.05 PL) + (0.50 PL)$$

$$W = (0.05 PL) + (0.50 PL)$$

| T    | [V/PL] / 1.4 |       |       |       |       |
|------|--------------|-------|-------|-------|-------|
| 0.10 | 0.121        | 0.099 | 0.049 | 0.039 | 0.021 |
| 0.20 | 0.121        | 0.099 | 0.049 | 0.039 | 0.021 |
| 0.30 | 0.121        | 0.099 | 0.049 | 0.039 | 0.021 |
| 0.40 | 0.121        | 0.099 | 0.049 | 0.039 | 0.021 |
| 0.50 | 0.121        | 0.099 | 0.049 | 0.039 | 0.021 |
| 0.60 | 0.121        | 0.099 | 0.047 | 0.037 | 0.021 |
| 0.70 | 0.121        | 0.099 | 0.040 | 0.032 | 0.018 |
| 0.80 | 0.107        | 0.090 | 0.035 | 0.028 | 0.016 |
| 0.90 | 0.095        | 0.080 | 0.031 | 0.025 | 0.014 |
| 1.00 | 0.086        | 0.072 | 0.028 | 0.022 | 0.013 |
| 1.10 | 0.078        | 0.066 | 0.026 | 0.020 | 0.011 |
| 1.20 | 0.071        | 0.060 | 0.023 | 0.019 | 0.011 |
| 1.30 | 0.066        | 0.056 | 0.022 | 0.017 | 0.010 |
| 1.40 | 0.061        | 0.052 | 0.020 | 0.016 | 0.009 |
| 1.50 | 0.057        | 0.048 | 0.019 | 0.015 | 0.008 |
| 1.60 | 0.054        | 0.045 | 0.018 | 0.014 | 0.008 |
| 1.70 | 0.050        | 0.043 | 0.017 | 0.013 | 0.007 |
| 1.80 | 0.048        | 0.040 | 0.016 | 0.012 | 0.007 |
| 1.90 | 0.045        | 0.038 | 0.015 | 0.012 | 0.007 |
| 2.00 | 0.043        | 0.036 | 0.014 | 0.011 | 0.006 |
| 2.10 | 0.043        | 0.034 | 0.013 | 0.011 | 0.006 |
| 2.20 | 0.043        | 0.033 | 0.013 | 0.010 | 0.006 |
| 2.30 | 0.043        | 0.032 | 0.012 | 0.010 | 0.005 |
| 2.40 | 0.043        | 0.032 | 0.012 | 0.010 | 0.005 |
| 2.50 | 0.043        | 0.032 | 0.012 | 0.010 | 0.005 |
| 2.60 | 0.043        | 0.032 | 0.012 | 0.010 | 0.005 |
| 2.70 | 0.043        | 0.032 | 0.012 | 0.010 | 0.005 |
| 2.80 | 0.043        | 0.032 | 0.012 | 0.010 | 0.005 |
| 2.90 | 0.043        | 0.032 | 0.012 | 0.010 | 0.005 |
| 3.00 | 0.043        | 0.032 | 0.012 | 0.010 | 0.005 |

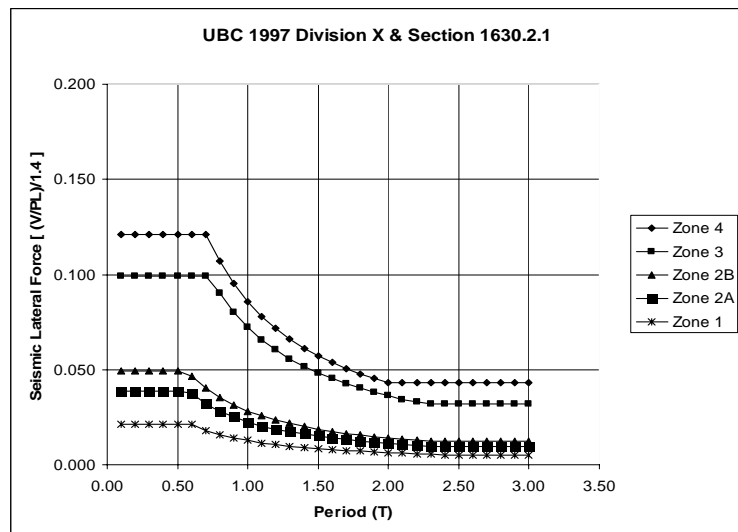


Figure 4-5 Determining seismic coefficients using UBC 1997, Division X.

## 4.5 MEMBER DESIGN

### 4.5.1 Shelf Design

Each rack manufacturer uses a unique and proprietary beam-to-column connector to join the horizontal and vertical members in the down-aisle direction. The connector usually is welded to each end of a shelf beam and engages slots or holes in the face and/or the side of the column (see Figure 1-3). Some connectors are bolted to the column, some have protruding rivets that slip into perforations in the column, and some have tabs formed from the connector that are inserted into slots in the column.

It is because of this multiplicity of connection types, the RMI standard requires the determination of the strength and stiffness of the connections by testing. Once the testing is completed for one product line, the results may be used for all projects using that type of connection. The



connection stiffness test produces a connection rotational spring constant,  $F$ . This rotational spring is then introduced between the end of the beam and the column and the structure analyzed.

When the load on the shelf is distributed by a pallet, the shelf beam bending is generally computed based on a uniformly distributed load for the entire length of the shelf beam. The beam may be designed as a simply supported beam, which AISC calls Type 2 construction, or may include the effects of the semi-rigid end connections, which AISC calls Type 3 construction. The span of the beam is generally considered to be the clear distance between the upright columns. Commentary Sec. 5.2 of the RMI standard provides guidance on including the effects of the connection stiffness in calculating the strength and deflection of the shelf for Type 3 construction. The strength for Type 3 construction is calculated with:

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

where

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

$E$  = the modulus of elasticity

$F$  = the joint rotational spring constant determined either by the cantilever test described in Sec. 9.4 of the RMI standard or by pallet beam in upright frames assembly test described in Sec. 9.3.2 of the RMI standard.

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

$L$  = the span of the beam

$W$  = the total load on each beam (including vertical impact loads)

and where

$$M_e = \frac{wL}{8}(1 - r_m) \text{ and } M_e = \text{the beam end moment.}$$

The deflection of a shelf beam under full design load is limited to 1/180 of the beam span. When considering semi-rigid joints, the following expression for maximum deflection  $\delta_{max}$  can be derived:

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

where

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

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

#### 4.5.2 Member Design in Down-Aisle Direction

Columns are designed in accordance with the rules of the AISC specification for hot-rolled members or the AISI specification for cold-formed members. There have been some modifications to those two specifications in areas that are unique to storage rack members. In particular, rack columns usually have a series of perforations regularly spaced for the entire column height that allow the mechanical attachment of shelf beams at many elevations. However, the elevation of the shelf beam from the floor and between beams is a critical limitation of the specific design applicable to an installation.

As discussed elsewhere, although the shelves can be located at any elevation, it is a function of the engineering of that system that determines whether the beam may be located at that spacing and still satisfy the design criteria.

The RMI standard requires testing to determine the effects of perforations punched in the column. Design limitations based on that testing then are prescribed.

Static stability, as well as seismic stability, in the down-aisle direction is dependant on the stiffness of the beam-to-column connections. For static stability, one suggested design method computes a column  $k$  factor based on the initial connector stiffness from testing. To determine the  $k$  factor, the nomographs from the commentary to the AISC specification are used with a beam stiffness modified by the connector stiffness. There are many simplifying assumptions that must be made in order for the nomographs to be applicable. Because of this, work is under way to permit use of a notional load and second order type approach for static stability.

Story drift limitations are not applicable to storage rack structures with the exception of making sure that the rack structure will not impact surrounding structures such as the warehouse building. In the absence of any other calculations, the model building codes require a separation of 5 percent of the overall height. The actual drift may be substantially less than this 5 percent value and the actual calculated drift at the design earthquake level may be used for the separation distance.

#### 4.5.3 Member Design in the Cross-Aisle Direction

Two design aspects are unique to cross-aisle storage rack frame bracing. First, the bracing can seldom be attached such that the lines of action of the structural components meet at a point. The commentary to the RMI standard defines what is an acceptable deviation from the ideal. Second, in many frame designs, the connections of the diagonals and horizontals to the column are designed on the basis of the compression in the members. It is not unusual for a bracing member to be designed first to fit into the available location (e.g., between the flange returns). Further, the bracing often is located such that it may take either tension or compression depending on the direction of the force. The diagonal just above a bracing panel often is oriented in the opposite direction.

Because the frame brace may have more capacity than required, it is frequently checked for adequacy by comparing the requirement to known, tested capacities.

When the two diagonals are in opposite directions, the force on the bracing system is limited to the compression capacity of one of the diagonals.

The RMI standard requires anchoring for all bases in every storage rack system. There may be no net base uplift calculated for the rack configuration but anchors are required regardless. Base plates frequently are sized to provide one or more locations for anchor bolts. There may be more than one hole even though there is only one anchor required. The size of the anchor required is based on the net uplift at the base under the worst of two conditions: (a) with the top load only in place and the appropriate seismic force resulting from that pallet applied at the top shelf level or (b) with a 350 pound lateral load applied horizontally at the top load level. The 350 pound force is applied in one location at a time and may be distributed over several uprights if they are adequately connected together. The uplift of the base plate under this 350 pound load is checked when there is no product load in any position of the uprights that are resisting this load.

A wide variety of post-installed anchors are used for storage rack systems. The selection of the appropriate anchor to resist the forces at the base is left up to the rack designer. However, no matter what anchor is used, it is important that the anchor be installed to meet the requirements of the anchor manufacturer and that the anchor loading not exceed the load capacity stated by the manufacturer. In some jurisdictions, the allowable anchor capacity may be required to be based on independent anchor evaluations such as those provided by the ICC Evaluation Service.

## Chapter 5

# PERFORMANCE EXPECTATIONS FOR STORAGE RACKS IN AREAS ACCESSIBLE TO THE PUBLIC

### 5.1 SCOPE

Presented in this chapter is a discussion of the seismic performance expectations for single selective steel storage racks that store contents stored at least 8 feet above the floor in areas accessible to the public.

### 5.2 SEISMIC SAFETY CONCERNS

During the past few decades, the number of large public warehouse stores (often referred to as big-box stores) across the nation has grown significantly, changing both consumer buying habits and the public's risk of injury during earthquakes. Whereas traditional retailers typically store goods and products outside the retail space in limited access storage rooms and warehouse facilities, big-box stores keep goods in close proximity to the consumer at all times. Typically, shoppers in these stores browse in aisles between steel storage racks, 14 to 18 feet in height, that hold pallets of inventory, some of which can be very heavy.

During an earthquake, occupant safety in a big-box store depends on both the structural performance of the building and on the performance of the storage racks and their contents. Earthquake ground motions can cause storage racks to collapse or overturn if they are not properly designed, installed, maintained, and loaded. In addition, goods stored on the racks may spill or topple off. Both occurrences pose a life-safety risk to the exposed shopping public.

### 5.3 SEISMIC PERFORMANCE OBJECTIVES

A seismic performance objective consists of one or more performance goals, each consisting of a target performance level coupled with a specific earthquake hazard. The explicit definition of seismic performance objectives provides an unambiguous basis for design and tools needed for seismic risk decision making. This project included a review of design standards currently in use with a view toward their improvement. Thus, the seismic performance objectives articulated in this chapter were developed within the framework of current code and standards procedures with the *NEHRP Recommended Provisions* serving as the base.

**5.3.1 Performance Expectations for Buildings.** The goal of the *NEHRP Recommended Provisions* is to present criteria for the design and construction of new structures subject to earthquake ground motions in order to protect life safety. For ground motions in excess of the design levels, the intent of the *NEHRP Recommended Provisions* is that a structure have a low likelihood of collapse.

Under the *NEHRP Recommended Provisions*, buildings and other structures are designed using design basis earthquake (DBE) ground motion parameters ( $S_{DS}$  and  $S_{DI}$ ) that are defined as 2/3 of the maximum considered earthquake (MCE) ground motion parameters ( $S_S$  and  $S_I$ ) modified by soil factors. For most regions in the country, the MCE ground motion is defined with a uniform probability of exceedance of 2 percent in 50 years. In regions of very high seismicity (ground motion parameter values greater than  $S_S = 1.5$  and  $S_I = 0.6$ ), the MCE is defined as the lesser of either the probabilistic value or 150 percent of the mean ground motion calculated for a deterministic characteristic earthquake on major active faults.

Buildings and other structures are designed to provide the life safety level of performance for the DBE. For buildings and building-like structures, the lower bound estimate of the margin against collapse is judged to be 1.5. Hence, collapse prevention performance is expected to be achieved for the adjusted MCE.

**5.3.2 System Performance Expectations for Storage Racks.** Storage racks are considered to be nonbuilding structures in the *NEHRP Recommended Provisions*. However, consistent with building design requirements, storage racks should be capable of providing life safety in the DBE and collapse prevention should be achieved in the MCE.

The seismic performance of storage racks consists of two components: the seismic performance of the rack itself and the response of 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 in the DBE is 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 supported 8 feet or more above the floor.

Collapse prevention performance in the MCE 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 injuries or even loss of life associated with contents shedding is probable at the collapse prevention performance level.

The guidance presented in this document represents the minimum recognized criteria similar to that contained in the model building codes. There may be special situations or risks where a higher level of performance is desired. Such situations would include a very high level or period of public use and, therefore, an increased level of life safety risk or the storage of contents of such a high value that a higher level of performance would be justified. The design and installation of storage racks to meet higher performance levels is not precluded in this document and, in fact, some of the guidance contained herein may be useful in meeting such goals.



## Chapter 6

### SUGGESTED IMPROVEMENTS TO CURRENT STORAGE RACK SEISMIC DESIGN PRACTICES

#### 6.1 INTENT AND SCOPE

This chapter recommends improvements to current storage rack seismic design practices. These improvements apply to single selective steel pallet type storage racks with the bottom of pallets or contents stored at least 8 feet above the floor and intended for use in areas accessible to the general public. Typical storage racks of the type considered herein are illustrated in Chapter 1.

Most structures including steel storage racks currently are designed for earthquake loads using an equivalent lateral force approach. This approach requires members to be sized based on elastic analysis using reduced seismic loads. The seismic loads are reduced to account for anticipated acceptable inelastic behavior of the structure, the type of structural system being used, and the level of detailing used in the design. Over the past 10 years, the probable ground motions used to establish the seismic forces, the reduction factors used in the analyses, and the required level of detailing required by building standards have changed significantly. However, the RMI standards used for storage rack design have not kept pace. Further, the differences in the approaches used in the design of storage racks and building structures have raised some concern on the part of building officials and structural engineers that the current storage rack seismic design criteria may not provide adequate life-safety protection. In addition, current building codes now are considering collapse prevention for maximum considered earthquake (MCE) ground motions as the key performance goal for structures, and the Rack Project Task Group was uncertain as to whether the current equivalent static force procedures for the seismic design of storage racks would result in racks that will not collapse at this level of ground motion.

This chapter includes the following:

- Recommendations concerning the basic seismic design criteria and seismic detailing requirements for storage rack assemblies, elements, connections, and anchorages that are consistent with the equivalent lateral force procedures of 2003 *NEHRP Recommended Provisions*.
- An optional displacement-based evaluation procedure is proposed for evaluating the adequacy of the moment connections and overall stability of racks for MCE ground motions in the down-aisle direction (longitudinal direction) based on connection cyclic test data that is currently under development.
- Optional displacement-based and limit state procedures are also recommended for the cross-aisle direction (transverse direction).



Specific suggested changes to the RMI standard and to the *NEHRP Recommended Provisions* based on the recommendations of this chapter are provided in Appendices B and C, respectively. The ultimate goal is that the recommendations made here be adopted in the next edition of the RMI standard so that it can serve as the reference standard for storage rack design in the codes and standards. In the interim, these recommendations will be submitted to those developing the 2008 edition of the *NEHRP Recommended Provisions* and the 2010 edition of ASCE 7. Based on ICC and NFPA code adoption schedules, the earliest that changes for the 2008 *NEHRP Recommended Provisions* and ASCE 7-10 would be reflected in the ICC or NFPA model building codes would be 2012. Since local jurisdictions normally take 18 months or so to adopt and enforce new model building codes, it will probably be 2014 before any changes would be in effect although voluntary use of the guidance by the rack industry could result in earlier adoption and use.

## 6.2 SUGGESTED SEISMIC DESIGN CRITERIA

**6.2.1 Performance Objectives.** The performance expectations and design intentions for structures designed in accordance with the 2003 *NEHRP Recommended Provisions* are stated in Sec. 1.1 of that document:

The design earthquake ground motions specified herein could result in both structural and nonstructural damage. For most structures designed and constructed to these provisions and constructed according to these *Provisions*, structural damage from design earthquake ground motion will be repairable although perhaps not economically so. The actual ability to accomplish these goals depends upon a number of factors including the structural framing type configuration, materials, and as-built details of construction.... For ground motions larger than the design levels, the intent of these *Provisions* is that there is a low likelihood of structural collapse.

These performance expectations and intentions for the structural design of steel storage racks have been interpreted by the Rack Project Task Group to mean:

To satisfy the intent of the *NEHRP Recommended Provisions*, the goals should be to demonstrate that storage racks located in occupancies accessible to the general public have a low likelihood of collapsing when subjected to maximum considered earthquake (MCE) level ground motions.

As noted above, most structures including storage racks currently are designed using equivalent lateral force procedures and collapse prevention at the MCE ground motions is not explicitly demonstrated but only inferred based on past experience. It is recommended that in addition to the equivalent lateral force design, collapse prevention be explicitly demonstrated using displacement-based and limit state design principles and cyclic connection testing for two reasons: (a) the inelastic behavior of rack structural system members and connections are significantly different from those of building structural systems (although the systems appear physically similar) and (b) storage racks do not have the added benefit of the diaphragms or secondary structural elements found in building structures. This demonstration is considered optional but, if performed, the selective design requirements (the  $I$  factor and  $0.14S_{DS}I$  minimum) of the equivalent lateral force procedure should be adjusted to provide an incentive.

**6.2.2 Design Earthquake Ground Motions.** Storage racks should be designed for the ground motions defined in Chapter 3 of 2003 *NEHRP Recommended Provisions*. Specifically, the

parameters  $S_S$ ,  $S_I$ ,  $F_a$ ,  $F_v$ ,  $S_{MS}$ ,  $S_{MI}$ ,  $S_{DS}$  and  $S_{DI}$  should be determined in accordance with Chapter 3 of the *NEHRP Recommended Provisions* for the precise location and soil profile of the site where the racks are to be located. If the site soil conditions are not known, Site Class D should be assumed. It is recommended that in areas of high seismicity, these values be determined for the longitude and latitude of the site using the CD of ground motion parameters provided with 2003 *NEHRP Recommended Provisions*.<sup>1</sup>

**6.2.3 Recommendation for Seismic Use Group, Occupancy Category, and Importance Factor.** In both the 2003 *NEHRP Recommended Provisions* and the 2002 RMI standard, an occupancy importance factor ( $I$ ) of 1.5 is assigned to steel storage racks for use in areas accessible to the general public. Seismic Use Group is needed to define the Seismic Design Category for a structure which is a key parameter used in specifying seismic code requirements.<sup>2</sup> While neither Sec. 1.2.1 nor 1.2.2 of the 2003 *NEHRP Recommended Provisions* assigns steel storage racks to a specific Seismic Use Group, it is recommended that, in accordance with Sec. 1.2.3 of the 2003 *NEHRP Recommended Provisions*, steel storage racks be assigned to Seismic Use Group I with the caveat that  $I = 1.5$ .

If a displacement-based evaluation is performed in a given rack direction in accordance with Sec. 6.5 of this document, it is recommended that the equivalent lateral force evaluation of Sec. 6.3 be permitted to use an  $I = 1.0$  for that direction since collapse prevention for the MCE is being explicitly evaluated rather than just inferred.

**6.2.4 Seismic Design Category.** The Seismic Design Category (SDC) for steel storage racks used in general public applications should be based on the ground motion parameters  $S_{DS}$  and  $S_{DI}$  of the site and Seismic Use Group I in accordance with Sec. 1.4 of the 2003 *NEHRP Recommended Provisions*. This means that steel storage racks for these applications can be assigned to SDC A, B, C, D or E.

**6.2.5 Structural System Description.** The requirements for steel storage racks supported directly by slabs on grade are defined in Sec. 14.3.5 in the nonbuilding structures chapter of the 2003 *NEHRP Recommended Provisions*. There are two types of nonbuilding structures -- those with structural systems similar to buildings and those with structural systems not similar to buildings. Steel storage racks are classified in the 2003 *NEHRP Recommended Provisions* as having structural systems that are similar to buildings because of the overall behavior of their structural systems.

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<sup>1</sup> There are many nominally priced commercially available GIS based programs and web sites that provide latitude and longitude for a given street address or mapped location (e.g., Microsoft Streets and Trips). Alternatively, parameters may be based on zip code provided the maximum value of the ground motion parameters  $S_S$  and  $S_I$  within the zip code are used; however, the caution about using zip code values provided on the CD should be observed, especially in areas with large variations in seismic hazard.  $S_S$ ,  $S_I$ ,  $F_a$ ,  $F_v$ ,  $S_{MS}$ ,  $S_{MI}$  are used to define the maximum considered earthquake ground motion while  $S_{DS}$  and  $S_{DI}$  are used to define the design earthquake ground motion.

<sup>2</sup> Note that in ASCE 7-05, the term “Seismic Use Group” has been replaced by the term “Occupancy Category” and in this context storage racks in areas accessible to the public should be assigned to Occupancy Category III, which will not affect the assignment of Seismic Design Category.

Two basic types of structural system are used in the design of single selective steel pallet storage racks. In the cross-aisle direction (transverse direction), steel braced frames typically are used for seismic applications. In the down-aisle direction (longitudinal direction), steel moment frames typically are used with special connections although braced frames sometimes are also used. The typical steel storage rack configuration used in areas accessible to the public is illustrated in Chapter 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.

The steel braced frames used in the transverse direction of racks are very similar in appearance to steel ordinary concentrically braced frames (OCBFs) as defined in Table 4.3-1 of the 2003 *NEHRP Recommended Provisions*. Table 4.3.1 references AISC Seismic (AISC 341-02), Part I, Sec. 14, for the detailing and design requirements for steel OCBFs. The requirements of AISC Seismic were specifically developed for building design applications and presume the use of AISC shapes and materials. While it is expected that the distribution of forces to braced steel racks will be very similar to that of steel OCBFs, the inelastic behavior of the members and connections may be significantly different. It should be recognized that AISI is currently developing seismic connection design criteria for cold-formed members which should be considered when available. The horizontal struts and bracing members are typically light gage open sections that are welded directly to open section columns. Gusset plates are not used for transferring loads between braces and beam and columns.

The inelastic response is somewhat dependent on the behavior 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 pallets may tend to slide at higher levels of ground motion which will introduce damping and reduce the effective seismic weight. For clarity, it is recommended that the AISC Seismic requirements not be referenced for design details and that instead a separate identifier – Steel Rack Concentrically Braced Frames – be used for steel braced frames in rack applications. Member design connection details and test requirements are to be based on RMI standard – ANSI MH-16.1-04. It is also recommended that the overall adequacy of the rack frame bracing connections be demonstrated using the optional rack testing and the displacement-based evaluation procedure described in Sec. 6.5 below.

The moment frames used in the down-aisle direction of steel storage racks, while appearing to be very similar to steel moment frames as defined in Table 4.3-1 of the *NEHRP Recommended Provisions*, behave inelastically very differently from ordinary, intermediate, and special steel moment frames for buildings whose details are provided in AISC Seismic. While the moment connections of AISC Seismic are designed to cause inelastic member deformations in the beams away from the connections, connections used in racks have their inelastic behavior occur directly in the rack beam to upright connections. For many connections, there is a significant difference in behavior between positive and negative moments because of the unsymmetrical geometry of the connections. In addition, although the system exhibits highly nonlinear behavior, up to a very large relative rotation, the system remains essentially elastic in the sense that the behavior 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 nonlinear 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 the 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. Since the detailed inelastic behavior of rack moment frames is so different from the behavior of the systems defined in Table 4.3-1 of the *NEHRP Recommended Provisions*, it is recommended that AISC Seismic not be referenced for detailing requirements and that a separate identifier – Steel Rack Moment Frames – be used in rack applications. It also 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 Sec. 6.5 of this Chapter.

Because the detailed behavior 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 nonlinear behavior of the racks and the necessary detailing that will ensure that the seismic response will be acceptable.

**6.2.6 Selection of Design Coefficients for Basic Seismic Force Resisting Systems.** Based on the discussion above, it is recommended that the following design coefficients be used for the basic seismic force resisting systems of steel storage racks for general public applications. These coefficients are needed to apply the equivalent lateral force approach to the design of structures.

| Structural System              | $R$ | $\Omega_0$ | $C_d$ | Height Limit<br>(with public access) |
|--------------------------------|-----|------------|-------|--------------------------------------|
| Steel Storage Racks            |     |            |       |                                      |
| - Concentrically Braced Frames | 4   | 2          | 3 ½   | 35 feet                              |
| - Moment Frames                | 6   | 3          | 5 ½   | 35 feet                              |

The  $R$  values in the above table are taken from ANSI MH-16.1-04, which appear to have been nominally confirmed by John Blume & Associates (Chen et al. 1980a) based on testing and analytical correlations with building structural systems performed in the late 1970s. The  $R$  values are a conversion of  $K$  values. A  $K$  value of 1.0 was suggested for the down-aisle direction (equals an  $R = 5.7$ ) and a  $K$  value of 1.33 was suggested in the cross-aisle direction (equals an  $R = 4.3$ ). It should be noted that the Blume report also expected that in the cross-aisle direction a load factor of 1.25 would be used, which means that really the  $K$  value was 1.67 (equals an  $R = 3.4$ ) and connections would not be designed using the 1/3 allowable stress increase. The values for  $\Omega_0$  and  $C_d$  and the height limits recommended above have been selected from the *NEHRP Recommended Provisions* based on similar basic seismic force resisting systems.<sup>3</sup>

<sup>3</sup> As noted above, the selection of the  $R$ ,  $\Omega_0$ , and  $C_d$  values are based on building structural systems and may not be appropriate for rack structural systems and connections. However, at this point, some of the Rack Project Task Group members are of the opinion that this is the best recommendation we can offer in providing design values. Note too that the height limitations also were taken from the *NEHRP Recommended Provisions* based on building structural system limitations although rack earthquake experience in public applications is limited to racks less than 20 feet in height which would be a more appropriate limitation.

**6.2.7 Drift Limitations.** It is recommended that the deformation requirements of Sec. 14.2.11 of the 2003 *NEHRP Recommended Provisions* apply to storage racks for use in areas accessible to the public. This section states:

The drift limits of Sec. 4.5.1 need not apply to nonbuilding structures if a rational analysis indicates they can be exceeded without adversely affecting structural stability of attached or interconnected components and elements (such as walkways and piping). P-delta effects shall be considered where critical to the function or stability of the structure. Structures shall satisfy the separation requirements as determined in accordance with Sec. 4.5.1 unless specifically amended by this Chapter.

For completeness, it is recommended that for concentrically braced frames the drift limits be checked although, as a practical manner, they will rarely govern. For the moment frame systems, if the recommended optional displacement-based approach discussed below is used to demonstrate structural stability, the drift limits need not be checked. With regards to separation requirements, it is recommended that Sec. 14.3.5.5 of the 2003 *NEHRP Recommended Provisions* be used to define separations. This section states:

Steel storage rack installations shall accommodate the seismic displacements of storage racks and their contents relative to all adjacent or attached components and elements. The assumed total relative displacement for storage racks shall not be less than 5 percent 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 percent will be computed in the cross-aisle direction by a simple frame analysis using equivalent static force procedures. The computed displacement should be determined in accordance with Sec. 5.2.6.1 of the 2003 *NEHRP Recommended Provisions*. Such computed displacements properly done are deemed to be properly substantiated analyses. In the down-aisle direction, a 5 percent calculated drift is possible and, therefore, additional analysis procedures may be required unless 5 percent is assumed.

## 6.3 STRUCTURAL ANALYSIS

**6.3.1 Equivalent Static Force Procedure.** It is recommended that the design of storage racks supported at grade utilize the equivalent lateral force procedure of Sec. 5.2 of the 2003 *NEHRP Recommended Provisions* for determining the seismic base shear along with certain modifications that are appropriate for racks. For purposes of determining the seismic base shear, the slab at grade can be assumed to be rigid from a seismic load perspective. In addition, an optional displacement-based approach is proposed to supplement the equivalent static force procedure. The optional approach assumes that a certified connection moment versus connection rotation curve for use in design will have been provided by the rack supplier for the connector. A typical connection curve is shown in Figure 6-1.

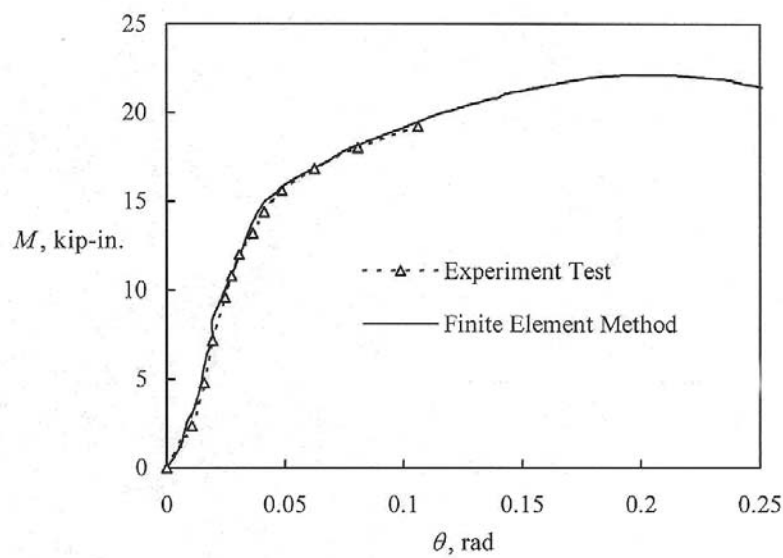


Figure 6-1 Comparison of the cantilever test and the analytical model  
(from Sarawit et al. 2003).

**6.3.1.1 Seismic Base Shear.** The seismic base shear,  $V$ , should be determined for both the down-aisle and cross-aisle directions in accordance with the following equation:

$$V = C_s W$$

where

$C_s$  = the seismic response coefficient determined in accordance with Sec. 6.3.1.2

$W$  = the effective seismic weight

$$= (0.67 \times PL_{RF} \times PL) + DL + 0.25 \times LL$$

$PL_{RF}$  = product line reduction factor; where storage racks are located in areas open to the general public, it is recommended that  $PL_{RF}$  be taken as 1.0 in down- and cross-aisle directions.

