Design Recommendations for the Improvement of the Seismic Performance of Steel Storage Racks

Barry J. Davidson, Anthony P. McBride
Compusoft Engineering Ltd, Auckland, New Zealand.

ABSTRACT: The use of steel storage racks has become more common in retail situations where their failure during an earthquake could give rise to “life safety” issues. The paper describes the differences between storage racks and more regular building frames and how these differences are expected to alter their seismic performance. The current seismic design criteria for storage racks are compared with that for building frames and it is concluded that these frames are typically designed to a much lower strength than what would be required of standard structures.

1 INTRODUCTION

1.1 Recent Failures

The recent Darfield and Christchurch earthquakes will be remembered for the collapse of many structures and the subsequent death of almost 200 people. They will also be remembered for the debilitating effect of liquefaction, giving rise to broken houses, sewers, roads and other facilities. For the structural engineering fraternity, the observed survival of engineered and retrofitted buildings is hopeful, but hidden behind mainly unscathed warehouse walls have been a large number of collapsed steel storage racks. These failures are noted with the sobering knowledge that had people been in the vicinity of the racks at the time of ground shaking, their failures could have caused additional deaths and injury.

While racking frames have been used for storage in warehouses for many years, they have over recent time become more common in modern supermarkets and retail mega stores. Their failure in these situations increases the likelihood of injury or worse to the public. It was with this in mind that BRANZ published a design guide “Seismic Design of High Level Storage Systems with Public Access” (BRANZ 2007) in 2007. The failures that occurred during the Darfield earthquake have instigated a review of this guide with the intent of improving it if necessary and extending its scope. This paper is intended to support that review.

Attempts have been made and will still need to be made to determine the causes of the failures of these frames. For example, to determine whether it was buckling of the columns or failure of the welds etc. that instigated a specific failure. While the results of these attempts may lead to changes in design and/or operational practice, it is the authors’ opinions that the results of these investigations will in the main be inconclusive. The reasons for this are; the physical condition of the frames and the weight of product supported at the time of the earthquake are unlikely to be determined, and, the actual seismic motions at a specific site, will in the main remain a speculation. In addition, immediately following the earthquake, to determine the actual failure that initiated a collapse from the conglomeration of product and distorted frames on warehouse floors is next to impossible. Investigators did not have the time nor resources to pick through the rubble and identify all the failed members. And even if they had, only a full analysis might provide an answer to what initialized the failure of a specific frame.

Consequently, the approach in this paper is to assume that engineering knowledge is sufficient to design, erect and operate a storage rack that will not fail during a design level earthquake. Based upon this assumption it is possible to review what could have gone possibly wrong.
As a result of the Darfield earthquake, there were a range of different forms of failures of racking frames ranging from complete collapse to situations where the rack survived, but product fell to the floor. At a higher level, a review will show that the reasons for these failures were:

1. Little or no design of the racking systems
2. Incorrect erection of the racking systems
3. Incorrect operation (over loading) of the racking systems
4. Insufficient maintenance of the racking systems (possible damage from forklifts)
5. A phenomenon in the seismic performance of racking frames that is not understood (this possibility is excluded in this paper)
6. Nonconservative design of the racking systems
7. A combination of the above.

What has been noticed from the surveys of damage, is that there are little or no reports of fully or overloaded racking frames adequately surviving the shake. This information has to be viewed in light of the knowledge that the buildings that housed the damaged racking frames were essentially undamaged during the earthquake.

This paper will concentrate on the possibility that some failures may have been a consequence of reason #6 and assume that the BRANZ Guidelines were used in the design of the racking system. With this assumption, the possibilities are that the Guidelines were either used incorrectly and/or the Guidelines themselves require improvement. This allows for a critical review of the Guidelines for both quality of information and clarity of presentation.

2 TYPICAL STORAGE RACK CONFIGURATION

![Figure 1 Typical Storage Rack Configuration (FEMA 460:2005)](image)

A sketch of a typical storage rack system is shown in Fig. 1. It is defined by a number of braced
frames placed in parallel in the “cross aisle” direction. These are connected together by beams which are parallel to the “along aisle” direction. The members of the braced frames are cold formed open sections with the columns highly perforated to allow a choice in height of the beams that support product. The braces may be spot welded as shown in Figure 2 or bolted to the columns allowing the possibility of eccentric connections. Almost in all cases brace forces will be required to transfer through the column section of the joint zone.

