Analysis and design of steel storage racks subjected to rocking

James R. Maguire
University of Wollongong

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Analysis and design of steel storage racks subjected to rocking

James R. Maguire

This thesis is presented as required for the conferral of the degree:

Doctor of Philosophy

The University of Wollongong
School of Civil, Mining and Environmental Engineering

&

The University of Auckland
School of Civil and Environmental Engineering

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This research has been conducted with the support of an Australian Government Research Training Program Scholarship.
Abstract

Selective storage racks are used worldwide as a lightweight structure for both temporary and long term storage of palletised goods. These structures are routinely designed for earthquake actions using the equivalent static method, which relies on the fundamental period of vibration to calculate design loads. Selective racks can uplift and rock in the cross-aisle direction during earthquake loading, however, leading to a period of vibration that is longer than the fundamental mode used in conventional design methods. This thesis investigates the uplift and rocking behaviour of selective storage racks and proposes a method to determine the effective period of vibration to improve the accuracy of the equivalent static method.

A series of snapback tests were conducted on a full-scale three-level selective rack in order to compare the free vibration/rocking characteristics of four different baseplate types. The period of vibration was shown to increase for increasing rack sway amplitude during rocking and the unanchored rack period could be predicted using Housner’s rocking block model. The damping ratio of the unanchored structure, which was 0.034, could be increased to 0.051 using ductile baseplates that dissipate energy during uplift.

A total of 29 shaking table tests were conducted on a similar rack to compare the seismic performance of the unanchored, ductile and heavy-duty baseplates. The rack with unanchored baseplates failed by overturning and the rack with heavy-duty baseplates failed by anchor pull-out; both at 1.5 times the magnitude of the design level ground motion. The ductile baseplate survived all tests, up to 2.3 times the design level ground motion.

A finite element model was developed to simulate the rocking behaviour of the upright frame in the cross-aisle direction. The model was validated against experimental testing and used to conduct a parametric study comparing the equivalent static method and the time history analysis method specified in NZS 1170.5:2004. A total of 15 rack configurations, from five baseplate types and 3, 5 and 7 level racks, were analysed. It is recommended that the effective natural period be computed in the Rayleigh method using nonlinear static analysis with the pallet loads included, where the Rayleigh lateral loads create an overturning moment equal to the restoring moment of the pallet loads.

Finally, the effect of short duration axial loads, caused by stomping during rocking, on upright members was investigated. Finite element models of 59 uprights were subjected to nonlinear inelastic static and dynamic analyses to determine the effects of length, bracing pitch, cross-section slenderness and torsional restraint on the residual capacity of the member. A typical cold-formed steel rack upright could sustain a 0.1 s stomping force at least 15 % greater than its static ultimate capacity without significant reduction in residual capacity, implying that designing against peak loads computed from the time history analysis method may be somewhat conservative.
Acknowledgements

I would like to express thanks to my supervisor, Lip Teh, for encouraging me to pursue a PhD and for his mentorship throughout. Without Lip I would have missed out on many great experiences. Thanks to my co-supervisors, Charles Clifton, James Lim and Timothy McCarthy, for their guidance and support during my studies.

Thanks also to my lab partner, Zhenghao Tang, whom I spent countless hours working and problem solving with. I would also like to acknowledge the structures lab manager, Lucas Hogan, and the lab technicians at The University of Auckland for facilitating the experimental components of this research.

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List of publications

This thesis has been prepared in journal article compilation style format. Below is the list of publications as they appear in the thesis.

Chapter 2 — Maguire, J. R., Teh, L. H., Clifton, G. C., Tang, Z., & Lim, J. B. P. (n.d.). Rocking behaviour of selective storage racks with different baseplate types. under preparation for submission to a well-established international journal


Statement of contributions

Below is a statement of contributions for each co-authored publication that appears in the thesis.

**Publication:** Maguire, J. R., Teh, L. H., Clifton, G. C., Tang, Z., & Lim, J. B. P. (n.d.). Rocking behaviour of selective storage racks with different baseplate types. *under preparation for submission to a well-established international journal*

**Contribution by PhD candidate:** The snapback laboratory test was conceptualised by GC Clifton, JR Maguire, Z Tang and JBP Lim. The snapback testing was planned and carried out by both Z Tang and JR Maguire, with the effort evenly split. The finite element model and analysis was done by J Maguire with input from LH Teh. The manuscript was drafted by JR Maguire and edited by both LH Teh and JR Maguire with a final review by all co-authors resulting in minor changes.

**Extent of contribution by PhD candidate:** 75 %


**Contribution by PhD candidate:** The shaking table laboratory test was conceptualised by all co-authors. The shaking table testing was planned and carried out by both Z Tang and JR Maguire, with the effort evenly split. The manuscript was drafted by JR Maguire and edited by both LH Teh and JR Maguire with a final review by all co-authors resulting in some additions.

**Extent of contribution by PhD candidate:** 70 %


**Contribution by PhD candidate:** Conceptualisation of the FEA model was done by J Maguire with input from LH Teh. The FEA model was created and analysed by JR Maguire. Analysis of the results was conducted by J Maguire with input from LH Teh. The manuscript was drafted by JR Maguire and edited by both LH Teh and JR Maguire with a final review by all co-authors resulting in minor changes.

**Extent of contribution by PhD candidate:** 80 %

Contribution by PhD candidate: Conceptualisation of the FEA models was done by J Maguire and LH Teh. The FEA models were created and analysed by JR Maguire. Analysis of the results was conducted by J Maguire and LH Teh. The manuscript was drafted by JR Maguire and edited by both LH Teh and JR Maguire with a final review by all co-authors resulting in minor changes.

Extent of contribution by PhD candidate: 70%

The undersigned hereby certify that:

- the above statement correctly reflects the nature and extent of the PhD candidate’s contribution to this work, and the nature of the contribution of each of the co-authors; and

- that the candidate wrote all or the majority of the text.

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## Contents

Abstract ii

Frontmatter iii
- Acknowledgements .................................................. iii
- Certification ......................................................... iv
- List of publications .................................................. v
- Statement of contributions ........................................... vi
- Contents ................................................................. x

1 Introduction 1

1.1 Background ........................................................... 1
  1.1.1 Selective storage racks .......................................... 1
  1.1.2 Current design methods ........................................ 3
  1.1.3 Rocking structures .............................................. 4
  1.1.4 Previous literature on storage racks ......................... 5

1.2 Aims and scope of this thesis .................................... 6

2 Free rocking response 7

2.1 Introduction .......................................................... 8

2.2 Experimental program .............................................. 8
  2.2.1 Test setup ...................................................... 8
  2.2.2 Baseplates .................................................... 10
  2.2.3 Instrumentation ............................................... 11
  2.2.4 Snapback testing procedure .................................. 13
  2.2.5 Experimental observations .................................. 14
  2.2.6 Test results .................................................. 15

2.3 Finite element analysis .......................................... 17
  2.3.1 Baseplate models ............................................. 18
  2.3.2 Snapback analysis ............................................. 20
  2.3.3 Ground motion analysis ..................................... 21

2.4 Conclusions .......................................................... 23

3 Shaking table testing 26
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>Introduction</td>
<td>26</td>
</tr>
<tr>
<td>3.2</td>
<td>Experimental program</td>
<td>27</td>
</tr>
<tr>
<td>3.2.1</td>
<td>Test setup</td>
<td>27</td>
</tr>
<tr>
<td>3.2.2</td>
<td>Baseplates</td>
<td>28</td>
</tr>
<tr>
<td>3.2.3</td>
<td>Instrumentation</td>
<td>29</td>
</tr>
<tr>
<td>3.2.4</td>
<td>Ground motions</td>
<td>30</td>
</tr>
<tr>
<td>3.3</td>
<td>Experimental results and discussions</td>
<td>33</td>
</tr>
<tr>
<td>3.3.1</td>
<td>General observations</td>
<td>33</td>
</tr>
<tr>
<td>3.3.2</td>
<td>Test results</td>
<td>35</td>
</tr>
<tr>
<td>3.3.3</td>
<td>Discussion of test results</td>
<td>39</td>
</tr>
<tr>
<td>3.4</td>
<td>Conclusions</td>
<td>40</td>
</tr>
<tr>
<td>4</td>
<td>Refined equivalent static method</td>
<td>41</td>
</tr>
<tr>
<td>4.1</td>
<td>Introduction</td>
<td>42</td>
</tr>
<tr>
<td>4.2</td>
<td>Rack Configurations and Modelling</td>
<td>42</td>
</tr>
<tr>
<td>4.2.1</td>
<td>Upright frame</td>
<td>42</td>
</tr>
<tr>
<td>4.2.2</td>
<td>Baseplate Types</td>
<td>43</td>
</tr>
<tr>
<td>4.2.3</td>
<td>Baseplate modelling</td>
<td>45</td>
</tr>
<tr>
<td>4.2.4</td>
<td>Model validation</td>
<td>46</td>
</tr>
<tr>
<td>4.3</td>
<td>Analysis Procedures</td>
<td>48</td>
</tr>
<tr>
<td>4.3.1</td>
<td>First stage</td>
<td>48</td>
</tr>
<tr>
<td>4.3.2</td>
<td>Second stage</td>
<td>50</td>
</tr>
<tr>
<td>4.4</td>
<td>Determination of the effective natural period</td>
<td>50</td>
</tr>
<tr>
<td>4.4.1</td>
<td>The computed natural periods of vibration</td>
<td>50</td>
</tr>
<tr>
<td>4.4.2</td>
<td>Design base shear</td>
<td>52</td>
</tr>
<tr>
<td>4.5</td>
<td>Implications of the refined procedure</td>
<td>55</td>
</tr>
<tr>
<td>4.6</td>
<td>Additional verifications</td>
<td>56</td>
</tr>
<tr>
<td>4.6.1</td>
<td>Pallet mass</td>
<td>56</td>
</tr>
<tr>
<td>4.6.2</td>
<td>Baseplate compression stiffness</td>
<td>57</td>
</tr>
<tr>
<td>4.6.3</td>
<td>Comparisons to shaking table experiments</td>
<td>57</td>
</tr>
<tr>
<td>4.7</td>
<td>Application of refined procedure</td>
<td>59</td>
</tr>
<tr>
<td>4.8</td>
<td>Conclusions</td>
<td>60</td>
</tr>
<tr>
<td>5</td>
<td>Residual capacity of uprights following stomping</td>
<td>62</td>
</tr>
<tr>
<td>5.1</td>
<td>Introduction</td>
<td>63</td>
</tr>
<tr>
<td>5.2</td>
<td>Stomping forces during rocking</td>
<td>64</td>
</tr>
<tr>
<td>5.3</td>
<td>Upright models</td>
<td>66</td>
</tr>
<tr>
<td>5.3.1</td>
<td>Cross-sections and boundary conditions</td>
<td>66</td>
</tr>
<tr>
<td>5.3.2</td>
<td>Material properties</td>
<td>67</td>
</tr>
<tr>
<td>5.3.3</td>
<td>Mesh refinement</td>
<td>68</td>
</tr>
<tr>
<td>5.3.4</td>
<td>Model validation</td>
<td>68</td>
</tr>
<tr>
<td>5.3.5</td>
<td>Model parameters</td>
<td>69</td>
</tr>
</tbody>
</table>
6 General conclusions

Bibliography
Chapter 1

Introduction

1.1 Background

Cold-formed steel storage racks are used worldwide as a lightweight structure for both temporary and long term storage of goods. The selective storage rack is a type of rack that holds palletised stock arranged in aisles. This layout has a lower floor space utilisation than other rack types, such as the drive-in rack, but allows direct access to each pallet by forklift or reach truck, making selective racks convenient general-purpose storage structures. An example of selective storage racking is shown in Figure 1.1.

1.1.1 Selective storage racks

Selective storage racks are assembled from multiple upright frames connected in the down-aisle direction by beams, as shown in Figure 1.2. The upright frames consist of

Figure 1.1: Selective storage racking in distribution warehouse.
two upright members connected by a pattern of laced bracing members. The upright members are perforated on both the rear flanges and web to accommodate bracing and beam connections. Bracing members are bolted to the upright rear flange and beams are connected to the upright web by a multiple hook connector welded to each end of each beam. Upright, bracing and beam members are designed to be modular and easy to assemble.

Upright frames in selective racks are typically tall and narrow. When subjected to lateral loading in the cross-aisle direction, which is the focus of this thesis, significant overturning moments can be developed. The rack resists these moments through two mechanisms: first, the self-weight of the loaded rack produces a restoring moment, given that the centre of mass of the rack is within its footprint, and second by the reaction forces transferred through the rack baseplates to the concrete floor.

The baseplate consists of a floor-plate, used to anchor the baseplate to the concrete floor, welded to a vertical stub that is connected to the base of the upright. For racks at risk of seismic loading, it is common to use ductile baseplates that allow the rack to uplift through a combination of bolt slip (through slotted holes in the vertical stub) and flexure of the floor-plate, shown in Figure 1.3(b). The low stiffness of these baseplate types reduces the tensile load demand on the concrete floor.

During the 2010 and 2011 Canterbury earthquakes, several selective rack collapses were the result of overturning in the cross-aisle direction (Crosier et al. 2010; Uma and Beattie 2011). Crosier et al. (2010) observed that, in a warehouse that experienced collapses of anchored racks, unanchored racks had survived undamaged after “walking”.

Figure 1.2: Three-level selective storage rack.
Although the rocking behaviour of racks in the cross-aisle direction can be modelled by nonlinear time history analysis, it requires a significant investment of time, computing facilities and expertise. In practice, rack designers commonly use equivalent static force methods allowed by seismic design standards such as NZS 1170.5:2004 (SNZ 2004).

1.1.2 Current design methods

Ductile baseplates types are used in seismic regions to improve the dynamic performance of selective storage racks. Since the upright frames are relatively stiff, the cross-aisle dynamic behaviour is sensitive to the baseplate characteristics. The use of flexible baseplates leads to uplift and rocking of the rack when subjected to ground motion accelerations in the cross-aisle direction.

Unlike common buildings and other non-rocking structures, rocking structures have a non-constant period (Housner 1963; Psycharis and Jennings 1983; Yim et al. 1980). For an uplifting selective storage rack, restoring moments (in the cross-aisle direction) are provided by the tension developed in the uplifted baseplate and the gravity load, with the restoring effect of the gravity load decreasing with increasing rack sway. Therefore, computing the natural period by elastic analysis can result in underestimation of the effective period during an earthquake when the structure rocks.

Current seismic design standards that are applicable to storage racks include ANSI MH16.1:2012 (RMI 2012), EN 16681:2016 (ECS 2016) and NZS 1170.5:2004 (SNZ 2004). The design methods detailed in these standards include the equivalent static force method, the modal response spectrum method and the time history analysis method. Both the equivalent static force method and the modal response spectrum method do not take into account the stomping and uplift that occur during rocking. ANSI MH16.1:2012 does not consider rocking at all, while EN 16681:2016 accounts for the rocking of the pallets.
The NZS 1170.5:2004 standard, which this thesis will focus on, is used to determine the seismic design loads for buildings, so does not necessarily account for the unique characteristics of cold-formed steel storage racks. Therefore, design guidelines such as the BRANZ design guide (Beattie and Uma 2012) suggest modifications to adapt the standards for use with storage racking. The BRANZ design guide, although providing modifications to the application of the standard appropriate for rack design, does not address cross-aisle rocking which can lead to significant differences in structural response.