$PL$  = rated rack capacity for pallet loads

$DL$  = dead load of rack

$LL$  = live load other than pallets or products stored on the racks (typically this is zero for general public applications)

**6.3.1.2 Seismic Response Coefficient.** The seismic response coefficient should be determined in accordance with the following equations:

For  $T$  less than or equal to  $T_s$

$$C_s = \frac{S_{DS}}{(R/I)}$$

For  $T$  greater than  $T_s$

$$C_s = \frac{S_{D1}(1/T)}{(R/I)}$$

except that  $C_s$  should not be taken less than  $0.14S_{DS}I$

and where  $S_I$  is equal to or greater than  $0.60 g$ ,  $C_s$  should not be taken less than

$$C_s = \frac{0.5S_I}{(R/I)}$$

where

$S_{DS}$  = the design spectral parameter in the short period range (see Sec. 6.2.2)

$S_{D1}$  = the design spectral parameter at a period of 1.0 seconds (see Sec. 6.2.2)

$I$  = the occupancy importance factor = 1.5 (see Sec. 6.2.3)

$R$  = the response modification factor as specified in Sec. 6.2.6

$T_s$  =  $S_{D1} / S_{DS}$

$T$  = the fundamental period of the rack structure in the direction under consideration determined in accordance with Sec. 6.3.1.3

It should be noted that the above seismic response coefficients are taken directly from the 2003 *NEHRP Recommended Provisions* and are reflected in the 2003 *IBC* and *ASCE 7-02*; therefore, they are code requirements for all jurisdictions that have adopted the *IBC* or *NFPA 5000* building codes. However, if the optional displacement-based evaluation procedures of Sec. 6.5 are utilized for a given direction, the  $I$  factor should be taken as  $1.0$  and the  $0.14S_{DS}I$  minimum should be replaced by a minimum of  $0.01$  for storage racks assigned to SDC B or above (which is the minimum for building structures in the 2003 *NEHRP Recommended Provisions*). Note that the minimum provided value for  $C_s$  remains unchanged.

**6.3.1.3 Fundamental Period.** The fundamental period used to determine the base shear coefficients in Sec. 6.3.1.2 in each direction should be determined in accordance with the following procedures. Since racks are nonbuilding structures, the upper limit on period (not exceeding the approximate period calculation by a factor,  $C_u$ ) specified in Sec. 5.2.2 of the 2003 *NEHRP Recommended Provisions* does not apply. This exemption is clearly implied in the requirements of Sec. 14.2.9 of the 2003 *NEHRP Recommended Provisions* that do not permit calculation of the fundamental period using the approximate period calculations.

The fundamental period in the cross aisle direction should be determined using either modal analysis procedures, Eq. 14.2-4 of the 2003 *NEHRP Recommended Provisions* (the Rayleigh procedure), 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. See Chen et al. (1980a)] for suggested adjustments. The seismic weight used in the period calculations should be equal to the effective seismic weight,  $W$ , specified in Sec. 6.3.1.1. In lieu of doing a period calculation, the period may be taken as equal to or less than  $T_s$ .

The computed period of steel rack moment frame systems can vary appreciably because the rotational stiffness of the moment connections varies 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-moment connections is consistent with the connection stiffness assumed in the analysis model. The determination of the fundamental period therefore may require an iterative procedure. Alternatively, the period may be determined utilizing the rotational spring constant,  $F$ , of the connection as determined in accordance with Sec. 9.4 of the commentary of ANSI MH-16.1-04. The period calculation procedure provided in Appendix A of this document may be used (for standard evenly spaced racks) to compute the fundamental period using either the rotation spring constant,  $F$ , as the connection rotation stiffness,  $k_c$ , or using the certified moment-rotation connection relationship. Other rational procedures may be used to compute the period but it should be demonstrated that the connection stiffness assumed does not result in computed moments from the base shear calculation that have a corresponding connection stiffness less than that assumed.

**6.3.1.4 Vertical Distribution of Forces.** The base shear forces should be distributed vertically to the rack structure in accordance with the procedures of Sec. 5.2.3 of the 2003 *NEHRP Recommended Provisions* (this is identical to that found in Sec. 2.7.4 of ANSI MH-16.1-04). When evaluating overturning, it is recommended that the fully loaded rack condition be considered in addition to the requirement found in Sec. 2.7.6 of the ANSI MH-16.1-04 where only the top loaded rack is considered.

Based on recent research by Filiatrault and Wanitkorkul (2004), the force distribution exponent factor,  $k$ , may be taken as 1.0 regardless of the fundamental period (i.e., triangular distribution permitted in both directions). This revised  $k$  factor should be considered in the next edition of the RMI standard, ANSI MH-16.1.



**6.3.1.5 Horizontal Distribution of Forces.** The horizontal shear forces at any level should be distributed in proportion to and at the location where the vertical loads are imparted to the rack. This approach is consistent with Sec. 2.7.5 of ANSI MH-16.1-04 and Sec. 5.2.4 of the 2003 *NEHRP Recommended Provisions*.

**6.3.2 Structural Analysis.** Steel storage racks generally do not have horizontal diaphragms or bracing and should therefore be considered as having flexible diaphragm structural systems. Thus, they should be analyzed as two-dimensional systems using flexible diaphragm loading assumptions for each brace and moment frame line of resistance.

**6.3.2.1 Braced Frame Systems.** Steel storage rack braced frames may be treated as statically determinate structures and hand analyses may be used to determine member design forces resulting from base shear applied forces. Computerized frame analyses also may 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 and the concrete slab.

**6.3.2.2 Moment Frame Systems.** It is typical to use the portal method to determine forces in storage rack members resulting from base shear applied forces. Computer frame analyses also may be used to determine member forces resulting from base shear applied forces. For such analyses, the rack members are treated as linearly elastic members, the connection stiffness may be assumed to be equal to  $F$  (see Sec. 6.3.1.3 above), and the bases are modeled to represent the semi-rigid connection of the base plates and the concrete slab. Alternatively, the connection stiffness may be based on the certified moment-rotation curve as discussed in Sec. 6.3.1.3.

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

**6.3.2.3 P-delta Effects.** Steel rack braced frame systems typically are relatively stiff structures (prior to brace buckling) and P-delta effects typically can be ignored in design. However, steel rack moment frame systems are very flexible structures and P-delta effects therefore may be significant.

If the certified moment-rotation connection properties have been determined using a portal test, 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 modeling and procedures.

## 6.4 DESIGN

**6.4.1 Load Combinations Including Seismic Loads.** The 2003 *NEHRP Recommended Provisions* presumes that ASCE 7 load combinations are used for design. The ASCE 7 load combinations include a term that accounts for vertical seismic effects. In Sec. 12.4 of ASCE 7-05, the seismic load combinations have been recast into a more convenient form. It is recommended that the load combinations presented below incorporating this more convenient form be used for the design of storage rack structures.

Many racks are currently designed using the alternate basic allowable stress load combinations that permit a 1/3 stress increase. It is anticipated that the alternate basic allowable stress load combinations will be eliminated from building codes and that only the ASCE 7 load combinations will be permitted for design. Thus, only the allowable stress basic load combination that does not permit the 1/3 stress increase is considered in this report.

#### 6.4.1.1 Basic Load Combination for Allowable Stress Design

$$(1.0 + (0.75 \times 0.14 S_{DS}) DL + 0.75(1.0 + 0.14 S_{DS}) PL + (0.75 \times 0.7) E_h + 0.75 LL$$

$$(0.6 - 0.14 S_{DS})DL + (0.6 - 0.14 S_{DS}) PL - 0.7 E_h$$

where

$DL$  = dead load of the rack

$PL$  = total pallet rated capacity

$E_h$  = effect (i.e., member forces) of horizontal earthquake resulting from application of base shear load ( $V$ )

$LL$  = see Sec. 6.3.1.1

$S_{DS}$  = The design spectral parameter in the short period range (Sec. 6.2.2)

Note that  $E_h$  generally also includes a factor for redundancy. Using the definition of redundancy found in the 2003 *NEHRP Recommended Provisions*, the redundancy factor for rack structures will always be determined as 1.0 (ignored). Also note that no increase in allowable stresses is permitted for the above load combination.

#### 6.4.1.2 Basic Load Combinations for Allowable Stress Design with Overstrength

$$1.0 + (0.75 \times 0.14 S_{DS}) DL + 0.75(1.0 + 0.14 S_{DS}) PL + 0.75 (0.7 \Omega_0 E_h) + 0.75 LL$$

$$(0.6 - 0.14 S_{DS}) DL + (0.6 - 0.14 S_{DS}) PL - 0.7 \Omega_0 E_h$$

where  $\Omega_0$  = the overstrength factor for the system (see Sec. 6.2.6).

Note that a 1.2 increase in allowable stresses is permitted for the above load combination.

#### 6.4.1.3 Strength Design Load Combinations. The basic combinations for strength design are:

$$(1.2 + 0.2 S_{DS}) DL + (0.85 + 0.2 S_{DS}) PL + 0.5LL + E_h$$

$$(0.9 - 0.2 S_{DS}) DL + (0.9 - 0.2 S_{DS}) PL - E_h$$

The above load combination is very similar to those found in Sec. 2.2 of ANSI MH-16.1-04 except that the vertical earthquake term is included.

The basic combinations for strength design with overstrength are:

$$(1.2 + 0.2 S_{DS}) DL + (0.85 + 0.2 S_{DS}) PL + 0.5LL + \Omega_0 E_h$$

$$(0.9 - 0.2 S_{DS}) DL + (0.9 - 0.2 S_{DS}) PL - \Omega_0 E_h$$

**6.4.2 Steel Design Approach for Members and Connections.** Steel storage rack structures may either be designed using the strength design method (load resistance factor design, LRFD) or the allowable stress design (ASD) method. Detailed design procedures for both methods are provided in ANSI MH-16.1-04.

**6.4.2.1 Allowable Stress Design.** For ASD, member stresses are determined using the basic load combinations of Sec. 6.4.1.1 and compared with allowable stresses provided in ANSI MH-16.1. No increase in allowable stresses is permitted for member design when the load combinations of Sec. 6.4.1.1 are used. Where required for connection design, connection stresses should be determined using the overstrength combinations for ASD and compared with connection capacities. When analysis is used to demonstrate connection adequacy, a 1.2 increase in allowable stresses is permitted with the overstrength combinations.

Alternatively, test results should be used to demonstrate that the storage rack structural system will maintain its structural stability when subjected to maximum considered earthquake ground motion. RMI has verbally committed to developing analysis and/or test procedures for evaluating moment connections and cross-aisle braced frame connections to satisfy this criteria. An alternate optional displacement-based procedure that may be used to evaluate and verify the adequacy of moment connections and cross-aisle frames is provided in Sec. 6.5.

**6.4.2.2 Load Resistance Factor Design.** For LRFD, member demands are determined using the basic load combinations in Sec. 6.4.1.3 and compared with member capacities provided in ANSI MH-16.1. Where required for connection design, connection forces should be determined using the overstrength combinations for strength design and compared with connection capacities.

Alternatively, test results should be used to demonstrate that the rack structural system will maintain its structural stability when subjected to maximum considered earthquake ground motion. As indicated above, RMI has verbally committed to developing analysis and/or test procedures for evaluating moment connections and cross-aisle braced frames connections to satisfy this criteria. An alternate optional displacement-based procedure that may be used to evaluate and verify the adequacy of moment connections and cross-aisle frames is presented in Sec. 6.5.

**6.4.3 Design Approach for Base Plates, Base Plate Connections, and Anchor Bolts.** Base plate analyses should be designed in accordance with rational methods that 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 overstrength. The literature has extensive guidance in this area.

It is recommended that the weld connecting the base plate to the upright be designed for the load combinations with overstrength or for the nominal yield capacity of the base plate, whichever is least.

It is recommended that post-installed anchors connecting the rack base plate to the slab on grade be designed for the load combinations with overstrength. However, the anchor bolt design forces need not exceed the nominal yield capacity of the base plate. The anchor bolt allowable forces and capacities should satisfy ICC Evaluation Services reports for the anchors determined to be acceptable for seismic applications.

**6.4.4 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. Procedures that are deemed acceptable by the City of Los Angeles are found in Los Angeles Department of Building and Safety Information Bulletin/Public Building Code Reference No. L.A.M.C. 91.1806, Document No. P/BC 2002-100, *Acceptable Design and Analysis Methods for Use of Slabs-on-Grade as Foundations*, effective 5-10-04. This document is included as Appendix D of this report.

**6.4.5 Design of Added Rack Components.** In some cases, such components as column extensions, chains, and screens are added to racks to secure contents. The design loads that these added components have on the rack structure should be evaluated by rational methods (see Chapter 8 of this document for additional guidance).

## **6.5 TENTATIVE PROCEDURE FOR DISPLACEMENT-BASED MCE EVALUATION OF PALLET-TYPE STEEL PALLET STORAGE RACK SYSTEMS**

Presented in this section is a recommended state-of-the-art procedure for evaluating steel storage racks that are accessible to the general public. Very few of the desired test data needed to implement the procedure exist at this time. The Rack Project Task Group believes that the fundamental theory provided by the procedures is technically sound and consistent with other displacement-based procedures and limited experimental testing. It is expected that the protocols for the test data needed by the procedure will be developed soon by RMI. However, many years of development and testing will be needed before the procedure described here will be ready for use as a mandatory code requirement.

Nevertheless, in the interim, design engineers are encouraged to utilize the method if they have sufficient information to do so, are confident of the test data, and find that the procedure confirms past storage rack performance in large earthquakes. Similarly, regulatory authorities should consider permitting this method as an “Alternate Means of Compliance” to support a construction permit application. It is recommended that if this optional displacement-based evaluation procedure is utilized for a given design direction with the equivalent lateral force procedure, the  $I$  factor for that direction should be taken as 1.0 rather than 1.5 and the minimum

base shear value of  $0.14S_{DS}$  should be taken as 0.01 since an explicit demonstration of down-aisle capacity and stability will have been made.

One of the important performance objectives for steel storage racks used in areas accessible to the public is to provide a structural system that will likely not collapse when subjected to maximum considered earthquake (MCE) ground motions. To demonstrate that storage racks will likely not collapse in the MCE, displacement-based evaluation procedures are provided for both the down-aisle (moment frame) and cross-aisle (braced frame) directions.

**6.5.1 Displacement-Based Procedure for Evaluating Collapse Prevention in the Down-Aisle Direction.** One alternative for evaluating whether a storage rack will likely not collapse in the moment frame direction is the optional displacement-based evaluation procedure. 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 (see Appendix A of this document) are that:

- The same moment connections are used throughout the moment frame system of the racks.
- All moment connections of the racks simultaneously experience very similar rotations at all times.
- The vast majority of inelastic behavior occurs at the moment connections.
- The overall seismic response can be reasonably modeled as a single-degree-of-freedom (SDOF) system.
- The connection moment versus connection rotation curves have been developed based on cyclic testing and the design curve has been adjusted for uncertainty in the cyclic data and provided to the design engineer for the racks.

Details of the test procedure still need to be developed but, for discussion purposes, it can be assumed that the procedures are similar to the RMI portal testing procedures found in Sec. 9.4.2 of the commentary of ANSI MH-16.1-04. For the purpose of the testing, it is presumed that the vertical load on the beams is the rated pallet load and that the testing will be cyclic. Details of a workable testing setup can be found in Krawinkler et al. (1979). Also for discussion purposes, the design curve is called a certified connection moment versus rotation curve for the connector. For illustrative purposes, it is assumed that the certified curve would be similar to that is Figure 6-1 (with  $M_{max} = 22$  in-kip and  $\theta_{max} = 0.20$  rad).

The following are the steps comprise the evaluation procedure:

Step 1 — From the certified connection curve, determine the rotation,  $\theta_{max}$ , at the maximum moment,  $M_{max}$ . Determine the minimum permitted connection stiffness  $k_c$  as:

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

It is assumed that the certified connection curve has been based on a cyclic portal test as described in Sec. 9.4.2 of ANSI MH-16.1-04 and the connection stiffness represents the average of the positive and negative moment connection stiffness and partially includes P-delta effects (the test procedure still needs to be developed by RMI). The test procedure should provide an adjustment to the maximum moment and stiffness that excludes the P-delta effects associated with the tests.

Step 2 — Determine the fundamental period,  $T_1$ , based on the following equation:

$$T_1 = 2\pi \sqrt{\frac{\sum_{i=1}^{N_L} W_{pi} h_{pi}^2}{g \left( N_c \left( \frac{k_c k_{be}}{k_c + k_{be}} \right) + N_b \left( \frac{k_b k_{ce}}{k_b + k_{ce}} \right) \right)}}$$

where

$W_{pi}$  = the effective horizontal seismic weight of the  $i^{\text{th}}$  pallet supported by the storage rack ( $0.67PL$ ),

$h_{pi}$  = elevation of the center of gravity of the  $i^{\text{th}}$  pallet with respect to the base of the storage rack,

$g$  = acceleration of gravity,

$N_L$  = number of loaded levels,

$k_c$  = rotational stiffness of each beam-to-upright connection,

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

$N_c$  = the number of beam-to-upright connections, and

$N_b$  = the number of base plate connections and where  $k_{be}$  is the beam end and  $k_{ce}$  is the base upright end rotational stiffness assumed to be given by:

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

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

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, and  $H$  = clear height of the upright.

The above formula is limited by the assumptions inherent in its derivation. One should be careful to respect these assumptions which include:

- All the connectors have the same rotation or that the displaced shape of the rack is linear with height (it is believed this assumption becomes more accurate at large displacements and less accurate at small displacements).
- The bays are uniform in width.
- The beam spacing is constant between levels.
- The columns are prismatic over their entire height.
- The beams are prismatic and their spans are the same at all levels.

Step 3 -- Compute the maximum displacement,  $D$ , without P-delta effects at the effective height of the SDOF system of the rack based on the period,  $T_1$ , for MCE ground motion using the following equation:

$$D = \frac{gS_{M1}(T_1/1.0\text{sec})}{4\pi^2 B}$$

where

$S_{M1}$  =  $1.5 S_{DI}$  = MCE spectral parameter at 1 second with site effects (Sec. 8.2.2.2),

$B$  = Damping coefficient as a function of the effective damping as follows (taken from Table 13.3-1 of the 2003 *NEHRP Recommended Provisions*)

| Damping    | $B$ factor |
|------------|------------|
| 5 percent  | 1.0        |
| 10 percent | 1.2        |
| 20 percent | 1.5        |
| 30 percent | 1.7        |

For purposes of this evaluation, it is acceptable to assume that the damping in the down-aisle direction is a function of the effective horizontal peak ground acceleration (EHPGA) of the MCE. The EHPGA may be taken as the peak spectral acceleration of the MCE divided by 2.5 and the peak spectral acceleration of the MCE is simply 1.5 times  $S_{DS}$ . Therefore, the EHPGA is simply  $0.6S_{DS}$ . The following values of equivalent damping (and associated damping factor  $B$ ) may be used for the MCE evaluation.<sup>4</sup> Larger values may be used if justified by testing.

| EHPGA = $0.6 S_{DS}$             | Damping | $B$  |
|----------------------------------|---------|------|
| Less than or equal to $0.1g$     | 5%      | 1.0  |
| $0.2g$                           | 10%     | 1.2  |
| $0.3g$                           | 15%     | 1.35 |
| $0.4g$                           | 20%     | 1.5  |
| $0.5g$                           | 25%     | 1.6  |
| Greater than or equal to $0.6 g$ | 30%     | 1.7  |

Step 4 – Adjust the displacement demand,  $D$ , to account for P-delta effects by multiplying by the factor  $(1 + \alpha)$ :

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

where

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

In the above equation,  $W_{pi}$  should not be scaled by the 0.67 effective horizontal seismic weight factor. The derivation of the P-delta adjustment factor is explained in Appendix A.

Step 5 – Determine maximum demand rotation,  $\theta_{demand}$  (drift angle), as follows:

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

where

$$h_{tot} = \text{height of center of gravity of highest pallet}$$

<sup>4</sup> These values of equivalent damping are based on experimental shake-table testing performed by Chen et al. (1980a) and on extrapolation of recent testing performed by Filiatrault and Wanitkorku (2004).



$0.72 h_{tot}$  = height of center of gravity of equivalent SDOF of rack (see Appendix A for derivation of 0.72 factor)

If one presumes high seismicity (i.e.,  $S_I$  greater than 0.5) and Site Class D ( $F_v = 1.5$ ) and therefore takes  $S_{DI} = S_I$  and if one assumes  $h_{tot}$  is specified in inch units and that  $g = 386.4$  inches per second squared and the damping factor  $B = 1.7$ , one can solve directly for  $\theta_{demand}$  as simply the following and skip Step 3 and the first half of Step 5:

$$\theta_{demand} = 12 (1 + \alpha) (T_I / 1.0 \text{ sec}) S_I / h_{tot}$$

Step 6 — Check whether maximum demand rotation,  $\theta_{demand}$ , is less the maximum rotation capacity,  $\theta_{max}$ . If demand rotation is less than maximum rotation capacity, connection design is ok and go to step 6. If the connection is not ok, provide an alternate connection design and go back to Step 1.

Step 7 — If the connections are ok, assume maximum column moment,  $M_c$ , and axial load,  $P_c$ , from seismic loads (not including gravity loads) are as follows:

$$M_c = 1.2 M^*_{max}$$

$$P_c = 1.2 N_L M^*_{max} / 0.5L \text{ (end columns)}$$

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

Evaluate the columns in accordance with the load combinations specified in Sec. 6.4.1 and the allowable capacities specified in ANSI MH-16.1-02. Note that the 1.2 factor is the presumed ratio between the maximum negative moment and the average maximum moment of the connection.

**6.5.2 Evaluation and Design Procedures for Cross-Aisle Direction.** One alternative for evaluating whether a storage rack will likely not collapse in the cross-aisle direction, the optional displacement-based procedure provided below is recommended for sites in areas of low seismicity (i.e., the MCE short-period spectral acceleration,  $S_S$ , is less than or equal to 0.258). It is assumed in this procedure that rack braced frames will be tested using a test procedure similar to that used by Krawinkler et al. (1979). It also is assumed that the storage rack unit will be loaded with pallets loaded vertically to their rated capacity at all levels and that the units will be cyclically tested. A certified force displacement curve is to be 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. It is expected that the cyclic testing will continue until the totally applied lateral force begins to decrease. The design details used in the testing including base plates should represent those found in actual operation. It is expected that the details of the required testing procedure will be developed by RMI. It also is expected that the results of the testing will be similar in general shape to that found in Krawinkler et al. (1979). The procedure assumes that the rack can be modeled as 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 based on a pushover type of analysis. The particular nonlinear characteristics of the bracing systems must be accurately represented in such a pushover analysis. The axial and rotational capacity of the connections and members must be able to resist without fracture the forces induced at the maximum displacements.

The displacement-based evaluation procedure does not explicitly account for sliding of pallets and shifting of contents. For sites where the MCE short period,  $S_s$ , ground motion levels are greater than  $0.25g$ , the procedure provided in Sec. 6.5.2.1 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 uprights and bracing. For such situations, it is recommended that the alternative limit state design approach suggested in Sec. 6.5.2.2 be used in lieu of the displacement-based procedure. This approach directly considers that the forces in the uprights are limited by pallet sliding. It is acknowledged that both of the approaches recommended in this section require significant development, testing, and verification before they are considered for use in the RMI design standards or the *NEHRP Recommended Provisions*.

### 6.5.2.1 Displacement Based Cross-Aisle Evaluation Procedure

Step 1 – From the certified force displacement curve, determine the maximum lateral force,  $F_{max}$ , applied at the top level that the storage rack 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 center of gravity of the top pallet are the same and the displacements increase linearly with height. Therefore, determine the displacement at the equivalent SDOF height,  $D_{equiv}$ , by scaling by 0.72 where 0.72 is the factor that converts  $h_{tot}$  to the height of the SDOF system (see Appendix A):

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

Step 2 – Determine the stiffness of the equivalent system as:

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

Step 3 – Determine the equivalent period as:

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

where  $W$  = the effective seismic weight on one rack unit (0.67 is the rated capacity but this may be less than  $0.67PL$  if demonstrated by test) and  $g$  = acceleration of gravity.

Step 4 – Determine the MCE spectral coefficient,  $S_a$ , given at site with  $S_{MS}$  and  $S_{MI}$ . Note that  $S_{MS} = 1.5S_{DS}$  and  $S_{MI} = 1.5S_{DI}$  and that  $T_s = S_{DI}/S_{DS}$ .

If  $T$  is less than or equal to  $T_s$ , then  $S_a = S_{MS}$

If  $T$  is greater than  $T_s$ , then  $S_a = S_{MI} (1.0 \text{ sec}/T)$

Step 5 – Compute the displacement demand,  $D_{demand}$ , of the SDOF equivalent system as follows:

$$D_{demand} = \frac{S_a W}{BK}$$

where

$B$  = Damping coefficient as a function of the effective damping as follows (taken from Table 13.3-1 of the 2003 *NEHRP Recommended Provisions*)

| Damping    | $B$ factor |
|------------|------------|
| 5 percent  | 1.0        |
| 10 percent | 1.2        |
| 20 percent | 1.5        |
| 30 percent | 1.7        |

For purposes of this evaluation, it is acceptable to assume that the damping in the cross-aisle direction is a function of the effective horizontal peak ground acceleration (EHPGA) of the MCE. The EHPGA may be taken as the peak spectral acceleration of the MCE divided by 2.5 and the peak spectral acceleration of the MCE is simply 1.5 times  $S_{DS}$ . Therefore, the EHPGA is simply  $0.6S_{DS}$ . The following values of equivalent damping (and associated damping factor  $B$ ) may be used for the MCE evaluation.<sup>5</sup> Larger values may be used if justified by testing.

| EHPGA = $0.6 S_{DS}$           | Damping | $B$  |
|--------------------------------|---------|------|
| Less than or equal to 0.1g     | 3%      | 0.8  |
| 0.2g                           | 5%      | 1.0  |
| 0.3g                           | 7.5%    | 1.1  |
| 0.4g                           | 10%     | 1.2  |
| 0.5g                           | 12.5%   | 1.28 |
| Greater than or equal to 0.6 g | 15%     | 1.35 |

Step 6 – Check whether displacement demand,  $D_{demand}$ , is less than the maximum displacement capacity,  $D_{equiv}$ . If displacement demand is less than the displacement capacity, cross-aisle direction is ok. If not the cross-aisle direction is not ok, provide an alternate cross-aisle design and go to Step 1.

### 6.5.2.2 Cross-Aisle Limit State Design Approach

It is recommended that the following limit state approach be used when  $S_S$  is greater than 0.25.

<sup>5</sup> These values of equivalent damping are based on experimental shake-table testing performed by Chen et al. (1980a) and on extrapolation of recent testing performed by Filiatrault (2001).

Step 1 -- 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 weight,  $PL$ , 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 as 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.

Step 2 -- Verify that the ratio of the section moduli of the beam minor to major axes is at least equal to the design coefficient of friction.

Step 3 -- Design the uprights, bracing, and floor connections to stay elastic for the design limit state forces.

Step 4 -- Determine the displacement of the uprights and beams for the design limit state forces.

Step 5 -- Estimate the sliding displacements of pallet contents to be sure they are within acceptable limits (see Sec. 8.3.6 and 8.3.7 for more guidance). When determining the sliding displacement demands, it is recommended that consideration be given to reducing the demands based on the concurrent displacements of the uprights and beams.

**6.5.3 Example Using Displacement-Based Evaluation Procedures.** The following example is used to illustrate the displacement-based evaluation procedures described in Sec. 6.5.1 and 6.5.2 above. It is based on the rack testing performed by Krawinkler et al. (1979). The example properties are taken from the same reference.

For this example, it is assumed that the seismic weight of each rack level is 3 kips which is the load that Krawinkler used in the second down-aisle test using Connection Type A. For this example, the data for Connection Type A also are used. For ground motion criteria, it is assumed that the site is in the plateau region of California (old *UBC* Zone 4 outside the near field) on Site Class D soil. Therefore,  $S_{DS} = 1.0$ ,  $S_{DI} = 0.6$ ,  $T_s = 0.6$  seconds.

### Down-Aisle Example

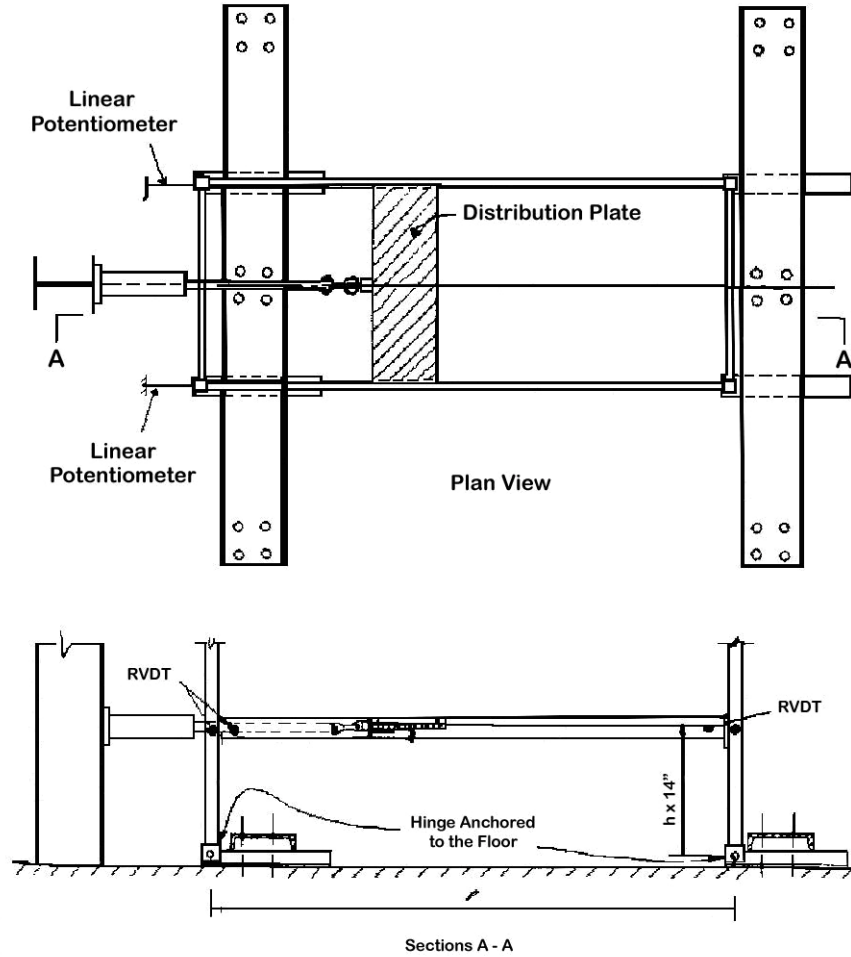


Figure 6-2 Portal test set-up (Krawinkler et al. 1979).

