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SEISMIC PERFORMANCE OF PALLET RACKING SYSTEM WITH FRICTION SLIPPER BASEPLATES IN CROSS-AISLE DIRECTION

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Abstract. *A novel component for significantly enhancing the seismic resilience of a pallet racking system has been developed by allowing the column to uplift with a friction slipper baseplate. A series of shaking table tests on full-scale 4.8-m-high selective racks with four different types of baseplates were conducted under three different ground motions. Each rack was subjected to a sequence of ground motions of increasing intensity up to failure or the shaking table's limit. The test results show that the friction slipper baseplate rack survived all the ground motions up to 2.3 times design level while the rigid heavy-duty baseplate rack failed due to foundation anchor pull-out at much lower design levels. Notably, the friction slipper baseplate rack experienced no damage to the column base, while the ductile baseplate suffered significant damage to both the baseplate and the column base. The test results also demonstrate that the friction slipper baseplates allow for a pre-set column uplift force, and keep the column uplift within a controlled range. These features allow engineers to design a racking system with less steel and in accordance with the available concrete slab pull-out strength.*

1 INTRODUCTION

Recent earthquake events have shown examples where storage racking systems inside steel buildings of similar age suffered severe damage, while the buildings themselves remained undamaged [1]. Often, a minor collision with a rack can lead to the total collapse of the rack system due to the dynamics of falling pallets. In such cases, a single rack frame collapse may trigger a domino effect, causing adjacent racks to collapse and resulting in the failure of racks throughout the warehouse, as depicted in Figure 1. Several past earthquakes, including the 1987 Edgecumbe Earthquake, the 2001 Nisqually Earthquake, the 2010 Darfield Earthquake, and the 2011 Lyttleton Earthquake, have resulted in significant economic losses due to the collapse of pallet racks and loss of contents [2][3][4]. Consequently, seismic resilience has become a critical consideration in the design of racking systems.

Figure 1: A minor collision can result in total collapse due to the dynamics of pallets falling

In conventional steel storage racking system design practice, engineers consider two directions independently, cross-aisle direction as a braced frame and the down-aisle direction as a moment frame. This paper focus on the cross-aisle direction seismic performance of cold-formed steel racking system.

The most commonly observed collapse modes of racking systems in the cross-aisle direction during earthquakes are: (a) collapse caused by upright buckling under high compression force, as shown in Figure 2 and Figure 3; (b) overturning due to large tension force in uprights, insufficient ground anchoring, or poor weld quality between the baseplates and the uprights, as shown in Figure 4.

Figure 2: Cross-aisle direction failure by column buckling [3]

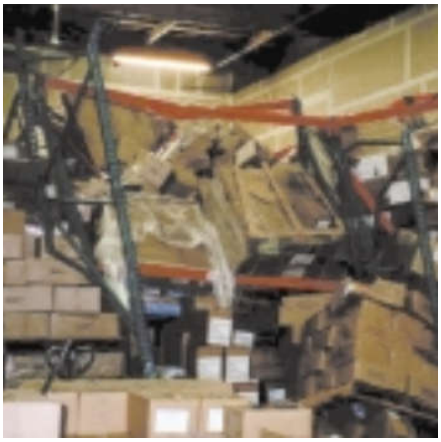


Figure 3: Cross-aisle direction collapse of pallet racking system [2]

Figure 4: Fracture of baseplate connection [4]

The weak link in the cross-aisle direction is the upright-base connection [5], which consists of an upright, a baseplate, and a set of anchor bolts. The behavior of an upright-base connection is largely influenced by the type of baseplate chosen. Conventional baseplates can be categorized into two types: rigid baseplates and flexible baseplates. While rigid baseplates are designed to be robust and resist seismic loads through their material and section properties, they may prove inadequate when exposed to excessive seismic energy, leading to collapse and overturning, as illustrated in the figures above. In contrast, flexible baseplates have been developed to enhance the efficiency of seismic energy dissipation in racking frames by allowing rocking behavior and steel yielding. This has been found to be a more cost-effective solution [6]. However, even in cases where robust and well-anchored, or flexible rocking frame racking systems survive severe earthquakes, the rack components or connections often suffer from fractures or plastic deformation, necessitating replacements. This outcome results in dismantling the racks, leading to significant rehabilitation costs and loss of business continuity.

