A Case Study on the Collapse of Industrial Storage Racks

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Abstract

The collapse of industrial storage racks poses a safety hazard to building occupants and can result in costly property damage and subsequent business interruptions. Similar to building failures, storage rack collapses are typically caused by a combination of causative factors, such as deficient storage rack design, construction, and materials; and/or improper maintenance and operational procedures.

A forensic case study of failure of industrial storage racks is presented herein. This case study will highlight forensic investigation methodology, industrial storage rack installation guidelines; and how a combination of improper installation, maintenance, and operational procedures led to a catastrophic failure of a warehouse storage rack system. Methodology for, and results of, structural analysis of the subject rack system is presented. The authors will demonstrate that current installation guidelines do not provide for adequate stability of storage rack systems and propose that a change from guidelines to prescriptive requirements will provide a safer environment for building occupants and protect against costly property damage and business interruptions due to storage rack collapses.

Introduction

In the early morning hours of July 5, 2007 a motion detector for the security system of a warehouse in Carrollton, Texas was triggered by the collapse of more than half of the industrial storage racks installed in the building (**Figure 1** and **Figure 2**). The collapse occurred approximately 30 minutes prior to the warehouse being operational for the day.

Two of the authors were retained to investigate the cause of the collapse of the industrial storage racks.

Figure 1: Collapsed Storage Racks **Figure 2:** Collapsed Storage Racks

Background

__**Storage Rack Design**__

Industrial storage rack systems are engineered structures consisting of cold-formed steel shapes, hot-rolled steel shapes, or a combination thereof. Such racks are usually designed as a series of two-dimensional planar frames with semi-rigid joints (Sputo and Turner, 2006). At the time of the authors' investigation of the storage rack collapse, the accepted industry standard for the design and maintenance of industrial racks was American National Standards Institute (ANSI) MH16.1 – 2004, *Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks*, published by the Rack Manufacturers Institute (RMI). This ANSI/RMI standard (RMI 2004b) prescribes that cold-formed steel elements in racks should be designed in conformance with the American Iron and Steel Institute (AISI) *North American Specification for the Design of Cold-Formed Steel Members* (AISI 2001b), as modified by the ANSI/RMI standard (Sputo and Turner 2006).

Industrial rack columns are designed as pure compression members that must resist pure flexural and flexural-torsional buckling (Sputo and Turner 2006). Flexural-torsional buckling is typically the governing critical buckling mode of rack structures (RMI 2004a). However, rack frame geometry must be considered, as eccentric loading conditions will cause a combined axial/bending effect, which may supersede flexural-torsional buckling as the governing failure mode.

A brief discussion of each of the aforementioned buckling modes is presented below (AISI 2001a; McCormac and Nelson 2003; Salmon and Johnson 1990).

- **Flexural (Euler) buckling** is the primary buckling mode of steel members with doubly-symmetric, closed, or symmetric cross-sections. This type of buckling occurs when the ratio of the unbraced length of a member to its stiffness is too large. Flexural buckling results in a straight member becoming bow-shaped. Such buckled members may lose all load-carrying capacity and fail under a sufficient compressive load.

- **Flexural-torsional buckling** may occur in columns with singly-symmetric or non-symmetric cross-sections. This failure mode is a combination of flexural buckling and torsional buckling. Flexural buckling is discussed above. Torsional bucking results in failure of columns by *twisting* about their longitudinal axis (without bending) due to torsional forces resulting from non-symmetry and/or imperfections in the member's cross-section. Therefore, flexural-torsional buckling results in a straight member simultaneously becoming bow-shaped *and* twisted about its longitudinal axis.

Eccentric loading conditions may cause an axial/bending failure of industrial rack columns. Eccentric loading occurs when the load applied to a column is not applied through the center, or centroid, of the cross section, inducing a *bending moment* in the column. Such a condition is induced when axially loaded members are out-of-plumb from vertical. When a column is subjected to a bending moment, it will be displaced laterally in the plane of bending. As a result, an even larger moment is applied (equal to the axial compression load times the lateral displacement, or eccentricity) (McCormac and Nelson 2003). This is termed the "p-delta (PΔ) effect," and often results in structural failure of the member.