### 1.1.3 Rocking structures

The first research into rocking structures was conducted by Housner (1963) to explain the seismic performance of tall slender structures. Housner developed a mathematical model that described the rocking motion of a rigid rocking block (shown in Figure 1.4), finding that the period of vibration increases as a function of its angle of rotation about the non-uplifting corner. The free rocking period of the block, when dropped from an initial angle of $\theta_0$, can be expressed as:

$$T = 4 \sqrt{\frac{I_0}{WR}} \cosh^{-1} \left( \frac{1}{1 - \theta_0/\theta_c} \right)$$

(1.1)

where $I_0$ is the moment of inertia, $W$ is the weight, $R$ is the distance from the point of rotation to the centre of gravity, and $\theta_c$ is the critical overturning angle of the block.

Housner’s rocking block model has been extended to consider overturning stability, com-
bined rocking and sliding responses, and effects of block geometry over a range of loads, including pulse, cyclic and earthquake loadings (Aslam et al. 1980; Pompei et al. 1998; Shenton III 1996; Spanos and Koh 1984). For rocking structures subjected to earthquake ground motions, an increase of loading intensity does not necessarily lead to an increase in the risk of overturning (Yim et al. 1980) as might be expected. Additionally, for some structures subjected to dynamic cyclic loading at given frequencies, better performance is achieved by removing anchors to allow rocking (Makris and Zhang 1999). Models have also been developed to account for differences in energy dissipation between real-world objects and Housner’s idealised model (Aslam et al. 1980; Ther and Kollár 2017).

The use of rocking structural systems for bridge piers, shear walls and steel frames is now common. In steel frames, similar to storage racks, it has been shown that a significant proportion of earthquake input energy can be dissipated by hysteresis damping from ductile uplifting baseplates and uplift of the frame centre of gravity (Azuhata et al. 2005). Rocking mechanisms in earthquake resistant steel frames have been shown to reduce the ductility demand due to reduction in base shear (Azuhata et al. 2007; Huckelbridge and Clough 1978; Midorikawa et al. 2005). More recently, rocking steel frame designs have been proposed that use post-tensioned cables to limit maximum displacements (Steele and Wiebe 2016), replaceable fuses to provide additional energy dissipation (Hall et al. 2010) or friction uplifting column bases (Freddi et al. 2017). Well designed self-centring rocking braced frames have also been shown to achieve lower floor spectra than for similar buckling-restrained braced frame buildings (Pollino 2015).

1.1.4 Previous literature on storage racks

Limited research has been done on the application of rocking design to cold-formed steel storage racking. The energy dissipation capacity of ductile baseplates was demonstrated for a storage rack upright frame subjected to quasi-static cyclic loading (Petrone et al. 2016). Cross-aisle shaking table testing of a storage rack was conducted by Sideris et al. (2010) to determine the response of the pallets during earthquakes. Other shaking table tests conducted on storage racking have been concerned with the down-aisle responses or use pseudo-dynamic tests that do not allow free rocking behaviour (Castiglioni et al. 2009; Filiatrault et al. 2007; Jacobsen and Tremblay 2017). Shaking table tests of other rack types, such as drive-in racks, have been carried out (Ahmed et al. 2016; Shaheen and Rasmussen 2017).

Slender steel framed buildings with ductile baseplates may be viewed as similar to selective storage racks in the way they behave. Shaking table tests of a steel braced frame with ductile baseplates, subjected to the El Centro 1940 ground motion, showed that uplift and rocking reduced the peak base shear and deformation within the structure (Midorikawa et al. 2006). Shaking table tests of another steel frame showed that ductile anchors that allowed the frame to rock reduced base shear by over 20 % (Trautner et al. 2018).
A challenge in the design of rocking structures, however, is managing the dynamic compression forces which arise from stomping (Azuhata et al. 2007; Priestley et al. 1978). Stomping refers to the impulse action of an upright (or column) of a rocking structure as it comes down to the ground periodically.

### 1.2 Aims and scope of this thesis

The primary aim of this thesis is to provide an improved procedure for the cross-aisle seismic design of selective storage racks that takes into account the rocking mechanism. The procedure should be easy to understand and implement for practising engineers.

Chapter 2 reports on the results of a series of snapback tests conducted to observe the free vibration/rocking response of a fully-loaded selective rack. Comparison between two ductile baseplates, a heavy-duty baseplate, and an unanchored baseplate are made. In this chapter, a 3-dimensional finite element model is developed and validated against the snapback test and used to run transient ground motion simulations for all baseplate types.

Chapter 3 presents the results of a series of full-scale shaking table tests on a selective rack for three baseplate types: ductile, heavy-duty and unanchored. Each baseplate was subjected to a sequence of ground motions increasing in intensity until failure.

In Chapter 4, a 2-dimensional upright frame finite element model is developed to compare base shear and upright loads obtained by the equivalent static method and time history analysis method of NZS 1170.5:2004. A total of 15 rack configurations, from five baseplate types (ductile, heavy-duty and three linear elastic baseplates of varying stiffness) with 3, 5 and 7 level rack frames, were considered. Analyses were carried out for each rack configuration to determine the effective period of vibration. A refined equivalent static method based design procedure is recommended which is shown to be suitable for selective storage racks up to 10 m in height.

Chapter 5 is concerned with the dynamic stomping forces experienced by the upright members during frame rocking. Finite element models are used to determine the residual capacity of a number of upright members after being subjected to one, or multiple, short-duration impulses. The effects of wall thickness, length and torsional restraints are investigated.

For continuity, some of the contents of each chapter has been modified from the original submitted papers. Specifically, the introduction sections have been shortened to avoid repetition and, for some chapters, additional paragraphs and sections have been added. A summary of changes to each paper is provided at the beginning of each chapter where applicable.
Chapter 2

Free rocking response

This chapter is a modified version of the paper:

Maguire, J. R., Teh, L. H., Clifton, G. C., Tang, Z., & Lim, J. B. P. (n.d.).
Rocking behaviour of selective storage racks with different baseplate types.
*under preparation for submission to a well-established international journal*

The introduction (Section 2.1) has been shortened to avoid repetition of information from Chapter 1. Some wording has been changed to maintain consistency with other chapters. Figure 2.4, which was not in the original paper, has been added to Section 2.2.2.

Abstract

A series of snapback tests were conducted on a full-scale cold-formed steel selective storage rack with different types of baseplates in order to study its free rocking behaviour in the cross-aisle direction. Eight tests were conducted, consisting of small and large initial sways for four baseplate types: standard ductile, alternate ductile, heavy duty and unanchored. The free rocking period was found to be a function of the rack’s initial sway, which is inconsistent with the assumption implicit in the equivalent static force method prescribed in seismic design standards. The free rocking period of the unanchored rack could be predicted using the Housner’s rocking block model. The standard ductile and alternate ductile baseplates increased the effective damping ratio from 0.034 (with the unanchored baseplate) to 0.051. The test results were used to validate a three-dimensional finite element model, which was then used to compare the ground motion responses of the racks with different baseplates to three earthquake records. The two ductile baseplates and the unanchored baseplate reduced the load demand on the structure under earthquake excitation, thus suppressing column failure, but resulted in larger peak sways, with greater risk of overall instability. The ductile baseplates offered the best seismic performance,
through limiting the load demand on the frame to prevent column failure while keeping the peak sway within the limits required for overall stability.

2.1 Introduction

Although common baseplate designs in seismic regions allow uplift that leads to rocking motion under seismic loading, it is largely ignored in the structural design procedure. The use of yielding baseplates that allow uplift was investigated in quasi-static tests (Petrone et al. 2016), and shaking table tests have been carried out showing that ductile baseplates improve seismic performance over rigid baseplates (Trautner et al. 2018).

This study aims to compare the free rocking responses of three-level two-bay selective racks with four baseplate types: standard ductile (SD), alternate ductile (AD), heavy-duty (HD) and unanchored (UA). Eight snapback tests were carried out, with small and large initial sways for each baseplate type. The experimental results were used to facilitate the development of a three-dimensional finite element model that was subjected to three unscaled ground motion records (El Centro 1940, Northridge 1994 and Kobe 1995). The performance of the rack during free rocking and under earthquake loading is compared for the four baseplate types.

2.2 Experimental program

2.2.1 Test setup

Snapback testing was conducted on a three-level two-bay cold-formed steel selective storage rack in the cross-aisle direction, shown in Figure 2.1. The rack rested on three concrete blocks cast from 40 MPa reinforced concrete fixed to the strong floor by post-tensioned rods. The rack was pulled to its initial sway for each test by a hydraulic actuator connected in series with a load cell, quick-release shackle and lifting sling. The lifting sling was wrapped around the far side of the central upright frame and the actuator was fixed to a strong wall.

To prevent the rack from accidental overturning, a set of six steel wires connected the rack to the strong wall. On the far side from the strong wall, six ropes connected the rack to two anchored steel frames. The steel wires and ropes were given enough slack to avoid interference with the rack motion during the tests.

The rack was assembled from a set of proprietary cold-formed steel members (minimum specified member properties are provided in Table 2.1):

- 90 mm deep perforated uprights,
Table 2.1: Rack structure member properties.

<table>
<thead>
<tr>
<th>Member</th>
<th>Depth (mm)</th>
<th>Width (mm)</th>
<th>Thickness (mm)</th>
<th>Min. yield stress (kN)</th>
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<tr>
<td>Upright</td>
<td>90</td>
<td>73</td>
<td>2.0</td>
<td>450</td>
</tr>
<tr>
<td>Bracing</td>
<td>30</td>
<td>25</td>
<td>1.8</td>
<td>450</td>
</tr>
<tr>
<td>Box beam</td>
<td>85</td>
<td>40</td>
<td>1.5</td>
<td>300</td>
</tr>
</tbody>
</table>

- 25×30 mm lipped channel bracing, and
- 85×40 mm box beams with five-hook-connectors at each end.

Each upright frame was assembled from two uprights connected by the bracing members. The frame had two X-braces with a pitch of 600 mm at the bottom followed by laced K-bracing to the top. The three upright frames were connected by beams in the down-aisle direction.

During all tests, the rack was loaded with six pallets. Each pallet, shown in Figure 2.2(b), consisted of two timber boxes filled with sandbags and resting on a timber pallet. The gross mass of each pallet was 783 kg (7.68 kN) with a centre of gravity approximately 450 mm above the beam level. The pallets were clamped to the rack beams to prevent sliding and shedding during the test. The clamped pallets provide a worst case scenario, assuming that shedding does not occur, as unclamped pallets are likely to slip on the beams, reducing the effective seismic mass of the structure.
2.2.2 Baseplates

The four tested baseplate types were: standard ductile (SD), alternate ductile (AD), heavy-duty (HD) and unanchored (UA), shown in Figure 2.3. Load-deflection curves for the SD, AD and HD baseplates are shown in Figure 2.4. It should be noted that the AD baseplate used in the baseplate cyclic axial load test had a 30 mm slotted hole, rather than a 15 mm slotted hole which was used for the snapback testing.

The SD baseplate consisted of a 3.5 mm 300 MPa hot rolled steel floorplate welded to a 235 MPa low carbon steel vertical stub. Welding consisted of two 25 mm 4×6 mm and one
40 mm 4×6 mm fillet welds. The SD baseplate was anchored to the concrete block with two sleeve anchors positioned diagonally across the floorplate. The vertical stub fit inside the upright rear flanges and was connected with a single M10 bolt. Under compression, the upright bears directly onto the floorplate. Under tension, the SD baseplate allows uplift by sliding of the bolt through the slotted hole in the baseplate stub, up to 7 mm, and once at the end of the slotted hole uplift continues by flexure of the floorplate.

The AD baseplate used the same baseplate design as the SD, but the two concrete sleeve anchors were positioned on the inner side of the upright frame, rather than diagonally. This configuration is sometimes used to avoid reinforcement in the foundation slab. The AD baseplate type behaves similarly to the standard ductile baseplate but has a lower stiffness during uplift.

The HD baseplate had a 10 mm 200 MPa hot rolled steel floorplate welded to a vertical stub with a 173 mm 5×5 mm fillet weld. The vertical stub fit around the outside of the upright and was connected to the upright web with four M10 bolts. As with the SD baseplate, the HD baseplate had two anchor bolts arranged diagonally.

The UA baseplate was the same as the HD baseplate but with no anchor bolts, allowing the upright to uplift freely. A steel block was fixed to the concrete along the outside edge of the upright frames, as shown in Figure 2.5, to prevent walking of the rack from its original position. Additionally, two sleeve anchors without the nut were installed for each baseplate to prevent the rack from walking in the down-aisle direction.

2.2.3 Instrumentation

Data collected during the experiments included:
• actuator load,
• rack displacement (sway) at each level,
• acceleration at each level,
• upright flange and rear flange axial strain for each upright, and
• uplift displacement for all uprights.

For all instrumentation, the recording rate was 50 Hz.

Rack sway was measured using draw-wire displacement sensors. The draw-wires were mounted to the rack using a steel angle bolted to the upright web so that displacement was measured approximately 100 mm above the beam level at each location, shown in Figure 2.6. The draw-wire tip was attached to the strong wall. Acceleration was measured at the same locations as the draw-wires using MEAS 4000A uniaxial accelerometers. The accelerometers were attached directly to the upright through two 2 mm diameter holes drilled into the upright web at approximately 75 mm above the beam level.
Upright axial strain was measured using four single-axis strain gauges per upright. The strain gauges were installed on both sides of each upright, 200 mm above the upright base on both the flange and rear flange, as shown in Figure 2.7.

The upright uplift was recorded using portal gauges, shown in Figure 2.5. The portal gauges were fixed to the floor using a metal wire fastened with epoxy into a 3 mm hole in the concrete block (or into the steel shear block for the UA baseplate). The other end of the portal gauge was connected by metal wire to a stud welded thread on the upright web 150 mm above the base.

### 2.2.4 Snapback testing procedure

The pullback forces, initial sway and initial uplift for each test is shown in Table 2.2. The target pullback force of 5 kN corresponds to the load required to just initiate uplift by overcoming the restoring moment of the pallet weight around the upright baseplate. The target pullback force of 10 kN corresponds to a state where the structure has significant uplift and development of tension through the baseplate. For the HD-02 test, a target displacement of 20 kN was used to develop uplift significantly larger than the HD-01 test.

For each test, the quick release shackle was used to connect the extended actuator to the lifting sling. The actuator was then retracted, pulling the rack towards the strong wall. At the target pullback force or sway, the quick release shackle was opened allowing the rack to respond by free vibration/rocking.