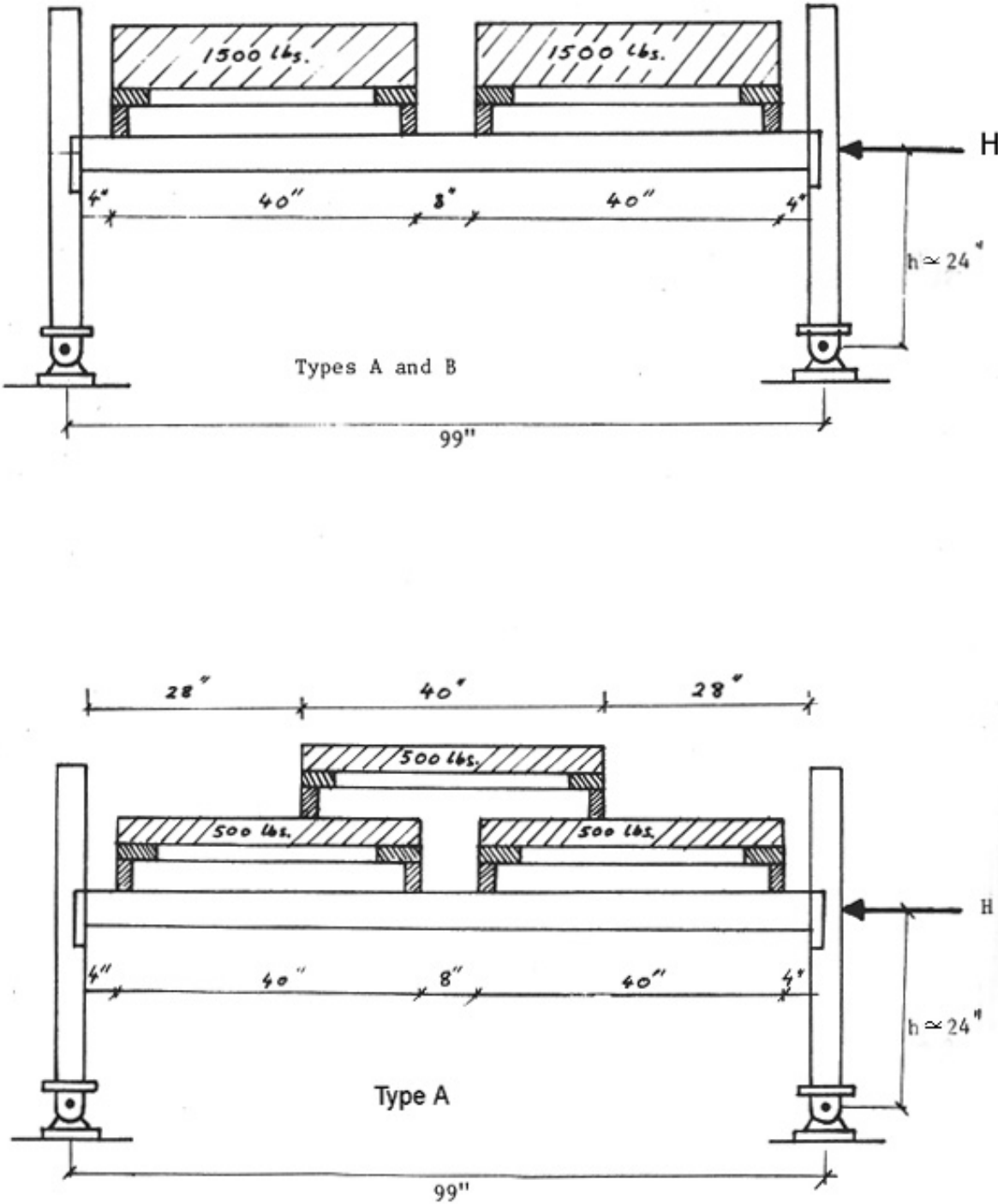


Figure 6-3 Portal tests – loading arrangements (Krawinkler et al. 1979).

1. From Krawinkler portal test data curves (see Figure 6-4):

Determine maximum resisting force per frame = 2.4 kips

Determine displacement at maximum resisting force = 3.0 inches

Weight on test frame = 3.0 kips

Compute  $\theta_{max} = 3 / 24 = 0.125$  (see test set up in Figure 6-3)

Compute maximum,  $M_{max}$ , moment per connection = 2.4 kips x 24 in/2 con

$$M_{max} = 28.8 \text{ in-kips}$$

Adjust  $M_{max}$  by removing the P-delta effects

P-delta moment per connection = (3 kips x 3 in.)/2 = 4.5 in-kips

Adjusted  $M^*_{max} = 28.8 + 4.5 = 33.3$  in-kips

Compute minimum connection secant stiffness,  $k_c$ ,

$$k_c = M^*_{max} / \theta_{max} = 33.3 / 0.125 = 266.4 \text{ in-kips/rad}$$

2. Using rack period formula in Step 2 of Sec. 6.5.1, compute the down-aisle period assuming  $k_b = k_c$ ,  $g = 386 \text{ in/sec}^2$  and the following example data:

Assume the rack is 3 levels high and 4 bays wide ( $N_c = 48$  and  $N_b = 10$ )

Weight per level,  $W_{pi} = 4 \times 3$  kips per bay/level = 12 kips

$L = 99$  inches  $H = 58$  inches  $I_c = 1.037 \text{ in}^4$   $I_b = 2.664 \text{ in}^4$

$h_{p3} = 3.5 \times 58 = 201$  in,  $h_{p2} = 2.5 \times 58 = 143$  in,  $h_{p1} = 1.5 \times 58 = 85$  in

Substituting the above data and  $k_c = 266.4$  in-kip/rad into the period formula, one obtains:

$$T_I = 2.40 \text{ seconds}$$

3. Using the P-delta adjust formula in Step 4 of Sec. 6.5.1, compute the adjustment factor  $(1 + \alpha)$ .

Substituting the data in Step 2 of this example into the formula in Step 4 of Sec. 6.5.1, one obtains:

$$(1 + \alpha) = 1.36$$

4. Compute the rotation demand,  $\theta_{demand}$ , using the simplified second equation in Step 4 of Sec. 6.5.1 noting that  $h_{tot} = h_3 = 201$  inches.

$$\theta_{demand} = 12(1 + \alpha)(T_1 / 1.0) S_I / h_{tot}$$

$$\theta_{demand} = 12 \times 1.36 \times 2.40 \times S_I / 201 = 0.195 S_I \text{ radians}$$

For  $S_I = 0.60g$

$$\theta_{demand} = 0.195 \times 0.60 = 0.117 \text{ radians} < 0.125 \text{ radians}$$

5. Since  $\theta_{demand}$  determined in Step 4 is less than  $\theta_{max}$ , design is ok and proceed to Step 6. Otherwise revise the connection design and go back to Step 1 and try again.
6. Since  $\theta_{demand}$  is less than  $\theta_{max}$ , and therefore connection design is acceptable. If acceptable, compute and check columns based on the following seismic member forces which do not include gravity loads:

Maximum end column moment,  $M_c = 1.2 M_{max}^* = 1.2 \times 33.3 = 40.0$  in-kips

Maximum end column axial load,  $P_c = 1.2 N_L M_{max}^* / 0.5L$

$$P_c = 2.4 \times 3 \times 33.3 / 0.5 \times 99 = 4.8 \text{ kips}$$

Maximum central column moment,  $M_c = 1.2 M_{max}^* = 1.2 \times 33.3 = 40.0$  in-k  
(at lowest level)

Maximum central column axial load,  $P_c = 0$  (at lowest level)

Complete evaluation of columns using the load combinations of Sec. 6.4.1 (including gravity loads) and the allowable capacities of ANSI MH-16.1-02 except for the phi factors may be taken as 1.0.



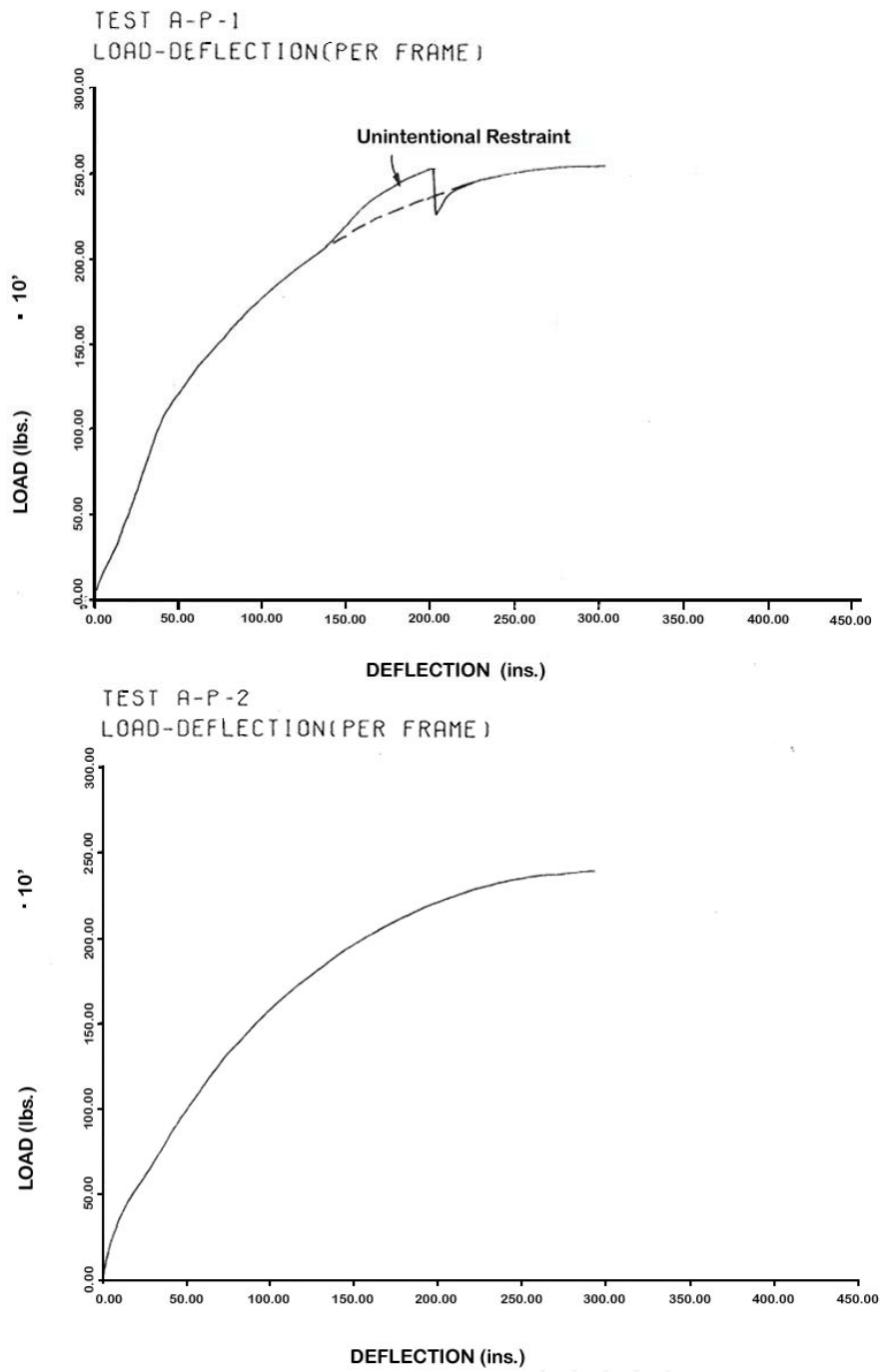


Figure 6-4 Portal tests –  $H - \sigma$  for Rack Type A (Krawinkler et al. 1979).

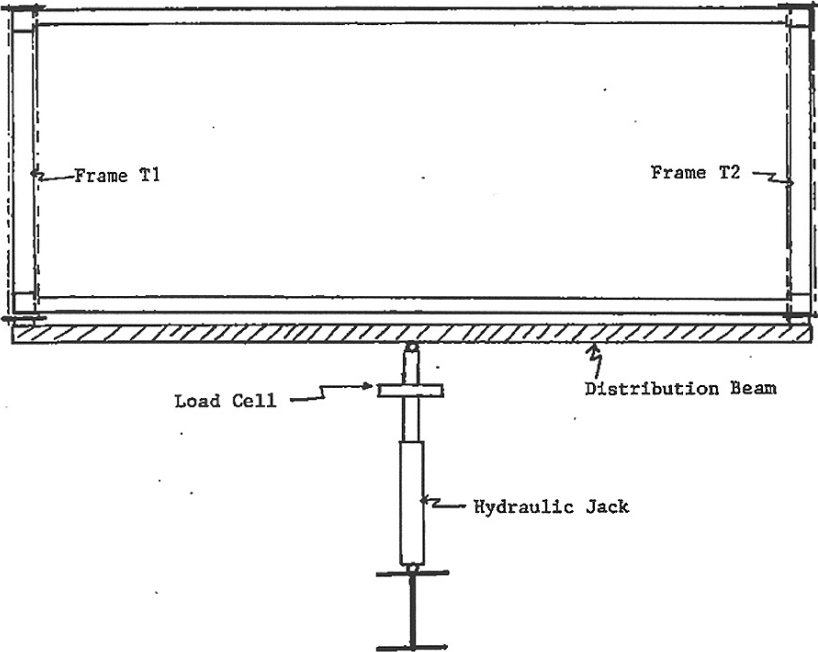


Figure 6-5 Experimental set-up – transverse tests (Krawinkler et al. 1979).

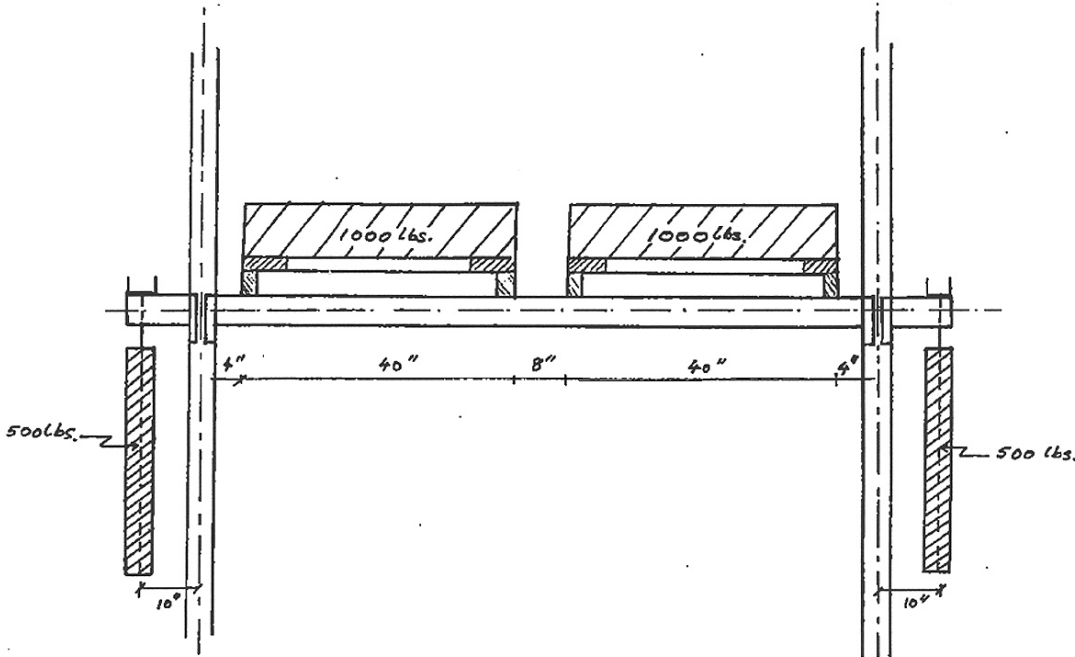


Figure 6-6 Loading arrangements per level per frame – Tests A-R-2 and B-R-2 (Krawinkler et al. 1979).

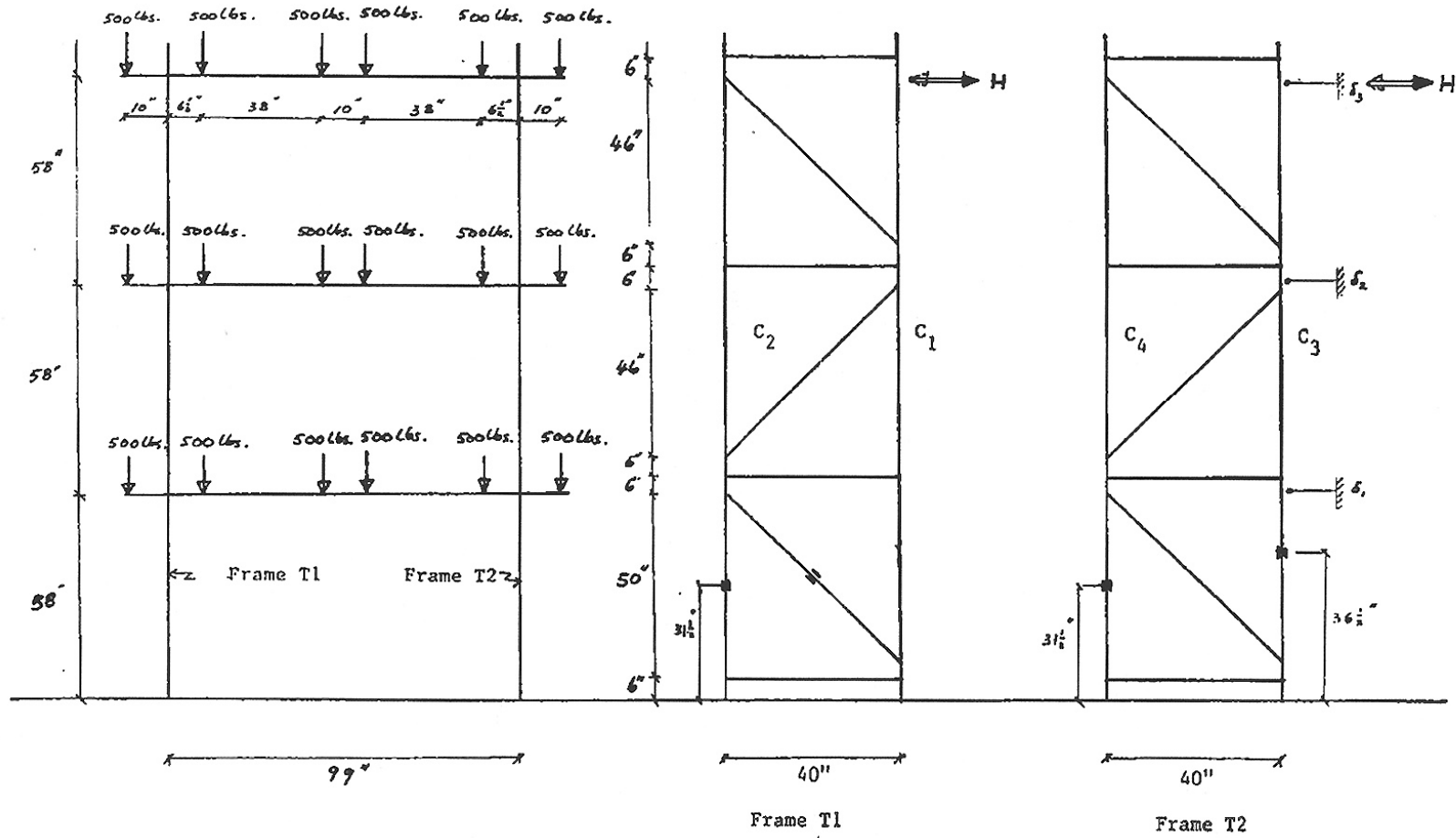


Figure 6-7 Loading summary and instrumentation – Test A-R-2 (Krawinkler et al. 1979)

**Cross-Aisle Example**

1. From Krawinkler transverse direction test curves (see Figure 6-8)

$$\text{Determine the maximum resisting force, } F_{max} = 2.7 \text{ kips}$$

$$\text{Determine the displacement, } D_{max} \text{ at } F_{max} = 1.8 \text{ inches}$$

Determine displacement capacity at equivalent SDOF height recognizing that  $h_{tot} = 201$  inches

$$D_{equiv} = 0.72 D_{max} = (0.72) (201) (1.8 / 174) = 1.50 \text{ inches}$$

2. Determine the effective secant stiffness of the equivalent system as

$$K = F_{max} / D_{equiv} = 2.7 / 1.50 = 1.8 \text{ kips / inch}$$

3. Determine the equivalent period noting the  $W = 3 \times 3$  kips per level = 9 kips (interior rack frame) and  $g = 386$  in/sec<sup>2</sup>

$$T = 2\pi (W/gK)^{1/2} = 6.28 ( (9 / 386) (1.80) )^{1/2} = 0.71 \text{ seconds}$$

4. Determine the MCE spectral coefficient  $S_a$  noting that  $T_s = 0.6$  seconds

$$\text{Since } T \text{ is greater than } T_s, S_a = S_{M1}(1.0 / T) = 1.5 S_{D1} / T = 1.5 (0.6 / 0.71) = 1.26g$$

5. Compute displacement demand,  $D_{demand}$ , of SDOF equivalent system as follows assuming the damping is 15 percent:

$$D_{demand} = \frac{S_a W}{BK} = \frac{(1.26)(9)}{(1.35)(1.80)} = 4.67 \text{ inches}$$

Since displacement demand,  $D_{Demand}$ , is greater than the displacement capacity,  $D_{equiv}$ , determined in Step 1, revise the design and go back to Step 1 and try again.

For this example, the above displacement-based procedure for the cross-aisle direction is shown for illustrative purposes only. As can be seen, it illustrates that the procedure does not demonstrate adequacy by a wide margin. This is the reason that the alternate limit state procedure is recommended for areas of moderate and high seismicity. For an assumed tested pallet coefficient of friction,  $\mu$ , of 0.50, the alternate limit state procedure would simply require a force,  $V_i$ , equal to:

$$V_i = 1.2\mu W_i = (1.2)(0.50)(3) = 1.8 \text{ kips}$$

applied at each level of each cross-aisle frame. The frame would then be evaluated by conventional means and checked against accepted member and connection allowable capacities.

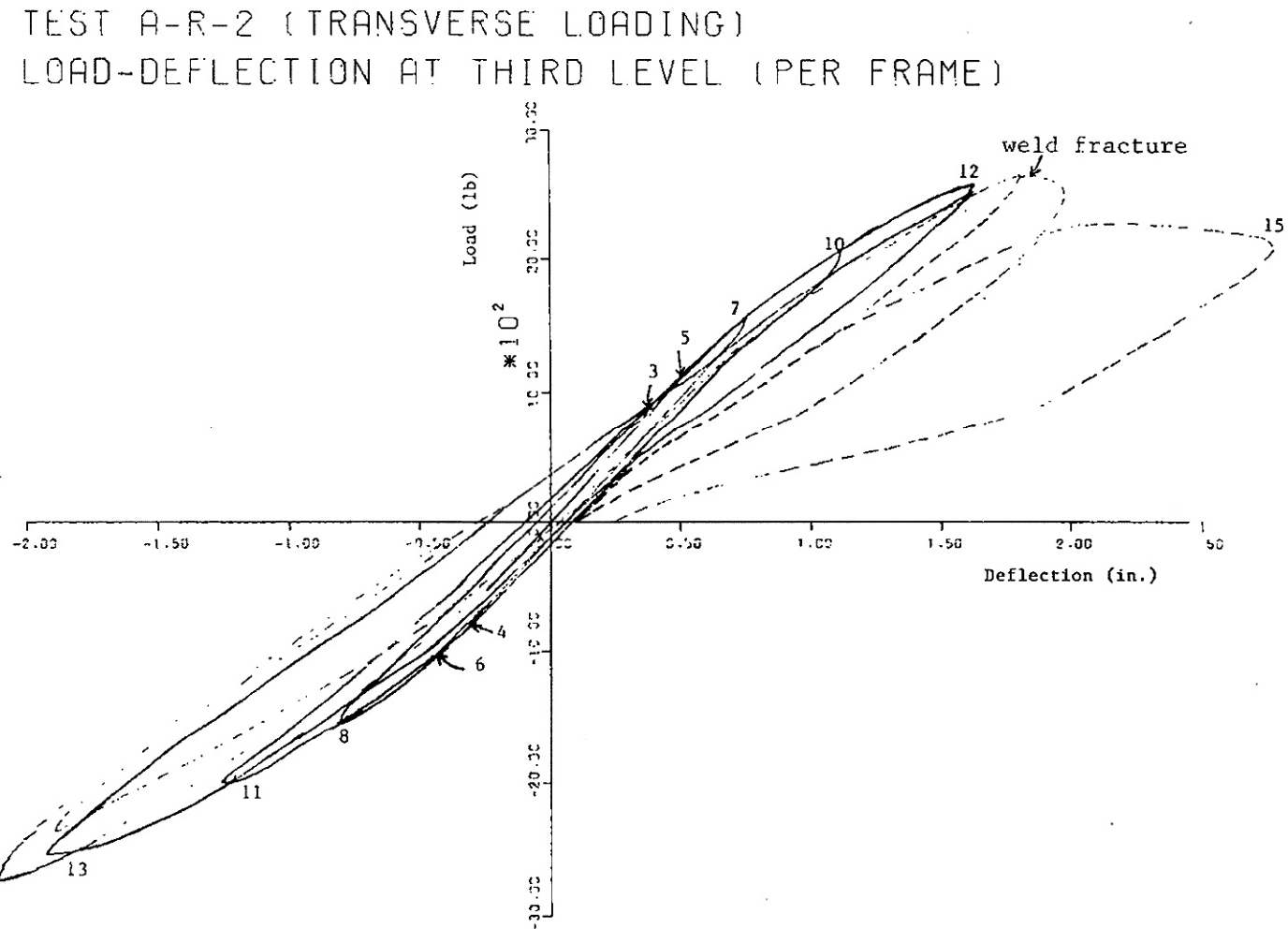


Figure 6-8 Load deflection at third level (per frame) – Test A-R-2, traverse loading (Krawinkler et al, 1979)

## Chapter 7

### GUIDELINES AND RESPONSIBILITIES FOR SPECIFYING, PURCHASING, AND INSTALLING PALLET STORAGE RACKS

#### 7.1 SCOPE

Pallet storage racks are essential infrastructure in the retail-operating environment. Proper design, application, and installation combined with appropriate training and housekeeping are necessary to ensure that rack systems will perform well. This chapter provides guidance on important factors and key responsibilities in the specification, purchase, and installation of steel pallet storage rack products and systems.

#### 7.2 SPECIFYING AND PURCHASING PALLET STORAGE RACKS

Pallet storage racks are one key component of the operating system. As such, it is important that a systems approach be taken when specifying this highly engineered product. Issues to be considered include but are not limited to the following:

- **The location of the rack installation.** The specific location (including latitude and longitude) will help to identify the codes, standards, fire safety requirements, and administrative practices appropriate for the site.
- **Details of the contents that will be stored.** The size, weight, commodity class, and packaging of unit loads as well as discrete items are needed to specify the basic rack structure as well as accessories that contribute to safety, fitness, and containment.
- **Details of the loading/unloading equipment.** The powered and non-powered industrial equipment that will be used to load and unload the storage racks will help define the footprint and aisle spacing necessary for the operation.
- **The operating characteristics of the system.** Knowing activity levels and zonation as well as the loading/unloading schedules and protocols will help with floor planning and selection of system components and accessories.
- **The design documents and normative references that govern the application.** Each storage rack installation will be governed by several design standards and codes documents. These documents include but are not limited to those issued by the American Institute of Steel Construction (AISC), the American Iron and Steel Institute (AISI), the American Society of Civil Engineers (ASCE), the Americans with Disabilities Act (ADA), American Welding Society (AWS), the Federal Emergency Management Agency (FEMA) including the *NEHRP Recommended Provisions*, the International Code Council (ICC) including the *International Building Code*, state or locally adopted codes,

manufacturers' drawings and plans, the National Fire Protection Association (NFPA), and the Rack Manufacturers Institute (RMI).

## 7.3 RESPONSIBILITIES

### DESIGN

- **Registered Design Professional.** Design professionals are responsible for designing to satisfy performance objectives desired by the owner or operator. The responsibility for communicating performance options and, in some cases educating the owner or purchaser with regard to rack performance also falls primarily to the design professional. Design responsibilities typically include the preparation of structural calculations and the preparation or review of rack installation drawings.
- **Rack Dealer or Manufacturer.** Since rack components vary by manufacturer, it is the manufacturer's responsibility to provide the design professional with sufficient information to describe the physical characteristics of the rack. This includes information on material and component properties sufficient to generate an analytical model of the rack system that will capture all relevant characteristics of its response. The rack dealer or manufacturer also may prepare rack configuration drawings.
- **Owner or Purchaser.** It is the owner's or purchaser's responsibility to provide information regarding intended rack usage and all potential loading requirements, to communicate the desired performance objectives to the design professional, and to make available structural drawings for the building, including the flooring subsystems. The owner or purchaser also is responsible for maintaining files of approved rack installation drawings, related permits, and any conditions to the permit.
- **Local Building Department.** The local building department or other regulatory body is responsible for enforcing code requirements. Where required, they also ensure, as a minimum, that the rack design has been reviewed and approved by a Registered Design Professional in the appropriate jurisdiction.

### MANUFACTURING

- **Manufacturer.** The pallet storage rack manufacturer is responsible for establishing and implementing a documented quality control program to ensure reliability of materials and fabricated components. Such a program should consist of documented procedures with records kept on file for review.

### INSTALLATION

- **Installer.** It is recommended that the installer establish and enforce a quality control program that ensures compliance with rack configuration drawings, rack manufacturer's installation requirements, and anchor manufacturer's installation requirements.

- **Owner or Purchaser.** An independent quality assurance program, separate from the installer's quality control program, will increase the reliability of the rack installation and is therefore recommended. This independent program may include hiring the registered professional responsible for the design or other qualified individuals or directly employing personnel with adequate rack installation experience to observe the installed rack system and implementing a testing program for the anchors.
- **Local Building Department.** The local building department or other regulatory body may be responsible for inspecting rack installations to the extent required by local codes to ensure compliance with approved rack drawings.