Eccentric loading conditions of columns will result in an additive **combination of axial forces and bending forces**. Such interaction of the two load effects occurring at the same time must be considered when analyzing a column subject to eccentric loading.

Storage Rack Design and Maintenance Guidelines

As stated previously, the accepted industry standard for the design and maintenance of industrial storage racks during the time of the authors' investigation was the ANSI MH16.1 – 2004, *Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks*, published by RMI. However, the guidelines and standards published by RMI are **not** prescriptive and/or code requirements. In fact, the disclaimer within RMI's publication states:

> *The acceptance or use of this specification is completely voluntary. Its existence does not in any respect preclude anyone, whether the specification has been approved or not, from manufacturing, marketing, purchasing, or using products, processes, or procedures not conforming to this specification.*

The following guidelines, selected based on their applicability to the case study presented herein, are stated in RMI's publication:

- Section 1.4.7 of the standard states *All rack columns should be anchored to the floor by anchors capable of resisting the forces caused by the horizontal and vertical loads on the rack*. Furthermore, the RMI/ANSI commentary states: *…all racks should be anchored to the floor. The anchor bolts should be installed in accordance with the anchor manufacturer's recommendations. Anchors serve* *several distinct functions:*

1. *Anchors fix the relative positions of, and distances between, neighboring columns.*
2. *Anchors provide resistance against horizontal displacements of the bottom ends of the columns. A tendency for such horizontal displacement may result from external lateral forces or from the horizontal reactions resulting from the rigid or semi-rigid frame action of the rack. If such shear forces would in fact cause horizontal displacements of the bottoms of the columns, this would reduce the load carrying capacity of the rack as compared to computed values.*
3. *For particularly tall and narrow racks, anchors may significantly increase the stability against overturning.*

- Section 1.4.9. of the standard states *Upon any visible damage, the pertinent portions of the rack shall be unloaded immediately by the user and the damaged portion shall be adequately repaired or replaced.* Furthermore, the RMI/ANSI commentary states:

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

  …this section addresses two possible ways to safeguard racks against the consequences of minor collisions…

  The first way is the provision for protective devices that will prevent trucks from hitting the exposed columns…

  A second method of safeguarding the rack upright is to reinforce the bottom portion of the front column and/or bracing in the frame.

- Section 1.4.11. of the standard states *To assure adequate plumbness, the maximum tolerance from the vertical is 0.5 inches in 10 feet of height.*

Case Study

Storage Rack Structural Configuration

Although a large portion of the storage racks within the subject warehouse collapsed, a portion of the racks remained standing. The racks still standing within the warehouse were observed and measured by the authors to serve as a general representation of the storage rack systems (**Figure 3**). The typical spacing of the aisles between the rack columns was approximately 56". The racks consisted of 3"x3" green painted vertical upright frames that were supported by horizontal and diagonal braces. The vertical frames had footplates that were pre-punched for connection to the floor. The footplates of the collapsed racks did not have any anchors connected to them. Also, no anchors were observed in the slab-on-grade foundation of the collapsed area. The racks that were still standing were fastened to the slab-on-grade with anchors. Additionally, the authors observed that only some of the uprights were anchored in the non-collapsed storage racks, and that none of the rack column legs were leveled with shims.

Orange coated horizontal beams were connected to the vertical frames by 3-point, teardrop connectors. The first level of shelving consisted of particleboard shelves supported underneath by 2x4s. The rest of the shelving levels were wire decking. Each rack column contained six shelving levels, and each rack row was 15 to 18 units long. No guardrails or bollards to protect the racks from equipment impact were observed, and no reinforcement of the front column or bracing in the frame was present.

At some locations, a metal bar was observed connecting abutting rack assemblies. These "back ties" were observed between the rack shelves at some of the non-collapsed racks. No back ties were observed at the collapsed racks.