![Figure 2 Typical Rack Bracing Members and Connections (FEMA460 2005)](image)

Base plates are welded to the columns and bolted to a concrete floor slab. In practice base plates may have one or two bolts configured in order to develop some moment support in the cross aisle direction and possibly the along aisle direction.

The braced frames are connected by beams in the along aisle direction, two for each level to provide a series of portals. The beams are typically thin walled closed box sections with end fittings that allow them to be “clipped” on to the columns at any height. This usually means that the height of the beams seldom coincides with height of “joints” of the braced frame. Product is typically stored on pallets, which are supported by the beams with two pallets per level per bay being usual.

3 DIFFERENCES BETWEEN A RACKING AND BUILDING FRAME

Despite initial apparent similarities between racking and building frames there are a number of essential differences that influence their seismic design and performance.

- Racking frames have no rigid diaphragm at each level. The concept of floor levels with rigid diaphragms is assumed in the design guidance provided by AS/NZS1170 and it is easy to overlook the simplicity that this assumption introduces to that Standard. Racking frames, depending upon
the pallet configuration and level of loading, may have bays with “no” diaphragms and others with a “flexible” diaphragms.

- The cross aisle braced frames of the racking system are constructed from open section thin walled members. The consequence of this is that the joint zones are more flexible than would be considered usual for a building frame.
- A major difference also occurs in the along aisle direction. The beams that help provide the portal action are “clipped” to the columns. The consequence of this erection procedure is that the “clipped” joint provides a large proportion of the flexibility of the frame to movement in that direction. Cyclic testing of a frame provides hysteretic curves that more closely resemble those of a “tension only” cross braced frame than a moment frame. The “clipping” connection also provides joints that are essentially “pinned” about a vertical axis with regard to the beam’s connection to the columns.
- The present practice of not securing the pallets to the beams allows for the possibility of pallet sliding. This introduces the possibility of a very complex (and “unpredictable”) seismic response.

4 FEATURES OF DESIGN TO THE BRANZ (2007) GUIDELINES

In this section, features of the BRANZ Guidelines that differ (or should differ) from standard practice are listed and discussed. This is done to highlight the design procedures for racking frames and identify possible inconsistencies in the current procedures.

- The performance expectations of a racking frame are stated by the Guide to be those required by the New Zealand Building Code (NZBC 2007):

(i) Life Safety: Interpreted to be
   (a) No loss of contents
   (b) No rack collapse
   (c) No rack overturning
(ii) Collapse Prevention: Interpreted to be
   (a) No rack collapse
   (b) No rack overturning

These follow closely the suggestions from FEMA (FEMA 2005) and referred to as the “Design Basis Earthquake (DBE)” and “Maximum Credible Earthquake (MCE)” design levels respectively. The parallel in NZS1170.5 nomenclature is that Life Safety is the “Ultimate Limit State (ULS)” design situation. “Collapse Prevention” is somewhat assumed to be achieved for the MCE event if you use that Standard for a “building” structure. There is no guarantee that the use of the Standard will provide collapse prevention for a “racking” frame.

- The Guidelines state that they have been developed with reference to the Loadings Standards AS/NZS1170.0 and 1170.1 and NZS1170.5, Material Standards AS/NZS4600:2005, NZS3403:1997 and AS 4048:1993. It is further stated that that the use of AS1538:1988 maybe used for the calculation of properties of perforated members. Through the Guide, reference is also made of the Rack Manufacturers Institute document “Specification for the Design, Testing and Utilization of Industrial Storage Racks” (RMI 1997) and it appears that formulae from FEMA 460:2005 are represented.

- The Guide requests the use of the AS/NZS1170 limit state equations,

\[ 1.2G + 1.5Q \]  
\[ G + \varphi Q + E_u \]  

(1)  
(2)

Where, “G” is the weight of the rack
“Q” is the “superimposed live load or “contents of the rack”,

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“E_u” is the load effects for the ultimate limit state earthquake, and
“ψ_c” is the a combination factor.

