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Equivalent static force method for selective storage racks with uplifting baseplates

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Abstract

This paper is concerned with the equivalent static force method for the seismic design of selective storage racks with uplifting baseplates. It proposes a new 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 proposed 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.

Keywords: cold-formed steel storage racks; equivalent static force method; natural period; seismic design; rocking

1 Introduction

A cold-formed steel selective storage rack consists of a row of upright frames, linked together in the down-aisle direction with clip-in beams at each storage level, as shown in Figure 1. The upright frames are typically tall and narrow in the cross-aisle direction. Cross-aisle lateral loads can cause significant overturning moments, which are resisted by the pallet weight and the

upright baseplates. The baseplates connect the base of each upright to the concrete slab by anchor bolts.

Figure 1: A selective storage rack

The design of selective storage racks in moderate to strong seismic regions presents some challenges to the structural engineer, especially one familiar with typical building seismic design. Standards for determining the seismic design loads are suited to buildings, but do not necessarily account for the unique characteristics of cold-formed steel storage racks. Design guidelines such as the BRANZ design guide (Beattie & Deam 2006) suggest modifications to adapt the standards for use with storage racking. The BRANZ design guide, however, does not address some fundamental differences in structural behaviour between multi-storey buildings and cold-formed steel storage racks.

Racks are often anchored to the concrete slab using ductile baseplates or other mechanisms that allow uplift of the uprights under cross-aisle seismic loading. This uplift leads to the rack to rock in the cross-aisle direction, thus isolating the superstructure from seismic base shear. Although rocking behaviour and its effect can be modelled by the nonlinear time history analysis method, it requires significant investment of time, computing facilities and expertise. In practice, rack designers therefore use the equivalent static force method mandated by seismic design standards such as NZS 1170.5 (SNZ 2004).

The use of rocking structural systems for bridge piers, shear walls and steel frames has become more 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). More recently, rocking steel frame designs have been proposed that use post-tensioned cables to limit maximum displacements (Steele & 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 a similar buckling-restrained braced frame building (Pollino 2015).

Some limited research has been done on the application of rocking design to cold-formed steel storage racking. Quasi-static cyclic testing of a storage rack upright frame in the cross-aisle direction was used to demonstrate the energy dissipation capacity of ductile baseplates (Petrone et al. 2016). While rocking can amplify the upright compression force (Azuhata, et al. 2007; Priestley, et al. 1978), cold-formed steel uprights are able to remain undamaged after experiencing stomping forces 15 % greater than their static compression capacities (Maguire et al. 2019).

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 only accounts for rocking of the pallet loads but not of the rack structure. NZ 1170.5:2004 requires a special study if rocking is to be used as an energy dissipation mechanism but gives no details of the procedure. Other seismic analysis methods of storage racks have been proposed including rigid-plastic analyses (Montuori et al. 2019).

In the equivalent static force 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, Bernuzzi et al. 2015). However, for a rocking rack, there are two fundamental problems in the application of the equivalent static force 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 a 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. 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.

2 Rack Configurations and Modelling

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 2 for a three level rack. The upright frame model consists of elastic beam-column elements as upright members, and truss elements as bracing members.

Figure 2: Finite element model of a three-level upright frame.

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/m³. 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 2) of the upright and bracing sections are shown in Table 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 in this paper is 9.825 m.

Table 1: Section properties of upright and bracing elements

Element	Rack levels	Area ($\times 10^{-6} \text{m}^2$)	I_z ($\times 10^{-9} \text{m}^4$)	Self mass (kg/m)
Upright	3	474	152	3.72
	5	611	337	4.80
	7	1028	1237	8.07
Bracing	3	13.8	-	1.19
	5	23.0	-	1.98
	7	32.2	-	2.77

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 3. The LS02, LS04 and LS40 baseplates allow uplift in tension at a linear stiffness of 2, 4 and 40 kN/mm, respectively. The stiffness of the LS02 and 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 (Maguire, et al. 2018).

Figure 3: Standard ductile (left) and heavy duty (right) baseplates

In tension, the SD baseplate had an initial stiffness of 1.75 kN/mm 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. The SD baseplate can undergo large displacements, allowing a storage rack to rock.

Figure 4: Ductile behaviour in tension of SD baseplate during cyclic axial testing.

The HD baseplate is significantly stiffer than the SD baseplate, with an elastic stiffness of 12.9 kN/mm. The HD baseplate was connected to the upright web with four bolts as shown in Figure 3, resulting in a shorter bolt slip of 2.5 mm at an axial tension of 1.5 kN.

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 5. All baseplates have a stiffness in compression of 40 kN/mm, which approximates the stiffness of the concrete slab as found from the experimental tests.

Figure 5: Baseplate response models

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 6 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 7. In Figure 6, the zero-length elements are shown to have a finite length only to illustrate the existence of the floor and upright base nodes.

- Upright node
- Floor node
- Zero-length element

Figure 6: Baseplate model constraints

- Node
- Rocking frame
- Baseplate element

Figure 7: Baseplate model uplift behaviour

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 8, 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.

Figure 8: Shaking table test setup

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-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 9. The finite element results match the experimental results reasonably well.

Figure 9: Validation of finite element model

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 force 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 $\mu = 1.0$ was used as nonlinear material properties were accounted for in the baseplate models.

