International Journal of Steel Structures
June 2013, Vol 13, No 2, 253-263
DOI 10.1007/s13296-013-2005-6

International Journal of STEEL STRUCTURES
www.springer.com/journal/13296

Cost Viability of a Base Isolation System for the Seismic Protection of a Steel High-Rack Structure

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Abstract

In this study the effects and costs of implementing a base isolation system for the mitigation of the seismic risk of an existing externally-braced steel frame rack structure are analysed by means of nonlinear static (pushover) analysis. Various plan asymmetric variants, with different realistic distributions of the payload mass and occupancy levels, have been investigated under two seismic intensities. The results obtained are presented as floor plan projection envelopes of the top displacements and as plastic hinge damage patterns of the superstructure. In the presented cost evaluation, the cost of the implementation of the proposed base isolation system is compared with the estimated costs of structural repairs to the damaged structural members of the superstructure, as well as with estimated expenses of the downtime period and content damage. The results have shown that base isolation is, in general, not economically feasible for lower ground motion intensities, whereas it could be of great benefit in the case of moderate and high intensities. A simple rough cost estimation study, based on the obtained plastic hinge patterns, showed that the inclusion of the downtime period costs and content damage costs might be important parameters, which - if taken into account - could make such an isolation system viable also for lower ground motion intensities. The other benefits brought by seismic isolation, such as savings on the building design costs, reductions in the threat to employees' lives, and others, were, however, not included in the presented study. The comparison is done only for two deterministic scenarios of seismic attack, e.g. for design ground motion intensity (a_g =0.175 g) and for increased intensity with a_g =0.25 g indicating the Maximum Considered Earthquake level.

Keywords: rack structures, base isolation, cost efficiency, mass eccentricity, repair costs, downtime costs, content damage costs

1. Introduction

Steel frame storage rack structures present special structures which can carry much larger live loads than its own self-weight, and can also be built to considerable heights, rising well in excess of 20 m. They are used in industry for storing various kinds of goods, which are usually stored on pallets that are inserted in the rack structure by means of a forklift or special automated elevators. Rack structures are very similar to the framed steelworks that are traditionally used for civil and commercial buildings, but there are large differences in the geometry of the members and in the connection systems. Such structures are usually made of thin-walled cold-formed steel sections, where the columns (uprights) are generally manufactured as open mono-symmetric (in some cases perforated) sections and the beams (stringers) are usually manufactured as closed, boxed cross-sections. The structural behaviour of such structures under seismic loading depends to a considerable extent on how the individual components such as the beam-to-column connections, the column bases and the structural members, interact with one another (Aguirre, 2005; Freitas *et al.*, 2010; Filiatrault *et al.*, 2008). One of the shortcomings of such structures is that the bracings can only be used to prevent longitudinal sway in the cross-aisle direction, whereas in the down-aisle direction bracing cannot be used since this would hinder access to the pallets containing the stored merchandise. An additional risk in seismic zones involves the so-called »contents spillage«, i.e. the possibility that stored merchandise may fall off the pallets, which could lead to financial loss as well as potentially the loss of life (Affolter *et al.*, 2009; Sideris *et al.*, 2010). Furthermore, due to the fact that the loads produced by the stored merchandise are usually substantially higher than the self-weight of the rack

Note.-Discussion open until November 1, 2013. This manuscript for this paper was submitted for review and possible publication on November 14, 2011; approved on March 7, 2013.