A minimal-damage and low-cost solution to improve the seismic resilience of the racking system is required which demands a more robust baseplate that dissipates energy but that does not require replacement after a severe earthquake. To address these challenges and improve the seismic resilience of racking systems, the design concept of controlled rocking in the cross-aisle direction has been developed. This concept permits the base of the structure or selected uprights to uplift from the foundation in response to severe lateral loading. To dissipate energy, a friction damping mechanism has been introduced to the system, resulting in the development of the Friction Slipper (FS) baseplate to implement this design concept.

A series of studies have experimentally investigated the local quasi-static behavior and global free-vibration response of a rack frame with friction slipper baseplate when uplifting [7]. It has been found to exhibit large ductility, high energy dissipation capacity, and adjustable upright uplift force through changes in bolt configurations, demonstrating the potential for a low-damage or damage-free design.

In order to further investigate and validate the benefits of this design concept and the friction slipper baseplate, especially concerning its ductility and dynamic behavior, a series of full-scale shaking table tests were conducted to evaluate the seismic performance of a steel racking system with four different types of baseplates, the friction slipper baseplate and three other types of baseplates for comparison under three different ground motions.

2 EXPERIMENTAL WORK

2.1 Shaking table

The shaking table in the Structural Test Hall of the University of Auckland was used for this test. It is a single degree of freedom shaking table with dimensions 4.5 x 3.6 m, and it is capable of a maximum displacement of +/- 180 mm, a maximum velocity of 0.987 m/s and a maximum acceleration of 16.7 m/s2 (1.7 g), when supporting a payload of 10 tons.

2.2 Frame assembly

The tested rack frame assembly design remained constant for all the shaking table tests, and a new rack was used for each test. As shown in Figure 5, each of the tested rack frames had 3 levels (1.4-m height per level) and 2 bays (1.35 m per bay) and was 0.9-m deep, erected on concrete slabs (150-mm-thick, 40 MPa commercial concrete) fixed to the shaking table, having six pallet places in total. Each pallet consisted of four steel billets, each billet weighing 175 kg. Total pallet mass was around 800 kg including the steel fasteners. The steel billets were placed on top of a lattice of hollow steel sections which act to increase the center of mass distance of the pallet above the beam level for 249mm to better represent a typical pallet in practice. Two smaller hollow sections welded on the bottom fitted inside the rack beams and acted as an interlocking key. Threaded bars and bolts were used to tighten the pallet mass and sections together as a whole rigid body to prevent the masses from jumping off the rails.

The section selections are: 90-mm-wide 2-mm-thick uprights; 85-mm-deep, 40-mm-wide double-skin box beams; 2 X-bracings at the bottom and then followed by K-bracing with a pitch of 600mm.