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 R 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 & Uma 2012):

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

where R_i , W_i and h_i are the Rayleigh lateral force, gravity load and height at level i respectively and 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

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

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

Using the period T_1 , the design base shear V_{ESM} (SNZ 2004) was determined using:

$$V_{ESM} = C(T_1) \frac{S_p}{k_\mu} \sum_{i=1}^n W_i \quad (3)$$

where $C(T_1)$ is the elastic site hazard spectrum, shown in Figure 10, 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 $\mu = 1$, $S_p = 1$ and $k_\mu = 1$ according to Clauses 4.4.2 and 5.2.1.

Figure 10: Elastic site hazard spectrum $C(T)$ for site subsoil class C (shallow soil)

Nonlinear time history analyses were carried out using a suite of 44 ground motion records with the period T_1 computed using Eqn (2). The ground motions were selected from the ATC-63 Far-Field Record suite developed for FEMA P695 (Applied Technology Council 2009). Each ground motion record was scaled to the target spectrum:

$$SA_{target} = \frac{1 + S_p}{2} C(T_1) \quad (4)$$

The scaling factor k_1 for each motion was determined by minimising the least square sum of $\log(k_1 SA / SA_{target})$ where SA 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 (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.

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 & Uma 2012):

$$F_i = V_{ESM} \frac{W_i h_i}{\sum_{i=1}^n (W_i h_i)} \quad (5)$$

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

4 Determination of the effective natural period

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 (2), were found to be significantly affected by the magnitude of the Rayleigh lateral loads applied, as shown in Figure 11 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.

Figure 11: Natural periods of three-level upright frames as determined by the Rayleigh method.

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

249 equating the overturning moment caused by the lateral load with the restoring moment of the
 250 pallet weights:

$$\frac{1}{2}D \sum_{i=1}^n (W_i) = \sum_{i=1}^n (R_i h_i) \quad (6)$$

251 where D is the frame depth as defined in Figure 2. For the load distribution described by Eqn
 252 (1), the uplift threshold can be determined by:

$$R_{uplift} = \frac{1}{2}D \sum_{i=1}^n (W_i) \frac{\sum_{i=1}^n (W_i h_i)}{\sum_{i=1}^n (W_i h_i^2)} \quad (7)$$

253 If the pallet weight at each level is constant:

$$R_{uplift} = \frac{1}{2}D W_t \frac{\sum_{i=1}^n (h_i)}{\sum_{i=1}^n (h_i^2)} \quad (8)$$

254 For the three rack heights considered in this study, the threshold Rayleigh lateral load is given
 255 in Table 2.

256 Table 2: Uplift thresholds

Number of levels	Level heights, h_i (m)	Uplift threshold, R_{uplift} (kN)
3	1.425, 2.825, 4.225	3.03
5	..., 5.625, 7.025	3.30
7	..., 8.425, 9.825	3.43

257

258 At Rayleigh lateral loads greater than the uplift threshold, the computed natural periods of the
 259 LS02 and LS04 baseplates lengthen at a high rate initially, but asymptote towards 0.92 s and
 260 0.68 s, respectively. The SD and HD baseplates have higher rates of lengthening of the natural
 261 period at the uplift threshold due to initial bolt slip. At higher Rayleigh lateral loads, the HD
 262 baseplate's computed natural period shortens due to its high stiffness, while the SD baseplate's
 263 period continues to lengthen due to its softening response as exhibited in Figure 6.

264 It is evident that the natural period of a frame with an uplifting baseplate, computed using the
 265 Rayleigh method, is not unique but is a function of the magnitude of the Rayleigh lateral load
 266 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.

4.2 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 12. 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.

Figure 12: Design base shears for period values determined using increasing Rayleigh lateral loads for a three-level rack with SD baseplates.

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 12 for the periods of 1.42 s, 1.61 s and 1.74 s came from the ATC63 121021 ground motion, shown in Figure 13. 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.

Figure 13: Spectra for the ATC63 121021 ground motion (Loma Prieta, 1989), with the scaling range of $0.4T_1$ to $1.3T_1$ highlighted.

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 14 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.

Figure 14: Design base shears for period values determined using increasing Rayleigh lateral loads for a three-level rack with LS02 baseplates.

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 15 for a three-level rack.

Figure 15: Design base shears for period values determined using increasing Rayleigh lateral loads for a three-level rack with HD baseplates.

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 Figure 16. 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.

Figure 16: Accuracy of the assumed natural periods based on the Rayleigh method

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 paper. Figure 16 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 same of the upright compression forces, as shown in the next section.

5 Implications of the proposed procedure

This section investigates the implications of the procedure proposed 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 force 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 17.

Figure 17: Ratio of ESM's design upright axial load to THA's median peak upright axial load.

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.

6 Additional verifications

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 18, the base shear obtained using the Rayleigh lateral loads at the uplift threshold is reasonable for the SD, LS02 and LS04 baseplates. For the racks with HD or LS40 baseplates, the equivalent static method's base shears V_{ESM} are unconservative regardless of the Rayleigh lateral loads. In general, the proposed procedure is more accurate than the conventional Rayleigh method.

Figure 18: Ratio of ESM's base shear to THA's median peak base shear with 1600 kg pallet loads.

6.2 Baseplate compression stiffness

As mentioned in Section 2.3, the baseplate stiffness in compression used for the rack models in Section 4 was 40 kN/mm, based on the experimental test results. In order 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 a stiffness of 400 kN/mm. The results

plotted in Figure 19 show that using the Rayleigh lateral load corresponding to the uplift threshold is still reasonable.

Figure 19: Ratio of ESM's base shear to THA's median peak base shear with baseplate compressive stiffness of 400 kN/mm.

7 Conclusions

This paper 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 actually 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 proposed 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.

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