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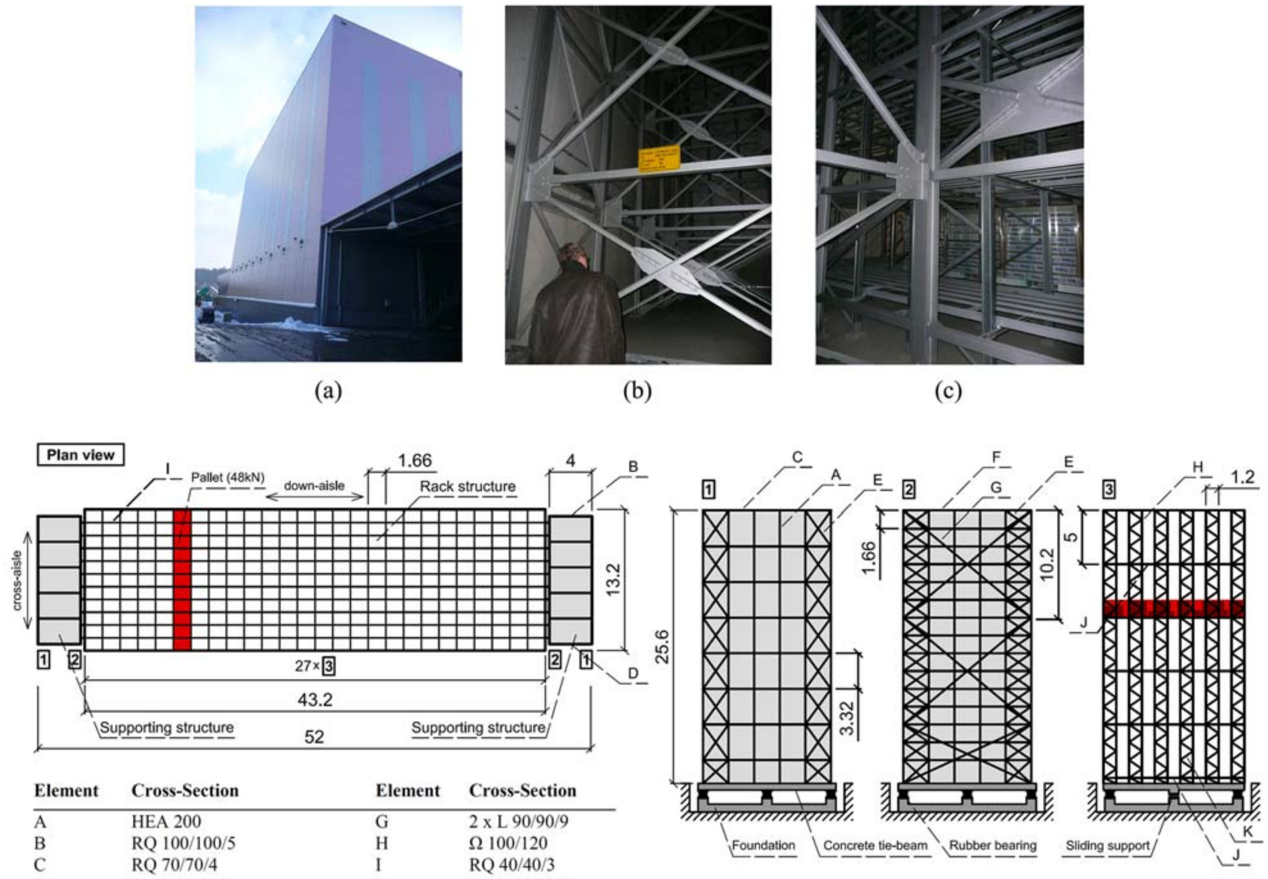

Element	Cross-Section	Element	Cross-Section
A	HEA 200	G	2 x L 90/90/9
B	RQ 100/100/5	H	Ω 100/120
C	RQ 70/70/4	I	RQ 40/40/3
D	2 x L 65/65/6	J	2 x L 50/50/8
E	2 x L 50/50/4	K	C 50/30/3
F	HEA 100	L	UPN 120/55

(d)

Figure 1. Analysed steel frame rack structure: (a) Outer view, (b) External supporting structure on the left side of the building, (c) Central rack structure with pallet guiders, stringers, uprights and K bracings, and (d) Plan view and cross sections with indicated pallet live loading (dimensions are given in metres). Proposed base isolation system is schematically presented.

structure, random rack loading patterns can lead to mass eccentricities greater than the 5% accidental design mass eccentricity which is incorporated in some building codes, e.g. Eurocode 8 (CEN, 2005b). It has been shown through our research that mass eccentricities which are higher than the maximum expected accidental eccentricity can lead to local instabilities, and pose an additional seismic risk for some essential parts of the structure.