The rack members, baseplates and concrete blocks were examined for damage between tests. For sequential tests using the same baseplate type (ie. SD-01 and SD-02), the same set of baseplates was reused. Between tests of different baseplate types, the rack was lifted by crane, using a spreader beam connected to the tops of each upright, and the baseplates were changed. For the anchored baseplate types (SD, AD and HD), the rack was offset.
<table>
<thead>
<tr>
<th>Test</th>
<th>Pullback force (kN)</th>
<th>Initial disp. (mm)</th>
<th>Uplift (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SD-01</td>
<td>5.6</td>
<td>26</td>
<td>4.1</td>
</tr>
<tr>
<td>SD-02</td>
<td>9.0</td>
<td>57</td>
<td>9.9</td>
</tr>
<tr>
<td>AD-01</td>
<td>5.5</td>
<td>33</td>
<td>6.5</td>
</tr>
<tr>
<td>AD-02</td>
<td>10.3</td>
<td>90</td>
<td>16.9</td>
</tr>
<tr>
<td>HD-02</td>
<td>22.5</td>
<td>91</td>
<td>7.9</td>
</tr>
<tr>
<td>HD-01</td>
<td>4.8</td>
<td>20</td>
<td>3.6</td>
</tr>
<tr>
<td>UA-01</td>
<td>4.9</td>
<td>45</td>
<td>9.2</td>
</tr>
<tr>
<td>UA-02</td>
<td>4.4</td>
<td>101</td>
<td>22.0</td>
</tr>
</tbody>
</table>

Table 2.2: Snapback test sequence.

Figure 2.8: Indentation on AD baseplate.

in the cross-aisle direction (either towards or away from the strong wall) and new sleeve anchor bolts were installed.

It should also be noted that test HD-02 was carried out before HD-01. For all other baseplate configurations, the 01 test was performed before the 02 test.

2.2.5 Experimental observations

For the SD and AD tests, no significant baseplate damage was observed. The base of the upright, particularly near the web, caused indentation of the floorplate, as shown in Figure 2.8. Indentation was found for both the central and outer upright frames. Some local deformation of the upright’s rear flange corners was seen during the UA baseplate tests, shown in Figure 2.9.

During pullback for the SD, AD and UA baseplates, rack sway was primarily due to rotation of the structure where the uprights on the far side were uplifted. Upon release, the structure responded in a rocking motion, alternating between uplift on either side in the cross-aisle direction. The dynamic response was governed by the rocking mode as the upright frame stiffness was significantly higher than the baseplate uplift stiffness. For
During the free rocking of the structure, global twisting of the rack was observed. Twisting usually occurred after one or two oscillations where the outer upright frames would become out of phase with the central upright frame. During the UA baseplate tests, twisting of the structure resulted in some down-aisle sway.

2.2.6 Test results

The load-displacement curves of the actuator pullback stage of the tests are shown in Figure 2.10.

Each of the racks with SD and AD baseplates had a loss of stiffness near 5 kN of pullback, when the pallets’ restoring moment was overcome resulting in uplift. However, the structure response stiffened once the bolts travelled the full length of the slotted holes in the baseplate stubs, after which the stiffness was due to the flexural stiffness of the floorplate.

Test HD-01 exhibited a reduction in the structure’s stiffness from test HD-02, but it was unclear what component of the structure had sustained damage. On visual inspection, no evidence of damage was identified for the baseplates, anchor bolts or the superstructure.

For the UA baseplate, a peak load of 4.9 kN was achieved. This load was required to overcome the restoring moment, of the pallet weight, to cause uplift. As the sway increased, the restoring moment reduced due to P-Delta effect.

The snapback sways during each test is shown in Figure 2.11. Comparisons between tests using the same baseplate show that larger amplitudes of rocking resulted in longer rocking periods.
Figure 2.10: Snapback test load-sway curves.

Figure 2.11: Snapback test sways.
The uplift behaviour for each test is shown in Figure 2.12. The portal gauges recorded small negative displacements when the upright and baseplate assembly was in contact with the concrete block. The negative displacement is largely influenced by compression of the portal gauge due to rotation of the rack during uplift of the far side upright rather than through compression of the base of the upright.

### 2.3 Finite element analysis

A three-dimensional finite element model of the rack described in Section 2.2.1 was developed using the OpenSees software framework (McKenna 2016), as illustrated in Figure 2.13. The upright frames consisted of elastic beam-columns elements (uprights) and truss members (bracing). The beams and pallets were elastic beam-column elements with a rigid link connecting the pallet load (at the centre of mass) to the pallet base.

Each upright had nodes spaced at 600 mm, starting at 112 mm above the base. Additional nodes were inserted at 1425 mm, 2825 mm, and 4225 mm, corresponding to the beam levels. Bracing elements, making up the two X-braces followed by laced K-braces, connected the two uprights to form an upright frame. The 1350 mm beams had nodes at the midspan and 75 mm from each end. These nodes allowed the pallet load to be distributed over three points on each beam in a similar manner to the tested structure. The pallet elements formed a grid that supported the pallet weight of 7.68 kN located at the centre of mass. The initial upright axial loads at the base due to gravity loading were 12.2 kN and 5.7 kN for the central and outer upright frames, respectively.
2.3.1 Baseplate models

The baseplates were modelled as zero-length elements with several material models applied in series and parallel to allow for rocking motion. These boundary conditions are shown in Figure 2.14, where the zero-length elements have been elongated to separate the $i$ and $j$-nodes for illustration. The $i$-nodes, representing the concrete floor, of the baseplate elements on the left-hand side ($x = 0$ mm plane) were restrained for all degrees-of-freedom (DOF). On the right hand side ($x = 900$ mm plane), the $i$-nodes were restrained for all rotational DOFs and against translations in the $y$ and $z$ directions. The $j$-nodes, representing the base of the upright, were restrained against translation in the $y$ axis and rotations about the $x$ and $z$ axes. Under these boundary conditions, the upright frame could rotate in-plane during rocking without causing artificial internal stresses.

An elastic-no-tension (ENT) material was assigned to the zero-length element for all baseplate types to allow free uplift in the positive $z$-direction and to support the upright in compression.

The SD, AD and HD baseplate models had two elastic-perfectly-plastic-gap materials arranged in parallel with the ENT material in the $z$-axis. The parallel materials defined the hysteretic behaviour including the gap response due to slotted holes in the upright-to-baseplate connection.

The initial stiffness, uplifting stiffness, bolt slip displacement and yield point for each baseplate model were determined from the experimental force-displacement curves shown in Figure 2.4. The baseplate model uplift force-displacement curves are shown in Figure 2.15.
Figure 2.14: Model boundary conditions.

Figure 2.15: Baseplate uplift responses.
2.3.2 Snapback analysis

Corotational coordinate transformation was used to handle large displacement and small strains. The snapback simulation procedure was completed in two consecutive steps: static pullback and transient snapback.

Pullback was initiated by applying a displacement constraint to the node on the central upright frame at the third beam level. The displacement constraint was incrementally increased until the target displacement was reached. At the target displacement, the constraint was removed and a transient analysis with a time step of 0.01 s was conducted.

Damping

The three sources of damping during rocking are baseplate yielding, stomping impact, elastic damping, and other connection damping within the rack structure. The baseplate hysteresis was modelled explicitly as shown in Figure 2.15, while the other sources of damping were taken into account using Rayleigh damping. A Rayleigh damping ratio $\zeta$ of 0.034 was determined by trial and error to match the experimental results of the (unanchored) UA rack, shown in Figure 2.16.

The effective damping ratios of the experimental and simulated snapback was computed using:

$$\zeta = \frac{1}{2\pi j} \ln \frac{u_i}{u_{i+j}}$$  \hspace{1cm} (2.1)
where $\zeta$ is the damping ratio, $u_i$ is the $i$th local peak sway and $j$ is the number of oscillations across which the measurement is taken (Chopra 2012). In the present work, $u_i$ was taken as the first positive local peak sway, and three oscillations were used ($j = 3$). The results are shown in Table 2.3.

### Rocking period

The free rocking periods of the rack structures with different baseplates can be compared against each other in Figure 2.17. At low overturning ratios (which are proportional to the initial sways), the free rocking periods for all baseplate types closely match the theoretical value for a rigid rocking block given by Eqn (1.1). At higher overturning ratios ($\theta/\theta_c > 0.04$), only the UA rack continues to match the Housner model. The ductile baseplate (SD and AD) racks have shorter rocking periods than the Housner model due to the baseplate yielding.

#### 2.3.3 Ground motion analysis

Transient analyses were performed on the finite element model to study the rack performance during earthquake loading. The rack model was subjected to three unscaled...
earthquake ground motion records: El Centro 1940, Northridge 1994, and Kobe 1995. The ground motion records are shown in Figure 2.18. The sway and upright base axial load time history results are shown in Figure 2.19 and Figure 2.20, respectively with a summary of the peak values given in Table 2.4.

In general, the peak sways for the SD and AD racks for a given ground motion record were similar, with the AD rack having a peak sway 96% to 135% that of the SD rack. The
HD rack typically had the smallest peak sway which was 56% to 101% that of the SD rack. The UA rack typically had the highest peak sway, with the exception being the El Centro ground motion where it had a smaller peak sway than the SD and AD racks. The SD, AD and HD racks had relatively low sensitivity to the ground motion record compared to the UA rack, having values of 99 mm, 473 mm and 135 mm for El Centro, Northridge and Kobe records respectively.

The peak upright compression for the AD rack was within 83% to 104% of that of the SD rack for the three ground motion records. The HD rack had the largest peak upright compression for all three records, which was 125% to 204% that of the SD rack. Except under the Northridge ground motion, the UA rack had the lowest peak compression. For all records, the UA rack had zero tensile axial load, due to uplift, and the peak compression occurred during stomping of the upright. After each stomping load, the axial force oscillated and eventually converged on 24.4 kN (the static load in an uplifted state).

### 2.4 Conclusions

This chapter has presented the results of eight cross-aisle snapback tests on a three-level two-bay cold-formed steel selective storage rack using four types of baseplates: standard
ductile (SD), alternate ductile (AD), heavy-duty (HD) and unanchored (UA). A three-dimensional finite element model capable of replicating the rocking response including energy dissipation has also been developed.

The Rayleigh damping ratio of the rack superstructure was found to be 0.034, with additional damping being provided by yielding of the anchored baseplates. The effective damping ratio was 0.051 for the SD and AD racks. The increase in damping due to yielding of the baseplates was modelled explicitly in the finite element model through hysteretic material models, providing good results.

For the SD, AD and UA racks, the rocking periods were longer for larger sways. The implication is that the rocking period is non-constant, which is inconsistent with the assumption implicit in the equivalent static force method prescribed in seismic design standards. The Housner rocking block model was an effective predictor for the UA rack, but overestimated the rocking periods of the SD and AD racks.

When subjected to earthquake loadings, the HD rack generally had the smallest peak sway, which was between 56 % and 101 % of the SD. The UA rack generally had larger peak sway than the SD and AD racks, with the exception of the El Centro record. However, the
UA rack had a significantly longer duration of rocking due to its low energy dissipation.

The UA rack generally had the smallest peak upright axial load compared to the other baseplates during the ground motion simulation. The HD rack, on the other hand, had the highest peak axial loading for all ground motion records, with about 2.5 times the peak tension and 1.5 times the peak compression of the SD rack.
Chapter 3

Shaking table testing

This chapter is a modified version of the published work:


The introduction (Section 3.1) has been shortened to avoid repetition of information from Chapter 1. Some wording has been changed to maintain consistency with other chapters. For continuity, the section of the original paper comparing shaking table results to equivalent static method results has been moved to Chapter 4 (Section 4.6.3).

Abstract

A series of single-axis shaking table tests were conducted on three full-scale selective storage racks in the cross-aisle direction. The uplifting and rocking behaviour of the racks was examined for three baseplate types: ductile, heavy-duty, and unanchored. Each rack was subjected to a sequence of ground motions of increasing intensity up to failure, with a total of 29 tests conducted. At 1.5 times the respective design level ground motions, the heavy-duty baseplates caused a foundation failure while the unanchored rack failed by overturning. The rack with ductile baseplates survived all tests up to 2.3 times the design level. The peak displacement, acceleration and upright base strain of all specimens are presented.

3.1 Introduction

This chapter compares the cross-aisle seismic response of selective storage racks with three different baseplate types through full-scale shaking table tests. A suite of three ground
motions (Kaikoura 2016, Northridge 1994, and Kobe 1995) was applied to the racks, with increasing intensity (up to 2.3 times the design level) until failure. Rack accelerations and displacements at each level were recorded, in addition to upright strain at the base.

3.2 Experimental program

3.2.1 Test setup

The test was conducted on a single-axis shaking table at The University of Auckland. The rack structure, shown in Figure 3.1, was three levels tall with two single-pallet bays in the down-aisle direction. To provide realistic baseplate anchoring conditions, the rack rested on three 150 mm thick reinforced concrete blocks. The concrete blocks were cast with 40 MPa concrete with pre-installed holes that allowed fastening to the shaking table by eight 12 mm threaded bars. The baseplates were anchored to the concrete blocks using drill-in anchor bolts.

The rack structure was assembled using three 4.8 m tall upright frames with a depth of 900 mm. The upright members, shown in Figure 3.2, were perforated to allow for beam and bracing connections and had a wall thickness of 2.0 mm. Bracing members were 25×30 mm lipped channels with a wall thickness of 1.8 mm. Bracing was installed with a pitch of 600 mm. Two pitches of X-bracing were provided at the base of the upright frame, with K-bracing above, which can be seen in Figure 3.3. The upright frames were connected in the down-aisle direction by 1.35 m (nominal length) 85×40 box beams.

The storage rack was loaded with six pallets during the shaking table tests. Each 800 kg pallet was made up of four 175 kg steel billets clamped to a welded steel pallet base, shown in Figure 3.4. The pallet base was designed to raise the pallet centre of gravity to 249 mm above the beam level to better represent a typical palletised stock unit. The pallets were prevented from sliding on the rack beams by clamping them using two ratchet straps. In addition to the ratchet straps, the pallet bases had two 50×50 mm steel hollow sections welded to their underside to prevent the pallets from sliding past the rack beam. Pallets...
shaking can occur during earthquakes but is outside of the scope of this study.

To allow for the racks to be tested to failure, a steel catch frame was put in place. The catch frame consisted of two large eccentrically braced frames connected by six beams as shown in Figure 3.3. The catch frame was dimensioned to allow clearance from the rack to be maintained until after overturning or collapse, allowing the rack to respond freely to the ground motion.

### 3.2.2 Baseplates

The shaking table tests were carried out for three racks, each with a different baseplate type: standard ductile (SD), heavy-duty (HD) and unanchored (UA), shown in Figure 3.5.

The SD baseplate had a 3.5 mm thick floorplate welded to a vertical stub that sat inside the footprint of the upright cross-section. The stub was connected by a single 10 mm bolt to the upright rear flanges. The floorplate was anchored to the concrete block with two
75×12 mm screw anchors positioned diagonally across the baseplate. In the cross-aisle direction, the SD baseplate allowed the rack to uplift by flexure of the floorplate.

The HD baseplate was made from a 10 mm thick floorplate connected to a vertical stub that wrapped around the outside of the upright web and flanges. The stub was connected to the upright web by a grid of four 10 mm bolts. The HD baseplate was anchored to the concrete slab using two 75×12 screw anchors arranged diagonally. The HD baseplates were stiffer in tension than the SD baseplates, preventing significant rack uplift during cross-aisle shaking.