(a):Frame configuration (b) The test setup

Figure 5: Frame configuration and a photo of the test setup

2.3 Baseplates

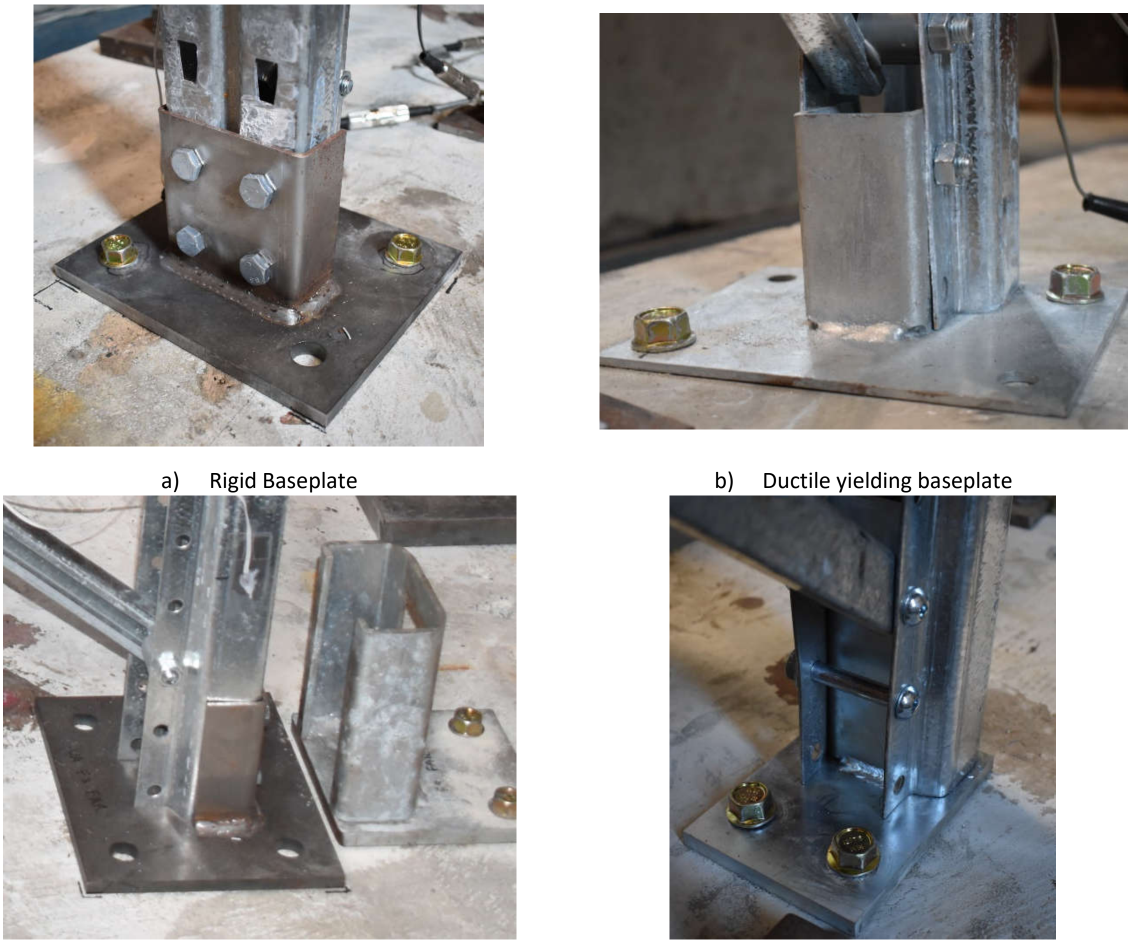
The uprights were fitted with 4 types of baseplate configurations connected to the concrete slabs with anchorage bolts, which enabled a direct comparison of their seismic and rocking behavior:

I: Rigid baseplate (RBP), this type of baseplates are wildly used throughout the world, which does not allow uplift, as shown in Figure 6a. This typical rigid baseplate is made from a 10-mm-thick steel floor plate with a 4-mm-thick vertical stub that fixed to the upright with 4 pieces of M8 bolts. This type of base connection has almost no energy dissipation capacity.

II: Ductile Yielding baseplate (DBP), which allows the frame to uplift at the base and dissipate energy through steel plate yielding, as shown in Figure 6b. This baseplate is made from 3.5-mm-thick ductile steel. With further increment of the axial load, the Ductile baseplate will dissipate seismic energy through developing its uplift and metal plate yielding.

III: Unanchored baseplate (UBP), as shown in Figure 6c, which can be consider as a free-to-rock base with horizontal shear resistance. Since there is not anchored, the energy dissipation through the baseplate is mainly due to the rocking behavior.