The purpose of this article is to analyse the effect of implementing a base isolation system for mitigating seismic risk in the case of different occupancy levels and mass eccentricities. A cost study was performed, comparing the costs of base isolation with the estimated repair costs of the damaged structural members and with the estimated downtime costs and costs of the damaged stored merchandise. Other positive effects of base isolation, such as savings on building design costs, possible reductions in the threat to employees' lives, and others, were, however, not considered.

2. The Investigated Steel Frame Rack Structure

2.1. Description of the analysed structure

The existing storehouse (Fig. 1a) is owned by the building construction joint-stock company Trimo and it is located in Trebnje in Slovenia. The storehouse construction system is a space frame consisted of uprights, stringers and K-bracings (in cross aisle direction only), which is externally braced by 3D moment-resisting supporting structures positioned on both of its outer sides. The two external supporting structures (Fig. 1b) consist of concentric diagonal bracings made of double L sections and columns (HEA sections) and beams (welded hollow square sections). The uprights of the central rack structure are made of specially designed »omega« cold-formed sections braced with K-bracings (Fig. 1c). The plan dimensions of the rack structure are 43.2 m×13.2 m, whereas it has a height of 25.6 m (Fig. 1d). Pallets containing stored goods with a maximum load bearing capacity of 6 tons (including pallet self-weight) can be inserted into the racks by means of a large automated elevator. The storey masses of the fully loaded rack structure amount to: 353 tons at the bottom storey, 530 tons at the intermediate storeys and 209 tons at the top storey. The fundamental periods of vibration of the fully loaded fixed-base rack structure amount to $T_x = 1.35$ s, $T_y = 1.25$ s and $T_z = 0.95$ s. More data about the investigated structure can be found in (Kilar *et al*., 2011), as well as in (Petrovčič and Kilar, 2012).

2.2. Modelling of the structure, mass eccentricities and seismic load

The existing fixed-base structure was originally designed taking into account a 70% occupancy level, assuming symmetric distribution of the merchandise. The rack structural part was initially designed taking into account the proposal Pr FEM 10.2.08 of the European Racking Federation (FEM, 2005), which does not enforce the usage of capacity design rules. Both of the supporting structures were designed in accordance with European building codes Eurocode 3 and 8 (CEN, 2005a; 2005b), applying a behaviour reduction factor of $q = 4$ and the design spectrum for soil class C scaled to the peak design ground acceleration of $a_g = 0.175$ g. For our parametric study the entire structure was taken as it was originally designed and modelled by means of the computer program SAP2000 (CSI, 2008). The response was observed in the cross-aisle direction only, in which it is possible to account for the effects of torsional twist due to the introduced asymmetries in the down-aisle direction. For the loading in the down-aisle direction the structural response is symmetric and therefore not interesting for the study of the effects of asymmetry considered in the paper. As it was shown in (Kilar *et al*., 2011), the mass eccentricity e_{max} due to the distribution of the stored merchandise (i.e. the payload mass) can be mathematically related with the occupancy level of the structure (ψ). We have assumed that each inserted pallet is fully loaded, and that a row of racks is considered to be »occupied« when it is filled with pallets throughout the whole height of the structure. In this case the maximum eccentricity of the rack structure (e_{max}) is achieved when the payload mass at a given occupancy level is distributed as far as possible from the gravity centre of the structure. By moving the payload mass toward the right hand side of the building plan we have gained additional torsional effects which increase displacements on the right hand building side (e.g. flexible side) and reduce displacements on the other side (e.g. stiff side). The derivation of the equation for the eccentricity e_{max} can be found in (Kilar *et al*., 2011). It was shown that larger eccentricities can be expected in the case of lower occupancy levels, except for occupancy level ratios of less than about 25%. Very low occupancies involve smaller payload masses, so that these cases are not of critical concern. Five different models were selected for the analyses, with e_{max} set equal to 0% (ψ=100%), 5% (ψ=85%), 10% (ψ=70%), 15% (ψ=55%), and 19.3% (ψ=27%) of B = 52.0 m. The eccentricity 19.3% represents the maximum possible mass eccentricity of the structure which can be obtained in the case of a rack occupancy level of 27%.