## 7.4 BUILDING CONSIDERATIONS

The pallet storage rack system will interact with the building structure in several important ways. It is important to understand those interactions and to ensure that proper engineering analysis has been made.

Among the factors to consider are:

- **Connections to the Building.** Most storage racks will be installed as free-standing items independent of the building structure. Any connection to the building requires knowledge of the forces that might be transferred between the building and the rack system to ensure that neither damages the other.
- **Building Floor Capacity.** The building floor and the soil that supports it must have adequate strength to carry the loads applied by the rack system. In some cases, when the lateral loads are high, the floor also may need to be of significant weight to resist uplift. The rack system specifier should provide the floor designer with sufficient information during design so that the floor can be designed to adequately support the rack system.
- **Rack System Anchorage.** Most rack systems are connected to the floor by anchors installed in holes drilled into the hardened concrete. The details of floor construction should be coordinated with the anticipated details for rack system anchorage. Where floors are either post-tensioned or reinforced with bars heavier than welded wire mesh, care should be taken in drilling anchor holes to avoid hitting reinforcement or tendons. Anchors that are near floor slab joints or edges must have adequate edge distance in order to carry required loads. Special inspection should be provided when required.
- **Occupancy Requirements.** The configuration and arrangement of racks also should comply with occupancy requirements related to such things as exiting and fire safety.



## **7.5 INSTALLATION**

Pallet storage racks should be installed by trained and experienced personnel working from installation drawings and instructions provided by the rack supplier. These instructions and drawings, when strictly followed, allow the supplier to warrant the installation for its as-designed condition. Also see Sec. 9.4 for additional information.

## **7.6 RECONFIGURED RACKING**

Most pallet storage racks are designed to permit the beam location to be changed. The capacity of the original installation will have been arrived at based on a specific arrangement of components. Any change from this as-designed condition or allowed deviations can have a significant impact on the safety and fitness of the rack system. The owner or operator should always consult the rack provider or a qualified design professional before undertaking any rack that is reconfigured beyond that allowed on the load application and rack configuration drawings for the rack system. Also see Sec. 9.5 for additional information

## **7.7 INTERMIXING COMPONENTS**

It is not uncommon for a facility to have pallet storage racks produced at different times by a single supplier or by different suppliers. While it may appear that components can be interchanged freely, this should be avoided unless a demonstrated capacity check of the integrated component is performed. This check might be performed by the supplier(s) or another qualified design professional. Even the slightest variation in design or manufacturing tolerance can have a significant and detrimental effect on rack integrity.

## **7.8 USED RACKING**

Previously used pallet storage racks should be installed under the same rigorous standards and guidelines as new storage racks. This would include capacity rating, installation, and satisfying codes and standards requirements.

Special care is called for to ensure that the prior use, disassembly, transport, and reassembly of the rack has not reduced its capacity. In the event that the capacity or the condition is not obvious, a qualified design professional should be consulted.

## Chapter 8

### SUGGESTED IMPROVEMENTS FOR SECURING CONTENTS WITHIN STORAGE RACKS

#### 8.1 SCOPE

Acceptable seismic performance of storage racks and their contents in public access facilities requires adequate restraint of contents. Contents falling from racks due to either operational mishap or earthquake shaking can potentially injure or even kill persons in the aisles. In this chapter, content restraint guidance is offered that is intended to prevent, or at least minimize the possibility of merchandise falling from storage racks during earthquakes.

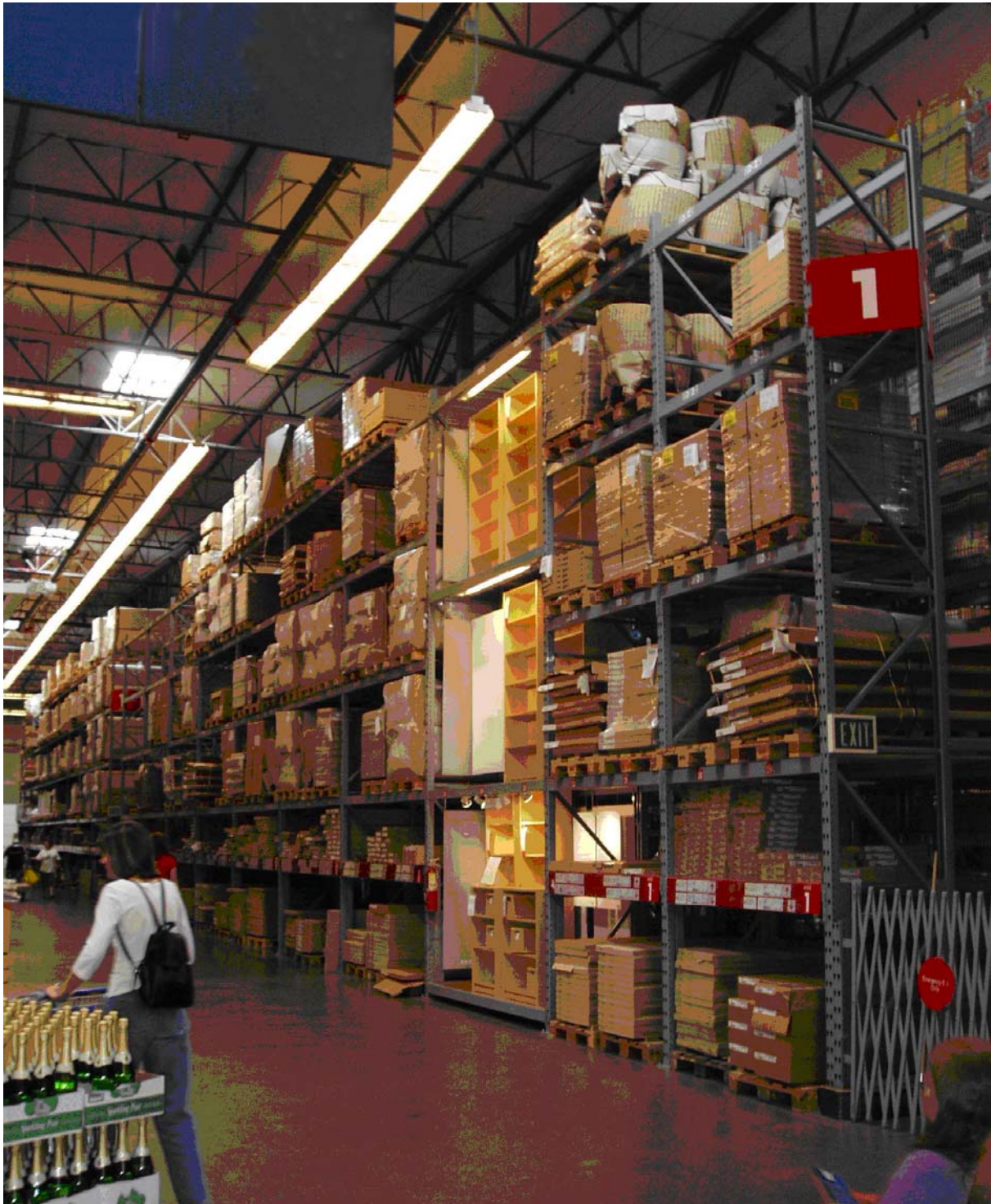
The guidance offered in this chapter is intended to be applicable to warehouse or big-box stores (Figure 8-1) located in the higher seismic regions of the United States. It is recommended that higher seismic regions be defined as locations where the design ground motion parameter  $S_{DS}$  is  $0.75g$  or greater. Under 1997 *UBC* provisions still in use in some states such as California, such values of  $S_{DS}$  correspond to sites in Seismic Zones 3 and 4. Seismic Use Group I structures (it is recommended in Chapter 6 that typically racks retail stores be designated as SUG I) located at site with such values of  $S_{DS}$  are assigned by the 2003 *NEHRP Recommended Provisions* to Seismic Design Category D or E.

The guidance presented here is considered interim in nature, pending the development of an industry-wide standard for restraint of merchandise. There currently are no applicable reference standards and, therefore, the recommendations contained herein are based on observed industry practices, current regulations, limited analytical research and testing data, and observations of the performance of storage rack contents during earthquakes.

#### 8.2 CURRENT RESTRAINT PRACTICES AND REGULATIONS

**8.2.1 Industry Practice with Regard to Securing Contents in Seismic Areas.** Pallets that arrive at warehouse stores loaded with merchandise must be able to withstand the rigors of shipping and handling. Generally, some reliable means of confinement is employed to keep the merchandise (frequently individual boxes and containers) from separating.

Once in the store, it is common practice to keep the pallets intact until broken open to provide individual items for display and sale. Fully loaded pallets also may be broken down, and their contents reshelfed on pallets with smaller loads. Pallet loads typically are secured by such means as banding or stretch-wrapping, a practice generally referred to as “blocking” in the industry. Merchandise blocking enables forklift operators to safely and efficiently move merchandise. Merchandise blocking also reduces the likelihood of individual merchandise falling off the pallets and/or off the racks and into the aisles. However, it also is common practice to break-down loaded pallets into individual components (e.g., boxes or containers) and to place these directly on rack shelves, generally at the lower levels of the rack to allow customer access to the merchandise.



*Figure 8-1 Rack contents on racks should be adequately restrained to prevent their falling into aisles.*

Because of the safety concerns regarding unrestrained individual items, State of California authorities recently enacted regulations intended to protect the public from falling contents in the event of an earthquake (Sec. 8.2.2 below). No specific industry standards or guidelines, other than the California regulations, require rack contents to be secured against earthquake shaking. However, as discussed above, industry practices that currently provide for safe, damage-free shipping and handling also mitigate pallet-related falling hazards.

**8.2.2 2001 California Regulations for Public Warehouse Facilities.** In 2001, the State of California enacted a law intended to protect the public in warehouse stores. This is believed to be the first attempt by any jurisdiction to eliminate or reduce the hazard of storage rack contents falling during an earthquake. This law reads as follows:

California Labor Code, Sections 9102(a) and 9102(b) state the following:

9102. (a) The owner, manager, or operator of a working warehouse shall secure merchandise stored on shelves higher than 12-feet above the sales floor. Methods of securing merchandise shall include rails, fencing, netting, security doors, gates, cables, or the binding of items on a pallet into one unit by shrink-wrapping, metal or plastic banding, or by tying items together with a cord.

9102. (b) All working warehouses shall comply with the provisions of this section on or before July 1, 2002.

This section of the California Labor Code is the direct result of California Senate Bill SB 629 and the California Seismic Safety Commission special hearing on the safety of storage racks and their contents in public areas held on April 12, 2001. A primary difficulty in enforcing these regulations, however, lies in the fact that they do not specify any strength requirements for the materials and methods used to secure merchandise to pallets. For example, how much and how strong should banding or shrink-wrapping be? Further, there is no guidance for the proper application of each method of restraint or merchandise “blocking.” Each of the suggested methods has a specific, useful application, but they are not universally applicable and the most appropriate method to use depends largely on the type of merchandise and the operating logistics behind the merchandise. Improper application of a restraint method may still yield unacceptable performance (e.g., using a chain to restrain stacked small boxes of merchandise).

Proper applications for the most commonly used methods of pallet and merchandise restraint are suggested later in this chapter to provide guidance on the use of the most common industry-accepted practices.

## **8.3 RECOMMENDED CONTENT RESTRAINING PRACTICES**

**8.3.1 Basic Approach.** Storage racks have shelves (or the equivalent) upon which merchandise is stored. Merchandise is typically either placed on pallets, that in turn, are placed on the shelves or merchandise in the form of individual boxes or containers is stacked directly on the shelves.

To prevent or minimize the falling hazard posed by stored overhead merchandise and to achieve the life-safety performance level articulated in this document, a dual approach is recommended:

- Prevent merchandise “fall-through” – Pallets and merchandise should not be permitted to fall downward through one shelf onto the shelf or the ground below.
- Prevent merchandise “toppling” – Pallets and individual merchandise should not be permitted to overturn or slide such that they fall from the shelves into the aisles.

Various techniques that can be used to achieve these two objectives and these are described below.

**8.3.2 Shelf Construction.** Fall-through of merchandise or pallets can be prevented if shelves are constructed in such a way as to keep the merchandise or pallet from falling through should it shift laterally in an earthquake. To accomplish this, uniform vertical support must be maintained at all times. Continuous support should be provided for all pallets placed above the floor in the rack structure. In seismic regions, it is considered poor practice for pallets to span between the front and rear rack beams without any intermediate vertical support (e.g., decking, boards, or plates). Pallets supported only on beams (Figure 8-2) are likely to shift, lose vertical support on one end, and fall between the beams during strong earthquake shaking. Common ways to prevent this fall-through include the use of the following types of shelf construction:

- Wire decking;
- Spaced wood boards;
- Spaced metal channels, angles or plates; or
- Perforated metal decking (fire codes generally prohibit use of solid decking).

Generally, wire decking of the waterfall type (decking hangs over the front and rear faces of the beams as shown in Figure 1-3) or decking firmly secured between beams in molded shelves are the recommended means for preventing fall-through. It is usually preferred over the use of spaced wood boards or metal plates to provide continuous support for pallets. Spaced boards may become dislodged during ground shaking and fall between the beams, compromising vertical support for the merchandise above. When spaced boards or plates are used, they should be positively secured to prevent their shifting or falling in an earthquake.

As a general practice, it is recommended that shelves of any level be constructed to prevent “fall-through.”

**8.3.3 Securing Merchandise Stored on Pallets.** For merchandise stored on pallets, toppling is prevented if the pallet is kept from overturning or sliding off the shelf and if the merchandise is secured to the pallet. At the present time, there is no publicly available evidence to prove that wood pallets placed atop one of the recommended shelf materials listed in (Sec. 8.3.2 above) will not slide off the shelf. However, based on earthquake performance to date, it generally is believed that wood pallets placed atop the common shelf construction will not slide sufficiently in an earthquake to fall off.



*Figure 8-2 Pallets in this home furnishings store are supported only on the beams and may slide off these supports during an earthquake.*

In addition to ensuring that pallets do not slide off or overturn on rack shelves, it is important that items of merchandise be secured to the pallets themselves. Common means to secure individual merchandise to pallets include but are not limited to the following:

- Stretch-wrapping (Sec. 8.4.1),
- Shrink-wrapping (Sec. 8.4.2),
- Banding (Sec. 8.4.3), and
- Integral box-pallet (Sec. 8.4.4).

**8.3.4 Securing Merchandise Not Stored on Pallets.** Some merchandise sold in public warehouse stores is not stored on pallets. Individual items or merchandise not stored on pallets but placed on shelves are at greater risk of toppling in an earthquake. The following are some of the most common means used to secure items of merchandise not stored on pallets:

- Restraining bars (Sec. 8.5.1),
- Restraining chains or cables (Sec. 8.5.2),
- Netting (Sec. 8.5.3), and
- Slip and overturning-resistant (SOR) individual containers or boxes (Sec. 8.5.4).

All merchandise should be evaluated based on weight and size to determine the most appropriate securing method. To be effective, the merchandise must be sufficiently large to engage the restraint. Netting can effectively restrain small boxes and containers, but a restraining bar, cable or chain may not. Recent shaking table tests (See Chapter 11) have shown that merchandise with high aspect ratios (the ratio of height to width) such as water heaters (Sec. 8.5.5) are particularly vulnerable to overturning if restraints are not provided.

**8.3.5 End Bay Uprights.** It is recommended that the ends of a longitudinal row of racks have upright frames or frame extenders that extend high enough above the topmost shelf to provide sliding and overturning restraint for the palletized or individually stored merchandise on the upper-most level. This is to prevent the merchandise from toppling into the main aisle-ways generally located at the end of a row of storage racks. If one end of the row abuts a wall, this frame need not have the frame extension.

**8.3.6 Maximum Height to Width Ratios for Uniformly Loaded Pallets.** To reduce the possibility of loaded pallets overturning, it is suggested that the height to least width ratio generally be limited to 2.0 or less for pallets placed higher than 8 feet above the floor when  $S_{DS} \geq 1.1g$  and generally be limited to 2.5 or less when  $S_{DS} \geq 0.75g$  but less than 1.1 g. (The pallet load is assumed to have a uniform weight distribution).

These aspect ratio recommendations are derived from some recent theoretical studies by Shao and Tung (1999), who studied overturning and sliding of rigid bodies subjected to 75 real earthquake time-history records. Using the data summarized in Table 1 of their work, it can be shown that rigid bodies, restrained against sliding and with a height-to-width ratio ( $H/D$ ) of 2 to 1, have about a 16 percent chance of overturning when subject to motions with peak shelf accelerations around 0.70g. If one assumes the amplification of motion from the floor to top shelf in the cross-aisle direction is on the order of 1.5 to 2.0, then earthquake motions with a peak ground acceleration (PGA) in the range of 0.35g to 0.50g may result in a 16 percent chance

of overturning. These PGA values correspond to an  $S_{DS}$  of approximately 1.1g. Similarly, when  $H/D$  is 2.5 or greater, motions with PGA in the 0.3g to 0.4g range may create a 16 percent overturning hazard. Motions in this range, using the amplifications assumed above, correspond to an  $S_{DS}$  of approximately 0.75g.

It should be noted that when a pallet slides, its tendency to overturn may be reduced, provided that it does not slide sufficiently to topple off the shelf.

**8.3.7 Plastic and Metal Pallets.** Recently, some merchandise vendors have begun to utilize alternative pallet materials such as plastic and metal instead of the traditional wood. Loaded pallets on pallet-shelf combinations with low frictional resistance are more prone to slide than those with greater frictional resistance. Currently, it is assumed that wood pallets have sufficient sliding frictional resistance to remain on shelves during earthquakes. Limited shake-table testing has shown this assumption to be true, but due to the limited scope of available test data, this is considered to be an interim assumption and needs to be confirmed by further research.

With the increasing use of plastic and metal pallets, combined with the lack of observation or testing of their sliding performance, there is a concern that these pallets could be more prone to slide than wood pallets and further research is recommended.

The reader is cautioned that pallet-shelf combinations with coefficients of friction of less than 0.20 may have a tendency to slide sufficiently under strong ground shaking to cause toppling. This caution is derived from the recent Shao and Tung (1999) theoretical work. For example, with an  $S_{DS}$  of 1.1g (PGA = 0.44g), the motions at the top of the racks in the cross-aisle direction, assuming a 1.5 to 2.0 amplification, would be in the range of 0.66g to 0.88g. Figure 6 of Shao and Tung (1999) indicates that motions in the range of 0.70g to 0.75g have about a 16 percent chance of resulting in relative sliding of 20 inches when the coefficient of friction is 0.20. This would be sufficient to topple a standard 40-inch pallet.

**8.3.8 Limiting Height for Unrestrained Merchandise.** Unrestrained merchandise on steel pallet racks can present a significant safety hazard to the public when high enough above the floor. The consensus recommendation of the Rack Project Task Group is that all merchandise above 8 feet on steel pallet storage racks should be restrained -- either on pallets as described in Sec. 8.4 or loose on the rack shelf as described in Sec. 8.3.4. The Task Group recognized that this height may present a problem as some retailers store loose merchandise on shelves up to 8 feet to facilitate restocking during the business day, which is preferable to conducting fork lift operations while the public is present. However, even in these situations, loose merchandise can present a hazard if it is heavy enough to cause injury.

## **8.4 SUGGESTED PRACTICES FOR RESTRAINING PALLET LOADS**

Suggested alternatives to meet pallet restraint criteria are described below.

**8.4.1 Stretch-Wrapping.** Stretch-wrapping involves wrapping the merchandise to the pallet with several layers of stretchable plastic film. To be effective, the stretch-wrap must not only bind the merchandise together but also overlap with and engage the pallet so as to form an integral, blocked unit. Stretch wrap should be pretensioned to industry accepted standards to provide adequate blocking of merchandise.



It is recommended that stretch-wrapped pallets be capable of passing the pallet tilt test described in Sec. 8.6.1.

**8.4.2 Shrink-Wrapping.** Shrink-wrapping involves wrapping the merchandise to the pallet with heat-sensitive plastic film that when heated, shrinks and tightens, thereby securing the merchandise to the pallet. Similar to stretch-wrapped merchandise, it should connect the merchandise together and overlap and engage the pallet. It is recommended that shrink-wrapped pallets be capable of passing the pallet tilt test described in Sec. 8.6.1.

**8.4.3 Banding.** Individual merchandise as well as merchandise in boxes or containers can be secured to pallets by banding. This involves securing the merchandise to the pallet with tight metal or plastic “bands” that wrap around the merchandise and connect it to the pallet, generally by passing through the pallet. To be effective, an appropriate number of individual bands should be used to ensure that all containers or boxes are effectively bound together. It is recommended that banded pallets be capable of passing the pallet tilt test described in of Sec. 8.6.1.

**8.4.4 Integral Box-Pallets.** Some merchandise is shipped in wood or heavy cardboard boxes constructed to be integral with the pallet. These integral pallet-box units can be quite effective in preventing toppling of merchandise.

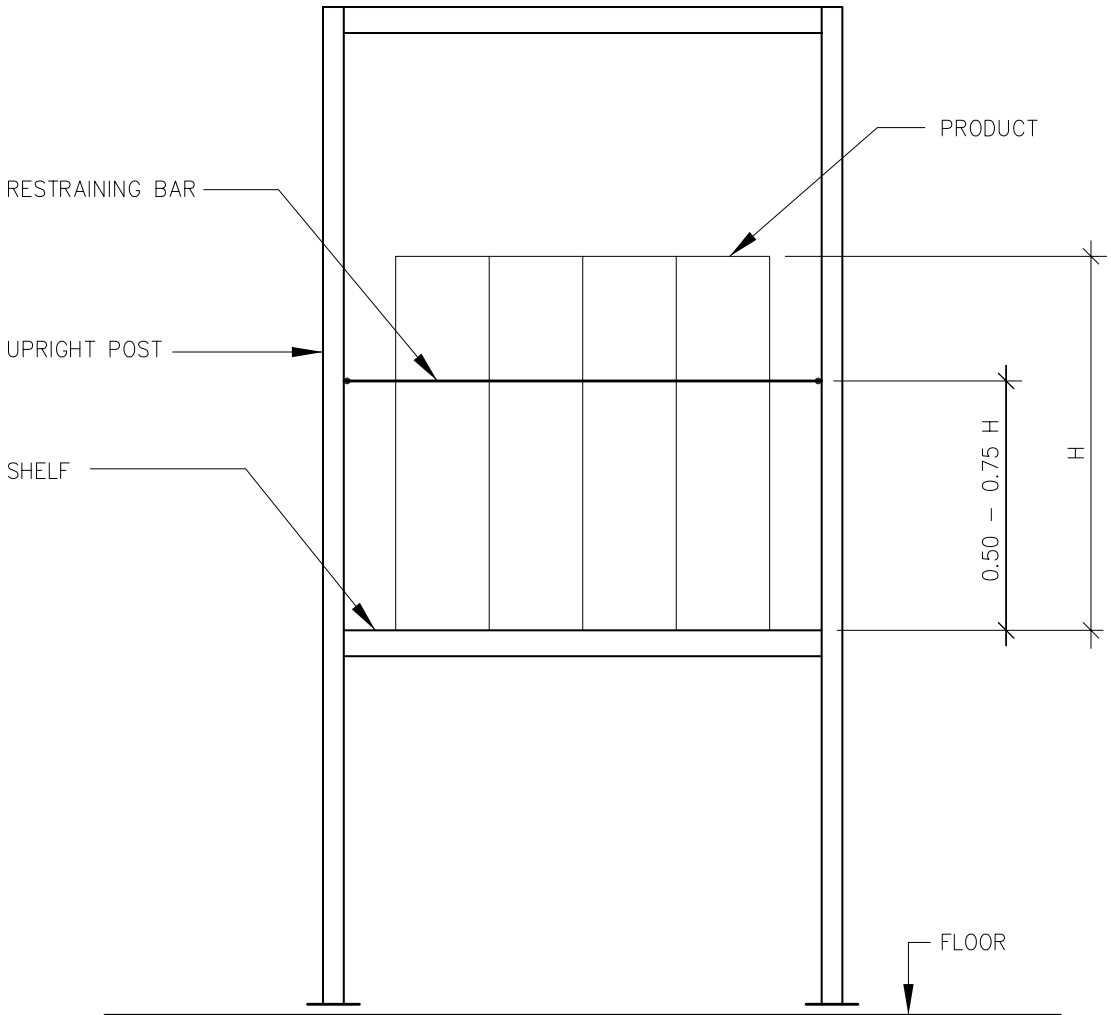
## **8.5 SUGGESTED RESTRAINT PRACTICES FOR NON-PALLETIZED MERCHANDISE**

The reader is cautioned that the following recommendations are based on the judgment of the Rack Project Task Group and are not the result of extensive research. However, unless restraints have sufficient strength relative to the stored merchandise, they may be ineffective.

**8.5.1 Restraining Bars.** Horizontal restraining bars can be installed on the front and rear of storage rack shelves to keep merchandise from falling off. Restraining bars can be effective for large boxes (e.g., water heaters) and other merchandises not placed on pallets. For back-to-back racks, restraining bars generally are required only at the front of shelves directly adjacent to the aisles.

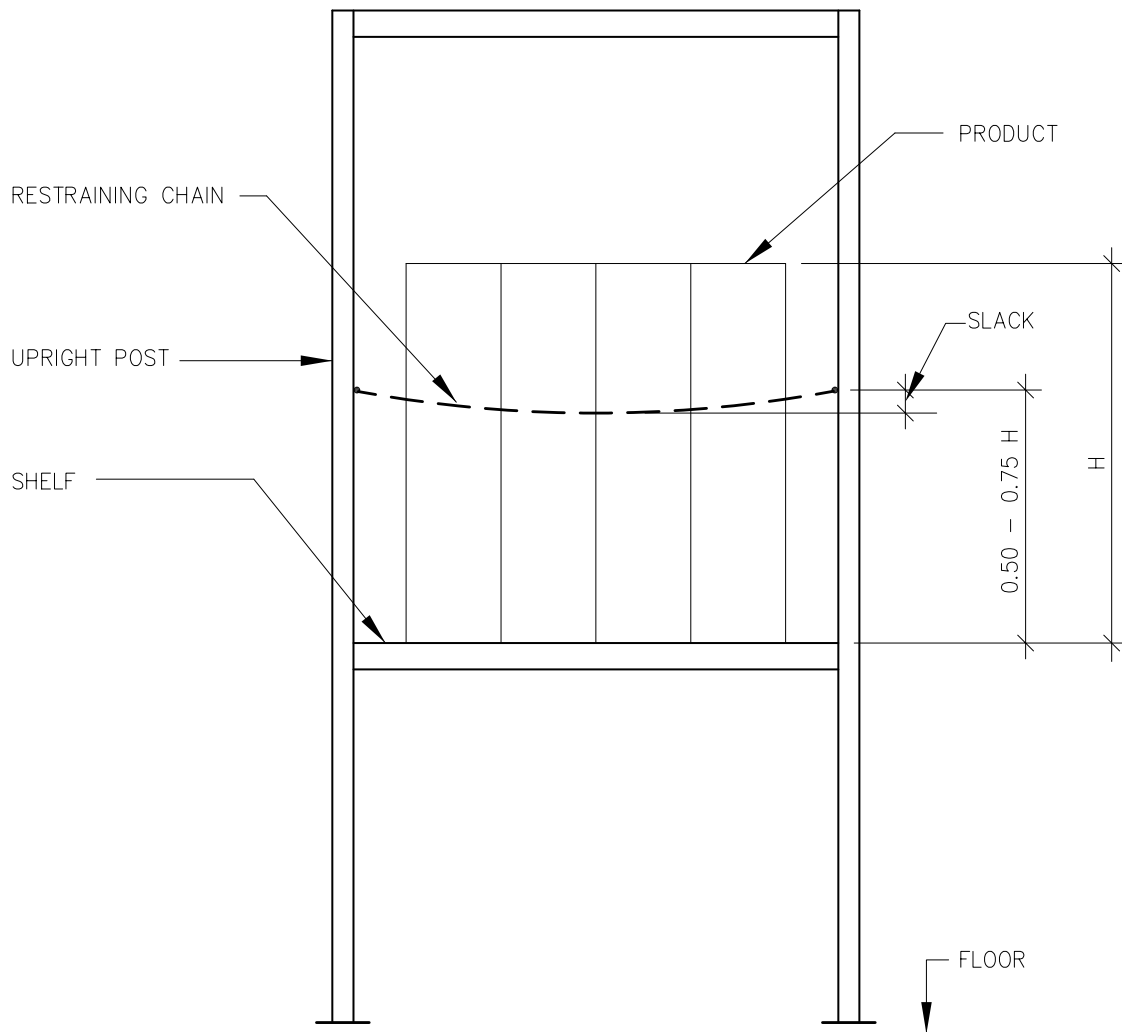
Bars should be placed as shown in Figure 8-3. At the present time, there are no industry-wide guidelines for design of restraints. As an interim measure for high seismic regions ( $S_{DS} \geq 0.75g$ ), it is recommended that these bars be designed to safely resist a uniform lateral force equal to 20 percent of the total weight of the merchandise to be restrained but not less than 50 pounds per foot of bar.

**8.5.2 Restraining Chains and Cables.** Restraining chains and/or cables can be installed on the front and rear of storage rack shelves to prevent merchandise such as large boxes or containers from toppling. Restraining chains are effective for merchandise in boxes or containers where load separation is not an issue. For back-to-back racks, restraining chains generally are required only at the front of shelves.



RESTRAINING BAR

Figure 8-3 A restraining bar is used to restrain tall, top-heavy merchandise that cannot be placed efficiently on pallets.



### RESTRAINING CHAIN

*Figure 8-4 Chain or cable can be used to restrain tall, top-heavy merchandise. When used, the chain or cable needs to be sufficiently taut to be effective. Slack in chains or cables should be no more than about 1/2-inch per foot of length.*

Chains or cables should be placed as shown in Figure 8-4. At the present time, there are no industry-wide guidelines for design of chain restraints. As an interim measure for high seismic regions ( $S_{Ds} \geq 0.75g$ ), it is recommended that chains or cables be designed to safely resist a uniform lateral force equal to 20 percent of the total weight of the merchandise to be restrained but not less than 50 pounds per foot of chain or cable.

**8.5.3 Netting.** Nylon or other types of netting can be installed across the exposed front of a storage rack bay to contain loosely stacked materials (e.g., small boxes, drums, cans, and containers). Figure 8-5 illustrates an example. Local fire regulations may require that nylon and other plastics be treated with an approved fire retardant. Typically, moveable netting is installed at the front of racks and, when necessary, fixed netting is installed at the rear.



Figure 8-5 Merchandise restrained by netting.