IV: Friction baseplate (FBP), as shown in Figure 6d. It dissipates seismic energy through the uplift displacement between uprights and the inner stub with a stable clamping force applied by bolts.



a) Rigid Baseplate

b) Ductile yielding baseplate

c) Unanchored Baseplate

d) Friction Baseplate

Figure 6: Four types of baseplates

2.4 Target spectrum & Ground motions

The following design criteria were used to develop the design ULS seismic spectrum.

Table 1: Target ULS design spectrum design criteria

Criteria	Value
Design working life	50 years
Importance level	2
Location	Wellington
Hazard factor	0.4
Site subsoil class	C (Shallow) soil
Distance to nearest fault	4 km
Structural ductility factor	3.0
Structural performance factor	0.7

Based on the requirements outlined in NZ 1170.5 [8] and the BRANZ design guideline [9], three strong motion records were carefully selected from the New Zealand Strong-motion Database published by GeoNet. The selection process focused on sites with subsoil class "C" and ensured that the chosen records met the specified target spectrum design criteria mentioned above. Moreover, the selected records were constrained to fit within the ultimate capacity of the shaking table. Table 2 presents the details of these three ground motions.

Table 2: Details of selected ground motions

Earthquake event	Event magnitude	Event fault type	Reason for being selected	The best fit scale factor	PGA (m/s2)
Kaikoura 2016 earthquake	M7.6	Oblique-slip	It is the best fit with the target spectrum in the period ranging from 0.5 s to 1.5 s which was observed in the free vibration response with a scale factor of 2.468.	2.46	1.38
Northridge 1994 earthquake	M6.7	Blind thrust	Its dominant period is the closest to those baseplates which have no uplift at the base (around 0.5s), with a scale factor of 0.988, this ground motion is at the design target level intensity	0.99	3.97
Kobe 1995 earthquake	M6.9	Strike-slip	Its considerable energy release in a long period range (1-1.5 s) which is a vulnerable period for the baseplates that enable racks to rock. With a scale factor of 0.983, this ground motion is at the design target level intensity.	0.98	2.35

The loading sequence applied is as follows: The 1st selected ground motion (Kaikoura 2016) was progressively scaled to 0.25, 0.5, 0.75 and 1.0 times the ULS design level intensity of the target spectrum. Following these four, the structure was subsequently subjected to gradually increasing ground motions from the Northridge 1994 and Kobe 1995 events, up to the ultimate capacity of the shaking table. The ultimate capacity of the shaking table corresponded to 1.75 times the design level intensity for the Northridge Earthquake and 2.3 times that for the Kobe Earthquake. In total, 12 ground motions were prepared, spanning the range of intensities that the shaking table could generate. Importantly, it should be noted that throughout each round of the 12 earthquake loadings, damage to the rack frame was accumulated until final failure or the completion of each test round. The precise loading sequence and intensity of the ground motions are itemized in Table 3. The displacement-time history of the three ground motions with a scale factor of 1.0 is illustrated in Figure 7.