The joints between members of the rack structure were assumed to act as hinged connections, whereas the joints in the supporting structure were modelled as rigid joints. Stiff horizontal overall diaphragm at the top of the structure was used to model the horizontal cross bracing on the roof. Additionally, rigid diaphragms were also considered at intermediate rack storeys which also exhibit a high in-plane stiffness due to double »L« horizontal bracings (element J in Fig. 1c-d) and at the base level due to the rigid concrete tie-beams. A bilinear elastic-perfectly plastic model was adopted for the structural steel, with the yield stress of the material set to 235 MPa and the steel elastic modulus (E) assumed as 210 GPa. The effects of material non-linearity were considered by conducting elasto-plastic analyses with plastic hinges. The influence of second order effects was verified by preliminary nonlinear static analysis of the fully loaded rack structure. According to Kim and Lee (2010) or Kang and Choi (2011) the P-D effect is highly affected by the axial load (which is relatively low in our case) and the stiffness of the first floor (which is relatively high due to the side supporting bracing). For these reasons the obtained second order effects were small and neglected in further analyses. Nonlinear static (pushover) analysis was used to assess the effect of mass eccentricity for the fixed-base (FB) and the base-isolated (BI) models of the investigated structure. In the seismic analyses of the investigated models two deterministic scenarios of seismic attack was taken into account. An increased intensity with $a_g = 0.25$ g (indicating the Maximum Considered Earthquake level (ICBO, 1997) was considered, as well as the design ground motion intensity ($a_g = 0.175$ g). A complete cost comparison should derive from a probabilistic evaluation accounting for the probability of different earthquake scenarios and of the consequent losses. A probabilistic approach in the seismic analysis of the building structures was used for example in (Erberik, 2008; Fajfar and Dolšek, 2012; Rajeev and Tesfamariam, 2012). In the present paper the N2 method was used, which was developed for fixed-base symmetric structures (Fajfar, 2000), but also extended to asymmetric structures (Fajfar *et al*., 2005). The target displacement is determined as the intersection between the idealized capacity curve of the structure and the inelastic demand spectrum curve. In the extended N2 method the results of the pushover analysis are further multiplied by correction factors, which can be obtained by means of elastic modal analysis for a given distance from the CM. Recently the N2 method has been applied, though with some modifications, to base-isolated

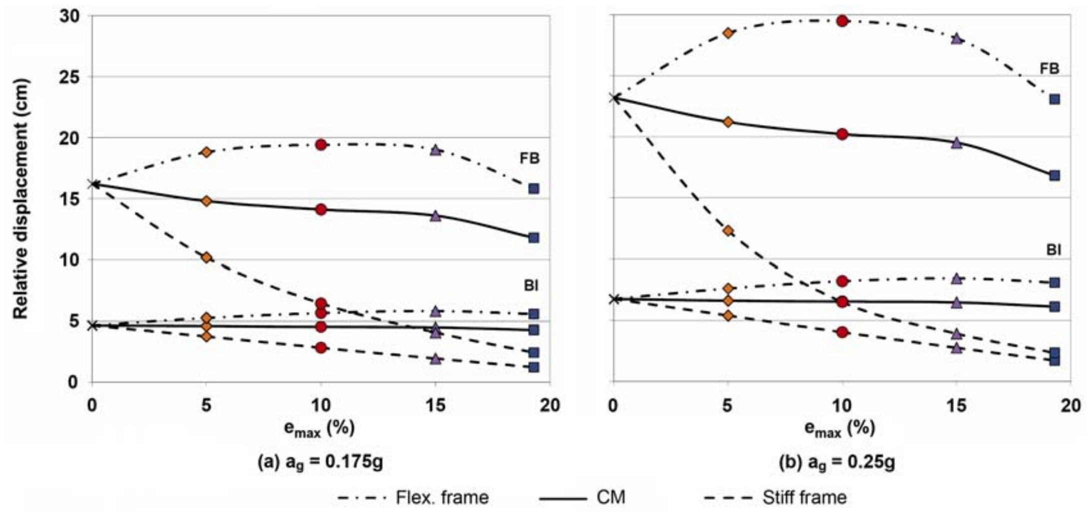

Figure 2. Relative displacements of the analysed models for different mass eccentricities.