Plumbness Testing

The authors observed that many of the columns were out of plumb from vertical *below the first beam of the racks* (**Figure 4** and **Figure 5**). However, column sections above the first beam appeared approximately plumb. Plumbness measurements were taken with a Stanley® SmartTool™, which is a four-foot level instrument, on the bottommost column sections (between the floor and the first beam) of the racks that were still standing at the south side of the warehouse. A reading of 90.0° indicates that the column is true and plumb with the vertical, whereas each 0.1° difference from 90.0° is equivalent to slightly more than 1/12" change over four feet from the vertical (i.e. 89.7° is approximately 1/4" out-of-plumb over four feet).

Using the plumbness measurements, the authors calculated the base displacement (Δ_{base}) about each axis, calculated from the geometry of the rack. Since the plumbness provisions of ANSI/RMI are expressed in inches of displacement over 10', and NAE measured plumbness of a 4' section, the measured displacement over 4' was extrapolated to an equivalent displacement over 10'. Using trigonometric relationships, the total (Δ_{total}) combined displacement from vertical (independent of axis) was calculated. The total displacement of the column base over 10' was checked against Section 1.4.11 of the ANSI/RMI standard for compliance.

Of the 58 columns on which plumbness measurements were recorded by the authors, 44 (approximately 76%) were not in compliance with the ANSI/RMI standard for plumbness. Ten of the measured members were out of plumb more than 0.8" in 4' feet, which extrapolated to 2" in 10'. The most severely leaning member was out of plumb 6.2" in 4', or 15.7" in 10'.