The UA baseplate configuration used the HD baseplate without any anchor bolts. To prevent the rack from walking in the cross-aisle direction, steel sections were anchored to the outside of the rack footprint, seen on the left hand side of Figure 3.5(c). The UA baseplates provided no resistance to uplift. Unlike in the snapback test, there were no unbolted anchors to resist walking of the rack in the down-aisle direction.

### 3.2.3 Instrumentation

The data collected for each of the shaking table tests included table acceleration, table displacement, rack accelerations, rack displacements, and upright flange axial strain. A
recording rate of 100 Hz was used.

Accelerations were measured using MEAS 4000A uniaxial accelerometers with a range of ±10 g. Three accelerometers were attached to the central upright frame 150 mm above each beam level. Two more accelerometers were attached to the outer upright frames 150 mm above the top beam level. A sixth accelerometer was attached to the shaking table in the direction of shaking to record the ground acceleration. Accelerometer locations on the rack are shown in Figure 3.3.

Rack displacement was measured using a set of five draw-wire displacement sensors. The displacements were recorded 150 mm above each beam level for the central upright frame, coinciding with the locations of the accelerometers, and at 150 mm above the first beam level for both outer upright frames. Draw-wire sensors for the central frame were mounted on a reference column fixed to the catch frame, and draw-wire sensors for the outer frames were mounted to the rack structure itself with wires connected to a strong wall behind the catch frame. Draw wire locations on the rack are shown in Figure 3.3. The table displacement was recorded using an LVDT built into the shaking table actuator.

Upright axial strain was measured using a set of single-axis strain gauges. Two strain gauges were installed 200 mm above the base of each upright, one on the outside centreline of each flange, as shown in Figure 3.6. Upright axial load was estimated by normalising the strain gauge readings using the initial strain and the strain at which the axial load was known to be zero (during uplift for the UA rack). The estimated axial load is given by:

\[ P(t) = \frac{|P_s|}{\varepsilon_{P=0}} \varepsilon(t) + P_s \]  

where \( P_s \) is the static upright axial load (11.7 kN for the uprights of the central frame), \( \varepsilon_{P=0} \) is the average strain recorded during the times at which the UA rack was known to have zero loading due to uplift (190 \( \mu \varepsilon \) for the central frame), and \( \varepsilon(t) \) was the strain recorded by the strain gauge during the test.

Additionally, video footage of each experiment was captured for both the cross-aisle view of the full rack and close-up cross-aisle view of the baseplates. The video footage can be accessed at https://www.youtube.com/playlist?list=PLrk23sPXDwJH1cBpIcfhcJZXdCv2IvFnnz.

### 3.2.4 Ground motions

For the purpose of selecting appropriate ground motions, a target spectrum was derived based on Clause 5.5.2 of NZS 1170.5:2004 (NZS 2004). The target spectrum was computed using a site in Wellington with subsoil class C. A return period factor of 1.0 and distance to the nearest fault of 4 km were assumed. The structural performance factor was 0.7. A
set of ground motions, consisting of the 44 record ATC-63 Far-Field Record suite (Applied Technology Council 2009) and the 756 record New Zealand Strong-Motion Database (Van Houtte et al. 2017), were considered for inclusion in the testing regime. Three ground motions were selected based on the following criteria after scaling to fit the target spectrum:

1. Displacement does not exceed the shaking table actuator working range of ±185 mm, and

2. Scale factor, when scaled to best fit the design target spectrum between 0.5 s and 1.5 s, is within $0.3 \leq k_1 \leq 3.0$ with $D_1 < \log (1.5)$ as required by Clause 5.5.2 of NZS 1170.5:2004 (NZS 2004). The scaling range of 0.5 s to 1.5 s was chosen to account for the initial period of the structure (0.6 s) up to a rocking period corresponding to a sway of about 85 mm. The larger period exceeds the recommended scaling range from NZS 1170.5:2004 which does not take into account the period increase due to the initiation of rocking.

The selected three ground motions were Kaikoura 2016, Northridge 1994, and Kobe 1995, shown in Figure 3.7. The Kaikoura record was chosen as the best fit to the target spectrum, the Northridge record was chosen as it had a strong spectral peak at 0.6 s, corresponding to the non-uplifting structural period of the rack, and the Kobe record was chosen as it had a large spectral peak at 1.2 s expected to correspond closer to the rack’s effective period during rocking. The ground displacement records for these three ground motions are shown in Figure 3.7 and the scaled spectra are shown in Figure 3.8.

Each rack specimen was subjected to a sequence of ground motions of increasing intensity. The testing sequence, shown in Table 3.1, began at 25% scaling of the design level Kaikoura ground motion. Following each ground motion test, the rack was inspected for damage. Provided the rack had not failed, it was subjected to the next ground motion, up
Figure 3.7: Unscaled ground motion displacement records.

Figure 3.8: Ground motion spectra scaled to the target spectrum.
Table 3.1: Test ground motion sequence.

<table>
<thead>
<tr>
<th>Ground motion</th>
<th>Scale factor</th>
<th>Design level</th>
<th>Test designation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaikoura 2016</td>
<td>0.617</td>
<td>25 %</td>
<td>SD-01 HD-01 UA-01</td>
</tr>
<tr>
<td>Kaikoura 2016</td>
<td>1.234</td>
<td>50 %</td>
<td>SD-02 HD-02 UA-02</td>
</tr>
<tr>
<td>Kaikoura 2016</td>
<td>1.851</td>
<td>75 %</td>
<td>SD-03 HD-03 UA-03</td>
</tr>
<tr>
<td>Kaikoura 2016</td>
<td>2.468</td>
<td>100 %</td>
<td>SD-04 HD-04 UA-04</td>
</tr>
<tr>
<td>Northridge 1994</td>
<td>0.988</td>
<td>100 %</td>
<td>SD-05 HD-05 UA-05</td>
</tr>
<tr>
<td>Kobe 1995</td>
<td>0.983</td>
<td>100 %</td>
<td>SD-06 HD-06 UA-06</td>
</tr>
<tr>
<td>Northridge 1994</td>
<td>1.235</td>
<td>125 %</td>
<td>SD-07 HD-07 UA-07</td>
</tr>
<tr>
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<td>SD-08 HD-08i UA-08</td>
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<td>150 %</td>
<td>SD-09 UA-09ii</td>
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<td>200 %</td>
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<tr>
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<td>1.729</td>
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<tr>
<td>Kobe 1995</td>
<td>2.258</td>
<td>230 %</td>
<td>SD-12</td>
</tr>
</tbody>
</table>

\[i\] Failure of concrete slab by baseplate anchor bolt pull-out
\[ii\] Failure by overturning of the rack

to a peak design level of 230 %. A new storage rack was constructed for each baseplate type (three racks were tested in total).

### 3.3 Experimental results and discussions

#### 3.3.1 General observations

**Standard ductile (SD) baseplate**

The SD rack showed little to no uplift during ground motion tests SD-01 through SD-03. During ground motion test SD-04, the uplift was a result of the bolt sliding within the slotted opening in the baseplate stub, but larger uplifts were seen during ground motion tests SD-05 and SD-06 resulting in flexure of the baseplate floorplate as shown in Figure 3.9(a).

Following the design level ground motions (up to SD-06), no visible damage to the baseplates or rack structure was found. During test SD-10, the floorplate was permanently deformed and small fractures were visible in the floorplate to stub welding, shown in Figure 3.9(b). Inelastic deformation and weld peeling increased during tests SD-11 and SD-12, the latter being 2.3 times the design level Kobe ground motion, but no damage was observed for the rack structure. Some damage to the concrete anchor bolts occurred in the forms of loosening and deformation of the bolt head, as seen in Figure 3.9(c).
Heavy-duty (HD) baseplate

No uplift was observed for the HD rack for ground motion tests HD-01 through HD-03. The HD rack survived the design level ground motions with no visible damage. During the HD-04 through HD-08 tests, the uprights were subjected to tension, but the uplift was limited to bolt-slip in the upright web perforations (a maximum of 11 mm).

The foundation concrete block of the central upright frame failed under anchor bolt tension at 1.5 times the design level Kobe ground motion, as shown in Figure 3.10. However, neither the rack structure nor baseplates were visibly damaged. The test was nevertheless discontinued.

By improving the foundation strength and anchoring detail it is possible that the HD rack could survive a stronger earthquake than HD-08. However, given that the same foundation and anchoring detail that was used for the HD rack was used for the SD rack, it is clear that the HD rack resulted in greater load demand on the baseplate-foundation connection.

For ground motion tests HD-05 through HD-08, the pallets were observed to slide in the cross-aisle direction despite being clamped to the beams. Occasionally the sliding pallets would catch on the inside of the beam causing a lateral impact load on the structure. It appeared that the concrete failure coincided with one of these pallet impacts.
Unanchored (UA) baseplate

Uplift was observed for the UA rack for ground motion tests UA-04 and above. During test UA-04, the rack ‘walked’ 100 mm in the down-aisle direction from its original position as shown in Figure 3.11(a). The rack was also observed to walk in the down-aisle direction during the UA-05 through UA-09 tests, but with smaller displacements due to the relatively shorter duration of the Northridge and Kobe records. The rack was re-centred to its original starting position between each test.

During the unanchored rack tests, the upright bases were observed to twist about the vertical axis, shown in Figure 3.11(b). The twisting occurred during rocking when the uplift was smaller than about 20 mm and reduced when rocking uplift was larger. The twisting motion appeared to be due to the eccentric load applied by the lowest bracing member which is connected to the upright 112 mm above its base. The end upright frames, each of which carried half the load of the central upright frame, had a visibly larger amount of twist than the uprights of the central upright frame.

While the HD rack failed by foundation concrete breaking up at 1.5 times the design level Kobe ground motion, the unanchored rack survived the same shaking. Failure of the UA rack only occurred during test UA-09 by overturning, at 1.5 times the design level Northridge ground motion.

3.3.2 Test results

The axial load and relative displacement time histories for the design level tests XX-05 and XX-06 (where XX is the baseplate type) are shown in Figure 3.12 for the Northridge and the Kobe ground motion tests. Axial loads were estimated from the strain gauge readings using the method described in Section 3.2.3. Larger axial loads were recorded for the HD rack.

The HD and SD racks had very similar displacement time histories for both ground motions,
(a) Displacement in down-aisle after test UA-04

(b) Twisting during far side uplift

(c) Overturned rack after UA-09

**Figure 3.11:** Response of rack with UA baseplates.

---

**Figure 3.12:** Upright base axial load and top level displacement (relative to shaking table) for Northridge and Kobe ground motions scaled to design level
but the SD rack did not go into uplift (upright tension) at any point of the design level tests. The UA rack had larger displacements than the SD and HD racks, and the axial loads alternated between zero and twice the static gravity load as the rack was rocking.

Displacement time histories of the SD rack are shown in Figure 3.13. The test results are arranged by ground motion record to highlight the effect of increasing the ground motion scaling factor. It can be seen that the larger scaling factors resulted in not only larger amplitudes, but also longer periods.

The peak relative displacements and accelerations at each level of the racks, in addition to the peak strains in both compression and tension, are given in Table 3.2.

The peak values for the top-level displacement of the centre frame, and the strain at the base of the front upright of the central frame, taken as the average value across the two strain gauges, are shown in Figure 3.14. For a given ground motion record, increasing the ground motion scaling typically increased peak displacement and upright base strain. The one exception was for test SD-09 (Northridge record at 150 % design level) which had a lower peak strain than SD-07 (Northridge record at 125 % design level). All tests
**Table 3.2:** Peak displacement, acceleration and strain recorded for each test.

<table>
<thead>
<tr>
<th>Test</th>
<th>Design level</th>
<th>Displacement (mm)</th>
<th>Acceleration (g)</th>
<th>Strain (µε)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Lvl1  Lvl2 Lvl3</td>
<td>Lvl1  Lvl2 Lvl3</td>
<td>Comp.  Tens.</td>
</tr>
<tr>
<td>SD-01</td>
<td>25 %</td>
<td>2      5      5</td>
<td>0.1   0.1  0.1</td>
<td>-478   -</td>
</tr>
<tr>
<td>SD-02</td>
<td>50 %</td>
<td>4      8      10</td>
<td>0.1   0.1  0.1</td>
<td>-547   -</td>
</tr>
<tr>
<td>SD-03</td>
<td>75 %</td>
<td>8      15     22</td>
<td>0.2   0.2  0.2</td>
<td>-326   -</td>
</tr>
<tr>
<td>SD-04</td>
<td>100 %</td>
<td>19     37     54</td>
<td>0.4   0.9  0.7</td>
<td>-552   -</td>
</tr>
<tr>
<td>SD-05</td>
<td>100 %</td>
<td>36     72     116</td>
<td>2.9   4.4  9.7</td>
<td>-681   -</td>
</tr>
<tr>
<td>SD-06</td>
<td>100 %</td>
<td>40     75     116</td>
<td>1.7   3.9  4.1</td>
<td>-576   -</td>
</tr>
<tr>
<td>SD-07</td>
<td>125 %</td>
<td>48     92     124</td>
<td>3.5   6.4  5.8</td>
<td>-690   -</td>
</tr>
<tr>
<td>SD-08</td>
<td>150 %</td>
<td>55     109    171</td>
<td>2.4   4.7  5.7</td>
<td>-704   19</td>
</tr>
<tr>
<td>SD-09</td>
<td>150 %</td>
<td>56     108    165</td>
<td>3.5   4.8  5.7</td>
<td>-908   13</td>
</tr>
<tr>
<td>SD-10</td>
<td>200 %</td>
<td>95     180    269</td>
<td>4.2   5.7  5.8</td>
<td>-848   129</td>
</tr>
<tr>
<td>SD-11</td>
<td>175 %</td>
<td>90     172    255</td>
<td>3.8   3.1  4.6</td>
<td>-802   5</td>
</tr>
<tr>
<td>SD-12</td>
<td>230 %</td>
<td>98     188    269</td>
<td>2.9   2.3  5.6</td>
<td>-824   169</td>
</tr>
<tr>
<td>UA-01</td>
<td>25 %</td>
<td>4      5      4</td>
<td>0.0   0.1  0.1</td>
<td>-250   -</td>
</tr>
<tr>
<td>UA-02</td>
<td>50 %</td>
<td>3      7      9</td>
<td>0.1   0.1  0.1</td>
<td>-496   -</td>
</tr>
<tr>
<td>UA-03</td>
<td>75 %</td>
<td>7      13     21</td>
<td>0.2   0.3  0.2</td>
<td>-335   -</td>
</tr>
<tr>
<td>UA-04</td>
<td>100 %</td>
<td>35     55     86</td>
<td>0.7   1.1  0.6</td>
<td>-460   -</td>
</tr>
<tr>
<td>UA-05</td>
<td>100 %</td>
<td>166    328    489</td>
<td>0.8   0.8  1.0</td>
<td>-447   -</td>
</tr>
<tr>
<td>UA-06</td>
<td>100 %</td>
<td>96     190    281</td>
<td>0.6   0.8  0.6</td>
<td>-367   -</td>
</tr>
<tr>
<td>UA-07</td>
<td>125 %</td>
<td>184    366    544</td>
<td>0.7   1.1  0.7</td>
<td>-428   -</td>
</tr>
<tr>
<td>UA-08</td>
<td>150 %</td>
<td>140    271    402</td>
<td>0.7   0.9  1.0</td>
<td>-492   -</td>
</tr>
<tr>
<td>UA-09</td>
<td>150 %</td>
<td>-      -      -</td>
<td>-     -   -</td>
<td>-375   -</td>
</tr>
<tr>
<td>HD-01</td>
<td>25 %</td>
<td>3      5      4</td>
<td>0.1   0.1  0.1</td>
<td>-274   -</td>
</tr>
<tr>
<td>HD-02</td>
<td>50 %</td>
<td>4      7      9</td>
<td>0.1   0.1  0.2</td>
<td>-321   -</td>
</tr>
<tr>
<td>HD-03</td>
<td>75 %</td>
<td>6      11     16</td>
<td>0.2   0.2  0.3</td>
<td>-415   -</td>
</tr>
<tr>
<td>HD-04</td>
<td>100 %</td>
<td>7      14     19</td>
<td>0.2   0.3  0.3</td>
<td>-492   47</td>
</tr>
<tr>
<td>HD-05</td>
<td>100 %</td>
<td>24     53     72</td>
<td>6.6   6.1  6.9</td>
<td>-707   331</td>
</tr>
<tr>
<td>HD-06</td>
<td>100 %</td>
<td>21     42     57</td>
<td>4.1   5.4  5.3</td>
<td>-545   103</td>
</tr>
<tr>
<td>HD-07</td>
<td>125 %</td>
<td>30     69     97</td>
<td>3.9   6.2  6.9</td>
<td>-915   500</td>
</tr>
<tr>
<td>HD-08</td>
<td>150 %</td>
<td>45     91     126</td>
<td>2.4   4.0  6.2</td>
<td>-907   298</td>
</tr>
</tbody>
</table>
remained within the elastic range of the upright members, as the yield strain was 2250 µε.