symmetric (Kilar and Koren, 2010) and asymmetric structures (Koren and Kilar, 2011). In our pushover analyses the lateral loads were always applied in the vertical plane through the centre of mass (CM) of the superstructure. In the case of FB variants a load pattern corresponding to a mass-proportional inverted triangular displacement distribution was considered, in which the normalized displacements of the stories have a linear distribution throughout the height of the structure and are multiplied by the mass of each storey (Kilar *et al*., 2011). In the case of BI structures an additional force (F_b), acting at the base level and proportional to the ratio between base mass and the mass of the superstructure, was considered (SEAONC, 1986). In all cases the target displacement needed for the N2 method was considered as the top displacement at the formation of a plastic mechanism on the flexible side of the structure (plastification of all columns at their bases). Such damage could lead to local instability, and should be considered as one of the limit states when designing high rack structures (Filiatrault *et al*., 2008; Affolter *et al*., 2009; Rodgers and Mahin, 2011). Comparisons with nonlinear dynamic (time-history) analyses are not given in this paper, but they can be found in (Kilar *et al*., 2011).

2.3. Implementation of the base isolation system

The implemented base isolation system was designed based on the condition that the fully (100%) occupied rack structure does not suffer any damage under the design seismic loading. The selected stiffness of the bearings thus brings the fully loaded symmetric superstructure exactly to the limit of its elastic range, and keeps the maximum ductility factor for the design load at a value smaller than or equal to 1.0. In practice, the designer would probably select a base isolation system that is a bit more flexible, in order to keep the design on the safe side. Rubber bearings with a diameter of 45 cm and a total height of 24 cm (including outer steel plates) were selected (FIP Industriale, 2012). They are made of soft rubber and have a horizontal stiffness of 620 kN/m, with damping equal to $\xi=10\%$ of critical damping. Their maximum allowed horizontal displacement is equal to 20 cm, which is about 200% of the height of the rubber. The base isolation system consists of 20 rubber bearings, which are distributed around the circumference of the structure's layout. The middle points of the layout are vertically supported by the use of sliding supports. To ensure a uniform distribution of stresses onto the base isolation system a RC slab with a thickness of 30 cm and a series of concrete tie-beams (b/h =40/60 cm), forming a 6 m×6 m grid, was added beneath the superstructure. This stiff diaphragm resulted in 633 tons of additional mass in the base storey. The centre of stiffness of the isolation system (CI) corresponds to the centre of stiffness of the superstructure (CS) (Kilar and Koren, 2009), as well as to the geometrical centre of the floor plan. The studied asymmetry effects were produced by shifting the centre of mass (CM) towards the right hand side of the building. The fundamental periods of vibration of the base-isolated symmetric structure amount to T_x =3.47 s, T_y =3.42 s and T_z =2.59 s.

3. The Effect of Mass Eccentricity on the Seismic Response of the FB and the BI Models

The relative displacements of the FB and the BI models for eccentricities ranging from zero to 20% are presented in Fig. 2. In the case of the BI structures the relative displacement was defined as the difference between the observed top (roof) displacement and the corresponding base displacement (measured at the isolation level). The displacements are presented for the outermost stiff and flexible frames of the asymmetric models, as well as for the CM. It can be seen that the displacements at the CM and on the stiff side decrease with increasing eccentricity. In the case of the FB structural models the displacements on the flexible side reach their maximum values in the case of eccentricities of around 10%, which corresponds to 70% storage occupancy. In the case of eccentricities greater than approximately 15%, the maximum obtained relative displacements are smaller due to the very low occupancy level, which in these cases drops below 50%. The same tendency, although less distinct, can be obtained in the case of the BI structural models. It can be seen that base isolation reduces the relative displacements by approximately 3 times for all the considered eccentricities and intensities, and that the effect of torsion is in general smaller for the BI structure than for the FB structure.