Netting may be connected to cables by carabineers or similar devices. At the present time, there are no industry-wide guidelines for design of netting restraints. As an interim, measure for high seismic regions ( $S_{DS} \geq 0.75g$ ), it is recommended that cables and their attachment posts be designed to safely resist the forces imposed on them but not less than 50 pounds per linear foot of net.

Merchandise also may be secured with moveable heavy wire mesh “doors” placed across the front of shelves. In single row racks, the backs of shelves should be similarly secured with fixed wire mesh or netting unless the rack abuts a wall.

**8.5.4 Slip and Overturning Resistant (SOR) Individual Containers or Boxes.** Some merchandise individually stored on shelves has a low aspect ratio and sufficient frictional resistance to be both slip and overturning-resistant (SOR). Such merchandise generally is not considered to be a falling hazard and, therefore, need not be secured to pallets or restrained by nets or other devices. In high seismic regions ( $S_{DS} \geq 0.75g$ ), the following interim guidelines may be used to establish under what circumstances the use of pallets and/or restraining devices is not required:

- The base of the container or box has a least dimension of at least 18 inches.
- The aspect ratio (height/width) is 2.0 or less for  $S_{DS} \geq 1.1g$ , and 2.5 or less for  $S_{DS} \geq 0.75g$  but less than 1.1g.
- The coefficient of friction between the merchandise and the deck is 0.3 or greater.
- The cartons or boxes in which merchandise is stored are sufficiently rugged as to not burst open due to earthquake shaking.

If all of the above guidelines are met, the individual SOR containers or boxes are considered to meet the life-safety performance objectives of these guidelines. Those items of merchandise that do not meet the guidelines should be secured by the methods discussed above or other industry accepted means.

**8.5.5 High Aspect Ratio Merchandise.** It is recommended that water heaters and other merchandise with height-to-width ratios greater than 2.5 stored at any level be positively restrained to prevent overturning. Recent shake-table tests (Filiatrault 2001) of loaded racks demonstrated the toppling vulnerability of unrestrained water heaters.

## 8.6 TESTS

**8.6.1 Pallet Tilt Test.** The pallet tilt test involves “proof” testing the binding (i.e., blocking) method used to secure the merchandise to the pallet. The purpose of the test is to ensure that the means used to secure the merchandise to the pallet are sufficiently strong to keep the merchandise from spilling during earthquake shaking.

The test is intended to establish the general adequacy of a particular binding method using a representative sample pallet load. It is not meant to imply that every blocked pallet in a store be individually tested. Rather it is intended to be a generic test used to establish an acceptable level of internal quality control that, once successfully performed, establishes the adequacy of the binding method for a particular type of merchandise and pallet combination.

Care must be taken in doing the tests so as to not cause the loaded pallet to overturn and damage the merchandise or injure personnel.

The basic test procedure is as follows:

Step 1 — The merchandise is bound to the pallet with an approved securing method (see Sec. 8.4)

Step 2 — The pallet is lifted on one side to a height that produces an angle of 20 degrees between the ground and the bottom surface of the pallet (see Figure 8-6).

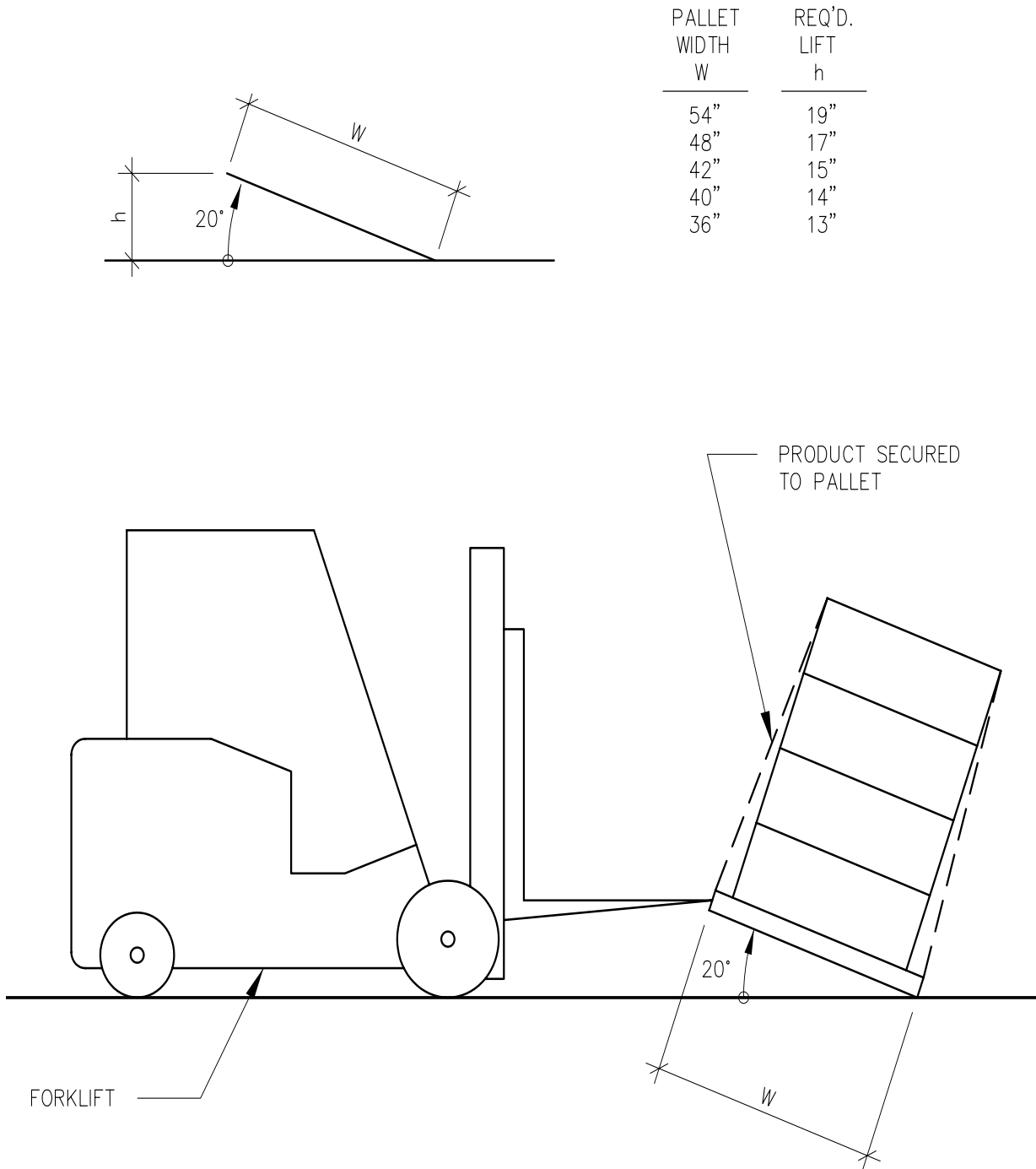
Step 3 — If the merchandise remains restrained in place for at least 5 minutes without appreciable movement, the load secured to the pallet is considered to have adequate confinement and passes the tilt test.

Step 4 — If the merchandise shifts appreciably or the securing material breaks, the merchandise must be resecured using an industry-approved method and retested.

## **8.7 ALTERNATE MEANS OF SECURING CONTENTS**

The guidance provided above for securing contents (i.e., individual merchandise and loaded pallets) to steel storage racks in high seismic areas draws heavily upon present practices that are known or believed to be effective. It is not the intent of these guidelines to prevent or discourage the use of means other than these described herein to prevent, or minimize, content falling hazards in earthquakes. If a scheme, system, or means not discussed herein can be shown to be effective in meeting the life-safety goals of Chapter 5, its use should be permitted.

The U.S. and the world-wide materials handling industry has produced, and no doubt will continue to produce, many storage innovations. Techniques for storage, and for preventing contents from falling, will continue to evolve, and innovative methods and practices are encouraged to ultimately produce safer content storage practices.



*Figure 8-6 The pallet tilt test is used to verify the strength of the means (e.g., stretch wrapping, banding) used to secure merchandise to pallets.*

## 8.8 FUTURE NEEDS

**8.8.1 Pallet Friction Testing.** To establish a more scientific basis for determining whether pallets (wood, plastic, or metal) will slide off shelves in earthquakes, testing needs to be performed to identify acceptable values for the static and dynamic coefficients of friction between different combinations of materials in surface contacts (e.g., wood to metal or plastic to metal)

**8.8.2 Confirmation of Pallet Tilt Test.** The adequacy and application of the pallet tilt test described in Sec. 8.6.1 should be verified. Further refinement of this test will help validate the effectiveness of the various securing techniques (e.g., stretch-wrapping, shrink-wrapping, and banding) in achieving the desired level of seismic performance set forth in this document.

**8.8.3 Confirmation of SOR Criteria for Containers and Boxes.** The adequacy of the slip and overturning resistant criteria described in Sec. 8.5.4 should be verified, and the scope of this concept should be expanded to include one or more levels of stacked boxes.

**8.8.4 Development of an Industry Standard for Content Restraint.** Due to the lack of available information on the earthquake performance of merchandise stored on storage racks, it is recommended that an industry-approved standard be developed under the leadership of RMI. This chapter is intended to serve as a basis for such a standard.





## Chapter 9

### SUGGESTIONS FOR STORAGE RACK INSTALLATION, OPERATION, INSPECTION, AND MAINTENANCE

#### 9.1 SCOPE

This chapter provides recommended guidance concerning the installation, operation, inspection, and maintenance of steel storage racks in facilities accessible to the public. The primary focus is the safe performance of storage racks and their contents during and immediately after earthquakes through proper installation, operation, inspection, and maintenance.

Even in high seismic areas, nonseismic requirements and safety considerations may govern the installation and operation of storage rack systems. The parties responsible for earthquake safety should be aware of and sensitive to critical nonseismic requirements, including those that may be outside the scope of the building code.

It is possible that more than one party may be responsible for specifying, purchasing, installing, operating, and maintaining a storage rack system. When more than one party is responsible, additional effort may be necessary to ensure coordination, enforcement, and documentation of storage rack system requirements.

#### 9.2 RESPONSIBILITIES OF INSTALLERS AND OPERATORS

**9.2.1 Qualification and Training of Personnel.** The storage rack system installer is responsible for the qualifications and training of personnel performing the following functions:

- Storage rack system installation, configuration, and anchorage;
- Storage rack system reconfiguration; and
- Storage rack system additions and alterations.

The storage rack system operator is responsible for the qualifications and training of personnel performing the following functions:

- Handling of materials stored on or near the racks,
- Securing of materials stored on or near the racks,
- Operation of vehicles and equipment in the vicinity of the racks,
- Inspection of racks, and
- Maintenance and repair of racks.

**9.2.2 Documentation.** It is recommended that the storage rack system installer have access to the following documentation:

- Storage rack system load application and rack configuration drawings;
- Building floor original design criteria;
- Building floor reinforcing steel details, including post-tensioning, if applicable; and
- Storage rack system requirements for anchorage and floor capacity.

This documentation is invaluable for evaluating future additions, alterations, and reconfigurations of the storage rack system. Such documentation may be maintained by another responsible person.

It is recommended that the storage rack system operator maintain or have access to the following documentation:

- Storage rack system load application and rack configuration drawings,
- Storage rack system requirements for anchorage and floor capacity,
- Requirements for storage rack system inspection and maintenance,
- Allowable limits for structural deficiencies that need not be corrected,
- Applicable prequalified procedures for repair of storage rack system damage, and
- Requirements for securing storage rack system contents.

This documentation is invaluable for establishing and verifying compliance with requirements and recommended practices.

As part of an overall risk management program, it may be desirable for the storage rack system operator to maintain the following documentation in some manner:

- Training requirements and records for persons responsible for storage rack system operation, inspection, maintenance, and repair;
- Records of storage rack system inspections and maintenance; and
- Records of storage rack system damage and repairs.

This documentation may be valuable in identifying problems and for improving operations.

## **9.3 INITIAL INSTALLATION**

The storage rack system installer is responsible for the proper installation of the storage rack system. The storage rack system installation should conform to approved construction documents and the rack manufacturer's recommendations.

**9.3.1 Storage Rack System Configuration.** The configuration and arrangement of racks should comply with the applicable load application and rack configuration drawings for the system. See Sec. 1.4.4 and 1.4.5 of RMI Standard -- ANSI MH 16.1-04 *Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks*.

**9.3.2 Building Floor Capacity.** The building floor and the soil that supports it must have adequate strength to carry the loads applied by the storage rack system. See Chapter 6 for guidance.

**9.3.3 Storage Rack System Anchorage.** Most storage rack systems are connected to the floor by anchors installed in holes drilled into the hardened concrete. See Chapter 6 for guidance on anchorage.

**9.3.4 Occupancy Related Requirements.** The configuration and arrangement of racks also should comply with nonstructural requirements including those concerning exiting and fire safety.

**9.3.5 Plaques.** All storage rack system installations should display in one or more conspicuous locations a permanent plaque that legibly identifies the allowable loads and load configurations. See Sec. 1.4.2 of the RMI Standard -- MH 16.1-04 *Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks*.

**9.3.6 Permits.** When required by the authority having jurisdiction, a permit should be obtained prior to the installation of the storage rack system.

## **9.4 ADDITIONS, ALTERATIONS, AND RECONFIGURATIONS**

Additions, alterations, and reconfigurations should meet the same requirements as the original installation. Additions, alterations, and reconfigurations that are outside the limits of the original load application and rack configuration drawings should be designed or approved by a registered design professional knowledgeable about storage rack systems.

In many jurisdictions, the installation of storage racks requires a permit which entails the submittal of construction drawings and structural calculations, typically stamped and signed by a registered professional engineer. At the time of application, the racks should have been designed for specific rated loads applicable to specific structural configurations. Each of these conditions should be identified by appropriate load application and rack configuration drawings. For each configuration, the rack should be designed to support its rated load without exceeding its capacity.

After the initial installation, it may be necessary to reconfigure some of the racks to store merchandise differently than originally envisioned. One of the significant safety concerns with storage racks, particularly in warehouse retail stores, is that store personnel will reconfigure the racks without the benefit of oversight by the original designer or another similarly qualified professional. Although some changes to a rack configuration can be trivial, such as moving beams up or down a few inches, other changes can be quite significant. For example, if beams are removed from the lower levels, the seismic moments on the columns, beams, and connections can increase substantially and the seismic safety of the storage rack system can be seriously compromised. For this reason, additions, alterations, and reconfigurations that are outside the limits of the original load application and rack configuration drawings need to be designed or approved by a registered design professional.

**9.4.1 Documentation.** Revised load application and rack configuration drawings should be provided for additions, alterations, and reconfigurations that are outside the limits of the original load application and rack configuration drawings. See Sec. 9.4.2.

**9.4.2 Plaques.** Revised storage rack system plaques should be provided for storage rack system loading conditions that exceed the limits of the original storage rack system plaques. See Sec. 9.4.6.

**9.4.3 Permits.** When required by the authority having jurisdiction, a permit should be obtained for storage rack system additions, alterations, and reconfigurations that are outside the limits of the original load application and rack configuration drawings. The permitting requirements of the authority having jurisdiction for such situations should be clearly understood by those responsible for ordering the addition, alteration, or reconfiguration.

## **9.5 STORAGE RACK SYSTEM OPERATION**

The storage rack system operator is responsible for the safe operation of the storage rack system. For additional guidance, see the RMI Standard -- ANSI/MH 16.2 *Standard for the Use of Industrial Steel Storage Racks*.

**9.5.1 Material Storage.** All material stored on, under, or adjacent to a storage rack system should be dealt with in a safe manner. Requirements concerning exiting and fire safety should be met. Blockage of exits and other means of egress may be a fire code violation and also may indicate that the storage rack system capacity is inadequate.

**9.5.2 Securing of Contents.** Items that can slide or overturn during normal operations or during an earthquake should be secured in manner that prevents sliding and overturning. See Chapter 8. The methods and procedures for securing contents should be documented. Persons responsible for storing the contents should be trained to properly secure the contents in accordance with the documentation.

**9.5.3 System Protection from Damage.** It is recommended that wherever a storage rack system is particularly susceptible to damage by vehicles or other equipment, collision protective devices, such as bollards or guards, should be provided. The protective devices should be appropriate for the vehicles and other equipment operated near the racks. See Commentary Sec. 1.4.9 of the RMI standard -- ANSI MH 16.1-04 - for additional guidance regarding resistance to minor impacts.

**9.5.4 System Protection from Vandalism.** Wherever a storage rack system is susceptible to damage by theft of essential components, defensive measures (e.g. such as installation of tamper-resistant fasteners or increased frequency of inspection) should be implemented.

**9.5.5 Reporting of Observed Deficiencies.** Storage rack system operators who observe deficiencies outside of scheduled inspections should document the problem and report it to the designated person responsible for initiating the correction of deficiencies. The intent of this recommendation is to encourage storage rack system operators to be aware of the condition of the rack and to initiate actions to correct obvious deficiencies in a timely manner.

## 9.6 INSPECTION AFTER INITIAL INSTALLATION

The storage rack system operator should establish and implement a program of regularly scheduled storage rack system inspections. The inspections should be performed by a qualified person retained or employed by the storage rack system operator. The inspections should follow a written inspection plan, and each inspection should be documented.

**9.6.1 Inspection Schedule.** The time between scheduled rack inspections may vary, depending on the conditions of use. All storage rack systems should be inspected at least annually and after each damaging earthquake or other damaging event. The inspection schedule should be documented.

### 9.6.2 Scope of Inspection

The inspector should inspect the entire storage rack system for conformance to approved load application and rack configuration drawings and the conditions of any applicable permits.

The inspector should inspect the entire storage rack system for the following structural deficiencies:

- Missing, loose or damaged anchor bolts;
- Missing or damaged connection locking devices;
- Missing or damaged rack guards;
- Missing or damaged rack accessories;
- Damaged, dented, buckled, or bent rack framing members;
- Weld fractures that are detectable by visual inspection;
- Corrosion and other deterioration that may adversely affect structural capacity;
- Rack frame misalignment;
- Rack beam spreading or excessive deflection;
- Improper beam installation; and
- Floor damage that may adversely affect the support or anchorage of the storage rack system.

The inspector should examine the entire storage rack system for the following operational deficiencies to the extent that they are observable at the time of inspection:

- Storage rack system overloading and other improper material storage,
- Use of pallets that are inappropriate for the storage rack system,
- Damaged pallets,
- Rack contents that are improperly or inadequately restrained,
- Missing or inadequate storage rack system plaques, and
- Obstructed aisles, inadequate lighting, or poor housekeeping.

Storage rack system deficiencies should be documented and reported to a designated person responsible for initiating the correction of deficiencies. The storage rack system operator is responsible for initiating the correction of deficiencies.

Configuration deficiencies should be corrected in a way that conforms to Sec. 9.3.2 or 9.4.

Structural deficiencies should be corrected in accordance with Sec. 9.7.

Operational deficiencies should be corrected by the storage rack system operator. Additional training of personnel should be provided as necessary or appropriate.

## **9.7 STORAGE RACK SYSTEM MAINTENANCE AND REPAIR**

The storage rack system operator is responsible for the maintenance and repair of the storage rack system. For additional guidance, see the RMI standard -- ANSI/MH 16.2 *Standard for the Use of Industrial Steel Storage Racks*.

**9.7.1 Allowable Structural Deficiencies.** Allowable limits for structural deficiencies that need not be corrected should be as specified by the storage rack manufacturer or as determined and documented by a registered professional engineer familiar with storage rack systems. Structural deficiencies that exceed allowable limits should be corrected.

**9.7.2 Repair and Replacement of Damaged Components.** Damaged components that can be readily replaced should be removed from service and replaced with undamaged components. Damaged components that cannot be replaced should be repaired by qualified persons following qualified and documented procedures. Repaired components should be at least as strong as undamaged components. In general, the details of repairs should be designed by the rack manufacturer or by a registered professional engineer familiar with storage rack systems.

## **9.8 INSPECTION AFTER EARTHQUAKES**

Storage rack systems should be inspected after each earthquake that causes damage to structures within the general vicinity of the storage rack system location. Specific guidance on post-earthquake inspections is provided in Chapter 12. The first and primary purpose of post-earthquake inspections is to protect lives and property. However, they also may be valuable in documenting storage rack system performance so that demonstrated good performance and potential areas for improvement can be identified.

**9.8.1 Storage Rack System Repair.** Damaged storage rack system components should be inspected and repaired or replaced in accordance Sections 9.6 and 9.7.

**9.8.2 Documentation of Storage Rack System Performance.** Earthquake damage and repairs should be documented in accordance with Sec. 9.2.3. Whenever possible, the storage rack system operator is encouraged to share the storage rack system performance data with third parties who have a legitimate interest in improving the seismic performance of storage rack systems. The intent of this recommendation is to facilitate the collection of earthquake performance data in a manner similar to that used by the Earthquake Engineering Research Institute (EERI). RMI may be the appropriate

organization to collect this data. Pre-earthquake agreements with nondisclosure provisions may be necessary to allow timely data collection without revealing sensitive or confidential information.





## Chapter 10

### SUGGESTED CONSIDERATIONS FOR QUALITY ASSURANCE PROGRAMS

#### 10.1 SCOPE

This chapter provides guidance for quality assurance programs for use during the design, manufacture, installation, and operation of steel storage racks in areas accessible to the public. The intent is to create a set of reference procedures and documents that will ensure the life safety needs are met for steel storage racks located in such areas. See Chapter 9 for additional requirements that should be integrated with a quality assurance program.

#### 10.2 MINIMUM COMPONENTS OF A QUALITY ASSURANCE PROGRAM

It is recommended that each entity involved with the design, manufacture, installation, or operation of steel storage racks have a documented quality assurance program. The minimum requirements for these programs includes:

- Written quality control procedures with detailed activities. These documents should establish the acceptable criteria as related to a specific feature or aspect of the storage rack systems for which the responsible party has control or obligation.
- A written list of quality assurance procedures (e.g. testing and visual inspection).
- A specific schedule for implementing quality assurance procedures.
- A list of responsible parties, preferably by job title, of the internal and external (if applicable) personnel responsible for execution of the quality assurance program.
- A method of recording and verifying compliance with the quality assurance program for each production run or installation.
- A detailed inspection and audit plan to ensure compliance with the quality assurance program.

#### 10.3 QUALITY ASSURANCE FOR RACK SYSTEM DESIGN

Each storage rack system should be designed by or through a registered design professional. Drawings, specifications, and structural calculations for the rack system should be provided to the owner of the racking system and, where appropriate, to the authority having jurisdiction (AHJ) in accordance with applicable building code and jurisdiction requirements.

#### **10.4 QUALITY ASSURANCE FOR STORAGE RACK SYSTEM MANUFACTURE**

Quality assurance for rack system manufacture is the responsibility of the rack system manufacturer. The rack system specifier or purchaser should select a manufacturer that has a documented and verifiable quality assurance program that ensures manufacture in accordance with the rack system design requirements and RMI standards.

#### **10.5 QUALITY ASSURANCE FOR STORAGE RACK SYSTEM INSTALLATION**

The owner or purchaser should implement a quality assurance program for rack system installation. Where required, building permits from the AHJ should be obtained prior to rack system installation. The quality assurance program should be created by the rack system owner or purchaser in conjunction with the manufacturer and the registered design professional. At a minimum, the quality assurance documentation should contain the following items:

- Installation and erection instructions from the provider.
- Additional installation and anchorage instructions from the registered design professional.
- Requirements of special inspection required by the AHJ as a condition of permitting.
- Written documentation stating that the racking is installed in accordance with all requirements. This document and a final inspection should be prepared by a qualified person employed by the owner.
- Limits on rack reconfiguration and loading beyond which the involvement from the registered design professional is required.

#### **10.6 QUALITY ASSURANCE FOR STORAGE RACK SYSTEM OPERATION**

The storage rack system operator should implement a quality assurance program for rack system operation and maintenance. The quality assurance program and its quality control components and procedures, including documentation requirements, should be created through a collaborative effort of the rack system manufacturer, owner or operator, and registered design professional.

## Chapter 11

### REVIEW OF PAST SEISMIC RESEARCH ON STORAGE RACK SYSTEMS AND SUBASSEMBLIES

#### 11.1 SCOPE

Experimental and analytical studies of the seismic performance of storage racks has been scarce and the results often are proprietary; and consequently, they have not significantly influenced the development of codes and regulations related to storage rack systems. In this chapter, available results from experimental and analytical investigations on the seismic response of storage racks are briefly reviewed. Gaps in knowledge requiring further research studies also are identified.

#### 11.2 REVIEW OF EXPERIMENTAL RESEARCH

Experimental research related to the seismic behavior of storage racks can be categorized into different types of testing procedures:

- *Cantilever* testing of subassemblies in which quasi-static cyclic loads are applied to beam-to-upright connections.
- *Portal* testing of subassemblies in which beam and uprights portal subassemblies are loaded laterally to simulate seismic loading.
- Quasi-static testing of storage rack systems for which completely loaded storage racks are loaded laterally to simulate seismic loading.
- In-situ dynamic testing of storage rack systems with small shakers or under ambient vibrations in order to obtain their dynamic characteristics (e.g., natural periods and damping).
- Shake-table testing of storage rack systems for which completely loaded storage racks are excited by recorded or artificially generated ground motions.
- Testing of cold-formed steel members and structures from which most storage racks are built.
- Testing of merchandise.

Table 11-1 lists the experimental investigations documented in the public literature that have used the various testing techniques. The major conclusions obtained from these experimental investigations are reviewed briefly below.

**Table 11-1 Investigations of Storage Rack Systems and Subassemblies**

| Year | Authors                    | Testing Types (Number of Specimens)  |
|------|----------------------------|--|
| 1973 | John A. Blume & Associates | In-situ dynamic tests (19)   |
| 1979 | Krawinkler et al.          | Cantilever tests (20), portal tests (6), quasi-static tests of storage rack systems (4), dynamic tests (2) |
| 1980 | Chen et al.                | Shake-table tests (4), merchandise tests (2)   |
| 2001 | Bernuzzi and Castiglioni   | Cantilever tests (22)  |
| 2001 | Filiatrault                | Shake-table tests with real merchandise (5)  |
| 2003 | Castiglioni et al.         | Shake-table tests (4)  |
| 2004 | Higgins                    | Cantilever tests (22)  |
| 2004 | Filiatrault                | Shake-table tests (4)  |

**11.2.1 Cantilever Testing of Storage Rack Subassemblies.** The lateral stiffness of storage rack systems in the down-aisle (moment-resisting frame) direction is greatly influenced by the distortions that occur at the beam-to-upright (post) connections. For analytical modeling purposes, these distortions often are represented by simple rotational spring elements inserted between the beam ends and the upright center line. The rotational spring constant to be used in a numerical model can be obtained from moment-rotation relationships between a beam end and an upright using the so-called cantilever test method (RMI 2002).

Krawinkler et al. (1979) performed cantilever tests on 20 different beam-to-upright subassemblies of standard pallet racks. In all connections, the beam ends were welded to angle connectors that, in turn, permitted connection to perforations on the upright through either hooks (Type A) or button grips (Type B). The experimental results indicated that because of local deformations at the beam-to-upright connections, moment-rotation hysteretic loops have a pinched shape similar to that obtained for reinforced concrete elements with high shear. Low cycle fatigue phenomena also may affect the strength and ductility of beam-to-upright and upright-to-floor (base plate) connections. The strength of the Type A subassemblies was limited by the capacity of the hook-type grips that started to pull out of the upright perforations. In the Type B subassemblies, fracture of the fillet weld between the beam and the connection angle limited the moment capacity.

More recently, Bernuzzi and Castiglioni (2001) performed a series of 11 monotonic and 11 cyclic tests on two different types of beam-to-upright connections used in Europe. The experimental results obtained from the monotonic tests indicated that the connections were characterized by ductile behavior. None of the test specimens failed before the maximum achievable rotation by the testing equipment was reached. This maximum rotation was way beyond practical design values. The results of the cyclic tests exhibited pronounced pinching behavior associated with slippage and plastic deformations of the connectors leading to significant reduction energy dissipation capacity with increasing number of response cycles.

Quasi-static testing was recently conducted on 22 different types of interior beam-to-upright subassemblies (Higgins 2004). The test data indicate that beam-to-upright connections exhibit

very ductile and stable behavior with rotational capacities beyond the values observed during shake-table tests and expected from a design seismic event. The hysteretic responses of some of the tested beam-to-upright connections, however, exhibited significant pinching similar to those tested by Bernuzzi and Castiglioni (2001).

Sarawit and Peköz (2003) recently proposed a new beam-to-upright connection test to replace the cantilever test method. The actual frames bending moment-to-shear force ratio is better represented in this proposed test method than the current cantilever test.

**11.2.2 Portal Testing of Storage Rack Subassemblies.** The moment-rotation characteristics obtained from the cantilever test method described in the previous section is highly dependent on the bending moment-to-shear force ratio applied to the beam-to-upright connection. In fact, this ratio varies continuously during the application of seismic lateral loading to a storage rack in its down-aisle direction and loaded with merchandise. In order to better represent the bending moment-to-shear force ratio in a beam-to-upright connection, the portal test method has been proposed. In this testing procedure, a portal assembly of generally one beam connected to two uprights is solicited by static gravity loads on the beam and by lateral loads on one of the upright at the elevation of the beam. Moment-rotation at both beam-to-upright connections can be monitored during the tests.

Krawinkler et al. (1979) performed six portal tests on three different beam-to-upright subassemblies of standard pallet racks. Four types of beam-to-upright connections were investigated. In all connections, the beam ends were welded to angle connectors which in turn permitted connection to perforations on the upright through either hooks (Type A) or button grips (Types B and C). In Type D connection, additional connectors were used to join the connector angles to the uprights. It was found that the global hysteretic response of the subassemblies was relatively insensitive to the gravity loads induced by the merchandise. When the moment-rotation hysteretic loops of the beam-to-upright connections were compared to that of the cantilever tests described in the previous section, it was observed that the loops from the portal tests exhibited a significantly higher initial stiffness. This result confirmed that moment-rotation characteristics of beam-to-upright connections depends on the bending moment-to-shear force ratio that is significantly higher in the portal tests due to the presence of the vertical merchandise loads.

**11.2.3 Quasi-Static Cyclic Testing of Complete Storage Rack Systems.** Quasi-static cyclic testing of complete storage racks represents an efficient experimental procedure to study the interaction between beams, uprights and connections under gravity (merchandise) loads and simulated seismic lateral loads. The beams of the racks are loaded by either concrete blocks on pallets or real merchandise and hydraulic actuators apply lateral loads to the uprights at the various beam levels. With this testing procedure, racks can be tested separately in their down-aisle or cross-aisle directions.