The peak displacement of the UA rack was sensitive to the ground motion record. The peak displacements recorded during the Kaikoura, Northridge and Kobe records at 100% design level were 86 mm, 489 mm and 281 mm, respectively. The corresponding peak strains were 270 µε, 209 µε and 237 µε, which did not vary significantly from each other. Conversely, the peak displacement of the SD or HD rack was not sensitive to the ground motion record, but their peak strain was.

### 3.3.3 Discussion of test results

The fact that the HD rack experienced foundation failure at 1.5 times the design level Kobe ground motion, while the UA rack survived, highlights the potential benefit of not anchoring a rack compared to using a heavy-duty baseplate. Foundation failure due to anchor pull-out is expensive to repair, so leaving the rack unanchored not only eliminates the cost of anchoring, but also avoids the high cost of foundation repair.

Even better performance can be achieved by employing an appropriately ductile baseplate, as the SD rack survived 1.5 times the design level Northridge ground motion, which caused the UA rack to overturn. At 2.3 times the design level Kobe ground motion, the damage of the SD rack was confined to the baseplates, with the weld between the floorplate and the stub about to fracture completely. At a stronger ground motion, failure of the baseplate
would avoid the anchor bolt damaging the foundation concrete of the SD rack.

The displacement of the UA rack was sensitive to the specific ground motion record applied. This is evident from the significantly different peak displacements recorded for all three ground motions at 100% design level, as shown in Figure 3.14. The overturning stability of rocking blocks and structures subjected to ground motion shaking do not follow any systematic trends (Yim et al. 1980), so may not be relied upon for design purposes.

3.4 Conclusions

This chapter has presented the results of a series of 29 shaking table tests performed on three full-scale selective storage racks in the cross-aisle direction. Three baseplate types, being standard ductile (SD), heavy-duty (HD), and unanchored (UA), were tested up to 2.3 times the design level, or until failure.

The SD and UA racks experienced rocking at design level (and above) ground motions, and therefore had extended periods of vibration, which increased their resilience. The HD rack did not rock during the tests, but experienced foundation concrete failure at 1.5 times the design level Kobe ground motion (due to anchor pull-out), which was survived by both the SD and UA racks.

While the UA rack survived the simulated ground motions better than the HD rack, it was subject to substantial displacements under certain ground motions. The UA rack overturned at 1.5 times the design level Northridge ground motion, while the SD rack survived 2.3 times the design level Kobe ground motion. For stronger ground motions, the weld between the floorplate and the stub of the SD baseplate would fail, preventing failure of the foundation concrete. Using the SD baseplate, therefore, appears to be the most advantageous.
Chapter 4

Refined equivalent static method

This chapter is a modified version of the published work:


The introduction (Section 4.1) has been shortened to avoid repetition of information from Chapter 1. Some wording has been changed to maintain consistency with other chapters. Section 4.6.3, which comes from Maguire et al. (2020b), has been included which compares the shaking table test results to the equivalent static method for both NZS 1170.5:2004 and the refined procedure.

Abstract

This chapter is concerned with the equivalent static method for the seismic design of selective storage racks with uplifting baseplates. It proposes a refined procedure for determining the effective natural period of such racks in the cross-aisle direction for use in the NZS 1170.5 equivalent static method. The procedure is derived by comparing the base shear results of trialled Rayleigh methods against nonlinear time history analysis results involving 15 upright frame configurations, comprising 5 baseplate types for three, five and seven level racks. The time history analyses use a suite of 44 ground motion records. It is recommended that the effective natural period be computed in the Rayleigh method using nonlinear static analysis with the storage loads included, where the Rayleigh lateral loads create an overturning moment equal to the restoring moment of the storage loads. The refined procedure leads to more efficient designs of storage racks with uplifting baseplates compared to the conventional procedure based on the use of linear analysis in
the Rayleigh method for determining the natural period, but results in more conservative designs compared to the use of nonlinear time history analysis.

4.1 Introduction

In the equivalent static method, the magnitudes of the horizontal loads are computed based on the (first) natural period of the structure. In design practice, the natural period is most commonly determined using the Rayleigh method (SNZ 2004). However, for a rocking rack, there are two fundamental problems in the application of the equivalent static method. First, the rocking period is not the natural period of the structure before rocking. Second, the natural period computed from the Rayleigh method depends on the magnitude of the Rayleigh lateral loads applied to the structure subject to uplifting.

If the lateral loads assumed in the Rayleigh method do not cause uplift of the upright on the “windward” side, then the computed natural period is that of the structure that does not rock, and is therefore too short (conservative). It should be noted that the use of linear analysis in the Rayleigh method invariably leads to such an outcome. On the other hand, if the assumed lateral loads are high enough to cause uplift (in a nonlinear analysis), then the computed natural period can be much longer, although it is unknown whether it would lead to an unsafe design.

The present study aims to derive a procedure for determining the effective natural period of a selective storage rack that is subject to rocking during earthquake, to be used in the equivalent static method prescribed in Section 6.2 of NZS 1170.5:2004 (SNZ 2004). For this purpose, 15 upright frame configurations comprising 5 baseplate types for three, five and seven level racks have been studied using nonlinear time history analyses under a suite of 44 ground motion records from the ATC-63 Far-Field Record suite (California State University, Chico 2015). The criterion for the new procedure is that it must lead to more efficient designs compared to the conventional procedure of using linear analysis in the Rayleigh method, but cannot underestimate the design forces obtained in the nonlinear time history analysis.

4.2 Rack Configurations and Modelling

4.2.1 Upright frame

Planar finite element models of selective rack upright frames in the cross-aisle direction were developed using the OpenSees software framework (McKenna 2016), as illustrated in Figure 4.1 for a three-level rack. The upright frame model consists of elastic beam-column elements as upright members, and truss elements as bracing members.

The upright nodes are vertically spaced at a distance equal to the bracing pitch (600 mm)
with a frame width of 900 mm. Two pitches of X-bracing are provided, starting at the frame base, with a K-bracing pattern above continuing to the top. Two horizontal braces are provided at the top of the upright frame.

Additional upright nodes are placed at each beam level, with the first beam level at 1.425 m and each subsequent level 1.4 m above the previous. At each beam level, a node is located in the middle of the frame connected to a node at the pallet’s centre of mass, 249 mm above the beam level, by a rigid link element. The node at the pallet’s centre of mass has a nodal mass in the vertical and horizontal directions, equal to the mass of the pallet.

Steel material properties applied to the upright and bracing elements are: elastic modulus of 200 GPa, shear modulus of 80 GPa, yield stress of 450 MPa and density of 7850 kg m$^{-1}$. Three sets of upright and bracing profiles were selected to match the respective pallet loads of the three, five and seven level racks studied in the present work. The area and the second moment of area (about the Z-axis as shown in Figure 4.1) of the upright and bracing sections are shown in Table 4.1.

Clause 6.1.3.1 of NZS 1170.5:2004 (SNZ 2004) restricts the application of the equivalent static method to regular structures with natural periods up to 2 s, or up to 0.4 s for irregular structures, unless the structure is not taller than 10 m. The tallest model is 9.825 m.

4.2.2 Baseplate Types

The five baseplate types included in the present study are a standard ductile (SD), a heavy-duty (HD) and three linear spring (LS02, LS04, LS40) baseplates. The SD and HD baseplates correspond to Dexion 90 MD and Dexion 90 XHD baseplates, shown in Figure 4.2. The LS02, LS04 and LS40 baseplates allow uplift in tension at a linear stiffness of 2 kN mm$^{-1}$, 4 kN mm$^{-1}$ and 40 kN mm$^{-1}$, respectively. The stiffness of the LS02 and
<table>
<thead>
<tr>
<th>Element</th>
<th>Rack levels</th>
<th>Area ($\times 10^{-6}$ m$^2$)</th>
<th>$I_z$ ($\times 10^{-9}$ m$^4$)</th>
<th>Self mass (kg/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upright</td>
<td>3</td>
<td>474</td>
<td>152</td>
<td>3.72</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>611</td>
<td>337</td>
<td>4.80</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>1028</td>
<td>1237</td>
<td>8.07</td>
</tr>
<tr>
<td>Bracing</td>
<td>3</td>
<td>13.8</td>
<td>-</td>
<td>1.19</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>23.0</td>
<td>-</td>
<td>1.98</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>32.2</td>
<td>-</td>
<td>2.77</td>
</tr>
</tbody>
</table>

Table 4.1: Section properties of upright and bracing elements.

(a) Standard ductile baseplate.  
(b) Heavy-duty baseplate.

Figure 4.2: Baseplate types.

LS04 baseplates were chosen as two intermediate values between the effective stiffnesses of the SD and HD baseplates. The load-displacement relationships of the SD and HD baseplates were determined by cyclic axial loading tests, shown in Figure 2.4.

In tension, the SD baseplate had an initial stiffness of 1.75 kN mm$^{-1}$ up to an axial load of 1 kN, where bolt slip began. The baseplate stub of the SD baseplate had a slotted bolt opening that allowed about 3.5 mm of slip. The baseplate then continued to deform elastically at its initial stiffness before yielding at 11 kN. After yielding, the baseplate plastically deformed as shown in Figure 4.3. The SD baseplate can undergo large displacements, allowing a storage rack to rock.

The HD baseplate is significantly stiffer than the SD baseplate, with an elastic stiffness of 12.9 kN mm$^{-1}$. The HD baseplate was connected to the upright web with four bolts as shown in Figure 4.2(b), resulting in a shorter bolt slip of 2.5 mm at an axial tension of 1.5 kN.
4.2.3 Baseplate modelling

Each baseplate was modelled as a zero-length element connecting the node at the base of the upright to a vertically restrained ‘floor node’ at the same location. The five baseplate types were differentiated by the response models applied to the zero-length element in the vertical axis, shown in Figure 4.4. All baseplates have stiffness in compression of 40 kN mm$^{-1}$, which approximates the stiffness of the concrete slab as found from the experimental tests.

The SD and HD models were built using elastic perfectly-plastic gap materials in parallel to simulate the bolt-slip behaviour. The HD baseplate model used an elastic multi-linear material in addition to the gap materials.

Frame rocking was achieved in the model by applying the boundary conditions shown in Figure 4.5 to the floor nodes. The floor node on the left-hand side was restrained in all DOFs while that on the right-hand side was restrained in the vertical and rotational...
DOFs. The floor node on the right-hand side was free to move in the horizontal direction to allow for rotation of the structure during uplift, shown in Figure 4.6. In Figure 4.5, the zero-length elements are shown to have a finite length only to illustrate the existence of the floor and upright base nodes.

4.2.4 Model validation

The finite element methodology was validated against full-scale shaking table tests of a three-level, two-bay selective rack shown in Figure 4.7, using time history analyses. The rack was loaded with six 800 kg pallets with a centre of gravity at 249 mm above the beam level. The pallets were clamped onto the rack beams to prevent sliding and pallet shedding. Suitable baseplate connection conditions were achieved by fixing three 40 MPa reinforced concrete blocks to the shaking table with post-tensioned bars and anchoring the baseplates to the concrete blocks with two 75×12 mm concrete screw anchors.

The experiment showed that half of the structure weight was carried by the central upright frame, with the two outer frames carrying a quarter of the structure weight each. The finite element model represents the central upright frame in the experiment.

The cross-aisle displacement at the central upright frame was measured using a wire transducer mounted on a rigid frame and connected to the upright just above the upright-
Figure 4.7: Shaking table test setup.

Figure 4.8: Validation of finite element model.

to-beam connection of the top level. Comparisons of the experimental and the time history analysis results for the rack with the SD baseplate under the Kobe 1995 and Northridge 1994 ground motions are shown in Figure 4.8. The finite element results match the experimental results reasonably well.
4.3 Analysis Procedures

The present study consisted of two stages. The first stage aimed to derive a Rayleigh method based procedure for determining the effective natural period of a rocking rack to be used in the equivalent static method specified in Clause 6.2 of NZS 1170.5:2004 (SNZ 2004). The effective natural period should result in a design base shear comparable to the mean base shear obtained in the nonlinear time history analyses under a suite of 44 ground motion records.

The second stage compared the maximum compression forces of the uprights between the nonlinear time history analyses and the equivalent static method using the effective natural period.

In all analyses, full gravity loading was applied in advance and P-delta effects were taken into account. Gravity loading was applied to the nodes at the centre of gravity of each pallet weighing 7.85 kN. The self-weight of each frame member was applied at the member’s centre of gravity.

For the purpose of determining the site hazard spectrum, a hypothetical site in Wellington with subsoil class C was chosen. A return period factor of 1.0 and distance of 4 km to the nearest major fault was assumed. Ductility factor was used as nonlinear material properties were accounted for in the baseplate models.

4.3.1 First stage

Each upright frame model was subjected to a series of Rayleigh analyses to determine the natural period, beginning with a total Rayleigh lateral load $R$ of 1 kN at increments of 0.5 kN until overturning failure in the nonlinear static analysis. Each Rayleigh lateral load was distributed over the height of the frame in the same manner as the seismic base shear in accordance with Clause 4.6 of the BRANZ Design Guide (Beattie and Uma 2012):

$$R_i = R \frac{W_i h_i}{\sum_{i=1}^{n} (W_i h_i)} \tag{4.1}$$

where $R_i$, $W_i$ and $h_i$ are the Rayleigh lateral force, gravity load and height, at level $i$. $R$ is the total Rayleigh lateral load. The height $h_i$ of each level was taken at the pallet’s centre of mass, not the beam level. The Rayleigh lateral forces $R_i$ were applied at the same location.