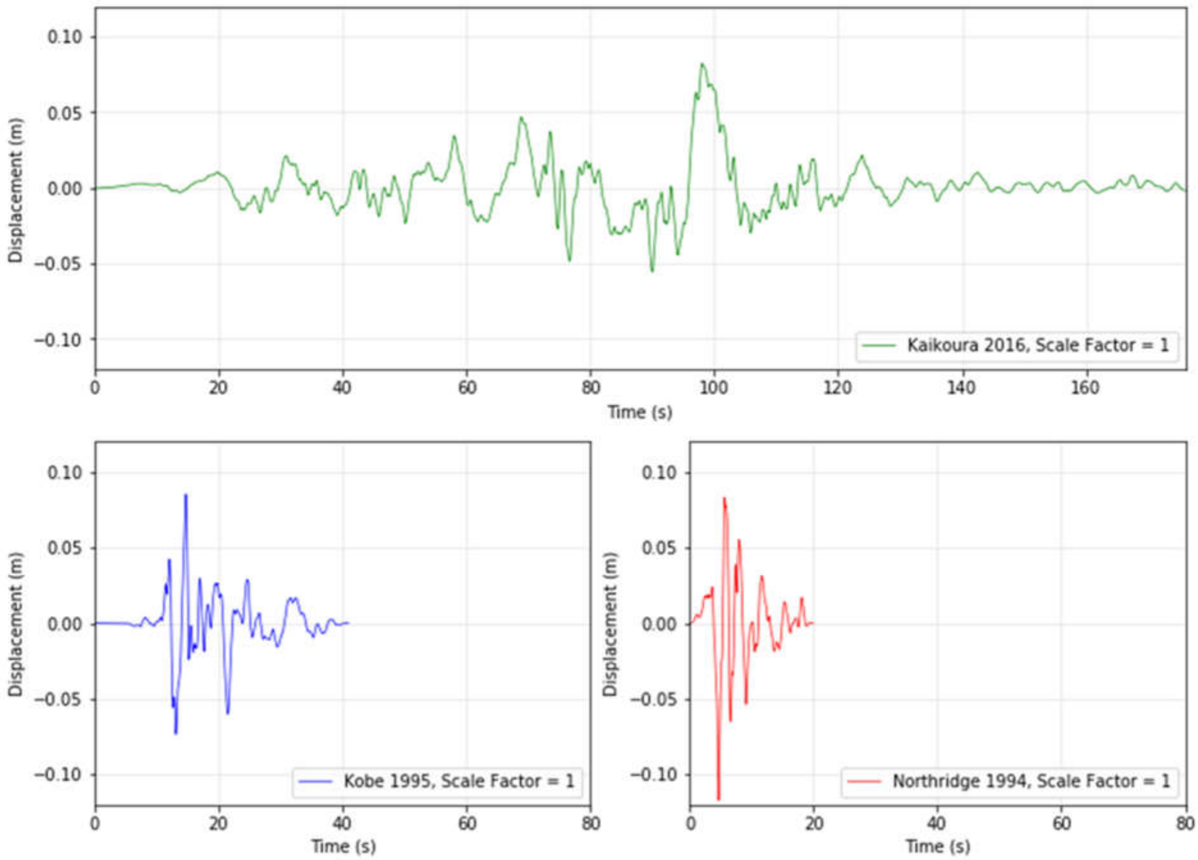


Figure 7: The displacement-time history of three selected ground motions with the scale factor = 1.0

Table 3: Test order, scale factor and the PGA of the generated ground motions

Test order	Scale factor	Ratio of design level	Ground motion	Max acc. (m/s^2)	g
1	0.617	0.25	Kaikoura2016	0.345	0.035
2	1.234	0.5	Kaikoura2016	0.689	0.070
3	1.851	0.75	Kaikoura2016	1.034	0.105
4	2.468	1	Kaikoura2016	1.379	0.141
5	0.988	1	Northridge1994	3.912	0.399
6	0.983	1	Kobe1995	2.170	0.221
7	1.235	1.25	Northridge1994	4.890	0.498
8	1.475	1.5	Kobe1995	3.256	0.332
9	1.482	1.5	Northridge1994	5.868	0.598
10	1.966	2	Kobe1995	4.340	0.442
11	1.729	1.75	Northridge1994	6.846	0.698
12	2.261	2.3	Kobe1995	4.992	0.509