4. Cost Analysis

Although seismic isolation has been shown to be very effective in improving the dynamic characteristics and behaviour of structures under seismic loads, its economic viability still remains questionable. In other words, the question always rises as to whether the costs of seismic isolation do, or do not, exceed the costs of the seismic damage (and the post-earthquake repair) of a fixed-base structure. Recent lifecycle cost analyses of buildings in seismic areas (Taflanidis and Beck, 2009; Goda *et al*., 2010) have shown that the use of an appropriate seismic isolation can reduce the expected lifecycle costs by about up to 20%, in comparison with a fixed-base structure, depending on the design level of the superstructure. This seismic risk reduction cost should be more than sufficient to compensate for the required design/construction/installation costs of base isolators. In such a case, seismic isolation technology is cost-effective and should be adopted. The essence of base isolation is not a saving on building design costs, but rather the reduction in the number of possible deaths, downtime, repair costs, and potential costs of damaged content, after an event has occurred. The designer's targets of interest should be therefore those related to damage costs (structural repair costs and costs of the stored merchandise suffering damage) and loss of function (downtime costs). According to (Medina and Krawinkler, 2005; Krawinkler, 2011) the performance measures used in the preliminary design and seismic performance assessment could be expressed by the three D's: "Dollars" (direct economic loss), "Downtime" (loss of operation/occupancy) and business interruptions, and "Death" (injuries, fatalities, collapse). Given that the preservation of the life of the occupants represents an intangible value, the quantity assessments in this paper were performed only for the structural repair, downtime and eventual damaged content costs. In order to obtain a general insight to the problem, a simple analysis and comparison of the expected costs was performed, which were based on the available price information obtained from warehouse management and designers/specialists for steel structures. The additional assumptions and simplifications which were needed in order to prepare a reasonable case study will be explained below.

In general terms, seismic isolation systems enable structures to remain elastic during the design earthquake, so that no permanent (inelastic) damage is caused to their structural elements. Fixed-base structures which are designed according to modern seismic design codes are, on the other hand, generally expected to undergo some permanent damage during the design earthquake. Although the stiffness and strength of such a structure is preserved to some degree (so that the structure does not collapse), the behaviour and safety of the structure in some future earthquake event is uncertain so that post-earthquake repairs are required.

In order to obtain a credible comparison of costs, the fixed-base structure must be considered to be fully repaired i.e. it has to be returned into its initial state. In general this can be achieved by two approaches. One is by performing some adequate local repairs of the damaged parts of the structural elements, whereas the other is by simply replacing the damaged elements by new ones. Whereas the former approach is suitable in the case of heavy cross-sections and complicated connections (e.g. for moment-resisting frames), the latter seems to be a more reasonable solution when the cross-sections are relatively small, and the connections are predominantly simple (e.g. braced frames).

The presented cost analysis was carried out by considering the following basic assumptions:

- all connections are designed as full strength joints (except pinned ones), so that all permanent damage is considered to occur to the structural elements only, whereas the joints remain undamaged;
- the configuration and nature of the joints make possible relative simple replacement of elements;
- as the cross-sections of the analysed structure are small (the largest section is HEA 200) the replacement of damaged elements is considered to be the most convenient solution.

For the subsequent calculation of the repair costs the damaged structural elements were arranged into the following groups (see also Fig. 1):

(i) Short diagonals in the supporting structures (element type E): the short diagonals of the 'X' braces are considered to be active both in tension and compression. The results of cyclic tests of braces in compression have shown that such elements suffer heavy losses in stiffness, see e.g. (Bruneau *et al*., 1998). Thus diagonals showing inelastic displacements have to be replaced. Considering the cyclic nature of the seismic load both diagonals of an 'X' brace are equally damaged and both have to be replaced.

(ii) Long diagonals in the supporting structures (element type G): due to their high slenderness ratios the large