For each Rayleigh analysis, the (first) natural period of vibration $T_1$ was computed from

$$T_1 = 2\pi \sqrt{\frac{\sum_{i=1}^{n} (W_i d_i^2)}{g \sum_{i=1}^{n} (R_i d_i)}} \tag{4.2}$$

where $g$ is the gravity acceleration and $d_i$ is the horizontal displacement at level $i$. 

48
Using the period $T_1$, the design base shear $V_{ESM}$ was determined using:

$$V_{ESM} = C(T_1) \frac{S_p}{k_\mu} \sum_{i=1}^{n} W_i$$

(4.3)

where $C(T_1)$ is the elastic site hazard spectrum, shown in Figure 4.9, which is the product of the hazard factor ($Z = 0.4$), return period factor ($R = 1.0$), near-fault factor, and the spectral shape factor. Since $\mu = 1$, $k_\mu = 1$ and according to Clauses 4.4.2 and 5.2.1.

Nonlinear time history analyses were carried out using a suite of 44 ground motion records with the period $T_1$ computed using Eqn (4.2). The ground motions were selected from the ATC-63 Far-Field Record suite developed for FEMA P695 (California State University, Chico 2015). Each ground motion record was scaled to the target spectrum:

$$SA_{target} = \frac{1 + S_p}{2} C(T_1)$$

(4.4)

The scaling factor for each motion was determined by minimising the least square sum of $\log(k_1SA/SA_{target})$ where is the 5% damped ground motion spectra. For ground motions that result in a scaling factor between 0.33 and 3.0, time history analysis using the scaled ground motion was carried out to determine the peak base shear, $V_{THA}$. Ground motions falling outside of the range between 0.33 and 3.0 were not used as recommended by NZS 1170.5 (SNZ 2004).

Each time history analysis was conducted with a time step of 0.01 s. If the analysis failed to converge for a given time step, the time step was reduced to 0.005 s for the next 2.0 s, and then reset to the default 0.01 s.

The effective natural period is the period which, when used in Eqn (4.3), leads to a design base shear $V_{ESM}$ that is representative of the median base shear obtained in the nonlinear
time history analyses under the 44 ground motion records. Since the NZS 1170.5 (SNZ 2004) time history analysis procedure only requires the selection of three ground motions, the median base shear obtained over the 44 ground motion suite remains conservative. The derivation of the Rayleigh based procedure for determining the effective natural period is detailed in Section 4.4.

4.3.2 Second stage

In the second stage, the effective natural period of each upright frame model determined in the first stage was used to conduct the equivalent static method in accordance with Clause 6.2 of NZS 1170.5:2004 (SNZ 2004), using linear static analysis. The design base shear $V_{ESM}$ was distributed over the height of the frame in accordance with Clause 4.6 of the BRANZ Design Guide (Beattie and Uma 2012):

$$F_i = V_{ESM} \frac{W_i h_i}{\sum_{i=1}^{n} (W_i h_i)} \quad (4.5)$$

where $F_i$ is the equivalent static force applied at level $i$.

4.4 Determination of the effective natural period

4.4.1 The computed natural periods of vibration

The computed natural periods of vibration of the frames with uplifting baseplates, determined by the Rayleigh method as expressed by Eqn (4.2), were found to be significantly affected by the magnitude of the Rayleigh lateral loads applied, as shown in Figure 4.10 for the three-level upright frames. For low lateral loads that do not cause uplift, all the frames with uplifting baseplates have the same natural period of approximately 0.4 s.

When the lateral load is large enough to produce uplift, the computed natural period begins to lengthen with increasing lateral load. The point at which uplift occurs can be determined by equating the overturning moment caused by the lateral load with the restoring moment of the pallet weights:

$$\frac{1}{2} D \sum_{i=1}^{n} (W_i) = \sum_{i=1}^{n} (R_i h_i) \quad (4.6)$$

where $D$ is the frame depth as defined in Figure 4.1. For the load distribution described by Eqn (4.1), the uplift threshold can be determined by:

$$R_{uplift} = \frac{1}{2} D \sum_{i=1}^{n} (W_i) \frac{\sum_{i=1}^{n} (W_i h_i)}{\sum_{i=1}^{n} (W_i h_i^2)} \quad (4.7)$$

If the pallet weight at each level is constant:
At Rayleigh lateral loads greater than the uplift threshold, the computed natural periods of the LS02 and LS04 baseplates lengthen at a high rate initially, but asymptote towards 0.92 s and 0.68 s, respectively. The SD and HD baseplates have higher rates of lengthening of the natural period at the uplift threshold due to initial bolt slip. At higher Rayleigh lateral loads, the HD baseplate’s computed natural period shortens due to its high stiffness, while the SD baseplate’s period continues to lengthen due to its softening response as exhibited in Figure 4.4.

It is evident that the natural period of a frame with an uplifting baseplate, computed using the Rayleigh method, is not unique but is a function of the magnitude of the Rayleigh lateral load applied to the frame.
As the stiffness in compression and tension is constant for the LS40 baseplate, the natural periods of the frames having these baseplates are not affected by the magnitude of the Rayleigh lateral loads. For frames having these baseplates, the natural period can be determined by the conventional linear elastic analysis. For the nonlinear baseplate models, the increase in period after uplift is influenced by the storage load and therefore cannot be determined by linear elastic analysis.

4.4.2 Design base shear

The design base shears for a three-level rack with SD baseplate, determined using the equivalent static method (ESM) of Clause 6.2 of NZS 1170.5:2004 (SNZ 2004) and those from the time history analysis (THA), are shown in Figure 4.11. At the shorter assumed natural periods of 0.41 s and 0.40 s (corresponding to Rayleigh lateral loads of 1.0 kN and 2.5 kN, respectively), the equivalent static method is conservative compared to the time history analysis. It should be noted that the natural period should remain constant before the uplift threshold, but small variations arise due to the Rayleigh method only approximating the true period. As the assumed natural period increases from 0.80 s to 1.42 s, the equivalent static method’s design base shear decreases and better matches the median base shear given by the time history analyses. The equivalent static method becomes unconservative as the assumed natural periods increase beyond 1.42 s.

Selection of the period of vibration $T_1$ has a significant effect of the design base shear in the equivalent static method, due to the shape of the elastic site spectrum, which decreases for longer periods such that a structure with a 0.4 s natural period has a design base shear nearly 10 times that of a structure with a 4.0 s natural period.

For time history analysis, the period selection has a smaller effect on the resulting base
shear of the structure since the selection only influences the ground motion scale factor $k_1$. The ground motion is scaled by fitting its spectra to the target spectrum over the period range of $0.4T_1$ to $1.3T_1$. Given that the shapes of the ground motion spectra are typically similar to the target spectrum, time history analysis is less sensitive to the selected period of vibration than the equivalent static method.

However, several ground motion records used in the present study were found to result in scale factors that were sensitive to the assumed natural period $T_1$. For example, the three outliers seen in Figure 4.11 for the periods of 1.42 s, 1.61 s and 1.74 s came from the ATC63 121021 ground motion, shown in Figure 4.12. The ATC63 121021 record produced a low response for periods above 1 s, resulting in an 80 % increase in scale factor when using a $T_1$ of 2.21 s, compared to a $T_1$ of 1.32 s.

For the three-level rack having the LS02 and LS04 baseplates, which are linearly elastic in tension, the base shear values of the equivalent static method and the time history analysis method agreed well with each other as higher natural periods were assumed, as evident in Figure 4.13 for the rack with LS02 baseplates. However, as with the rack with SD baseplates discussed previously, the equivalent static method based on the lower natural periods led to overestimation of the design base shear.

The equivalent static method and the time history analysis method happened to produce consistent base shears across a large range of Rayleigh lateral loads for the racks with HD baseplates, as shown in Figure 4.14 for a three-level rack.

The ratios of the design base shear $V_{ESM}$ determined by the equivalent static method to the median base shear obtained by the time history analyses are summarised for all racks in

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure4.12.png}
\caption{Spectra for the ATC63 121021 ground motion (Loma Prieta, 1989), with the scaling range of $0.4T_1$ to $1.3T_1$ highlighted.}
\end{figure}
Figure 4.13: Design base shears for period values determined using increasing Rayleigh lateral loads for a three-level rack with LS02 baseplates.

Figure 4.14: Design base shears for period values determined using increasing Rayleigh lateral loads for a three-level rack with HD baseplates.

Figure 4.15. The figure shows that the optimal periods for the equivalent static method correspond to Rayleigh lateral loads somewhere between the uplift thresholds and 7.5 kN. For taller racks, the optimal period tends to correspond to the uplift threshold.

For practical purposes, using the natural period computed based on the Rayleigh lateral loads corresponding to the uplift threshold appears to be reasonable, and is therefore proposed in this chapter. Figure 4.15 shows that such an equivalent static method gives design base shears that are more efficient with respect to the time history analysis than those based on the conventional Rayleigh method, yet mostly conservative.

The racks with LS40 baseplates have a constant natural period, and therefore were not affected by the period selection. There was a good match between the equivalent static method and the time history analysis for the three-level rack with LS40 baseplates, but the
equivalent static method gave lower base shear as the rack increased in height. However, the underestimation of the design base shear does not necessarily mean the same for the upright compression forces, as shown in the next section.

4.5 Implications of the refined procedure

This section investigates the implications of the refined procedure in the preceding section for determining the effective natural period of a rack with uplifting baseplates. It should be noted that an underestimation of the design base shear in the equivalent static method does not necessarily lead to underestimation of the upright design force. During ground motion (time history analysis), the peak upright force may not coincide with the peak base shear. The ratios of the design upright compressive force in the equivalent static method to the median peak upright force in the time history analyses are shown in Figure 4.16.

The upright design force given by the equivalent static method $P_{ESM}$ tends to approach the time history analysis median value as the Rayleigh lateral load used to determine the period of vibration increases. At low Rayleigh lateral loads for all rack heights, the baseplate
design force $P_{ESM}$ is overestimated by the equivalent static method by up to 240\% for the more flexible baseplate types (SD, LS02 and LS04), while the stiff baseplates (HD and LS40) remain mostly constant.

In any case, it can be concluded that determining the effective natural period of vibration at the uplift threshold is significantly more accurate than the conventional Rayleigh method based on linear analysis. The periods calculated using the refined procedure are compared to the conventional method in Table 4.3.

4.6 Additional verifications

4.6.1 Pallet mass

An additional suite of simulations was conducted on the three-level rack with double the pallet load (1600 kg per level). As seen in Figure 4.17, the base shear obtained using the Rayleigh lateral loads at the uplift threshold is reasonable for the SD, LS02 and LS04
Table 4.3: Rack effective period, using conventional and refined approaches.

<table>
<thead>
<tr>
<th>N. levels</th>
<th>Period (s)</th>
<th>Conventional SD</th>
<th>LS02</th>
<th>LS04</th>
<th>HD</th>
<th>LS40</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.41</td>
<td>0.82</td>
<td>0.51</td>
<td>0.45</td>
<td>0.69</td>
<td>0.41</td>
</tr>
<tr>
<td>5</td>
<td>0.78</td>
<td>1.47</td>
<td>1.07</td>
<td>0.92</td>
<td>1.36</td>
<td>0.78</td>
</tr>
<tr>
<td>5</td>
<td>1.32</td>
<td>2.45</td>
<td>1.88</td>
<td>1.57</td>
<td>2.24</td>
<td>1.32</td>
</tr>
</tbody>
</table>

The conventional method is unaffected by baseplate type.

Figure 4.17: Ratio of ESM’s base shear to THA’s median peak base shear with 1600 kg pallet loads.

As mentioned in Section 4.2.3, the baseplate stiffness in compression used for the rack models in Section 4.4 was 40 kN mm\(^{-1}\), based on the experimental test results. To assess the sensitivity of the analysis results to the baseplate compression stiffness, another suite of simulations on the three-level rack was conducted with stiffness of 400 kN mm\(^{-1}\). The results plotted in Figure 4.18 show that using the Rayleigh lateral load corresponding to the uplift threshold is still reasonable.

4.6.2 Baseplate compression stiffness

As mentioned in Section 4.2.3, the baseplate stiffness in compression used for the rack models in Section 4.4 was 40 kN mm\(^{-1}\), based on the experimental test results. To assess the sensitivity of the analysis results to the baseplate compression stiffness, another suite of simulations on the three-level rack was conducted with stiffness of 400 kN mm\(^{-1}\). The results plotted in Figure 4.18 show that using the Rayleigh lateral load corresponding to the uplift threshold is still reasonable.

4.6.3 Comparisons to shaking table experiments

The peak upright axial forces observed during the design level shaking table tests, in Chapter 3, and those given by the equivalent static method (ESM) are compared in Figure 4.19. The design axial forces were obtained by two equivalent static method variants: the conventional method, as prescribed in Clause 6.2 of NZS 1170.5:2004 (NZS 2004), and the refined procedure. Nonlinear static analysis was used in both variants for
Figure 4.18: Ratio of ESM’s base shear to THA’s median peak base shear with baseplate compressive stiffness of 400 kN mm$^{-1}$.

Figure 4.19: Comparison of design axial load for rack uprights with the peak experimental upright load

the purpose of determining the design upright forces under the respective equivalent static forces.

In the case of the UA rack, the structure has a negative static stiffness once uplift occurs. For the present work, the design upright axial force is simply taken as the axial force at the point of uplift, which is equal to the upright frame’s total gravity load (or twice the upright’s normal load at rest). There is, therefore, no difference between the two ESM variants as far as the UA rack is concerned.

In determining the frame’s natural period of vibration, the Rayleigh loads were distributed across the levels in accordance with the BRANZ design guide (Beattie and Deam 2006), i.e. the Rayleigh loads varied linearly with the level heights. The equivalent static forces were then determined for the design parameters given in Section 3.2.4, using a ductility factor $\mu = 1.25$. 

58
It can be seen from Figure 4.19 that, while both ESM variants led to conservative estimates of the upright’s design axial forces for the SD and HD racks, the values given by the refined variant are much closer to those obtained in the shaking table tests under the three design level ground motions. The design axial forces given by the refined method are 6% and 11% higher than the peak axial forces obtained during shaking table testing for the SD and HD racks, respectively.

As a matter of interest, under the design level Kaikoura and Northridge ground motion tests, the peak upright axial forces of the UA rack were found to be about 20% higher than the upright frame’s total gravity load (which is twice each upright’s load at rest). However, the assumption of the frame’s total gravity load to be the upright’s design axial force is not necessarily unconservative. Chapter 5 will show that a typical cold-formed steel upright can sustain 0.1 s stomping forces that are at least 15% greater than its static ultimate capacity.

4.7 Application of refined procedure

This section provides instructions on the application of the refined procedure. Equations previously shown are repeated here for convenience. The key difference between the conventional method and the refined method are the determination of the period of vibration.

To determine the period of vibration the Rayleigh Method is used, but rather than an elastic structural model, a nonlinear baseplate model must be used and gravity load must be considered. Consider the example model shown in Figure 4.20.