Krawinkler et al. (1979) performed four quasi-static tests of complete three-story storage racks. Two tests were performed in the down-aisle direction and two others were performed in the cross-aisle direction. The first two rack specimens contained hook beam-to-upright connectors, while the two others incorporated button grip connectors. The lateral load was applied only at

the top level of the racks. The global hysteretic response of the racks along the down-aisle direction was characterized by a nonlinear pinched hysteretic shape similar to that obtained in reinforced concrete flexural members with high shear. It was observed that for constant lateral displacement amplitudes the second load cycle led to a significant decrease in energy dissipation capacity while the third cycle was practically identical to the second one. Failure in the down-aisle direction was typically initiated by weld cracking between the beam ends and the connector angles. It was noted that this weld cracking in the cyclically loaded racks occurred at smaller lateral displacements than in the monotonically loaded racks. When weld cracking was averted, the ductility of storage racks in their down-aisle direction depended strongly on the axial load ratios in the uprights. For small axial load ratios, a very ductile behavior characterized by flexural plastic hinging in the uprights was achieved. It also was found that second order (P- $\Delta$ ) effects greatly affected the lateral strength and stiffness of storage racks in the down-aisle direction. Finally, it was found that the ductility and energy dissipation capacity of storage racks is much larger in the down-aisle moment-resisting direction than in the cross-aisle braced frame direction.

**11.2.4 Dynamic In-Situ Testing of Storage Rack Systems.** The first published in-situ dynamic investigation of storage racks was performed in the mid 1970s at various distribution centers in the San Francisco Bay Area (John A. Blume & Associates 1973). Ambient and man-made vibration measurements were applied to representative steel industrial storage racks of the standard pallet, drive-in and drive-through, cantilever, and stacker crane types for the purpose of obtaining range of natural periods and damping ratios. The ambient vibration and man-made excitation measurements generated average response accelerations at the top of the racks on the order of 0.005g and 0.015g, respectively. The experimental results indicated that the fundamental translational period of storage racks could not be predicted well by the empirical formula used to characterize building periods in the 1973 edition of the *Uniform Building Code*. Measured fundamental periods over a range of actual merchandise loading conditions averaged 0.6 sec in the down-aisle direction, and 0.2 sec in the cross-aisle direction. Torsional periods averaging 0.4 sec were identified in many of the rack configurations. It was noted that these period values would increase at least by 20 percent under response amplitudes representative of a large earthquake. Measured structural damping ratios for storage racks averaged 2 to 3 percent of critical at root-mean-square response acceleration levels of 0.01-0.02g. It was noted that these damping values would increase at least by a factor of 2 under response amplitude representative of a large earthquake for which significant energy dissipation would occur due to rocking, slippage and interaction of stored merchandise. Thus, it was concluded that a damping ratio of 5 percent of critical would be a reasonable value for storage racks under seismic excitations.

Krawinkler et al. (1979) subjected two full pallet rack assemblies loaded with gravity loads to forced and free vibration tests to obtain information on natural frequencies, mode shapes and damping characteristics in the down-aisle and cross-aisle direction. Measured fundamental periods averaged 0.7 sec in the down-aisle direction, and 0.5 sec in the cross-aisle direction. The vibration decay obtained from the free-vibration tests in the down-aisle direction exhibited a textbook example of Coulomb-type friction decay. At large amplitudes, the friction between the grip-type connectors and the perforations in the uprights caused significant damping (on the order of 2.5 to 3.5 percent of critical). Once the connectors locked up at smaller amplitudes, the damping dropped drastically to a very small value (on the order of 0.7 percent of critical). The

damping characteristics in the cross-aisle direction was more constant with vibration amplitudes (on the order of 1 to 2 percent of critical).

**11.2.5 Shake-Table Testing of Storage Rack Systems.** Shake-table testing for which complete storage rack systems are loaded with real merchandise represents the most direct experimental procedure to assess their seismic behavior. Unfortunately, this type of testing is expensive compared to other testing procedures and only a very limited number of shake-table studies on storage racks have been performed to date.

The first published shake-table studies on storage racks in the United States, was performed in the late seventies on the 20-ft-square shake-table at the University of California, Berkeley (Chen et al. 1980a, 1980b, 1981). Four types of full-scale industrial steel storage racks were subjected to scaled ground motions actually recorded during two California earthquakes (1940 El Centro and 1966 Parkfield earthquakes). The ground motions were scaled so that the resulting base shear coefficients about equaled the design base shear coefficients found in the 1979 edition of the *Uniform Building Code* of ordinary moment frames (with  $K=1.0$ ) for buildings in the down-aisle directions and ordinary brace frame for buildings (with  $K=1.33$ ) in the cross-aisle direction. The types of storage racks tested were: single standard pallet racks, back-to-back pallet racks, drive-in racks, and stacker racks. In general, the storage racks performed well during the tests with the exception of the drive-in stacker racks in the cross-aisle direction, for which considerable buckling was observed in the first story diagonal members. The fundamental periods of vibration ranged from 2-3 sec for the standard pallet and drive-in racks in the down-aisle direction and 0.5-1.0 sec for the standard pallet, drive-in, and stacker racks in the cross-aisle direction. The first mode damping values were much larger in the down-aisle direction (on the order of 3 to 9 percent of critical) than in the cross-aisle direction (0.5 to 3 percent of critical). It was also observed that the ductility and energy dissipation capacity of the racks were much larger in the down-aisle, moment resisting frame direction than in the cross-aisle, braced frame direction. Therefore, the racks could undergo significant inelastic deformations without suffering major damage in the down-aisle direction, but could only develop limited amount of inelastic deformations in the cross-aisle direction. Second order (P-delta) effects contributed significantly to the response of the racks in the down-aisle direction.

More recently (Filiatrault 2001), five different back-to-back pallet racks loaded with real merchandise were tested on a uniaxial shake-table under a single component, scaled at various amplitudes, of the ground motion recorded at Canoga Park during the 1994 Northridge earthquake in California. Three of the tests were conducted in the cross-aisle direction, while the two other tests were conducted in the down-aisle direction. In general, the racks performed well. Significantly more flexibility, ductility, and energy dissipation capacity were observed in the down-aisle direction than in the cross-aisle direction. The fundamental periods of vibration averaged 1.4 sec in the down-aisle direction and 0.6 in the cross-aisle direction. No structural damage occurred in any of the rack configurations for peak ground motion amplitudes less than 0.42g.

Castiglioni et al. (2003) performed shake-table tests of four full-scale steel storage pallet racks loaded by concrete blocks mounted on pallets simulating content merchandise. The four specimens were chosen among six structures designed by two different European manufacturers



based on the Eurocode 8. The experimental results indicated that sliding of pallets occurred for ground motion intensities less than the design levels. Also, the diagonal bracing configuration in the down-aisle and cross-aisle directions has a significant influence on the seismic response of steel storage pallet racks. In particular eccentric bracing configurations can lead to significant torsional response. The authors stressed the importance of a regular configuration of bracing systems.

A shake-table study was recently conducted at the University at Buffalo on four different pallet rack configurations incorporating bolted beam-to-upright connections (Filiatrault et al. 2004). All racks were tested in the down-aisle direction. The main objectives of the tests were to:

- Determine the variations of in-plane dynamic characteristics of the industrial storage racks during service life.
- Deduce average beam-to-upright rotational stiffness from measured in-plane fundamental period of racks.
- Evaluate the response of storage racks under strong ground motions.

The test results indicated that the rotational stiffness of beam-to-upright connections is the main factor influencing the down-aisle seismic response of pallet racks. The initial values obtained for the rotational stiffness of beam-to-upright connections are two to three times higher than values used in current design. These values reduce with the amplitude of vibration but generally return to their initial values once the pallets rack return to its undeformed position. Furthermore, very ductile seismic behavior was observed in the down-aisle direction with peak interstory drifts exceeding 7 percent without any sign of incipient collapse.

**11.2.6 Experimental Research on Cold-Formed Steel Members.** The lateral load-resisting systems of storage racks often include cold-formed steel bracing members. Therefore, research information related to the behavior of cold-formed steel structural members and systems have influenced the design of storage racks (RMI 2002). The reader is referred to more specialized references regarding the general behavior of cold-formed steel members and systems under seismic loading (Rhodes 1991, AISI 1996). In this section, only investigations available in the public domain on the seismic behavior of cold-formed steel members typically used in storage racks are briefly reviewed.

Cheng (1973) performed axial load tests on cold-formed steel open sections used as primarily load carrying structural members in storage racks. It was observed that local torsional flexural buckling is the primary mode of failure for axially loaded perforated open section segments. An analytical expression was proposed to predict the axial load carrying capacity of these members.

Kotha and Peköz (2000) studied the behavior of cold-formed pallet storage racks with semi-rigid beam-to-upright connections and with flexible upright bases through finite element analyses. A general moment-rotation relationship was established to model the beam-to-upright connection stiffness of pallet storage racks. Also, the upright base flexibility caused by base plate bending

was quantified. Guidelines were provided to carry out nonlinear finite element analysis of storage racks accounting for these influencing parameters.

**11.2.7 Testing of Merchandise.** As part of the shake-table testing conducted by Chen et al. (1980a), the seismic response of a two-story, one bay wide, one bay deep rack was obtained for two different cases: (a) the case in which the merchandise was tied with metal straps to the rack beams, and (b) the case in which the merchandise was not fastened to the rack. For the purpose of this investigation, eight types of merchandise with different geometry and weight were considered. The tests were conducted with scaled ground motions that resulted in base shear coefficients that were about the same as code level base shear coefficients for buildings. The results of the tests indicated that neither merchandise nor wood pallets fell off the racks during the tests. Only some uppermost paper products and canned goods moved slightly during the tests in the down-aisle direction. It was found, however, that the seismic response of the racks without the merchandise tied up to the beams was smaller compared to that of the same rack with the contents rigidly attached to the beams. This response reduction was particularly obvious during the strongest portion of the response. Low amplitude free vibration tests also indicated that the damping was slightly higher for the case where the merchandise was not tied up to the rack.

The shake-table tests conducted recently by Filiatrault (2001) also included real merchandise. It was clearly demonstrated that the use of plastic wrap was a very efficient mean of restraining the merchandise from falling off the racks. No fallen merchandise was observed for ground motion amplitude less than 0.30g. Significant slippage of heavy merchandise on pallets was observed but no pallet overturned during any of the tests. However, tall merchandise items (e.g., water heaters) were shown to be vulnerable to overturning and need of restraint.

### 11.3 REVIEW OF ANALYTICAL AND NUMERICAL RESEARCH

As evidenced by the experimental research reviewed previously, the seismic response of storage racks in the down-aisle direction is strongly affected by the nonlinear moment-rotation response of the beam-to-upright connections. In the cross-aisle direction, on the other hand, the seismic response of storage racks depends on the characteristics of the bracing members used in the truss configuration. Therefore, numerical models that have been used to predict the seismic response of storage racks incorporate these different lateral load-resisting systems to various degrees. The analytical and numerical research related to the seismic behavior of storage racks can be divided into two different types of models:

- *Linear models* for which the moment-rotation response of beam-to-upright connections is linearized by simple linear rotational springs representing secant properties at the anticipated response level of the storage racks. For dynamic analysis, an equivalent linear viscous damping model is also used to represent the energy dissipation of these same connections during inelastic actions.
- *Nonlinear models* for which the nonlinear response of beam-to-upright connections is followed over the time-history response of storage racks by the use of nonlinear moment

rotation hysteretic rules. This nonlinear modeling is used mainly for research purposes and rarely in design situation.

**11.3.1 Linear Modeling of Storage Rack Systems.** John A. Blume & Associates (1973) developed and analyzed equivalent lumped mass numerical models representing selected storage racks in order to compare their predicted fundamental periods to in-situ measured values. Pinned upright bases were assumed for all rack configurations except for the cross-aisle direction of cantilever racks. Rigid beam-to-upright connections were assumed in the down-aisle direction. Reasonable agreement was achieved between measured and computed storage fundamental periods.

Chen et al. (1980a) conducted frequency analyses of linear mathematical models in order to compare calculated periods of vibration and mode shapes with those observed during low-amplitude shaking table tests and pull-release free-vibration tests that they conducted, as described in Sec. 6.2.5. These calculated periods and mode shapes then were used to perform response spectrum analyses. The calculated fundamental periods of vibration were also used to determine the base shear coefficients according to the 1973 edition of the *Uniform Building Code* and to the ATC-3 procedure (ATC 1978). It was found that two-dimensional models with minimum net section properties and centerline dimensions were adequate for practical purposes. Modeling parameters such as semi-rigid beam-to-upright and base connections should be considered in theoretical predictions of rack response. It was also found that in the down-aisle direction, the lateral forces determined by the 1973 edition of the *Uniform Building Code* were roughly equivalent to those obtained from response spectrum analyses with intensity levels slightly more than 50 percent of the 1940 El Centro and 1966 Parkfield earthquake records. In the cross-aisle direction, however, the code lateral forces were approximately equivalent to 25 percent to 50 percent of the El Centro and Parkfield records. In the cross-aisle direction the lateral forces predicted by the *UBC* were higher than those predicted by the ATC-3 (ATC 1978) procedure. Opposite results were obtained in the down-aisle direction.

John A. Blume & Associates (1987) performed static and response spectrum analyses to investigate the applicability of the eccentric braced frame concept (Roeder and Popov 1978) to storage racks in order to improve their seismic behavior in the cross-aisle direction. The results of the study indicated that aside from a considerable savings in steel material, the eccentric bracing system could undergo significantly more inelastic deformations without structural instability than conventional bracing systems. Although the analytical results were promising, the authors recommended also that experimental investigations needed to be conducted before implementing the eccentric bracing system in storage racks. Such experimental results are not available to date.

**11.3.2 Nonlinear Modeling of Storage Rack systems.** Chen et al. (1980b) developed also two-dimensional nonlinear numerical models for standard pallet racks in both down-aisle and cross-aisle directions and compared their predictions to the results obtained from shake-table tests, as described in Sec. 6.2.5. In the down-aisle direction, semi-rigid beam-to-upright connections and semi-fixed upright bases were assumed in evaluating their initial stiffness. Bilinear moment-rotation hysteretic rules were considered for the semi-rigid beam-to-upright connections. Second order (P-delta) effects also were considered in the time-history dynamic analyses. Good

agreement was observed between the predicted and experimental results. In the cross-aisle direction, the stiffness of the connections between the beams and the uprights was adjusted in the model to simulate the observed local deformations. Again, the responses predicted by the model were in good agreement with the experimental results.

Baldissino and Bernuzzi (2000) conducted a numerical study on the lateral-load response of steel storage pallet rack systems commonly used in Europe. The results confirmed that the nonlinear rotational behavior of beam-to-upright connections influenced significantly the response of storage rack systems in the down-aisle direction. Also, the numerical investigation confirmed the significant influence of the base plate connections on the overall rack response in both directions. The authors pointed out the need for test data on the nonlinear moment-rotation behavior of base upright connections.

## **11.4 RESEARCH NEEDS**

As presented in this brief review, the current engineering knowledge base concerning the earthquake safety and vulnerability of storage racks is 20 to 30 years old and is limited to contents and racks unlike many modern applications. The retail industry and the state-of-the-art of the design of storage racks have changed considerably in the interim. Large chains of stores now routinely invite the public to shop in physical environments that formerly were only found in warehouse occupancies, racks have more complex configurations and are taller, and their contents have become heavier. These facts clearly pinpoint to urgent research needs related to the seismic behavior of storage rack systems. In this section, experimental and analytical research that is perceived to be the most urgently needed is briefly listed.

**11.4.1 Experimental Research Needs.** Only two full-scale shake-table testing investigations of storage racks fully loaded with contents have been performed in the United States (Chen et al. 1980a, 1980b, 1981; Filiatrault 2001) and only three reported worldwide. There is an urgent need to increase the experimental database on the seismic response of complete storage rack systems through shake-table testing. The main variables that need to be investigated in such experimental programs are:

- The layout and types of storage racks representing current construction practices and innovative systems such as eccentric bracing.
- The layout and types of merchandise contents.
- The types of seismic restraints (e.g., plastic wraps, screens, ledges, etc.) for contents.
- The structural interaction between neighboring racks.
- The direction of the horizontal seismic input, relative to the rack's orientation (transverse, longitudinal, or non-orthogonal).

- The characteristics of the input ground motions, including consideration of whether vertical accelerations must be characterized and near-field motions, and relating these input motions to seismic hazard mapping and codes in use in the United States.

As demonstrated by available experimental and analytical results, the seismic response of storage racks in their down-aisle direction is strongly affected by the nonlinear response of the beam-to-upright and base plate connections. Since numerous variables enter in the design of these connections, an experimental parametric study on the cyclic response of beam-to-upright and base plate connections is urgently needed.

While the needs of the down-aisle direction are urgent, testing needs in the cross-aisle are even more urgent since the understanding of this directions behavior is even less understood. Failures of racks in earthquakes are most commonly reported as cross-aisle failures.

The information on the seismic response of merchandise contents installed in storage racks is very limited. There is an urgent need to conduct shake-table studies of merchandise. For this purpose, shake-table testing could be used to simulate the motions experienced by various levels of storage racks during earthquakes. A robust numerical model would be required to develop these input motions. Various merchandise items could be mounted on the shake-table via a rigid assembly representative of the level on which they are mounted. Various types of merchandise contents would be investigated experimentally under a large number of input motions representative of several seismic hazard levels. With this procedure, experimental fragility curves could be constructed for various types of merchandise content. Furthermore, these fragility curves could be compared with the ones generated when various types of seismic restraints are introduced. With this information, clear recommendations could be provided on the types of seismic restraint to be used for a particular type of merchandise content.

**11.4.2 Analytical Research Needs.** There is a need to develop a general purpose computer-based numerical model for the prediction of the seismic response of storage racks and contents. The development of such a general-purpose model would require close coordination and interaction with experimental work.

## Chapter 12

### INSPECTION OF STORAGE RACKS AND RECOVERY OF MERCHANDISE AFTER EARTHQUAKES

#### 12.1 SCOPE

Storage racks located in seismic regions of the United States will periodically be subjected to earthquake ground shaking. Earthquakes causing ground motions at design levels or greater will test the limits of rack structural systems and may even cause the collapse of some racks and the toppling of contents. This chapter provides guidance for the post-earthquake safety evaluation of storage racks and their contents. The purpose of such inspections is to identify unsafe conditions and prevent entry into hazardous areas. This chapter also provides guidance for the recovery of merchandise from damaged racks.

The post-earthquake inspection guidelines given here are intended to be used by building inspectors and engineers working for the local building department and making ATC-20 safety evaluations of warehouse stores after a damaging earthquake. ATC-20, sponsored in part by FEMA, is the de facto standard used for U.S. post-earthquake inspections. ATC-20, *Procedures for Post-Earthquake Safety Evaluation of Buildings*, provides procedures and guidelines for making on-the-spot evaluations and decisions regarding continued use and occupancy of earthquake damaged buildings. Written specifically for volunteer structural engineers and building inspectors, the report includes rapid and detailed evaluation procedures for inspecting buildings and posting them as INSPECTED (apparently safe, green placard), LIMITED ENTRY (yellow placard) or UNSAFE (red placard).

Also included are special procedures for evaluation of essential buildings (e.g., hospitals), evaluation procedures for nonstructural elements and geotechnical hazard, and guidance on human behavior following earthquakes. Rapid Evaluations are performed very quickly after an earthquake, are often cursory in nature, and are designed to identify dangerous conditions immediately. Detailed Evaluations usually begin a day or two after the earthquake when sufficient members of structural engineers and specialists become available.

The merchandise recovery guidelines also provided in this chapter are intended to be used by qualified store personnel to evaluate the condition of and recover merchandise from damaged and collapsed racks.

#### 12.2 POST-EARTHQUAKE HAZARDS

Earthquakes are typically followed by aftershocks. These are smaller but still potentially dangerous earthquakes that follow the main shock. Aftershocks can happen minutes, hours, days or even weeks after the main event. They can cause additional damage to damaged racks, and may cause shifted or damaged contents to topple. Damaged racks can become further damaged or even collapse, and unstable piles of merchandise in and near collapsed racks can shift dangerously.

During an earthquake, pallets and merchandise on decks as well as the restraints used to secure them, may become damaged. This can create a falling hazard even when the racks are otherwise undamaged.

## **12.3 POST-EARTHQUAKE INSPECTION OF STORAGE RACKS**

**12.3.1 Scope of the Inspections.** Immediately following a damaging earthquake, local jurisdictions will conduct safety evaluations of buildings in the damaged areas. This is normally done according to the ATC-20 procedures. Inspections are typically made by building inspectors and engineers from the local building department, often supplemented by volunteer structural engineers or staff obtained from other jurisdictions through mutual aid pacts. After an inspection and safety evaluation, buildings are posted with placards reading INSPECTED, RESTRICTED USE or UNSAFE depending on the kind and severity of damage experienced. This is done to let the owner, occupants and public know that the building has been inspected, and what its safety status is. Unsafe areas are typically barricaded. Barricading is typically done with yellow caution tape and, in the more hazardous situations, with wood barricades or chain link fencing.

ATC-20 inspections of warehouse stores will entail examination of both the building and the storage racks for both structural (e.g., wall-roof separations) and nonstructural hazards (e.g. falling hazards associated with damaged light fixtures or merchandise that has shifted on racks). Because there is generally a shortage of structural engineers following earthquakes, many inspections are made by building inspectors working for the local building department, often supplemented by volunteer structural engineers. In general, the inspections are likely to be made by personnel not very experienced in the design of the structure (or racks) that they are inspecting.

The procedures given below are intended to be used solely for the inspection of storage racks (ATC-20 provides procedures for buildings) and are intended to be used by those individuals doing an ATC-20 rapid evaluation of a warehouse store. (These procedures also can be used by qualified store employees to identify and deal with unsafe storage racks prior to the time an ATC-20 inspection is done.)

Because of the difficulty of first finding and then judging the significance of structural damage to rack systems, particularly beam-column connections, a structural engineer experienced in rack design may be needed to evaluate damaged racks, or racks that have been subjected to high levels of ground shaking. This latter inspection would correspond to an ATC-20 detailed evaluation, which is the second and higher level of inspection according to the ATC-20 procedures.

**12.3.2 Step-by-Step ATC-20 Rapid Evaluation Procedure for Racks.** The following steps should be followed when conducting an ATC-20 rapid evaluation of steel storage racks.

Step 1 -- Keep the public away:

- After a strong earthquake, the store manager should be requested to close the store to the public, until the building and racks can be examined and their safety assessed.

- All but essential store personnel should be kept out of rack areas (and other potentially unsafe areas).

Step 2 -- Get an overview of the situation:

- Walk around outside of the store. Make sure the building is safe before attempting entry.
- Survey the entire inside of the warehouse to get an overview of the situation. Do this before examining individual racks in detail.
- Be on the lookout for spills of hazardous or flammable materials. Treat spills of unknown materials as hazardous.
- Do not go into areas with imminent falling hazards (e.g., racks with unstable merchandise).

Step 3 -- Examine each rack:

- Examine the structural system of each rack. Refer to Figures 12-1, and 12-2 for guidance on where to look for damage. (Figure 12-3 shows inspection points for detailed evaluations but also may be used.)
- Observe the merchandise on the rack. Look for falling and other hazards (e.g., shifted merchandise that may fall in an aftershock).
- Repeat this process for all racks in the store.

Step 4 -- Actions to be taken:

- If serious structural damage is observed or suspected, call for an ATC-20 detailed evaluation by a structural engineer.
- In case of a spill of a known or suspected hazardous material, notify the local fire department.
- Barricade aisles with spilled merchandise. Keep all but essential personnel out of the area.
- Barricade aisles adjacent to leaning or seriously damaged racks.
- In general, stores with damaged racks should be posted RESTRICTED USE. The UNSAFE posting should be restricted to stores with collapsed racks, or racks still visibly moving more than half an hour after the latest aftershock, that threaten the safety of the entire building. Stores with a few damaged or collapsed racks may be posted RESTRICTED USE, with the unsafe areas barricaded. Stores with undamaged racks, but with spilled merchandise, can be posted Inspected provided there are no falling hazards



present and unsafe areas have been barricaded. (Storage racks may reenter limited service even if they are damaged so long as proper precautions and protective vehicles are used.)

- Be sure and note on the rapid evaluation assessment form and the ATC-20 placards (e.g., INSPECTED, RESTRICTED USE, or UNSAFE) that the racks, as well as the building, have been inspected. Indicate their status and any required further actions (e.g., an ATC-20 detailed evaluation of questionable racks). (ATC-20 assessment forms and placards can be downloaded free from [www.atccouncil.org](http://www.atccouncil.org)).
- Inform the local jurisdiction and the store manager of the posting, including any recommendations for handling unsafe situations or the need for a subsequent Detailed Evaluation.

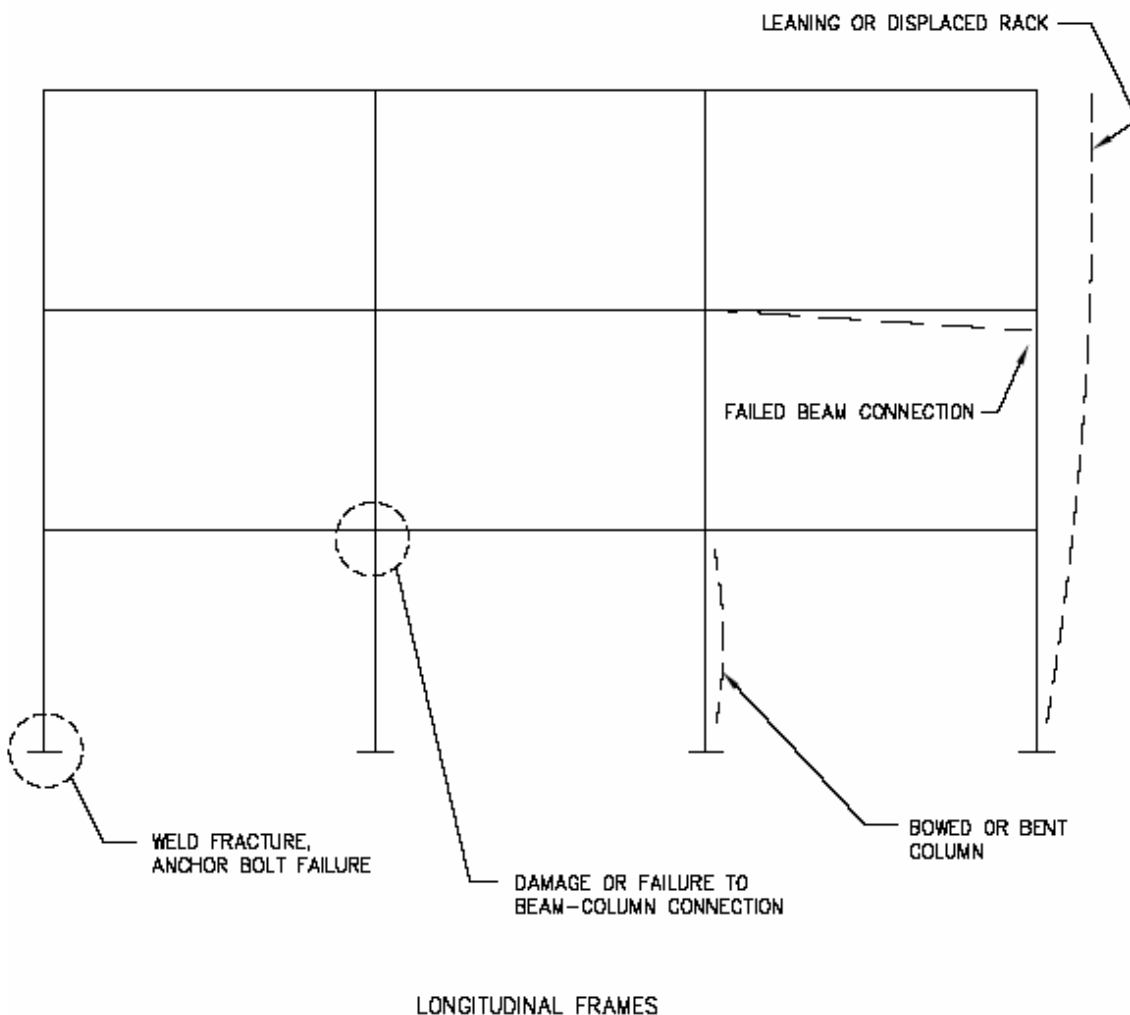


Figure 12-1 Indicated above are the recommended post-earthquake inspection points for down-aisle (longitudinal) rack frames. Both front and back frames should be examined.

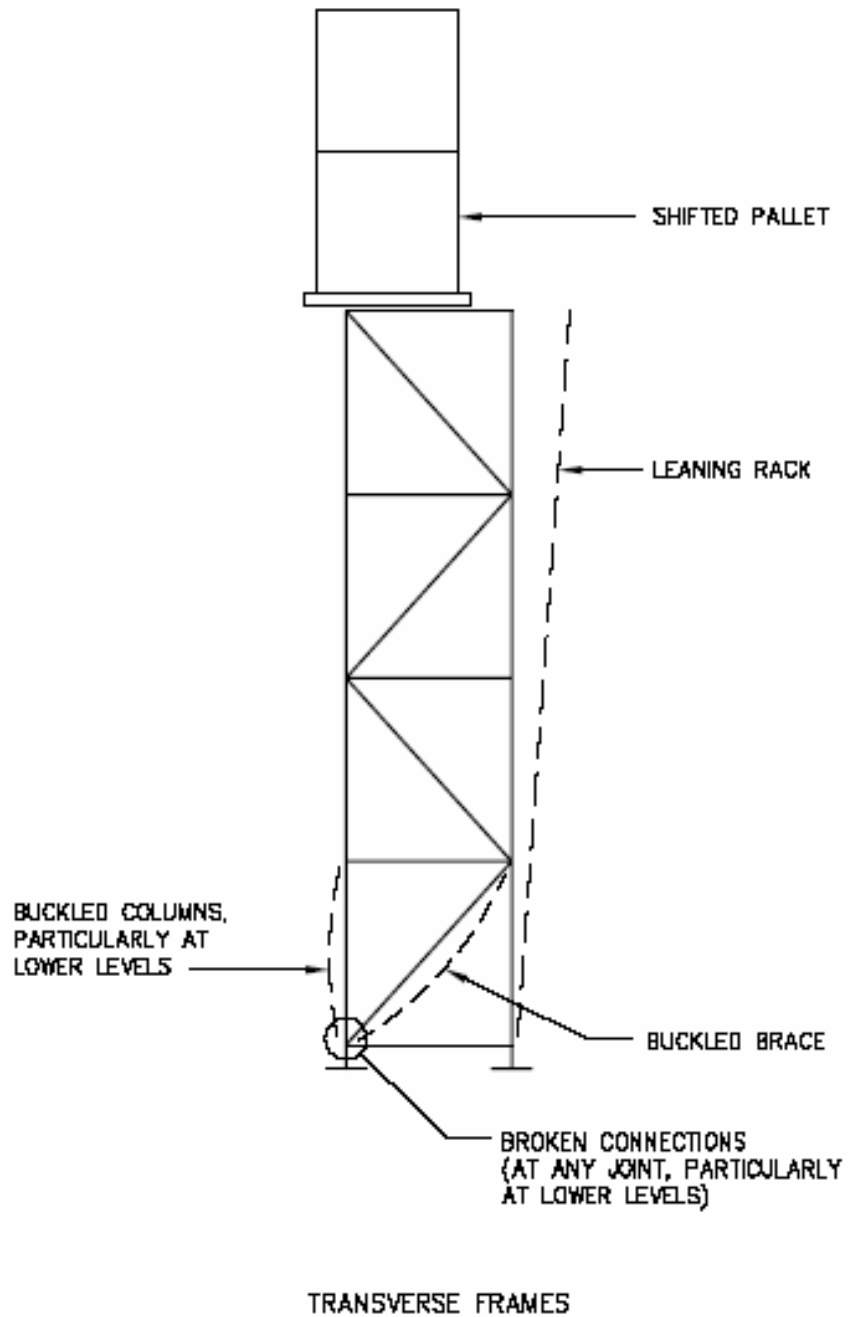


Figure 12-2. Indicated above are the recommended post-earthquake inspection points for cross-aisle (transverse) rack frames. Each frame should be examined.