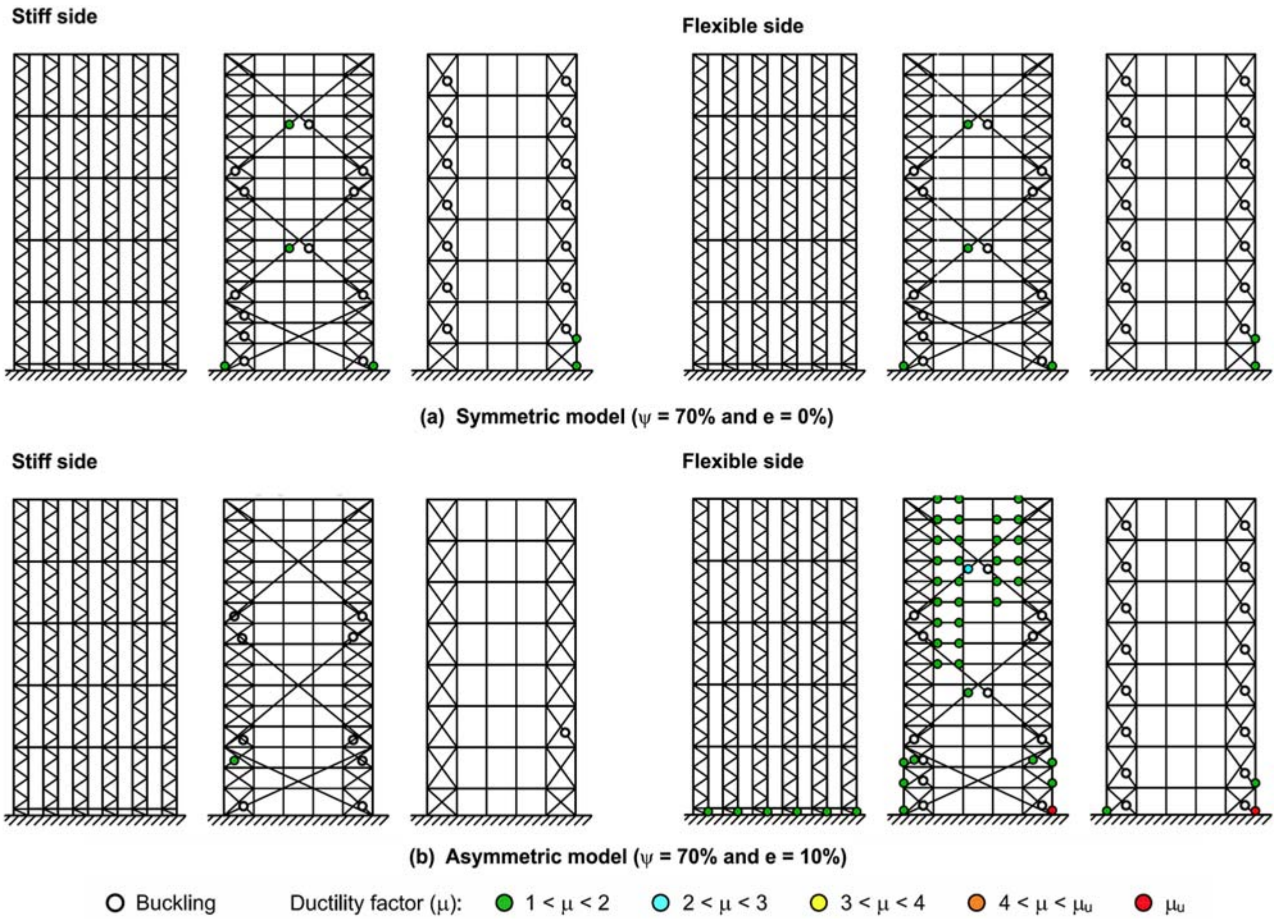

Figure 3. The formation of plastic hinges in the superstructure as obtained by the extended N2 method for the ground motion intensity a_g =0.25 g.

diagonals are considered to resist tension forces only. Tension braces are widely used as seismic energy-dissipative elements. However, when a large inelastic axial deformation occurs during the loading, then considerable sag of such element remains after the structure is unloaded and returns to its initial shape. The stiffness of long tension diagonals is thus reduced and they have to be replaced.

(iii) Beams in the supporting structures (element type F): taking into the fact that both the spans as well as the cross-sections of the beams are small, these elements have to be replaced in every case when a plastic hinge is formed either at one end or at both ends.

(iv) Columns in the supporting structures (element type A): some local damage repair can be considered in cases when only a single plastic hinge is formed at the column-base. Most of the analyses, however, show that several hinges are formed on columns from the base upwards. In such a case, the bottom part of the columns (i.e. up to the splice) has to be replaced.