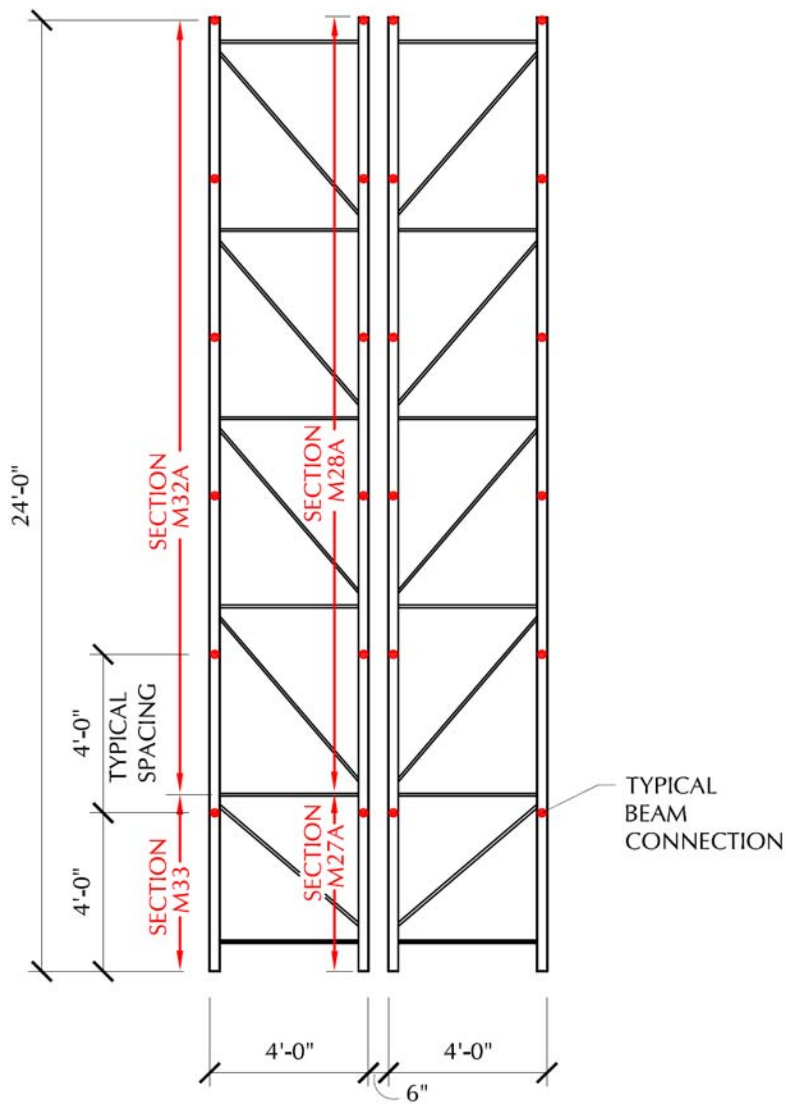

Figure 3: Storage Rack Frame Configuration

Figure 4: Out-of-plumb Column

Figure 5: Out-of-plumb Column

Structural Analysis

The authors performed structural analyses of the critical column section of the subject racks and a series of rack frame configurations. The basis for those analyses and results thereof are presented below.

Critical Column Section Analysis. The authors calculated geometrical cross-section properties in accordance with the provisions of Section 3.3.2 of the *AISI Manual: Cold-Formed Steel Design* (AISI 2002) and based upon field measurements of the members. The structural analysis was performed in accordance with the provisions of the AISI specification (AISI 2001b), as modified by the ANSI/RMI standard (RMI 2004b).

The following were considered in selecting the column section most likely to fail under load, or the *critical section*:

- **Unbraced Length.** Since all of the subject column sections exhibited identical cross-sectional properties, under equivalent loading the section with the largest unbraced length was most likely to fail.

- **Applied Load.** Loads were applied to the subject columns at successive points along the length of each member. Given equivalent cross-sectional properties and unbraced lengths of each section, the section subject to the greatest loading was most likely to fail.

The section of the subject columns with both the largest unbraced length and the greatest loading was the bottommost section, extending from the base of the column to the first beam connection point. Thus, the critical section of each column extended upward from the base of the column to the first beam connection point.

The yield stress of the steel was taken as 50 kips (1 kip = 1000 lb) per square inch (ksi), as the columns were to be formed of high-strength, low-alloy steel, based upon the manufacturer's literature. The unbraced length of the critical section about the critical buckling axis was taken as 48" (L_x = 48"). While typical first beam height was measured as 48" from the floor, some measurements in the range of 56" to 90" were also obtained by the authors. Beams installed at an elevation greater than 48" would result in a longer unbraced length of the column, thus reducing the load carrying capacity of the column.

The effective length factor for flexural buckling about the same axis was taken as 1.7 (K_x = 1.7), as per the ANSI/RMI standard, Section 6.3.1.1 for frames not braced against sidesway. The unbraced torsional length was taken as 41" (L_t = 41"), the length of the critical section unsupported against twisting. The effective length factor for torsional buckling was taken as 0.8 (K_t = 0.8), as per the ANSI/RMI standard, Section 6.3.3.2. The authors measured the thickness of steel comprising the upright columns as 0.079", but used 0.075" (t = 0.075"), the thickness of 14 gage metal specified by the American Institute of Steel Construction (AISC), to account for paint on the cross-section.

The strength of an axially loaded column is proportional to the cross-sectional area of the column. Perforations, such as those found in rack columns, reduce the effective cross-sectional area of the column, and thus reduce the load-carrying capacity of the column. The effect of perforations on the load-carrying capacity of these columns is accounted for by RMI's modification of the AISI specification by use of an *effective area* (RMI 2004b).

The effective area used in the authors' analysis was approximated as the gross cross-sectional area minus the area of the perforations at the critical section as calculated by a computer-aided drafting software program. The effective cross-sectional area was taken as 0.60 sq. in. (A_e = 0.60 sq. in.).

The authors' calculated a flexural buckling load ($P_{flexural}$) of 40,365 lbs, a yielding load ($P_{yielding}$) of 30,000 lbs, and a flexural-torsional buckling load ($P_{tors/flex}$) of 17,798 lbs. The critical failure mode ($P_{critical}$) for the concentrically loaded column was flexural-torsional buckling. Each column, if axially loaded through the centroid of the member cross section, could have supported approximately 17,800 lb without failing. The allowable design load for each column ($P_{allowable}$, $P_{critical} \div$ safety factor) was 9,888 lb.