**12.3.3 Guidance for ATC-20 Detailed Evaluations of Racks.** An ATC-20 detailed evaluation of storage racks should be done by a structural engineer, preferably one with rack design experience. Alternatively, the store can retain a structural engineer to perform the ATC-20 Evaluation and report the results to the local jurisdiction.

Rack structural systems are relatively easy to inspect because the structural components are not hidden from view by architectural finishes as in buildings. Because the structural members and connections are relatively small compared to building systems, inspectors must look closely at members and, particularly, connections to be able to detect significant structural damage. For example, fillet welds may be only 1/8-inch in size and cracks may be difficult to find.

For the down-aisle (longitudinal) direction, it may be necessary to ask the store to unload decks and disassemble some of the beam-column joints to fully assess whether structural damage has occurred (e.g., sheared pins, incipient block shear failures at pin holes).

For the cross-aisle (transverse) direction, bracing and bracing connections need to be closely examined. Major concerns are weld failures at bracing connections to columns and local buckling of braces. Anchor bolts and column to base plate connections need to be checked for tension (uplift) failures. It is not uncommon for mechanical expansion anchors to loosen slightly even in the absence of earthquake forces. Anchors that retighten with one or two turns of the nut have not failed and should remain in service.

Structurally damaged racks, including racks with a residual lean, should be repaired or replaced. When this has been done, the RESTRICTED USE or UNSAFE posting can be removed, provided of course that there are no other unsafe conditions left unmitigated.

The following guidelines are intended to assist the engineer doing a detailed evaluation in determining if there is any serious hidden damage which might cause a future collapse if the racks remain in service.

The racks should be standing straight and true to 1 part in 100 or roughly 1-1/4 inches deviation from an ideal line in 10 feet of dimension. Thus, a 15 foot tall rack may be almost 2 inches out of plumb either down aisle or cross aisle and remain in service if otherwise undamaged. Indeed, as the racks are unloaded, they may straighten up. Displaced pallets should be repositioned properly.

The following inspection points are shown in Figures 12-1, 12-2, and 12-3:

Point 1, Posts -- Free of severe dents greater than approximately 4 times the material thickness, or dents in corners. The post should be straight, even if leaning, and not deviate more than 1 part in 100 from straight.

Point 2, Base plate -- In firm contact with the slab, no ruptured or fractured welds.

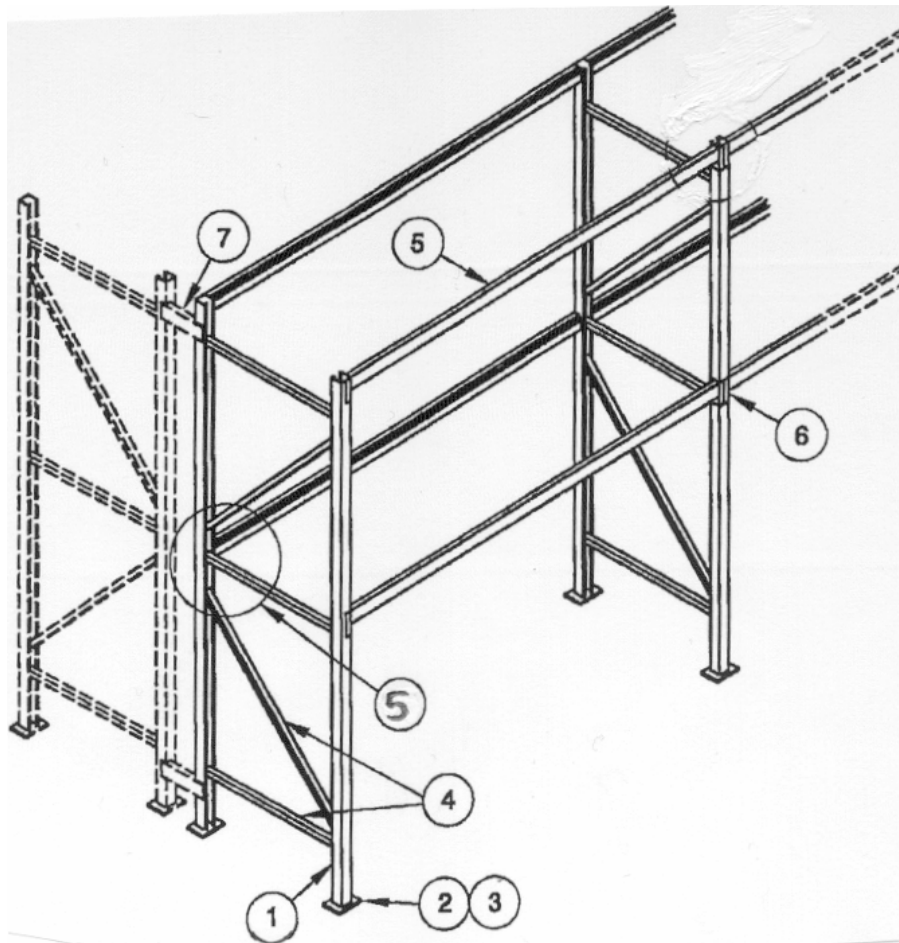
Point 3, Anchors -- Present and preferably with tight nuts or no looseness allowing uplift of the post. Checking all anchors for tightness is highly encouraged.<sup>2</sup>

Point 4, Braces -- Straight and true to 1 part in 120, with no deformations more than 4 times the material thickness.

Point 5, Beams -- Straight and true in the 1 part in 180, no dents in the top surface of more than 3/8 inch in the vertical or bottom surface.

Point 6, End connectors -- Not visibly bent or distorted, no ruptured or fractured welds, no sheared or bent pins, locking pins are present and operational. Portions of posts receiving connector pins should not be distorted more than 1/16 inch, and no block shears can be present.

Point 7, Bracing connections -- No ruptured or fractured welds or bending of the elements more than 4 times the thickness of the material.



*Figure 12-3 Inspection points for an ATC-20 detailed evaluation of a rack. Numbers refer to the inspection points listed above.*

Portions of racks failing the above criteria should be unloaded carefully from the top down if possible, and tagged as unserviceable. Generally, the rest of the rack may remain in service, provided access is restricted to persons with proper protection either by lift vehicles or other means. Large open aisles (wider than twice the rack is tall) may be accessed by unprotected persons, and materials may be brought out into these areas by the lift vehicles for picking if desired and then replaced into the racks.

## **12.4 RECOVERY OF MERCHANDISE FROM DAMAGED RACKS**

This section deals with the recovery of merchandise from racks that have been damaged or have collapsed. The advice furnished below is intended to be used by those individuals directing the store recovery effort and the removal of merchandise from damaged or failed racks.

**12.4.1 Recovering Merchandise.** Storage racks are unique in that they are normally serviced by lift vehicles (e.g., forklifts) that enclose the operator in a robust protective cage. Consequently, racks may frequently be accessed in a damaged state without undue risk to personnel. This may be important because the racks may contain material essential to post disaster relief such as food, medicines, building supplies, and tools. As with any potentially hazardous situation, those involved must balance the life-safety risk of operating a storage rack which has some damage against the public benefit of having the material available for disaster recovery.

Storage racks are also redundant and tough structures. For example, it is not uncommon for an entire front post of a rack to be destroyed, yet the rack is standing with no other signs of distress due to interconnections between the uprights. If a rack is standing reasonably straight after an earthquake, it generally may remain in service to persons operating in OSHA approved lift vehicles that have protective cages. Areas of the racks which fail this inspection criteria may still be unloaded and marked as unserviceable with the remaining areas of undamaged rack kept operational.

**12.4.2 Dealing with Collapsed Racks.** When loaded racks collapse the racks are often embedded within a pile of stored material, having been severely bent, twisted, and broken in the process of failure. The adjacent aisles maybe completely blocked, and lift vehicles may not be able to enter the area. The pile may still be moving, or may shift during aftershocks. Such racks cannot reenter service and will need unloading by specialist crews and equipment.

In this situation, the main concerns are how far to cordon off the collapsed racks, and whether it presents a hazard to the building. The store inspector should not underestimate the energy contained in such piles. The forces they can generate are easily capable of severely damaging or even destroying a building. There are no universal guidelines for these cases. However, if the pile is unstable and still shifting about, the entire building should be evacuated. If the pile appears stable, and the building is not otherwise damaged, it is recommended that the pile be cordoned off for a horizontal distance approximately twice its height. The area beyond may then remain in service.

## Chapter 13

### SUMMARY AND RECOMMENDATIONS

#### 13.1 OVERVIEW OF THE STORAGE RACK PROJECT

The Rack Project Task Group consisted of individuals with backgrounds in rack design and manufacturing, structural engineering, public safety, and warehouse store management. The Task Group made a significant effort over a limited time (approximately nine months) to examine those factors that most affect the safety of the public in warehouse-type stores during earthquakes. These are stores where the public has access to merchandise stored on steel storage racks typically on pallets.

The project brought together individuals with diverse backgrounds with a common interest in rack safety. Fortunately, the Task Group had access to recent full-scale shake-table tests done at State University of New York (SUNY) Buffalo and limited access to tests done at the University of California (UC) San Diego as well as some early shake-table tests sponsored by the National Science Foundation (NSF) and RMI and done at the University of California (UC) Berkeley in the late 1970s. Also, several individuals contributed knowledge of actual performance of racks in earthquakes, including damage photos and facts relating to the kinds of damage racks have experienced. This information allowed the group to make thoughtful considerations regarding recommendations for guidance.

The Task Group addressed the two principal seismic safety concerns with storage racks: (a) whether the present rack design requirements are sufficient to reasonably prevent the collapse of racks in earthquakes; and (b) whether present techniques for securing merchandise in racks are adequate to reasonably prevent merchandise from toppling out of racks. In delving into these two concerns, the Task Group found that storage rack seismic safety also is dependent on rack construction, operation, and maintenance. It is important that racks be designed properly, be constructed according to approved plans, not be loaded beyond design limits, be inspected periodically, and be well-maintained once in use. The Task Group findings on these topics are summarized below. In addition, recommendations for future improvements are also provided.

#### 13.2 SUMMARY

**13.2.1 Adequacy of Present Storage Rack Seismic Design Requirements.** The Task Group found no evidence that the seismic design force levels given in current design codes and standards (i.e., the 2003 IBC, ASCE 7-02, and 2003 *NEHRP Recommended Provisions*) are inadequate, but it did find that the behavior of beam-column connections in the down-aisle direction and brace-column connections in the cross-aisle direction is much different than that anticipated or predicted by the structural analysis procedures commonly used in rack design. However, the Task Group was uncertain whether the

current seismic force levels and design practices were adequate to prevent rack collapse for the maximum earthquake ground motion levels (maximum consider earthquake levels) as is the case for buildings. The Task Group developed and has proposed in Chapter 6 an optional displacement-based evaluation procedure to verify storage rack stability when subject to maximum earthquake ground motions.

Racks are presently designed using an equivalent static seismic design criteria approach similar to that for steel buildings. Recent shake-table tests at UC - San Diego and SUNY - Buffalo have revealed that the dynamic behavior of racks can be quite different from that for buildings principally because of the types of connections used behave in a semi-rigid nonlinear manner. The differences are most significant in the down-aisle (longitudinal) direction. Racks in this direction behave in nonlinear fashion and with periods much longer than that determined from the approximate code period formulas.

Rack behavior in the cross-aisle (transverse) direction is similar to that of braced steel frame structures except that the connections are frequently light-gage steel and these can behave quite differently under extreme lateral loads than the structural steel shapes used in buildings. In the case of racks, light-gage connections sometimes fail at welds at the brace-to-column connection or braces buckle locally. When this happens, the stability of the rack is of serious concern and depends on flexural action in the horizontal braces and columns without participation of the lower level braces. Another concern is failure of welds at post-to-base plate connection because this can lead to overturning of the rack. Procedures for evaluating brace-frame connections need to be developed.

The current RMI standard (2002 edition), subsequently adopted as American national standard ANSI MH 16.1 (2004), is out of date with regard to the seismic hazard maps and load combinations and needs to be updated to be in conformance with the 2003 *NEHRP Recommended Provisions*.

**13.2.2 Securing Merchandise in Racks.** Toppling merchandise is possibly the greatest life-safety risk associated with storage racks in areas accessible to the public. Unsecured items can fall from shelves and strike shoppers in aisles below.

Other than the 2001 California Labor Law, the Task Group did not find any publicly available guidelines for securing merchandise to prevent it from toppling in earthquakes. The California law requires that merchandise on shelves 12-feet and higher be secured to pallets or be netted or otherwise restrained. The Task Group felt that the 12-foot height limit was too high and that a lower height should be used. The figure of 8 feet was agreed upon since heavier items falling from that height can result in injury.

The Task Group also found that the California law was fairly brief, and its provisions could be expanded into a more general guideline for racks in high seismic regions. Several methods are recommended in Chapter 8 to secure merchandise above 8 feet. These include various techniques to fasten the merchandise to pallets as well as the use of restraining devices such as netting, restraining bars, and chains to restrain individual containers not on pallets.

**13.2.3 Dangers of Unregulated and Unpermitted Racks.** Ideally, storage racks in a public access warehouse store are installed according to drawings prepared by a registered design professional and have received a plan check and construction permit from the local jurisdiction. However, the Task Group heard commentary that this is not always done, although this issue is believed to not be of concern in public warehouse applications involving large retail chain stores. It was felt that unregulated rack installations (i.e., those installed without a construction permit from the local jurisdiction) were more likely result in unsafe situations. The Task Group believes that this practice needs to be controlled and that the responsibility for this rests with both the store operator and the local jurisdiction.

**13.2.4 Benefit of Periodic In-Use Inspections.** Once a storage rack is installed in a public access warehouse store, there are typically no further safety inspections by the local jurisdiction unless the design is significantly altered beyond that permitted on the design drawing. If the design is significantly altered, building codes generally require that drawings and calculations be submitted for plan check and permit before the modifications can be made.

It was reported that many large chains have adopted as standard practice the requirement that periodic safety inspections be performed. Audits of these inspections are done at least annually. Depending on the thoroughness of the inspections, this can be a satisfactory practice to ensure that no unsafe modifications have been made and that the racks have not experienced significant structural damage while in use.

**13.2.5 Post-earthquake Rack Performance Verification.** There is a paucity of publicly available information on the performance of racks in earthquakes. Since the 1971 San Fernando earthquake, much has been learned through post-earthquake reconnaissance on the behavior of buildings, nonstructural elements and building contents. Groups such as the Earthquake Engineering Research Institute (EERI) have established Learning from Earthquakes programs to obtain and then disseminate information of the performance of a wide variety of building-related issues. Steel and concrete industry trade organizations have regularly reported on buildings built with their materials, and organizations such as NSF and FEMA have sponsored studies to advance seismic design and mitigation practices. However, there has been no similar progress in understanding storage rack performance.

The Task Group found that there have been few, if any, formal publicly available post-earthquake evaluations of storage racks and virtually no analysis or documentation of either successes or failures. Individual stores, their structural engineers, and store owner management may know about the performance of individual racks, but in general the storage rack design profession and public safety officials do not have access to that detailed information.



## 13.3 RECOMMENDATIONS

### 13.3.1 Improvement in Storage Rack Seismic Design

- The RMI standard should adopt the 2003 *NEHRP Recommended Provisions* seismic hazard maps, design ground motion parameters, base shear equations, and ASCE 7-02 load combinations.
- The RMI standard should include evaluation procedures and design examples for beam-column connections, brace-column connections, and base plate design.
- A cyclic testing standard to test proprietary down-aisle moment connection systems and cross-aisle bracing system connections should be developed and included in the RMI standard.
- Full-size shake-table testing of storage rack systems should be encouraged and the results incorporated into the RMI criteria. Testing should include simultaneous horizontal and vertical motions and simulate design basis and maximum considered earthquake levels and durations of shaking.
- An optional maximum seismic displacement check of beam-column moment connections should be included in the RMI standard for the down-aisle (longitudinal) direction. If such checks are performed in accordance with Sec. 6.5 and the design satisfies the criteria, the  $I_p$  factor of 1.5 may be taken as 1.0 and the  $0.14 S_{DS}$  minimum force coefficient may be taken as 0.01 in the equivalent static force design.
- An optional maximum seismic displacement check of the brace frame system should be considered for inclusion in the RMI standard for the cross-aisle (transverse) direction. If such checks are performed in accordance with Sec. 6.6 and the design satisfies the criteria, the  $I_p$  factor of 1.5 may be taken as 1.0 and the  $0.14 S_{DS}$  minimum force coefficient may be taken as 0.01 in the equivalent static force design.

### 13.3.2 Improvement of the Process of Procuring, Specifying, and Installing Storage Racks

- The guidance of Chapter 7 should be used as the basis for the procurement, specification, and installation of storage racks.

### 13.3.2 Improvement for Securing Contents

- The guidelines of Chapter 8 for securing contents should be used for an interim period until a nationally applicable standard is developed.

- Various means for securing contents in racks should be tested by shake-table testing. The testing should stimulate both vertical and horizontal “in rack” motions on a variety of decks.
- A joint industry/government effort to write nationally applicable guidelines for securing contents in high seismic regions should be undertaken in the near future. The need for guidelines for securing contents in other than high seismic regions of the United States should be studied and be incorporated if deemed necessary.

### **13.3.3 Eliminate Unpermitted Racks**

- Local jurisdictions must take steps to enforce building code seismic provisions with regard to rack construction.
- If not already done, individual store operators should require that all new storage rack installations or significant alterations beyond those allowed in the design drawings be approved by the local authority having jurisdiction, and that significant alterations not be done without the oversight of a registered design professional.

### **13.3.4 Maintain Strong Maintenance and Operation Programs**

- Store operators are encouraged to maintain strong maintenance and operations programs and to follow the guidance provided in Chapter 10.

### **13.3.5 Require Periodic Safety Inspections**

- Periodic rack safety inspections should be made by store operators.
- Annual audits of the safety inspections should be made store operator quality assurance management.
- State and local safety and health agencies and other authorities having jurisdiction are encouraged to adopt and enact inspection criteria based on these storage rack seismic design practices to support existing safety and health laws and other local ordinances to protect workers and citizens.

### **13.3.6 Arrange for Post-earthquake Reconnaissance to Verify Rack Performance**

- There needs to be systematic scientific studies of rack behavior in earthquakes, and this needs to be publicly reported (to the rack manufacturing, structural engineering, and building safety communities). Such parameters as the load on the racks at the time of the event, the condition of the racks, and the type, configuration and sizes of the rack structural system need to be documented as well as the motions the racks received.

- A pre-arranged joint industry/government post-earthquake reconnaissance team, perhaps under EERI leadership, should be established. Prior arrangements with retail companies and store operators need to be made to permit quick team access after an event, and the results of the reconnaissance need to be published in appropriate technical publications.
- Selected stores in high seismic regions in the vicinity of active faults should be instrumented with strong-motion recorders to measure and record free-field ground motions. When strong-motion records for such installations are obtained, the records can be used to correlate rack design with actual performance.

### **13.3.7 Maintain Strong Quality Assurance Programs**

- Store operators are encouraged to maintain strong quality assurance programs and to follow the guidance provided in Chapter 10.

## Chapter 14

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## APPENDIX A

### DEVELOPMENT OF AN ANALYTICAL MODEL FOR THE DISPLACEMENT-BASED SEISMIC DESIGN OF STORAGE RACKS IN THEIR DOWN-AISLE DIRECTION

#### A1 OBJECTIVES

This Appendix provides a simple analytical model that captures the seismic behavior of storage racks in their down-aisle direction. More specifically, the model is aimed at developing simplified equations for the fundamental period, the base shear, and the top lateral displacement of storage racks in their down-aisle direction as a function of the beam-to-upright rotational characteristics at a given target design lateral displacement.

#### A2 ASSUMPTIONS

In order to derive simplified expressions for the down-aisle fundamental period, base shear and top lateral displacement of storage racks, the following assumptions based on observations are made:

- The same beam-to-upright connections are used throughout the moment-resisting frame system of the rack.
- The beams are spaced relatively uniformly with height.
- All moment-resisting connections of the racks experience simultaneously similar rotations at all times. This assumption implies that the connection rotational stiffness is somewhat smaller than the rotational stiffness of the beams and uprights.
- All inelastic deformations occur at the beam-to-upright moment-resisting connections and base upright-to-slab connections.
- The overall seismic response can be reasonably modeled as a single-degree-of-freedom system corresponding to an assumed first down-aisle mode of deformation of the rack.
- Connection moment versus connection rotation curves have been developed based on cyclic testing and these design curves have been adjusted for uncertainty in the cyclic test data.

Figure A-1 illustrates the assumed lateral first mode deformation of a three-level storage rack in its down-aisle direction according to the assumptions listed above. It is assumed that the following properties of the beam-to-upright connectors are known at the target displacement  $\Delta$  :



- The rotational moment ( $M_c$ ),
- The secant rotational stiffness of the connectors at that moment ( $k_c$ ), and
- The rotation of the connectors at that moment ( $\theta_c$ ).

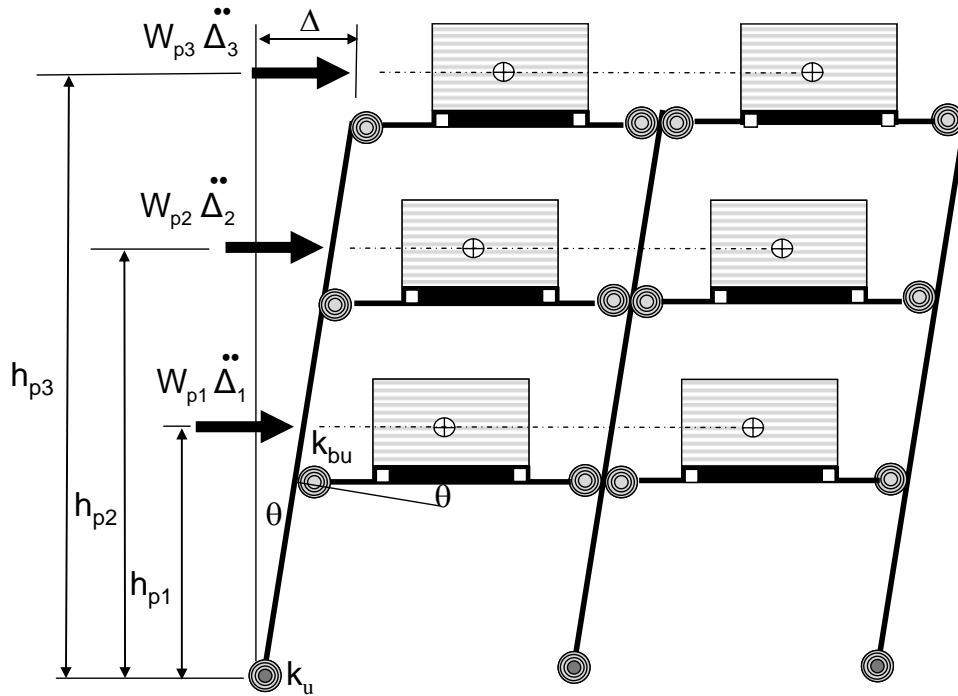


Figure A-1 Kinematics assumptions for storage racks in down-aisle direction.

The total rotational stiffness between the beams and uprights ( $k_{bu}$ ) indicated in Fig. A-1 is the sum in series of the secant rotational stiffness of the connector ( $k_c$ ) and of the flexural rotational stiffness of the beam-end ( $k_{be}$ ):

$$k_{bu} = \frac{k_c k_{be}}{k_c + k_{be}} \quad (\text{A-1})$$

Similarly, the total rotational stiffness at the base of each upright ( $k_u$ ) indicated in Figure A-1 is the sum in series of the secant rotational stiffness of the base plate ( $k_b$ ) and of the flexural rotational stiffness of the base upright-end ( $k_{ce}$ ):

$$k_u = \frac{k_b k_{ce}}{k_b + k_{ce}} \quad (\text{A-2})$$

### A3 SIMPLIFIED EQUATION FOR FUNDAMENTAL PERIOD OF VIBRATION

The applied moment about the base ( $M_{bi}$ ) caused by the lateral inertia forces is given by:

$$M_{bi} = \sum_{i=1}^{N_L} \frac{W_{pi}}{g} \ddot{\Delta}_i h_{pi} = \frac{1}{g} \sum_{i=1}^{N_L} W_{pi} h_{pi}^2 \ddot{\theta} \quad (\text{A-3})$$

where ( $W_{pi}$ ) is the weight of the  $i^{\text{th}}$  pallet supported by the storage rack, ( $h_{pi}$ ) is the elevation of the center of gravity of the  $i^{\text{th}}$  pallet with respect to the base of the storage rack,  $g$  is the acceleration of gravity, and ( $N_L$ ) is the number of loaded level.

The resisting moment about the base ( $M_{br}$ ) is given by:

$$M_{br} = -(N_c k_{bu} + N_b k_u) \theta \quad (\text{A-4})$$

where ( $N_c$ ) is the number of beam-to-upright connections and ( $N_b$ ) is the number of base plate connections.

Equating the applied moment to the resisting moment yields the equation of motion for the equivalent single-degree-of-freedom system:

$$\frac{1}{g} \sum_{i=1}^{N_L} W_{pi} h_{pi}^2 \ddot{\theta} + (N_c k_{bu} + N_b k_u) \theta = 0 \quad (\text{A-5})$$

The fundamental period of vibration ( $T_1$ ) is then expressed as:

$$T_1 = 2\pi \sqrt{\frac{\sum_{i=1}^{N_L} W_{pi} h_{pi}^2}{g(N_c k_{bu} + N_b k_u)}} \quad (\text{A-6})$$

Substituting A-1 and A-2 into A-6 yields:

$$T_1 = 2\pi \sqrt{\frac{\sum_{i=1}^{N_L} W_{pi} h_{pi}^2}{g \left( N_c \left( \frac{k_c k_{be}}{k_c + k_{be}} \right) + N_b \left( \frac{k_b k_{ce}}{k_b + k_{ce}} \right) \right)}} \quad (\text{A-7})$$

The beam end and the base upright end rotational stiffness are given by:

$$k_{be} = \frac{6EI_b}{L}; \quad k_{ce} = \frac{4EI_c}{H} \quad (\text{A-8})$$

where  $E$  is the Young's modulus of the beams,  $I_b$  is the moment of inertia about the bending axis of each beam,  $L$  is the clear span of the beams,  $I_c$  is the moment of inertia of each base upright, and  $H$  is the clear height of the upright.

#### A4 SIMPLIFIED EQUATION FOR BASE SHEAR

Alternatively, the resisting moment about the base ( $M_{br}$ ) can be written as:

$$M_{br} = -(N_c M_c + N_b M_b) \quad (\text{A-9})$$

where  $M_c$  is the bending moment at each beam-to-upright connection and  $M_b$  is the bending moment at of each base plate connection.

Equating the applied moment (Eq. A-3) to the resisting moment (Eq. A-9) yields an equation for the rotational acceleration causing the moments  $M_c$  and  $M_b$ :

$$\ddot{\theta} = \frac{(N_c M_c + N_b M_b)}{\frac{1}{g} \sum_{i=1}^{N_L} W_{pi} h_{pi}^2} \quad (\text{A-10})$$

By horizontal equilibrium, the base shear ( $V_b$ ) must be equal to the sum of the inertia forces:

$$V_b = \frac{1}{g} \sum_{i=1}^{N_L} W_{pi} \ddot{\Delta}_i = \frac{1}{g} \sum_{i=1}^{N_L} W_{pi} h_{pi} \ddot{\theta} \quad (\text{A-11})$$

Substituting Eq. A-10 into Eq. A-11 yields:

$$V_b = \frac{\sum_{i=1}^{N_L} W_{pi} h_{pi}}{\sum_{i=1}^{N_L} W_{pi} h_{pi}^2} (N_c M_c + N_b M_b) \quad (\text{A-12})$$

The only unknown in Eq. A12 is the bending moment at each base plate connection ( $M_b$ ) arising when  $M_c$  occurs at each beam-to-upright connector. Since the beam ends rotational stiffness act in series with the beam-to-upright connectors, the total rotation at a

beam-to-upright connection ( $\theta$ ) is the sum of the rotation experienced by the beam-to-upright connector( $\theta_c$ ) and the rotation experienced by the beam end( $\theta_{be}$ ):

$$\theta = \theta_c + \theta_{be} = \frac{M_c}{k_c} + \frac{M_c}{k_{be}} = M_c \left( \frac{k_c + k_{be}}{k_c k_{be}} \right) \quad (\text{A-13})$$

From Figure A-1, the same total rotation is assumed to be occurring at the base of each upright and is also the sum of the rotation experienced by the base plate connector ( $\theta_b$ ) and the rotation experienced by the base upright-end ( $\theta_{ce}$ ):

$$\theta = \theta_b + \theta_{ce} = \frac{M_b}{k_b} + \frac{M_b}{k_{ce}} = M_b \left( \frac{k_b + k_{ce}}{k_b k_{ce}} \right) \quad (\text{A-14})$$

Equating Eq. A13 and A14 yields an expression for the bending moment at each base plate connection ( $M_b$ ) as a function of all know quantities:

$$M_b = M_c \left( \frac{k_b k_{ce}}{k_c k_{be}} \right) \left( \frac{k_c + k_{be}}{k_b + k_{ce}} \right) \quad (\text{A-15})$$

Substituting Eq. A-15 into Eq. A-12 yields an explicit expression for the base shear ( $V_b$ ) as a function of only known quantities:

$$V_b = \frac{\sum_{i=1}^{N_L} W_{pi} h_{pi}}{\sum_{i=1}^{N_L} W_{pi} h_{pi}^2} \left( N_c + N_b \left( \frac{k_b k_{ce}}{k_c k_{be}} \right) \left( \frac{k_c + k_{be}}{k_b + k_{ce}} \right) \right) M_c \quad (\text{A-16})$$

#### A5 SIMPLIFIED EQUATION FOR TOP LATERAL DISPLACEMENT

The lateral displacement at the top of the rack ( $\Delta$ ) is simply given by the total rotation at the base of the uprights ( $\theta$ ) multiplied by the height of the rack ( $h_{tot}$ ). Using Eq. A-13 yields:

$$\Delta = M_c \left( \frac{k_c + k_{be}}{k_c k_{be}} \right) h_{tot} \quad (\text{A-17})$$

#### A6 CONSIDERATION OF SECOND ORDER (P-DELTA) EFFECTS

An amplification factor for second order (P-delta) effects on the top lateral displacement (see Eq. A-17) can be obtained by considering a deformed rack under the gravity loads caused by the pallet weights, as shown in Figure A-2.