3 OBSERVATIONS

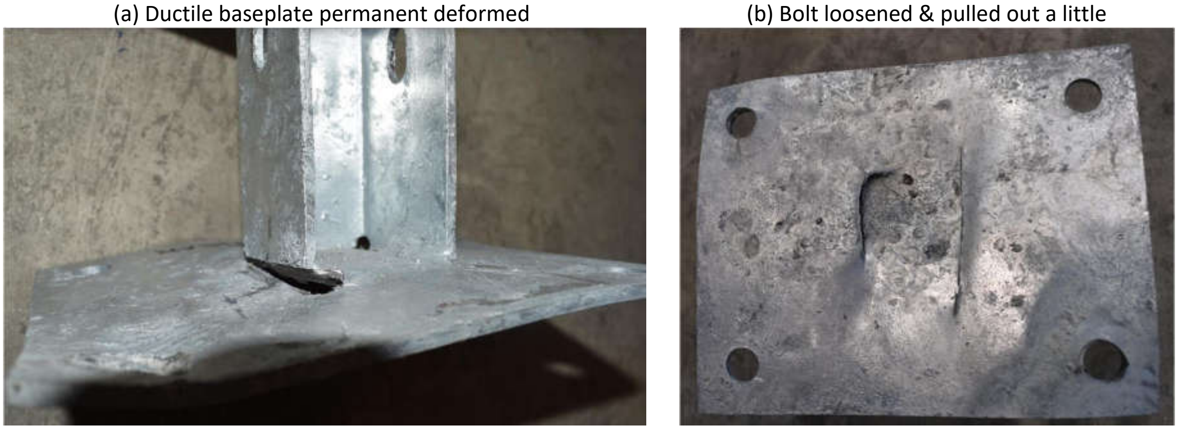
After each shake, the rack frame was carefully inspected for damage. The observations are summarized below:

Ductile yielding baseplate

The rack frame with the ductile yielding baseplate survived all the 12 ground motions with no structural failure observed, even at the ultimate ground motion intensity of 2.3 times the design level earthquake.

The observed damage was primarily concentrated at the baseplate-floor connection. Starting from 1.25 times the design level ground motion, slight permanent bends were evident in the floor plates. Additionally, some uprights exhibited localized twisting or bending, measuring 1-2 mm relative to the floor plates. As the ground motion intensity reached 1.5 times the design level, cracks were observed at the corners of the welds between the stub and the floor plates. As the ground motion intensity increased further, the bending of the floor plates, twisting between uprights and baseplates, and fractures at the welds continued to develop, leading to an accumulation of damage. Following the completion of all 12 ground motions, the floor plates showed extensive yielding and upward bending, as depicted in Figure 8 (a). Many floor plates were torn off at the weld connection to the stub, and some fractures were observed to penetrate through the floor plates, as shown in Figure 8 (c) and (d). In certain cases, the anchor bolts experienced minor pull-out, with a few millimeters of displacement, as shown in Figure 8 (b), but they did not lose their complete pull-out resistance.

(a) Ductile baseplate permanent deformed (b) Bolt loosened & pulled out a little



(c) Ductile baseplate fracture at welding (d) Fracture through the floor plate

Figure 8: Ductile yielding baseplate damage after all 12 ground motions

Unanchored Baseplate

The unanchored rack exhibited noticeable rocking behaviour starting from the 4th ground motion, which corresponds to the Kaikoura 2016 earthquake at 1.0 times the design level ground motion. This behaviour was followed by the other two ground motions, both at 1.0 times the design level, taken from the Northridge 1994 and Kobe 1995 earthquakes. Although these three ground motions had the same design intensity (all at 1.0 times the design level), the response of the unanchored rack frame varied significantly.

After successfully surviving all four gradually increased intensity Kaikoura Earthquake ground motions, as well as the first two Northridge and first two Kobe Earthquakes, the rack overturned during the 3rd Northridge ground motion, which was at 1.5 times the design level earthquake, as shown in Figure 10.

Figure 9: Unanchored baseplate: Restraining cross-aisle movement while allowing frame uplift

Figure 10: Unanchored baseplate frame overturned at design level 1.5x Northridge earthquake

Rigid baseplate

The rack frame with a rigid baseplate survived the first 7 ground motions, up to 1.25 times the design level earthquake, without any noticeable damage. However, during the 8th ground motion, which was at 1.5 times the design level earthquake (Kobe Earthquake), the anchor bolts into the concrete were pulled out due to the very large pull-up force at the upright and baseplate connection. As a result, the concrete slab around the baseplate area was destroyed, as illustrated in Figure 11, down to the full depth of the anchor bolt (80 mm depth). Subsequently, the damaged slab was replaced with a new concrete foundation slab for the remaining tests.