An issue the authors’ have with the approach of the Guidelines is that the pallet loading is considered a “live” (by implication “floor”) load, whereas it our opinion that it is more appropriate to consider it as superimposed “dead” load. Thus for Eq. 1, the factors “1.2” and “1.5” have been developed using statistics based upon building self weight and floor loading surveys. NBS577 (NBS577 1980) describes their development. As the weight of racking frames are known far more precisely than the weights of buildings, and pallet weights are far more accurately known for a specific application than the live loads on floors, it can be resolved that these factors are greater than required. Alternative values need to be determined, and, if expected values were chosen for these terms, one might consider
\[ 1.1G & 1.2Q_{sl} \]  
(3)
to be a more appropriate form of the first equation.

For the use of the second equation the Guide recommends:
1. For cross aisle frames and along aisle frames with one or two bays, \( \psi_c = 1 \).
2. For along aisle bays with three or more bays, \( \psi_c = 0.6 \).

The reasoning for these choices appear to be influenced by the Loadings Standard values and the confusion with live “floor” loads. As the beams are “clipped” to the columns, they maybe considered “pinned” so the expected weight effect of the pallet is more likely to be equal to the maximum design loading of the frame at the time of an earthquake. As a consequence of this, the authors’ would prefer \( \psi_c = 1 \) used in both directions.

- The BRANZ guideline (Section 3.3) recommends that for inertia considerations, the seismic weight at each beam level “i” is to be calculated from
\[ W_i = G_i & \psi_E \psi_M Q_i \]  
(4)

Where \( \psi_E \) is an “area” reduction factor
- \( \psi_E = 1 \) for cross aisle direction and along aisle direction with 1 or 2 bays
- \( \psi_E = 0.8 \) for along aisle for 3 or more bays

\( \psi_M = 0.67 \) is a rigid mass factor for friction loss.

In addition, it is stated in the Guide that the seismic weight supported by a beam attached to the frames at 300mm or less from the floor need not be accounted for.

It is understood that the choice of \( \psi_E = 1 \) for across aisle is made assuming that it is imposed as a consequence of the flexible diaphragm assumption. Thus the choice of \( \psi_E = 0.8 \) in the along aisle direction inherently accepts a rigid diaphragm action in that direction. This may be reasonable, though flies against the recommendation in section 5.1 of the Guide warning that “the loss of a single rack member can cause failure of the entire racking system”. Consequently, the authors would suggest \( \psi_E = 1 \) for both directions.

In the opinion of the authors’, the recommendation of \( \psi_M = 0.67 \) cannot be justified. It appears that it may follow a recommendation from the FEMA 460 report which recommends a similar factor, provided that the Equivalent Static Method (ESM) of analysis is used in design process. From a comparison of base shears calculated from the ESM with the more accurate Response Spectrum Method (RSM) there is some basis for that recommendation. However it is in conflict with the procedures spelt out in NZS1170.5. The concept to use friction to limit the lateral inertia forces to the frames is attractive but unsubstantiated. The presentation in the Guide is inconsistent with our knowledge of the expected behaviour of the frames for the following reasons;
1. In regions of low seismicity, the design accelerations may be insufficient to initiate sliding, so the full inertia forces are required.
2. In regions of high seismicity, sliding is unlikely to occur at the lower beams levels.

In addition, apart from the observation that after an earthquake it is common to observe that pallets have slid off the beams, spilling their contents and possibly initiating some form of collapse; there is little evidence that supports the proposal that a design approach can take advantage of friction. There is insufficient knowledge to allow designers to calculate the amount of sliding; the sliding that limits the demands on the frames, with a sufficiently small amount of “slide” so as not to threaten life safety. It is the opinion of the authors, that further research is required to take advantage of this design approach. Until a defensible design methodology has been presented, the recommendation is that $\psi_E = 1$ should be used in design. Further, either pallets or the rack beams should be modified to eliminate sliding in the cross aisle direction.

The anomaly that allows the non-inclusion of the weight supported by beams that are less than 300mm above the floor in the calculation of seismic weight is inconsistent with NZS1170.5. The weight on these beams will cause an increase in the shear in the column, and as it is unlikely that there is bracing at that level, an increase of column and base plate moment.

- The recommendation of the Guide for the determination of the Return Period Factor $R_u$ is to use a design life of 25 years and an Importance Level of 2. When considered with respect of the Performance Expectations stated at the beginning of the Guide, this recommendation is in stark contrast with the design of the structure that is housing the racking system. For life safety, the envelope structure is designed to resist an earthquake that is 33% stronger than the design level used for the racking system. The recommendation of the authors is to increase the design life of racking systems accessible to the public and those that failure could lead to loss of life to 50 years.