Global Frame Analysis. The authors conducted a second-order, non-linear analysis of the vertical frames using RISA-3D, a commercial structural analysis software package. Such computerized analysis allows simultaneous consideration of all possible failure modes of the frame and accounts for second-order ($P\Delta$) effects, which often dictate the failure mode of frames.

The global frame geometry was modeled based on field measurements; custom shapes were defined with geometrical and structural properties calculated from field measurements. Geometrical and structural properties used for the frame analysis were the same as used for the aforementioned *critical column section analysis*.

Two back-to-back frames were modeled, identical to the configuration observed at the site of the collapse. The bases of the frames were modeled as restrained against translation about all axes and free for rotation about all axes. This is a conservative assumption, as the bases were not anchored to the ground, and thus allowed to translate once frictional forces were overcome. Furthermore, some rotational resistance was provided by the geometry of the baseplate. The frames were modeled as restrained against side-sway in-plane and unrestrained against side-sway out-of-plane, as per Section 6.3 of the ANSI/RMI standard. The beams provided a degree of bracing against out-of-plane buckling, so out-of-plane translational restraint was applied at each point on the end frame columns at which a beam-column connection occurred. In some model cases, member deflections caused a significant out-of-plane force at the first beam-column connection from the ground. In those cases, the translational restraint was removed at that point.

The specified maximum planned loading for the racks was 108, 30-pound boxes, or 3240 lb., per level. Distributing this load evenly to each of two beams results in a load of 1620 lb. per beam. As such, an 810 lb. load was applied to each column from each beam. For interior frames, two beams framed into each column at the same location, thus a 1620 lb. point load was applied to the model at each point on the end frame columns at which a beam-column connection occurred. The self-weights of the members were included in the analysis.

Two sets of eight structural models were analyzed. The first set of models included backties installed between the rack frames at every 66" interval from the floor. The second set of models did not include backties. Each set included one frame pair modeled as perfectly plumb (vertical). Each set also included one frame pair with the base of one bottommost column section (between the floor and the first beam) displaced out-of-plane in 1" increments from 1" to 7", representative of the authors' field observations and measurements. Purely out-of-plane displacement was assumed for model simplicity and is a conservative approach, since most members were displaced both in-plane and out-of-plane, resulting in a total displacement greater than that of the out-of-plane displacement alone. The authors also conservatively assumed that only one column per pair of frames was out of plumb.

Results of Analysis. The stresses in each member under the applied load were calculated through the structural frame analysis and compared to the allowable stresses, as determined according to the provisions of the *AISI Specification for the Design of Cold-Formed Steel Structural Members* (AISI 2001b). Shear and bending stresses were evaluated, as were all three axial buckling modes: flexural, torsional, and flexural-torsional. The combination of bending and axial stress was considered, and was found to be the failure mode of the critical column section, and thus of the frames.

The results of the authors' structural analysis are summarized in **Table 1**, in the form of the ratio of applied stress to allowable stress. A value greater than 1.00 indicates that the applied stress is greater than the allowable (design) stress for that section, but *does not indicate physical failure of that section*. The allowable stress is the ultimate (failure) stress of the section divided by a safety factor. Due to the interaction between bending and axial stresses, the safety factor for this stress condition is a hybrid of the safety factor for each individual condition, Ω_{axial} = 1.80 and $\Omega_{bending}$ = 1.67. The safety factor applied to the ultimate stress for the combined axial and bending stress condition may be conservatively taken as 1.67. Thus, an applied stress to allowable stress ratio of greater than 1.67 indicates likely physical failure of the subject section and resulting failure of the rack frame. Refer to **Figure 3** for the location of each frame section.