First, the Rayleigh lateral load should be calculated so that it provides an overturning...
moment equivalent to the restoring moment of the gravity load \( F_{BP-L} = 0 \). Taking the sum of moments about the point \( O \) gives:

\[
\frac{1}{2} D \sum_{i=1}^{n} (W_i) - \sum_{i=1}^{n} (R_i h_i) = 0
\]  

(4.9)

where \( R_i \) is given by the distribution recommended in the BRANZ Design Guide:

\[
R_i = R_i \frac{W_i h_i}{\sum_{i=1}^{n} (W_i h_i)}
\]  

(4.10)

Hence, the total Rayleigh lateral load \( R_t \), from which \( R_i \) can be determined, is:

\[
R_t = \frac{1}{2} D \sum_{i=1}^{n} (W_i) \frac{\sum_{i=1}^{n} (W_i h_i)}{\sum_{i=1}^{n} (W_i h_i^2)}
\]  

(4.11)

By static analysis of the model, the displacements at each level \( d_i \) should be determined. The Rayleigh period can then be determined from:

\[
T_1 = 2\pi \sqrt{\frac{\sum_{i=1}^{n} (W_i d_i^2)}{g \sum_{i=1}^{n} (R_i d_i)}}
\]  

(4.12)

From here, the conventional equivalent static method can be applied to determine the design loads for each member. An elastic model with pinned baseplates should be used for this step.

### 4.8 Conclusions

This chapter has presented a comparison of the equivalent static method and the nonlinear time history analysis method, both as prescribed in NZS 1170.5:2004, as applied to the seismic structural analysis of selective storage racks in the cross-aisle direction. The racks had baseplates that were subject to uplift and rocking during earthquake, and therefore did not have constant natural periods of vibration, which are required to determine the equivalent static forces. A further complicating factor is that, according to NZS 1170.5:2004, the ground motion record used in the time history analysis method should be scaled based on the natural period of the structure.

It was found that using the natural period of vibration determined based on linear analysis in the Rayleigh method results in significant over-estimations of the equivalent static forces.
for racks with standard uplifting baseplates, preventing the benefits of using uplifting baseplates from being realised in the design. Based on the comparison of the base shear results between the time history analysis method and the trialled Rayleigh methods, it is proposed that the effective natural period of vibration be determined in the Rayleigh method using nonlinear static analysis, and be computed for the Rayleigh lateral loads that just result in uplift of the baseplate.

The refined method for determining the effective natural period leads to storage racks with uplifting baseplates that are more efficient than the conventional method of using linear analysis in the Rayleigh method, but tends to be more conservative than the time history analysis method.
Chapter 5

Residual capacity of uprights following stomping

This chapter is a modified version of the published work:


The introduction (Section 5.1) has been shortened to avoid repetition of information from Chapter 1.

Abstract

When selective storage racks are allowed to rock in the cross-aisle direction during an earthquake, the uprights are subjected to short duration high axial forces at stomping. In this chapter, the amplitude of the stomping force needed to compromise the upright’s residual capacity is assessed for 59 configurations using nonlinear inelastic static and dynamic analyses. Parametric studies are performed to investigate the effects of upright length, bracing pitch, section slenderness, torsional restraints and multiple impulses on the residual capacity of an upright due to rocking. Uprights that fail in the flexural-torsional buckling mode perform better than those that fail by local-distortional buckling as the stomping causes permanent local-distortional deformations rather than sweep (torsional deformation). A rack upright that has a greater length, greater thickness and lower torsional restraint tends to have a higher residual capacity (relative to the undamaged capacity). A typical cold-formed steel rack upright can sustain a 0.1 s stomping force that is at least 15% greater than its static ultimate capacity without significant reduction in residual capacity. An implication is that an unanchored upright that survives an earthquake through rocking
may double its storage load during the post-earthquake emergency period. The present shell element analysis results can be used to plan an experimental program for optimising the resilience of storage rack uprights against stomping.

5.1 Introduction

Rocking can amplify axial forces of the uprights when the uplifted side of the structure drops and impacts with the foundation (Azuhata et al. 2007; Priestley et al. 1978). For such a short duration of peak loading, it is of interest to determine the magnitude of impulse that will cause damage leading to a significant reduction in the member’s static ultimate capacity.

Literature on the behaviour of steel members subjected to high strain rates is available in the field of blast impacts, but the impact load is typically applied laterally to the member. High strain rate axial loading of steel tubes has been investigated during axial crushing for applications in the automotive industry (Mamalis et al. 1984). These studies are concerned with the energy absorption of the material rather than the member’s ability to maintain its static ultimate capacity after impulse loading. At the time of writing, literature on the effect of axial impulse loads in the order of the member’s static ultimate capacity has not been found.

During impulse loading, it is important to consider the effects of strain rate on the member’s material properties. At a very high rate of loading, the mechanical properties of cold-formed steel are substantially changed. The yield stress increases with strain rate, leading to increased importance of local buckling due to an increase in cross-section slenderness (Chen and Liew 2005). For impact design, the increase in yield stress is characterised by a dynamic increase factor $DIF_y \geq 1.0$. High strain rates also increase the ultimate tensile strength, but do not affect the modulus of elasticity (Gilsanz et al. 2013).

The dynamic increase factors for the yield stress and the ultimate strength of Grade 350 reinforcing steel bars at a strain rate of $\dot{\varepsilon} = 0.01 \text{ s}^{-1}$ are $DIF_y = 1.18$ and $DIF_u = 1.06$, respectively (DOD 2008). For A36 and A572 grade structural steels (with a measured yield stress of 345 MPa) at a strain rate of 0.1 s$^{-1}$, the values of $DIF_y$ are in the range 1.10–1.12 and those of $DIF_u$ are between 1.00 and 1.05 (Gilsanz et al. 2013). The $DIF_y$’s for tension coupons cut from rectangular hollow sections with a thickness of 3 mm and yield stresses between 450 MPa and 550 MPa were consistently $>1.1$ for strain rates in the order of 0.1 s$^{-1}$ (Ritchie et al. 2017a).

The magnitude of the stomping load on a steel frame can be controlled using friction damping devices at the rocking joint (Wiebe et al. 2013), but the increase in cost may not be suitable for the competitive storage racking industry. If the uprights can be shown to sustain short duration axial impulses with amplitudes significantly greater than their static
ultimate capacity, lighter members can be used for projects where stomping forces govern the design. Information on the attributes of an upright that increase its resilience against stomping forces during a strong earthquake will also be useful.

This chapter aims to assess the magnitudes of stomping forces that can be sustained by typical cold-formed steel rack uprights without significant reductions in their residual capacities. Parametric finite element studies have been conducted to investigate the effects of upright length, bracing pitch, section slenderness and torsional restraint on its resilience against stomping forces during earthquake (rocking). The analysis results can also help devise an experimental program for optimising the resilience of a storage rack that rocks during earthquake, to provide more comprehensive experimental verification to that obtained through the shake table testing.

5.2 Stomping forces during rocking

Stomping forces during rocking were observed during rocking in Chapters 2 and 3. In Chapter 3, a three-level selective rack with two bays in the down-aisle direction was loaded with six 800 kg welded steel pallets, as shown in Figure 5.1, and was subjected to three ground motion records shown in Table 5.1: Kaikoura 2016, Kobe 1995 and Northridge 1994. The ground motions were scaled according to NZS 1170.5 (SNZ 2004) for a typical Wellington site with subsoil class C. The ground motions were applied in the cross-aisle direction, resulting in rocking of the upright frames.

The axial loads of the uprights during the shaking table tests were recorded indirectly using two single-axis strain gauges per upright. The 90-20-L upright cross-section (see Figure 5.3(a)) was used, with the strain gauges installed in line with the neutral axis on the flange outer wall and 200 mm from the base. The strain data sample rate was 100 Hz. It
Figure 5.2: Axial loads of rack upright subjected to Kaikoura 2016 ground motion in the cross-aisle direction.

Table 5.1: The three largest axial loads recorded for each ground motion as a ratio of the peak axial load.

<table>
<thead>
<tr>
<th>Ground motion</th>
<th>Impulse 1</th>
<th>Impulse 2</th>
<th>Impulse 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaikoura</td>
<td>1.00</td>
<td>0.71</td>
<td>0.70</td>
</tr>
<tr>
<td>Kobe</td>
<td>0.75</td>
<td>0.73</td>
<td>1.00</td>
</tr>
<tr>
<td>Northridge</td>
<td>1.00</td>
<td>0.60</td>
<td>0.59</td>
</tr>
</tbody>
</table>

was found that the peak axial loads occurred as (short duration) impulses. The impulses typically had a duration around 0.1 s and could be approximated by a half-sine wave, shown in Figure 5.2.

Table 5.1 shows the three largest relevant stomping forces experienced by the uprights, during each of the authors’ shaking table tests, as ratios of the respective largest stomping forces. The first ratio, which is unity, refers to the largest stomping force. The second and third ratios correspond to the next largest stomping forces after the largest one, as any damage in the form of inelastic deformations would be added after the largest stomping force. The three relevant stomping forces for the Kaikoura earthquake are indicated in Figure 5.2. However, for the Kobe ground motion the maximum peak occurred close to the end of the shaking duration, so the second and third largest forces shown in the table for the present study occurred before the peak.

In any case, it can be seen that the second and third largest stomping forces never exceeded 75% of the largest stomping force for each ground motion tested. This information is used in Sections 5.3.5 and 5.4.8.
5.3 Upright models

5.3.1 Cross-sections and boundary conditions

Four different upright sections, based on the existing ones used in the storage rack industry, were considered, as shown in Figure 5.3. The first number in each section name indicates the web depth in millimetres, the second the wall thickness, and the letter indicates whether it has a rear flange stiffener (H) or not (L). All sections had slotted web perforations for hooked beam-end connectors and circular rear flange perforations for bolted bracing connections, as illustrated in Figure 5.4. Perforations were spaced at 50 mm for the whole length of the upright. The 100-30-H and 120-30-H uprights had circular flange perforations in addition to the rear flange perforations, as shown in Figure 5.4(b).

The upright members were modelled using S4R shell elements (ABAQUS 2014). All element edges at the top and bottom of the upright were constrained to a reference point at the cross-section neutral axis with multi-point constraint (MPC) beam constraints, as shown in Figure 5.5. The top reference point was restrained for all translational degrees of freedom (DOF) and the rZ-DOF. The X, Y and rZ degrees of freedom for the bottom reference point were restrained, and the compressive axial load P was applied to the Z-DOF. Each upright was therefore simply supported.

The upright was considered to be part of an upright frame with an X-bracing pattern, as illustrated in Figure 5.6. Braces were connected at the rear flange perforations of the uprights with a uniform pitch of 600 mm. The element edges on both rear flanges were tied to a reference point at the centreline of the cross-section, which was restrained in the Y-DOF to simulate the restraint provided by the bracing.
5.3.2 Material properties

A bilinear steel material with rate dependency was applied to all elements. The modulus of elasticity was set to $E = 200$ GPa, the yield stress $f_y = 450$ MPa, and the ultimate strength $f_u = 480$ MPa. These nominal properties correspond to G450 sheet steel (Standards Australia 2011) commonly used for cold-formed steel rack uprights in Australia. The plasticity of the steel material was handled through the von Mises yield criterion and the Prandtl-Reuss flow rule with isotropic hardening. Without the means to experimentally determine an appropriate value, a conservative dynamic increase factor based on the results compiled by Ritchie et al. (2017b) was used. This was $DIF_y = 1.1$ applied at a loading rate of $0.4 \, s^{-1}$. The yield stress as a function of strain rate is interpolated using the Cowper-Symonds overstress power law (Cowper and Symonds 1957).
5.3.3 Mesh refinement

Appropriate mesh sizes for the upright models were determined by a series of linear buckling analyses with varying mesh sizes. Using approximately square quadrilateral shell elements of mesh sizes larger than 4 mm, it was found that the elastic buckling eigenvalues were significantly lower than expected. The low eigenvalues were due to a reduction in stiffness caused by the lack of detail of the rounded corners, which were reduced to a single element. It was found that only when the rounded corners were modelled with mesh sizes equal to or smaller than 1 mm, the eigenvalues converged to the expected value.

The final mesh used for the upright models is shown in Figure 5.7. The mesh was generated using approximate mesh sizes of 1 mm for rounded edges, 3 mm for flange and rear flange edges with circular perforations, 5 mm for web edges with slotted perforations, and 10 mm for flat edges with no perforations along the length of the upright. The longitudinal mesh approximate size was 10 mm.

5.3.4 Model validation

To validate the finite element modelling methodology, the upright compression test results obtained by Koen (2008) were used for verification. Interested readers may find details of the geometry and material properties by downloading the reference from https://ses.library.usyd.edu.au/handle/2123/3880.

The three nominally identical specimens tested by Koen (2008) had an average static ultimate capacity of 268 kN, which was closely matched by the ultimate limit load of 266 kN found by the present finite element model. The failure mode of the finite element model also resembles that of the tested uprights, as evident in Figure 5.8.
5.3.5 Model parameters

The parameters investigated in the present work are the upright length, bracing pitch, section slenderness and torsional restraint. The reference value of each parameter was: length = 1400 mm; bracing pitch = 600 mm; section thickness as given in Figure 5.3; cross-section scale = 100%. These values can be assumed for all upright models unless indicated otherwise.

Four upright lengths were considered for each section, being 1000 mm, 1400 mm, 1700 mm and 2000 mm. As mentioned in Section 5.3.1 and indicated in Figure 5.9, the bracing points are restrained translationally in the cross-aisle direction.

The reference value of bracing pitch, which is 600 mm, is typical for an X-brace configuration used for cold-formed steel storage racks in seismic zones. During earthquake, X-bracing at the lower end of an upright frame reduces the bending moment about the upright’s weak axis caused by base shear. However, the K-bracing configuration was also considered in the present study. The pitch of a K-braced upright frame is double that of an X-braced one, being 1200 mm.
Four section thicknesses were considered for the 90-?-H, 100-?-H and 120-?-H uprights, being 2.0 mm, 2.5 mm, 3.0 mm and 3.5 mm. For the 90-?-L upright, only the first three thicknesses were included. Four cross-section scaling factors were tested for the 1400 mm 100–30-H upright, being 90 %, 100 %, 110 %, and 120 %. All cross-section dimensions, including thickness, were scaled uniformly, but the perforation dimensions remained at 100 %.

Unless otherwise noted, each model was subjected to a single impulse in each analysis to determine the residual capacity of the upright, as described in the next section. However, to investigate the effects of multiple impulses during rocking of the upright frame, two series of multiple impulses were analysed, each of which involving three consecutive impulses. In the first series, each upright was subjected to three consecutive impulses of equal amplitude. In the second series, it was subjected to three consecutive impulses with the second and third impulses at 75 % of the first amplitude. The latter analysis is more realistic as indicated previously in Section 5.2.

5.3.6 Loading condition, imperfections and residual capacity

A rack upright is subjected to the maximum compression (and stomping force) when the two adjacent bays are fully loaded, in which case the typical upright is not subjected to bending moment in the down-aisle direction prior to the earthquake due to symmetric loading of the beams. For the purpose of the present work, the upright is assumed to be subjected to axial stomping forces only.