- The calculation of seismic base shear follows the NZS1170.5 formulae.

\[
V = C_d(T_r)W_t \\
C_d = \frac{C(T_r)S_p}{k_\mu} \\
C(T_r) = C_h(T_r)ZRN(T_r, D)
\]

(5)  
(6)  
(7)

To calculate the frame period in the cross aisle direction it is recommended by the Guide to allow for joint flexibility by using a brace stiffness equal to a value between $\frac{1}{7}$th to $\frac{1}{12}$th of their basic values, $EA/L$. Other than this there is no guidance as to which values to choose. There are further recommendations to use a ductility factor of 1.25, and $Sp$ according to the NZS1170.5 formula

\[
Sp = 1.3 - 0.3\mu
\]

which are consistent with NZS1170.5.

The guidance for design in the along aisle direction is confusing. There is a mandatory testing regime to determine an effective stiffness, essentially a secant stiffness, that must be used to calculate the along aisle first mode period. The implication that this period be used in the above equations (5 – 7) and the use of a ductility of “2” for design is suggested. This is inconsistent with NZS1170.5 and shown to be flawed in a later section of this paper.

- The Equivalent Static Method is discussed in the Guide. It is not stated that that method must be used and as the calculation of the first mode periods is most easily achieved from a simple model, it is most likely that the Response Spectrum Method could be used for the determination of member forces. This approach along with the $\psi_M$ recommendations described above, would lead to nonconservative designs. If designs require beams that are approximately 300mm above the floor level, it is the authors’ recommendation that frame analyses be performed with $\psi_M = 1$ and the Response Spectrum Method is used taking into account missing mass terms.
5 SEISMIC RESPONSE IN THE ALONG AISLE DIRECTION

Reproduced in Fig. 3 is a plot of typical test result as illustrated in the Guide. Drawn on this plot is a “secant” line (from the origin to the position, $M_{\text{max}}$, $\theta_{\text{max}}$) providing a joint stiffness “$k_c$” upon which the frame stiffness (and first mode period) is based.

Figure 3. Typical Cantilever Test Moment – Rotation Plot (BRANZ 2007)

The positive region of this plot is replicated in Fig.4 with a bounding bilinear curve representing the ASCE (ASCE 2006) approach. It can be seen from these typical results that the ratio of the secant stiffness to an equivalent “elastic” stiffness is in the order of $19559 / 61750 = 0.32$, which would result in the secant period to be $1/\sqrt{0.32}$ times the equivalent elastic period, or approximately $1.8$ Te. It is obviously non conservative to use the NZS1170.5 formulae based upon a period that is 80% longer than the intended value. The authors recommend that the ASCE approach is used in the evaluation of experimental results and the elastic stiffness calculated is then used in conjunction with the design procedures described in NZS1170.5.

Figure 4 Backbone Curve of test data with Bilinear “Fit”
6 CONCLUSIONS AND RECOMMENDATIONS

The paper has attempted to explain the concepts involved with the seismic behaviour and design of racking systems. A comparison of the life safety level design procedures currently recommended for racking systems with those for more standard structures, shows that the lateral strength required for a racking frame is significantly less than that required for similar looking structural frame system. The paper provides recommendations for improvement in the design of racking systems and discusses the reasons for them.

In summary, present design recommendations for racking systems may allow the following unjustifiable (in the authors’ opinion) strength reductions:

(i) $\psi_E$ for a “area” reduction factor
   
   $= 1$ for cross aisle direction and along aisle direction with 1 or 2 bays
   $= 0.8$ for along aisle for 3 or more bays

(ii) $\psi_M = 0.67$ is a rigid mass factor for loss friction

(iv) $R_u = 0.75$ (design life of 25 years)

(v) Period misinterpretation in the along aisle direction provides a 1/1.8 reduction (assuming the periods are in the constant velocity region of the design spectrum).

Thus a quick summary shows that in the cross aisle direction the design strength is approximately 50% ($0.67 \times 0.75$) of a standard frame, whereas in the along aisle direction the strength is much less.

A recommendation from the authors would be to align the revamped Guidelines more closely to NZS1170.

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