Importantly, the replacement slab had been cast at the same time as the others to ensure consistent concrete age in each foundation slab.

However, this particular failure mode for the pallet racking system is not commonly observed in previous real earthquake events. The reason for this discrepancy lies in the fact that, in reality, during a severe earthquake event, the pallets loaded on the rack frames are typically free to slide in between the beams. In contrast, in this experiment, the pallets are secured to the beams with interlocking keys to fix their position. The utilization of interlocking keys in the experiment restricts the movement of pallets, leading to higher earthquake-induced forces in comparison to regular rack systems where pallet sliding is permitted.

Figure 11: Rigid baseplate anchor bolt pulled out

Friction slipper baseplate

The rack frames equipped with friction slipper baseplates were initially subjected to 12 ground motions and successfully endured all of them without any visible damage. Subsequently, an additional 9 ground motions were applied to the rack frame, excluding the first three less intense ground motions from the original 12. Once again, no visible damage was observed in any of the cases.

4 DISCUSSION

The test results are summarized in Table 4:

Table 4 Test result summary

Baseplate Configuration	Peak Upright Axial Load (kN)*	Peak Top-Level Displacement (mm)	Resilience	Failure Mode
DBP	-88 kN	269	2.3 X design level	Survive all shakes with baseplates fractured
UBP	-39 kN	Overturned	1.5 X design level	Overturned
RBP	+81 kN	126	1.25 X design level	Anchor bolts pulled-out, concrete floor damaged
FBP	-28 kN	425	2.3 X design level	Survive all shakes with no damage found

* Positive value (+) as tension, negative value (-) as compression

A set of strain gauges was affixed to the base of each upright, situated 200 mm above the floor, to measure changes in strain. By analyzing the strain readings, the upright axial load was determined, providing an estimation of the axial load variation. Furthermore, a series of wire displacement transducers were installed at each level of the rack frames to record the rack's displacement. The peak upright axial load and the top-level relative displacement for the middle frame, at each tested frame, is presented in Table 4.

The rack with the Rigid baseplate shows the largest axial load compared to all other baseplate cases and failed at 1.5X design level by anchor bolt pull-out off concrete slab. During an earthquake event, the earthquake-induced energy will accumulate in the structure with a limited dissipation pathway, primarily converting between kinetic energy and potential energy. As the total energy in the system increases, the internal force also escalates until one or more structural members experience failure. This phenomenon was confirmed by this shaking table test.

The rack with the Ductile baseplate exhibited behavior of a free rocking frame with a hook element. The uprights forcefully impacted the floor and rotated around the upright base until restrained on the other side by the baseplate anchored to the floor. By the conclusion of the test, most of the baseplates were fractured, as illustrated in Figure 8. Due to the yielding and fracturing of the steel plate, a substantial amount of seismic energy was dissipated by the baseplate, thereby preventing the overturning and collapse of the rack frame up to 2.3 times the design level. Although this performance surpasses that of the rack with a rigid baseplate, it is important to note that, in the aftermath of a severe earthquake event, the replacement of the baseplate can incur significant time and financial costs, potentially leading to business disruptions. An even better performance was observed with the friction slipper baseplate.

The rack frames equipped with the friction slipper baseplate exhibited no damage after surviving all 12 + 9 ground motions, up to 2.3 times the design level. This is attributed to a stable and controlled friction force to dissipate seismic energy. The development of kinetic energy within the rack was effectively controlled, thereby limiting the speed of rack movement which led to a smaller stomping force when a upright impacts the floor.

5 CONCLUSION

A series of shaking table test were conducted to compare the seismic response of rack frames with 4 types of base connection. The friction slipper baseplate exhibits the best seismic performance while other base plates show different level of failure or damage. The business of the warehouse owner using friction slipper baseplate would not be interrupted after a severe earthquake event, and no replacement is required.

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