The applied second order moment about the base ( $m_{ba}$ ) is given by:

$$m_{ba} = \sum_{i=1}^{N_L} W_{pi} \Delta_i = \sum_{i=1}^{N_L} W_{pi} \frac{h_{pi}}{h_{tot}} \Delta \quad (\text{A-18})$$

The resisting second order moment about the base ( $m_{br}$ ) can be written as:

$$m_{br} = -(N_c m_c + N_b m_b) \quad (\text{A-19})$$

where ( $m_c$ ) is the second order bending moment at each beam-to-upright connection and ( $m_b$ ) is the second order bending moment at each base plate connection.

Equating the second order applied moment to the second order resisting moment and using Eq. A-15 to express ( $m_b$ ) as a function of ( $m_c$ ) yields:

$$m_c = \frac{\sum_{i=1}^{N_L} W_{pi} \frac{h_{pi}}{h_{tot}} \Delta}{\left( N_c + N_b \left( \frac{k_b k_{ce}}{k_c k_{be}} \right) \left( \frac{k_c + k_{be}}{k_b + k_{ce}} \right) \right)} \quad (\text{A-20})$$

Eq. A-20 provides an expression for the second order bending moment induced in each beam-to-upright connection when the rack is subjected to a first order top lateral displacement  $\Delta$ .

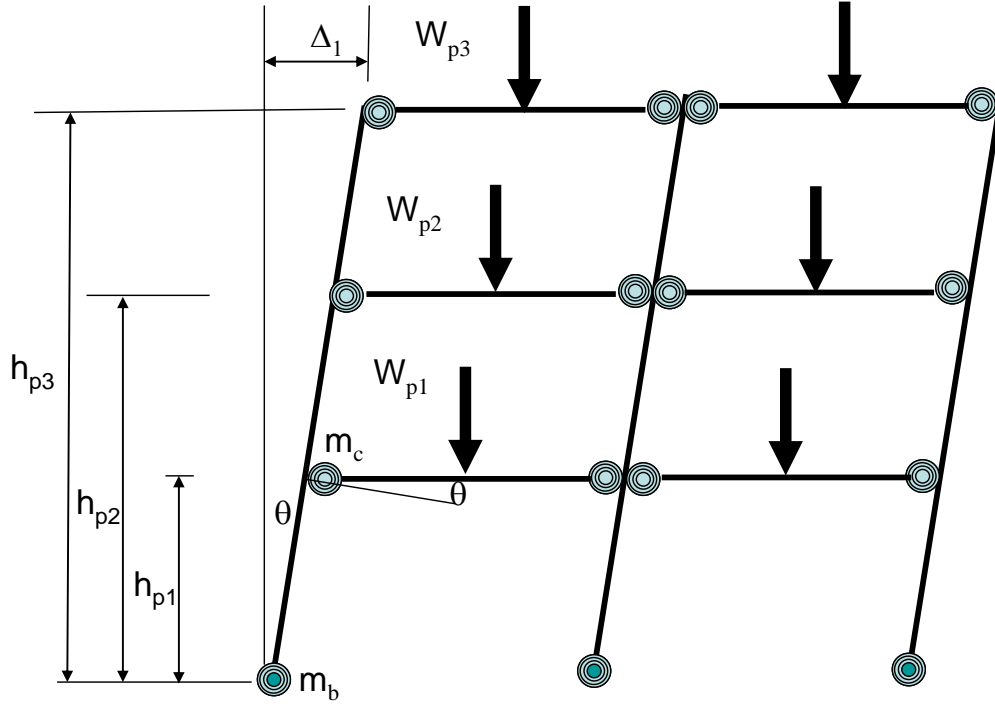


Figure A-2 Deformed rack subjected to gravity loads from pallet weights.

The second order top lateral displacement ( $\delta$ ) can be obtained from Eq. A-17:

$$\delta = m_c \left( \frac{k_c + k_{be}}{k_c k_{be}} \right) h_{tot} = \frac{\sum_{i=1}^{N_L} W_{pi} h_{pi} \left( \frac{k_c + k_{be}}{k_c k_{be}} \right)}{\left( N_c + N_b \left( \frac{k_b k_{ce}}{k_c k_{be}} \right) \left( \frac{k_c + k_{be}}{k_b + k_{ce}} \right) \right)} \Delta \quad (\text{A-21})$$

The total top lateral displacement ( $\Delta_{tot}$ ) is then obtained by summing the first order displacement (Eq. A-17) and the second order displacement (Eq. A-21) :

$$\Delta_{tot} = \Delta + \frac{\sum_{i=1}^{N_L} W_{pi} h_{pi} \left( \frac{k_c + k_{be}}{k_c k_{be}} \right)}{\left( N_c + N_b \left( \frac{k_b k_{ce}}{k_c k_{be}} \right) \left( \frac{k_c + k_{be}}{k_b + k_{ce}} \right) \right)} \Delta = \Delta \left( 1 + \frac{\sum_{i=1}^{N_L} W_{pi} h_{pi} \left( \frac{k_c + k_{be}}{k_c k_{be}} \right)}{\left( N_c + N_b \left( \frac{k_b k_{ce}}{k_c k_{be}} \right) \left( \frac{k_c + k_{be}}{k_b + k_{ce}} \right) \right)} \right) \quad (\text{A-22})$$

Therefore, second order effects can be taken into account by multiplying the first order top lateral displacement (Eq. A-17) by a second order amplification factor equals to  $(1 + \alpha)$  and given by:

$$1 + \alpha = 1 + \frac{\sum_{i=1}^{N_L} W_{pi} h_{pi} \left( \frac{k_c + k_{be}}{k_c k_{be}} \right)}{\left( N_c + N_b \left( \frac{k_b k_{ce}}{k_c k_{be}} \right) \left( \frac{k_c + k_{be}}{k_b + k_{ce}} \right) \right)} \quad (\text{A-23})$$

## A7 EXPERIMENTAL VERIFICATION

Experimental results obtained from uniaxial shake-table tests performed on two different steel pallet storage racks in their down-aisle direction are used to assess the predictive capabilities of the simple equations derived above. Only a brief description of the experimental study is described herein. Detailed information on the experimental study can be found elsewhere (Filiatrault and Wanitkorkul 2004).

## A8 DESCRIPTION OF TEST STORAGE RACK STRUCTURES

Figure A-3 shows the configuration of the three-level, two-bay test storage rack structures. The relevant dimensions and properties are also indicated. The uprights are C4x4.5 hot-rolled sections. Two different beam sizes were considered — C4x4.5 (Rack No. 1) and C5x6.7 (Rack No. 2).

## A9 EARTHQUAKE GROUND MOTIONS

The largest horizontal component recorded at Canoga Park during the 1994 Northridge earthquake was used for the seismic tests. This record, scaled to a peak ground acceleration (PGA) of 0.5 g, can be associated with an ordinary ground motion matching the NEHRP Recommended Provisions spectral accelerations for a probability of exceedence of 10 percent in 50 years for a soil type D (Krawinkler et al., 2000).

## A10 FUNDAMENTAL PERIOD COMPARISON

The pallet weight distribution term in Eq. A-7 can be calculated from Figure A-3:

$$\sum_i W_{pi} h_{pi} = (4)(3.3 \text{ kips}) \left[ (93 \text{ in}) + (153 \text{ in}) + (213 \text{ in}) \right] = 6058.8 \text{ kip-in} \quad (\text{A-24})$$

Considering the two parallel frames comprising the test storage rack, the number of beam-to-upright connections  $N_c = 24$  while the number of loaded levels  $N_L = 3$  and the number of base plate connections  $N_b = 6$ . Table A-1 shows the properties of the beams and uprights used for the two test racks.

Table A-1 Properties of beams and uprights of test storage racks.

| Properties                                 | Values               |                      |
|--|----------------------|----------------------|
|  | Rack No. 1           | Rack No. 2           |
| Young's Modulus, E                         | 29000 ksi            |                      |
| Beam Span, L                               | 95 in                |                      |
| Upright Height, H                          | 60 in                |                      |
| Beam Inertia, $I_b$                        | 3.65 in <sup>4</sup> | 7.40 in <sup>4</sup> |
| Upright Inertia, $I_c$                     | 3.65 in <sup>4</sup> |                      |
| Beam End Rotational Stiffness, $k_{be}$    | 6686 kip-in/rad      | 13719 kip-in/rad     |
| Upright End Rotational Stiffness, $k_{ce}$ | 7057 kip-in/rad      |                      |

Assuming  $k_b = k_c$  in Eq. A-7, the variation of the down-aisle fundamental period of the storage racks as a function of the rotational stiffness of the connectors can be obtained. The assumption that  $k_b = k_c$  is considered reasonable for installations where there is at least one bolt on opposite sides of the post in the down-aisle direction. Figure A-4 compares the predictions of the simplified model with the results obtained from finite element analyses of the test racks. Details of the finite element model can be found elsewhere (Filiatrault and Wanitkorkul 2004). Also shown on the graph are fundamental periods measured by low amplitude systems identification tests (Filiatrault and Wanitkorkul 2004).





*Figure A-3 Configuration of test storage rack structure.*

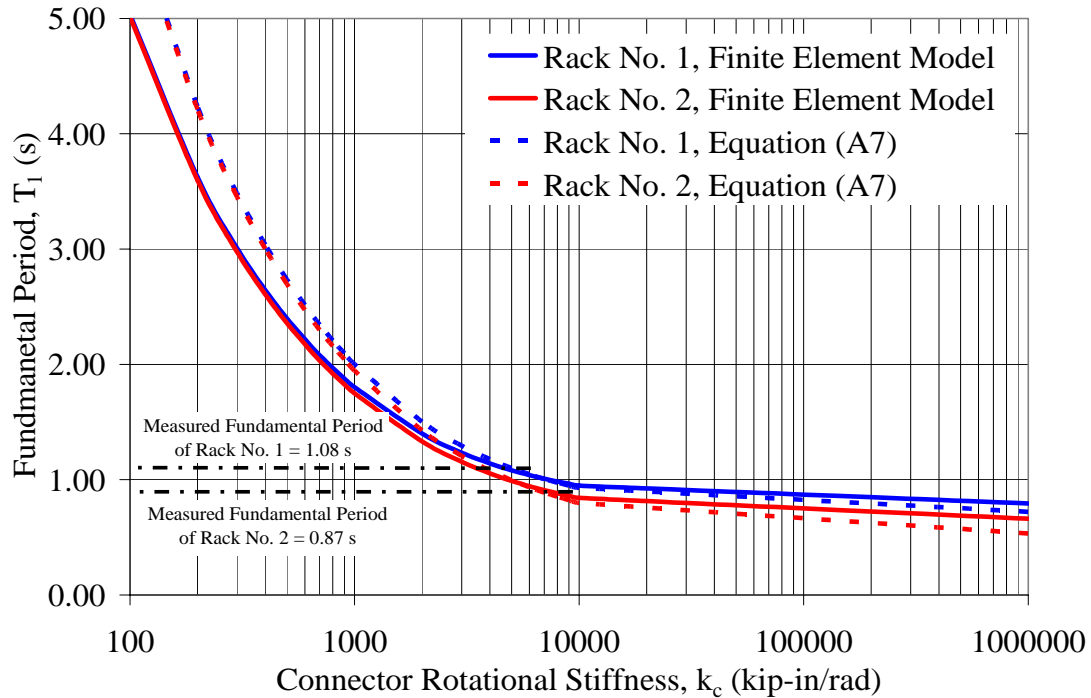


Figure A-4 Comparison of fundamental periods for test storage racks.

The results shown in Figure A-4 indicate that the simplified model overestimates the fundamental period of the racks for connector rotational stiffness values less than 1000 kip-in/rad. Also, the model slightly underestimates the fundamental period of the racks for connector rotational stiffness values larger than 10000 kip-in/s. The model, however, provides reasonable accurate estimate of the fundamental period of the storage racks in the range of beam-to-upright connector rotational stiffness values commonly encountered in steel pallet storage racks (1000 to 10000 kip-in/rad). The results presented in Figure A-4 indicate also that the beam size has a minor influence on the fundamental period of the test storage racks.

#### A11 BASE SHEAR COMPARISON

Assuming  $k_b = k_c$  in Eq. A-7, A-16, and A-17, the two main connector parameters ( $k_c$  and  $M_c$ ) can be estimated in an average sense based on the experimental data generated above. Once three parameters are known, a prediction of the base shear forces developed during the seismic tests of the test storage racks can be obtained. The various steps of this procedure are given below and the results obtained are presented in Table A- 2.

Step 1 — An estimate of the effective lateral stiffness of each rack ( $k_{eff}$ ) during a seismic test can be obtained by dividing the experimental base shear ( $V_{bexp}$ ) by the experimental

lateral displacement at the effective height of the loaded pallets. Assuming constant mass at each level and linear lateral displacement profile, it can be shown that the effective height is located at 72 percent of the total height of the rack. Therefore, the lateral displacement at the effective height of the rack can be made equal to 72 percent the experimental top lateral displacement ( $\Delta_{exp}$ ).

Step 2 — An effective fundamental period of vibration  $T_{1eff}$  can be obtained from  $k_{1eff}$  :

$$T_{1eff} = 2\pi \sqrt{\frac{W_{tot}}{gk_{1eff}}} \quad (A-25)$$

where  $W_{tot}$  is the total pallet weight on the rack (39.6 kips).

Step 3 — Using the values of  $T_{1eff}$  in Eq. A-7 and assuming  $k_b = k_c$ , the effective rotational stiffness for the beam-to-upright connectors ( $k_c$ ) can be estimated for each seismic test.

Step 4 — Substituting the values of the experimental top lateral displacement ( $\Delta_{exp}$ ) into Eq. A-17 and taking into account second order effects through Eq. A-23, the values of the bending moment in each beam-to-upright connector ( $M_c$ ) can be calculated.

$$M_c = \left( \frac{k_c k_{be}}{k_c + k_{be}} \right) \frac{\Delta_{exp}}{h_{tot}(1 + \alpha)} \quad (A-26)$$

where  $h_{tot} = 4.88$  m for the test racks. Finally, the base shear for each seismic test ( $V_b$ ) can be predicted by Eq. 16, again assuming  $k_b = k_c$ .

The comparison of the experimental and predicted base shear forces shown in Table A-2 indicates that the model predicts within 10 percent the experimental base shear for Rack No. 1. The predictions of the model are in even better agreement with the experimental results for Rack No. 2. Based on the results, it can be concluded that the proposed model is able to capture, with reasonable accuracy, the seismic behavior of steel pallet storage racks in their down-aisle direction.

Estimates of the moment-rotation envelope for each beam-to-upright connector can be obtained by plotting the values of  $M_c$  as a function of  $M_c/k_c$  in Table A-2. As shown in Figure A-5, the nonlinear characteristics of the connectors can be clearly seen.

Table A-2 Predictions of base shear for test storage racks

| Rack No. 1, Beams: C4x4.5 |         |                   |                     |                      |                |                |              |                |              |                |
|---------------------------|---------|-------------------|---------------------|----------------------|----------------|----------------|--------------|----------------|--------------|----------------|
| Test No.                  | PGA (g) | $V_{bexp}$ (kips) | $\Delta_{exp}$ (in) | $k_{leff}$ (kips/in) | $T_{leff}$ (s) | $k_c$ (kip-in) | $1 + \alpha$ | $M_c$ (kip-in) | $V_b$ (kips) | $V_b/V_{bexp}$ |
| 1                         | 0.07    | 1.47              | 1.36                | 1.50                 | 1.65           | 1628           | 1.15         | 8.14           | 1.42         | 0.96           |
| 2                         | 0.12    | 2.68              | 2.97                | 1.25                 | 1.80           | 1292           | 1.19         | 13.98          | 2.40         | 0.90           |
| 3                         | 0.24    | 4.33              | 5.55                | 1.08                 | 1.93           | 1097           | 1.21         | 22.48          | 3.98         | 0.92           |
| 4                         | 0.36    | 5.70              | 7.37                | 1.08                 | 1.94           | 1097           | 1.21         | 29.82          | 5.28         | 0.92           |
| 5                         | 0.47    | 7.06              | 9.04                | 1.08                 | 1.94           | 1097           | 1.21         | 36.73          | 6.67         | 0.92           |

| Rack No. 2, Beams: C5x6.7 |         |                   |                     |                      |                |                |              |                |              |                |
|---------------------------|---------|-------------------|---------------------|----------------------|----------------|----------------|--------------|----------------|--------------|----------------|
| Test No.                  | PGA (g) | $V_{bexp}$ (kips) | $\Delta_{exp}$ (in) | $k_{leff}$ (kips/in) | $T_{leff}$ (s) | $k_c$ (kip-in) | $1 + \alpha$ | $M_c$ (kip-in) | $V_b$ (kips) | $V_b/V_{bexp}$ |
| 1                         | 0.05    | 1.23              | 0.47                | 3.63                 | 1.06           | 4381           | 1.06         | 0.87           | 1.29         | 1.05           |
| 2                         | 0.10    | 2.85              | 1.54                | 2.57                 | 1.25           | 2788           | 1.09         | 1.92           | 2.90         | 1.02           |
| 3                         | 0.25    | 5.27              | 3.68                | 1.99                 | 1.42           | 2062           | 1.12         | 3.45           | 5.27         | 1.00           |
| 4                         | 0.34    | 8.45              | 7.82                | 1.50                 | 1.65           | 1416           | 1.16         | 5.10           | 7.80         | 0.92           |

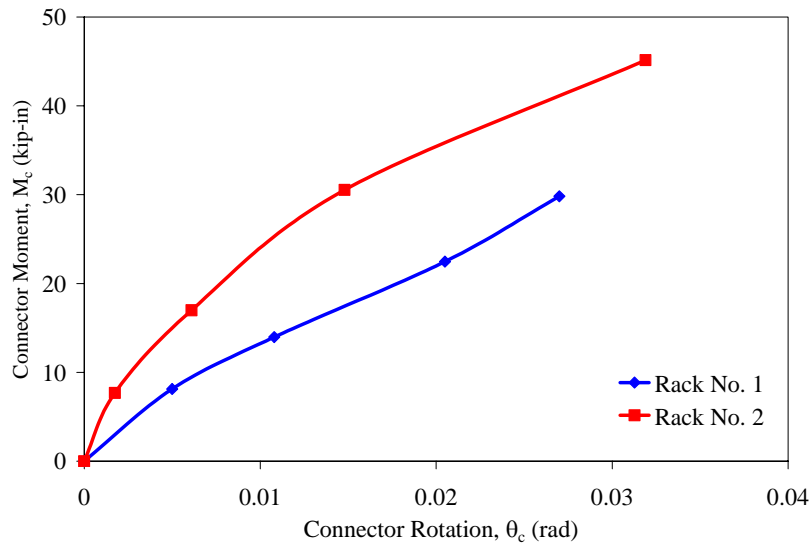


Figure A-5 Envelope of beam-to-upright connector moment-rotation relationship predicted by analytical model.



## APPENDIX B

### RECOMMENDED MODIFICATIONS TO ANSI MH 16.1-04, *SPECIFICATION FOR THE DESIGN, TESTING AND UTILIZATION OF INDUSTRIAL STEEL STORAGE RACKS* (RMI – 2002 Edition)

The following modifications are recommended to the ANSI MH 16.1-04 Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks.

Revise ANSI MH 16.1-04 as follows:

#### 2.1 Load Combinations For The ASD Design Method

Replace the Load Combinations of Section 2.1 with the Load Combinations of Section 6.3.1.1 of this guidance document.

#### 2.2 Load Factors and Combinations For The LRFD Design Method

Replace the Load Combinations of Section 2.2 with the Load Combinations of Section 6.3.1.2 of this guidance document.

#### 2.7.2 Minimum Seismic Forces.

Revise the following phrase:

However, for storage racks in areas open to the general public, e.g., in warehouse retail stores,  $I_p = 1.5$ ; unless a displacement evaluation of the type provided in Section 6.5 of this guidance document is performed in a given direction in which  $I_p$  may be taken as 1.0 in that direction.

Add note that  $PL_{RF} = 1.0$  for storage racks in areas open to the general public (e.g. in warehouse retail stores).

Need to add statement that unless used to store hazardous material, storage racks are deemed to be Occupancy Category I structures.

#### 2.7.3 Calculation of Seismic Response Coefficient

Replace the Seismic Response Coefficient equations of Section 2.7.3 with the Seismic Response Coefficients of Section 6.2.1.2 of this guidance document except do not include the term “ $T$ ”.

*A preferable option may be to revise the base shear equation  $V$  by deleting “ $I_p$ ” and leaving the “ $I$ ” in the equations. Also replace “ $I_p$ ” with “ $I$ ” in the definition.*

Add note following the clause:

Except that  $C_s$  should not be taken less than  $0.14 S_{DS} I$

Note: For storage racks in areas open to the general (e.g. warehouse retail stores) if a displacement evaluation of the type provided in Section 6.5 of this guidance document is performed in a given direction, the above value of  $0.14 S_{DS} I$  may be taken as 0.01 in that direction.

Definition of  $T$ :

Indicate  $T$  is to be consistent with the stiffness associated with the applied rack forces  $V$ . In the down aisle direction, this requirement is deemed satisfied when  $T$  is calculated using the moment rotational stiffness  $F$  determined in accordance with Section 9.4 and the Commentary to Section 9.4. In the cross-aisle direction provide a suitable reduction factor for the bracing area when computing period. Do not leave the requirements in this area gray.

Add somewhere in section 2.7.3 a displacement based evaluation option based on Section 6.5 of this guidance document.

### **2.7.3.1 Seismic Coefficients $C_a$ and $C_v$**

Replace Section and Title of Section with Section 6.1.2 in mandatory language.

### **2.7.4 Vertical Distribution of Forces**

Revise in accordance with Section 6.2.1.4 of this guidance document.

### **2.7.6 Overturning**

Revise in accordance with Section 6.2.1.4 of this guidance document.

## **7.1 Connections**

Provide procedure for explicitly evaluating connections including how to use test data to evaluate when dealing with seismic loadings.

## 9.0 TEST METHODS

Provide test methods to provide capacity data need to comply with the new optional performance-based procedure of Section 2.7.3 (Based on 6.5 of this guidance document) and the new connection evaluation procedures of Section 7.1.

### COMMENTARY

Revise commentary as necessary to implement the above changes.

In addition, it is recommended that the Horne-Davis method be eliminated from Commentary Section 2.7.3. It is recommended it be replaced by the period calculation equation of Appendix A of this guidance document for the down-aisle direction and using rotational stiffness  $F$ . It is also recommended that a period equation developed for the cross-aisle direction based on the reduced area of the diagonal brace permitted in the new Section 2.7.3 be provided.

*The Horne-Davis may be rational but is not normally used by earthquake engineers. A simpler more direct procedure is therefore recommended.*





## APPENDIX C

### RECOMMENDED MODIFICATIONS TO THE 2003 NEHRP RECOMMENDED PROVISIONS

The following modifications are recommended to the 2003 *NEHRP Recommended Provisions*. The modifications presume that the RMI developed ANSI MH 16.1-04 specification has been modified in accordance with and adopted with the recommendations of Appendix B of this document.

Revise 2003 *NEHRP Recommend Provisions*, Chapter 14 as follows:

#### Section 14.1.2.1 Adopted References.

ANSI MH 16.1-04      Adopt latest approved edition of RMI Specification ANSI MH16.1

**Section 14.3.5.6 RMI storage racks.** Steel storage racks supported at or below grade and designed in accordance with Section 2.7 of ANSI MH 16.1 shall be are deemed to satisfy the force and displacement requirements of these *Provisions*. For storage racks supported above grade, the value of  $V$  in Section 2.7.2 of the RMI shall not be taken less the value of  $F_p$  determined in accordance with Section 6.2.6 of these provisions where  $R_p$  is taken equal to  $R$ , and  $a_p$  is taken equal to 2.5. if all the following conditions are met:

DELETE CONDITIONS 1, 2 and 3.



## APPENDIX D

*The information presented below was issued by the City of Los Angeles Department of Building and Safety as Information Bulletin/Public-Building Code, Reference L.A.M.C. 91. 1806, Document P/BC 2002 to be effective May 10, 2004.*

### ACCEPTABLE DESIGN AND ANALYSIS METHODS FOR USE OF SLABS-ON-GRADE AS FOUNDATIONS

**PURPOSE:** This Information Bulletin establishes a list of acceptable analysis methods for slabs-on-grade (“SOG”) as foundations.

**ACCEPTABLE DESIGN METHODS:** The following methods of design and analysis for SOGs are acceptable:

- ACI Committee 360, “Design of Slabs-On-Grade - Reported by ACI Committee 360,” ACI 360R-92, 1997.
- Packard, Robert G., “Slab Thickness Design for Industrial Concrete Floors on Grade,” IS195.01D, Portland Cement Association, Skokie, Illinois, 1976.
- Departments of the Army and Air Force, Concrete Floor Slabs on Grade Subjected to Heavy Loads,” ARMY TM 5-809-12, Air Force AFM 88-3, Chapter 15, 1987.
- Department of Defense, “Engineering and Design: Rigid Pavements for Roads, Streets, Walks and Open Storage Areas,” TM-5-822-6, U.S. Government Printing Office, Washington D.C., 1977.
- Wire Reinforcement Institute, “Formulas for Success: Innovative Ways to Reinforce Slabs-On-Ground,” TF 705-R-03, 2003.
- Post-Tension Institute, “Design and Construction of Post-Tensioned Slabs on Ground,” Phoenix, AZ, 1980.

The following are additional acceptable methods presented to the SEAOSC membership during a series of seminars held in March 2003 and available from SEAOSC:

- Equivalent Footing - Analysis of allowable loads is modeled by assuming a “saw-cut” square unreinforced section using the conventional working stress method.
- Integral Footing - Analysis of SOGs strength using empirical equations developed by the American Concrete Institute. (Design and Construction of Concrete Slabs on Grade, ACI SCM-11(86), American Concrete Institute, Detroit, 1980).

- Empirical Method - Method of analysis based on studies that compare the load test results to computer analysis. (Shentu, L., Jiang, D., Hsu, T. (1997). "Load Carrying Capacity for Concrete Slabs on Grade." Journal of Structural Engineering, ASCE, January 1997, pp 99-103.)
- Evaluated the limitations of a chosen method for the intended application based on assumptions documented on the plans.

The following parameters must be given consideration in design and proper documentation supplied to justify assumptions of field conditions:

1. Specified Compressive Strength of Concrete,  $f'c$ :
  - a. assumed to be 2,000 psi., or
  - b. core tests conducted per Section 91.1920.2.3 of the Los Angeles Municipal Code ("LAMC").
2. Soil Capacities:
  - a. assume Allowable Soil Bearing Pressure to be 500 psf., or
  - b. assume Standard Modulus of Subgrade Reaction, K30, to be 60 pci., or
  - c. foundation investigation approved by the Department, or
  - d. verified by documentation of approved compacted fill, as determined by ASTM D1557
  - e. in Section 91.7011.3 of the LAMC and on file with the Department.
3. Reinforcement steel location, size, and spacing
  - a. SOG load capacity may be analyzed assuming unreinforced concrete, or
  - b. SOG reinforcement verified by documentation from drawings on file with the Department, or
  - c. SOG reinforcement verified by tests.
4. Slab thickness:
  - a. verified with information from drawings on file with the Department, or
  - b. test methods listed in the code, or
  - c. interior SOGs of commercial or industrial buildings may be assumed to be 3 inches thick
  - d. without testing, or
  - e. interior SOGs of residential buildings may be assumed to be 2 inches thick without testing.

**INSPECTION:**

1. Structural Observation: The Structural Observer shall use the criteria established on the plans to confirm the installation of structural members supported by SOGs relative to cracks, slab edges, piping, conduit and other slab openings. The Structural Observer shall report findings to the Building Inspector pursuant to Section 91.1702.5 of the LAMC.

2. Field Verification by the Building Inspector:

- a. The thickness of new SOGs will be confirmed during inspection of holes drilled in the concrete slabs for attaching anchor bolts shown on plans.
- b. The size, location and spacing of reinforcement in existing SOGs shall be confirmed using
- c. methods approved by the Department as indicated on the approved plans.



## APPENDIX E

### BSSC RACK PROJECT REVIEWERS AND WORKSHOP PARTICIPANTS (\* identifies participant in October 15, 2004, Rack Project Workshop in San Francisco, California)

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The **Building Seismic Safety Council (BSSC)** was established in 1979 under the auspices of the National Institute of Building Sciences as an entirely new type of instrument for dealing with the complex regulatory, technical, social, and economic issues involved in developing and promulgating building earthquake hazard mitigation regulatory provisions that are national in scope. By bringing together in the BSSC all of the needed expertise and all relevant public and private interests, it was believed that issues related to the seismic safety of the built environment could be resolved and jurisdictional problems overcome through authoritative guidance and assistance backed by a broad consensus.

The BSSC is an independent, voluntary membership body representing a wide variety of building community interests. Its fundamental purpose is to enhance public safety by providing a national forum that fosters improved seismic safety provisions for use by the building community in the planning, design, construction, regulation, and utilization of buildings.

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