Initial imperfections corresponding to the member and cross-section buckling modes are included in the nonlinear static and dynamic analyses as described in the next section. For open thin-walled sections the ultimate load can be sensitive to the imperfection magnitude (Pastor et al. 2014). Imperfection magnitude for this study is determined by Sections 3.3.2.4 and 3.3.2.5 of AS 4084:2012 (SA 2012), using values for undamaged members.

It is assumed that the reduction of an upright’s capacity under combined compression and bending is proportional to that under compression only, the latter being investigated in the present work.

5.4 Analysis and results

5.4.1 Analysis procedure

The residual capacity $P_{ur}$ of an upright member is defined as the static ultimate capacity of the damaged member following the application of an impulse (or impulses). The residual capacity for each upright member was determined using the following procedure:
Table 5.2: Static ultimate capacity $P_u$ of upright members.

<table>
<thead>
<tr>
<th>Length (mm)</th>
<th>Static ultimate capacity $P_u$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90-20-L</td>
<td>160 247 350 379</td>
</tr>
<tr>
<td>90-25-H</td>
<td>148 219 332 367</td>
</tr>
<tr>
<td>100-30-H</td>
<td>145 198 307 359</td>
</tr>
<tr>
<td>120-30-H</td>
<td>130 168 274 333</td>
</tr>
</tbody>
</table>

Figure 5.10: Impulse loading applied during explicit analysis.

1. Linear buckling analyses are performed to determine the lowest member and cross-section buckling modes, which are then superimposed onto the perfect member geometry as initial imperfections after being scaled by $\delta_o = L/1000$ and $s_o = 0.3t \sqrt{f_y/f_o}$ respectively, as specified by AS 4084:2012 (Standards Australia 2012). The upright model with initial imperfections is referred to hereafter as the undamaged upright.

2. The static ultimate capacity of the undamaged upright $P_u$ is determined by geometric and material nonlinear analysis using the arc length method (Riks 1979) in Abaqus/Standard (ABAQUS 2014). The static ultimate capacity for each upright member is provided in Table 5.2.

3. An axial impulse is applied to the undamaged upright, analysed in Abaqus/Explicit (ABAQUS 2014). The impulse is in the form of a half-sine pulse of duration 0.1 s and amplitude $P_A$ followed by a 2.9 s rest period of zero load, as shown in Figure 5.10. Determination of the analyses duration is explained in Section 5.4.2. The inelastic nodal deformations caused by the impulse are added to the initial imperfections to model the damaged upright.

4. The residual capacity $P_{ur}$ (static ultimate capacity of the damaged upright) is determined by geometric and material nonlinear analysis of the damaged upright model using the arc length method (Riks 1979).
5.4.2 Explicit analysis duration

Following the application of the impulse load in the explicit analysis, a rest period is required to allow elastic vibrations in the member to dissipate so that the inelastic deformations could be determined. A suitable rest period has been determined by comparing the analysis results using periods of 1 s, 2 s, 3 s, 5 s and 10 s. It was found that for a duration equal to or > 3 s, any elastic deformations imposed on the damaged upright model had negligible effects on the upright’s residual capacity. In order to ensure that only permanent (inelastic) deformations are imposed on the damaged upright model, the impulse load shown in Figure 5.10 is used in the present analysis.

5.4.3 Critical impulse amplitude

The critical impulse amplitude $P_{Acr}$ is defined as the amplitude that results in a residual capacity $P_{ur}$ equal to 95% of the undamaged static ultimate capacity $P_u$. If the residual capacity of the upright is lower than 95%, the upright is considered to have a significant capacity reduction. In the present study, each member was analysed using a range of impulse ratios from $P_A/P_u = \{1.00, 1.05, 1.10 \ldots\}$ until the critical amplitude was reached.

The geometric and material nonlinear analysis results for all impulses of the 1400 mm 90-20-L upright are shown in Figure 5.11 as an example. The critical impulse ratio, $P_{Acr}/P_u = 1.27$, was determined by linear interpolation. It should be noted that any residual capacity found to be greater than the static ultimate capacity $P_u$ was taken as equal to $P_u$.

5.4.4 Effect of upright length (with X-bracing)

The effect of upright length on the critical impulse ratio $P_{Acr}/P_u$ was determined by analysis of each upright length, given in Figure 5.9, for each upright section. The results plotted in

![Figure 5.11: Residual capacities and critical impulse ratio of 1400 mm 90-20-L upright](image)
Figure 5.12: Critical impulse ratios.

Figure 5.12(a) show a general increase in the critical impulse ratio with increasing upright length. Damage caused by the critical impulse load typically manifested as local and distortional deformations as seen in Figure 5.13. When subjected to the nonlinear static analysis to determine the residual capacity, the shorter 1000 mm and 1400 mm members failed in local-distortional buckling modes as shown in Figures 5.14(a) and 5.14(b). On the other hand, the 1700 mm and 2000 mm uprights failed in flexural-torsional buckling as evident in Figures 5.14(c) and 5.14(d). The shorter uprights are therefore more vulnerable to stomping damage. An exception was found for the 1700 mm 90–25-H member for which the impulse caused primarily flexural-torsional deformation, as shown in Figure 5.13(c).

5.4.5 Effect of bracing pitch (1400 mm upright)

Using the X-bracing rather than the K-bracing configuration was found to increase the critical impulse ratio for the 90-20-L, 90-25-H and 100-30-H upright sections, as shown in Figure 5.12(b). The smaller pitch of the X-bracing configuration tends to ameliorate the damage incurred by the stomping, which largely manifests in the form of local-distortional imperfections for the 1400 mm uprights. Local-distortional imperfections are prevented at bracing points.
5.4.6 Effect of section slenderness (X-braced 1400 mm upright)

Up to 3.0 mm, increasing the section thickness was found to mostly increase the critical impulse ratio, as shown in Figure 5.12(c). As the thickness increased to 3.0 mm, the upright failure mode changed from a local-distortional buckling dominated one to a member buckling dominated one, and the upright was therefore less susceptible to the local-distortional damage incurred during stomping.

The cross-section scaling study showed that decreasing the member slenderness (by increasing the cross-section scale factor) mostly led to decreasing critical impulse ratios, as shown in Figure 5.12(d). This outcome is consistent with all the preceding parametric results. It is noted, however, that the 90-20-L and 100-30-H uprights do not follow this trend between cross-section scale factors of 90 % and 100 %. It was found that the failure mode of each of these uprights at 90 % cross-section scaling was different from the common mode at the other scaling factors, as shown in Figure 5.15.
5.4.7 Bracing torsional restraint

Although torsional restraint at the bracing connection was not provided to the simulated uprights presented in the preceding sections, some bracing details in practice do provide partial torsional restraint. A subset of the previous models was re-analysed with full torsional restraint at each bracing point to gauge the effect of partial torsional restraint. Results of the simulations with and without torsional restraint at the bracing connections are compared in Figure 5.16.

The models with full torsional restraint at the bracing connections tend to have lower critical impulse ratios than the comparable models with torsional restraints only at the member ends. Torsional restraints had a negligible effect on the 1000 mm uprights, which had the two bracing connections close to the member ends. In general, the lack of torsional restraints tends to increase the critical impulse ratio as it facilitates the flexural-torsional buckling mode, which is hardly affected by the local-distortional damage incurred by stomping.

Furthermore, even for long uprights failing by member buckling rather than cross-section buckling, the absence of torsional restraint can change the buckling mode from flexural...
Figure 5.16: Critical impulse ratios of uprights with and without torsional restraint at bracing connections.

Figure 5.17: Failure modes of damaged 2000 mm 90–25-H members with and without torsional restraint at bracing connections.

to flexural-torsional, as illustrated in Figure 5.17 for the 2000 mm 90–25-H upright. An upright failing in the flexural mode is more susceptible to stomping damage than that failing in the flexural-torsional mode, since stomping damage is unlikely to manifest in a permanent sweep (torsional deformation).

It should be noted that a lower critical impulse ratio does not imply a reduction in the critical impulse amplitude. For example, the presence of full torsional restraints at the bracing points increased the static ultimate capacity $P_u$ of the 2000 mm 90–25-H upright from 168 kN to 210 kN, and the critical impulse amplitude $P_{Acr}$ from 261 kN to 265 kN.
5.4.8 Multiple consecutive impulses

When subjected to multiple consecutive impulses of a given amplitude in the first series (see Section 5.3.5), the residual capacity of the 1400 mm uprights often drastically decreased after each consecutive impulse, as shown in Figure 5.18. The 90-25-H and 100-30-H uprights failed ($P_{ur}/P_u < 0.95$) after two consecutive impulses of amplitude $P_A/P_u = 1.0$ only. The 90-20-L and 120-30-H uprights withstood two consecutive impulses up to $P_A/P_u = 1.10$, and three impulses up to $P_A/P_u = 1.05$. However, it can be seen that the residual capacity quickly decreased after each consecutive impulse of equal magnitude, especially at higher amplitudes.

For the second series, where the second and third impulses were reduced to 75% of the first, the residual capacity was not consistently decreased by subsequent impulses as in the first series. In any case, it can be seen from Figure 5.18 that in the more realistic condition encountered during real earthquakes, only the peak impulse is relevant for determining the residual capacity of an upright. This outcome justifies the analysis procedure described in Section 5.4.1, which was used to generate the results presented in the preceding sections.
5.5 Conclusions

The chapter has presented an extensive parametric study concerning the effects of stomping forces on the residual capacity of a cold-formed steel rack upright. The shell element analyses found that critical stomping of a rack upright causes damage mostly in the form of inelastic cross-sectional deformations. Damage in the form of inelastic flexural-torsional deformations may prevail only in relatively rare cases.

Therefore, cold-formed steel rack uprights that fail in the flexural-torsional mode tend to survive higher impulse amplitudes (relative to their static ultimate capacity) than those failing in the local-distortional mode. Adding a return flange to the section, increasing the member effective length, and/or increasing the wall thickness improve the upright’s resilience against stomping forces during rocking in an earthquake.

While bracing-to-upright connection detailing that provides torsional restraint increases the upright’s static ultimate capacity by a relatively small amount, it can reduce the critical impulse ratio significantly as it leads to more permanent deformations. Bracing detailing that provides low torsional restraint to the upright can increase the upright’s resilience against stomping forces during rocking in an earthquake without significantly reducing the static ultimate capacity.

It has been found that among the 59 upright models analysed, only 9 had a critical impulse ratio \( P_{Acr}/P_u < 1.15 \). Even so, eight of the nine uprights with critical impulse ratios \( < 1.15 \) had their cross-sections modified from actual uprights used in the racking industry by either cross-section scaling or altering thickness; with the ninth being the 2000 mm 90-20-L member with full torsional restraint applied at the bracing points. Overall, the average critical impulse ratio was 1.23. It can therefore be surmised that the majority of practical uprights will survive stomping magnitudes at least 15% greater than their respective static ultimate capacity and retain 95% or more of their undamaged capacity.

When a cold-formed steel rack upright is subjected to consecutive impulses each of a magnitude equal to the critical impulse magnitude, its residual capacity degrades successively. However, in most earthquakes, the maximum stomping forces following the most severe (peak) stomping are only up to 75% of the peak amplitude. If the peak stomping force in an earthquake is equal to or \(< 1.15 \) times the static ultimate capacity, then subsequent stomping will not likely cause further significant permanent deformations.

A practical implication is that, leaving aside the condition and original capacity of the other rack components, during the post-earthquake emergency period an unanchored storage rack upright that survives through rocking may be able to carry a storage load that is double what it sustains just prior to the earthquake. As one side of the upright frame lifts from the base during rocking, the compressed upright carries all the storage load that is
normally shared between the pair of uprights, for a duration of a few seconds or so. This condition means that the upright’s static ultimate (axial) capacity must be at least double its compression force under the storage load just prior to the earthquake. If the amplitude of the peak impulse is not >2.3 times the compression force under the storage load, then the upright’s residual capacity will not be compromised significantly.
Chapter 6

General conclusions

This thesis has presented four research projects that advance the field of knowledge around the seismic analysis and design of cold-formed steel storage racking, specifically for the dynamic rocking response in the cross-aisle direction.

The free vibration/rocking response of a three-level rack for four baseplate types was investigated through snapback tests in Chapter 2. It was determined that the damping ratio for the unanchored rack tested in the present work was 0.034. The damping ratio was significantly increased, up to 0.051, using a ductile baseplate that dissipated energy during uplift cycles while rocking. Additionally, the structure cross-aisle period was shown to be non-constant. During rocking, the period increased as a function of rack sway amplitude. For the stiff heavy-duty baseplates, uplift was relatively small and the effective period was closer to the period of the upright frame with pinned baseplate conditions.

Subjecting racks to shaking table testing in Chapter 3 showed that the concrete floor load demand limited the seismic resilience of the heavy-duty baseplate, with the concrete floor failing in tension at 1.5 times the design level ground motion. Overturning limited the performance of the unanchored rack, which also failed at 1.5 times the design level. The ductile baseplate performed the best, surviving 2.3 times the design level.

The unanchored rack tended to have peak displacements that were sensitive to the specific ground motion record, while the peak axial loads had low sensitivity to the ground motion. On the other hand, the heavy-duty rack peak axial load was sensitive to the ground motion, while peak displacement had low sensitivity to the ground motion. The ductile baseplate rack provided a good middle ground which resulted in greater earthquake resilience than the other rack types.

Using the results of the snapback and shaking table tests, a 2-dimensional finite element model of the upright frame was developed in Chapter 4. The model was used to carry out analysis of 3, 5 and 7 levels racks for five baseplate types (ductile, heavy-duty and...
three linear elastic baseplates of varying stiffness). The NZS 1170.5:2004 equivalent static method was found to be overly conservative for the racks with baseplates that allowed rocking.

It is proposed that the effective natural period of vibration be determined in the Rayleigh method using nonlinear static analysis, and be computed for the Rayleigh lateral loads that just result in uplift of the baseplate. This method resulted in design loads that were conservative with respect to the time history analysis method but were lower than the conventional equivalent static method.

The peak upright axial loading during rocking was the result of stomping, having a duration in the order of 0.1 s. Chapter 5 investigated the residual capacity of the cold-formed steel uprights when subjected to short-duration impulse loads greater than their static ultimate capacity. For standard upright cross-sections, it can be expected that the upright can survive a stomping load 15% greater than its static ultimate capacity with no significant reduction in static ultimate capacity. On average the uprights could sustain stomping loads 23% larger than their static ultimate capacity with no significant reduction in capacity.

The comparisons between the refined equivalent static method and the time history analysis method in Chapter 4 were conservative in two ways. First, comparison was made to the peak axial forces during time history analysis, but the uprights could sustain a short-duration stomping load at least 15% higher than a static axial load of the same magnitude. Second, the time history analysis method from NZS 1170.5:2004 requires the designer to use the maximum loading out of three ground motion records, with no guidance on the ground motion selection, while the comparison used the median values from a larger ground motion set. This provides confidence that the effective period method proposed in Chapter 4 remains conservative despite resulting in lower load demands than the conventional equivalent static method.
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