Table 1: Summary of Results of Structural Frame Analysis

(Applied Stress / Allowable Stress)									
		Without Backties							
Δ_{base} at M33		0"	1"	2"	3"	4"	5"	6"	7"
Section	M32A	0.96	1.33	**1.71**	**2.09**	**2.48**	**2.88**	**3.29**	**3.72**
	M33	1.00	1.33	**1.68**	**2.03**	**2.39**	**2.76**	**3.14**	**3.53**
	M28A	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
	M27A	0.99	1.00	0.99	0.99	1.00	1.01	1.02	1.04
		With Backties							
Δ_{base} at M33		0"	1"	2"	3"	4"	5"	6"	7"
Section	M32A	0.92	1.22	1.52	**1.79**	**2.01**	**2.19**	**2.32**	**2.41**
	M33	0.97	1.24	1.51	**1.75**	**1.96**	**2.12**	**2.24**	**2.33**
	M28A	1.03	1.03	1.05	1.07	1.10	1.14	1.18	1.23
	M27A	1.12	1.12	1.14	1.21	1.26	1.33	1.34	1.41

As shown in **Table 1**, the *vertical frames* were of sufficient strength to support the maximum specified loading under concentric and slightly eccentric loading conditions. Analysis showed that the frames *without backties* were structurally adequate to resist the maximum specified loading, providing that the base of one of the columns was not displaced more than approximately 1" from vertical. The frames *with backties* were analyzed to be structurally adequate to resist the maximum specified loading, providing that the base of one of the columns was not displaced more than approximately 2" from vertical. The backties controlled in-plane deflection of the racks caused by typical deformation under load, and thus increased the stability of the racks by minimizing the second-order (PΔ) effect of eccentric loading.

Conclusions

Based on observations and analysis, the authors came to the following conclusions regarding design, installation, and maintenance of the subject storage rack system:

- The *columns* were of sufficient strength to support the maximum specified loading *under concentric loading conditions*. The allowable design load for each column was 9,888 lb. Based on the maximum loading specified at the warehouse, each column would have been subject to a maximum 9,720 lb.

- The *vertical frames* were of sufficient strength to support the maximum specified loading under concentric and slightly eccentric loading conditions. Analysis showed that installation of backties between adjacent frames increased the stability of the frames.

- Backties were not installed on the collapsed racks. Installation of backties between adjacent frames would have increased the stability of the frames and may have prevented the collapse.

- Safeguards were not in place to protect the subject storage rack system from impact damage by equipment. Rack columns were out of plumb as a result of equipment impact.

- The failure to positively anchor column bases to the floor allowed column bases to be displaced, allowing for eccentricities, or out-of-plumbness, of the columns. Many column sections of the frames that remained standing were out of plumb and did not meet the provisions of the ANSI/RMI standard for plumbness.

- Out-of-plumb column sections caused eccentricities more severe than that which the frames were designed to resist. These eccentricities, combined with the absence of backties, caused the collapse of the racks.

Recommendations

Through their investigation, the authors determined that the collapse of the subject storage rack system was caused primarily by failure to meet construction and maintenance standards set forth in the American National Standards Institute MH16.1 – 2004, *Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks*, published by the Rack Manufacturers Institute.

The guidelines and standards published by RMI are **not** prescriptive and are not adopted or referenced by governing building codes. In fact, the disclaimer within the publication states that the "*acceptance or use of [the] specification is completely voluntary."*

It is the authors' opinion that industrial storage racks are structural systems which affect the safety and welfare of the public; and thus should be designed by a licensed and qualified structural engineer based on prescriptive requirements. The design engineer should provide for positive anchorage of the rack bases to the building foundation, and ensure adequate bracing for lateral stability. A structural analysis should be conducted for the global geometry of each rack installation to ensure adequate strength and stability of the configuration to be installed. Furthermore, protection of the bases of rack systems must be provided to prevent impact damage.

The authors recommend that ANSI/RMI and building code committees consider requiring prescriptive provisions for the design, analysis, and construction of industrial rack systems. A change of standards from *guidelines* to *requirements* will provide a safer environment for building occupants and protect against costly property damage and business interruptions caused by storage rack collapses.

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