(v) Columns in the racks (element type H): the inelastic deformations in the racks are the most delicate ones. Before any repairs can be performed on the rack structure the merchandise has to be unloaded. The seismic repair costs can be therefore significantly increased, so that the rack structure should not consist of seismic energy-dissipative elements (FEM, 2005). The results of the presented analyses have shown, however, that when the applied load is eccentric, particularly in combination with the stronger analysed earthquake (a_g =0.25 g), plastic hinges are formed in a large part of the racks (Fig. 3). The so damaged parts of the racks have to be replaced.

In Fig. 3 the rotational ductility factors obtained by the extended N2 method are presented for the symmetric and asymmetric (e_m =10%) fixed-base (FB) models for a_g = 0.25 g. The ductilities are presented for the stiff as well as for the flexible characteristic outer frames in the rack structure and in the supporting structure. The hinge colour indicates the value of the obtained ductility factor (μ), which is defined as the ratio between the achieved and yield plastic deformation (for braces) or rotation (for beams and columns). In case of symmetric superstructure, the damage patterns at the stiff side frames are the same as those at the flexible side. It should be noted that pushover analysis in one direction can detect only some plastic hinges, as well as the buckling of diagonals which are in compression for this direction of loading. For this reason it is necessary to apply pushover analysis in both (e.g. +Y and −Y) directions in order to obtain the actual plastic hinge pattern. It can be seen that, in the symmetric variant, no damage occurs to the rack structure. Some damage can be observed in the supporting structures, where a few diagonals buckle/yield and some plastic hinges develop at the bottom of the columns. In the corresponding asymmetric structure, however, a much greater concentration of damage occurs on the flexible side of the supporting structure. In this case the rack structure does not remain elastic, and some damage develops at the bottom of the columns on the flexible side frames of the rack structure. Such damage can lead to local structural collapse and should be avoided. Some damage was also recorded in the supporting structure on the stiff side. It should be noted that in the case of the BI structural models the behaviour of the superstructure was elastic.

Based on the damage patterns obtained in the above-described analyses, a simplified assessment of the repair costs, downtime costs and damaged content costs was performed. The repair costs (C_r) for a single damaged structural element were calculated by using the following simple expression:

$$C_r = m \cdot Cs \cdot \zeta \tag{1}$$

where m is the mass of the element (in kg), C_s is the general price (considering both material and erection costs) of structural steel (in EUR/kg), while ζ is the so-called dimensionless cost factor, by means of which the additional replacement costs with respect to the further discussed different groups of structural elements were taken into consideration. As the structural repairs give rise to certain additional costs (e.g. cutting out and removal of damaged elements, preparation of connections, etc.) the aforementioned cost factors (ζ-s) were introduced. A value of $\zeta=2$ was taken into consideration for the diagonals (types E and G) and beams. The replacement of the damaged parts of columns requires adequate temporary supporting of the structure, so a value of $\zeta=4$ was used for the columns. In the case of racks, however, the actual damage costs may become perceptibly higher and are in general difficult to define. In the present analysis a value of $\zeta=6$ was assumed for the racks. All considered values of ζ-s are the authors' assumption and should be re-evaluated for each specific case. Similarly, the used price assumptions quoted in the following text (see Eqs. (2) and (3)) and the repair time (see Tables 1 and 2) and recovery time values were obtained by consultations with warehouse managers and designers/specialists for steel structures.

The downtime costs (C_d) for a single damaged structural element were calculated by means of a simple expression, where downtime costs are defined as the rent cost of a surface equivalent to that of the damaged building:

$$C_d = (t_p + t_r) \cdot C_{rent} \tag{2}$$

where t_p is the preparation or recovery time which expresses the community resilience to an earthquake event, t_r is the repair time of the selected element (hours/pc) including fabrication and transport time of the steel profiles, and C_{rent} is rent cost of a surface equivalent to that of the building (in EUR/hour) as suggested by (Mezzi *et al*., 2011). The recovery time (t_p) is the period necessary to restore the functionality of a structure, or an infrastructure system (water supply, electric power, hospital building, etc., or a community) to a desired level that can operate or function equally well, close to, or better than the original one (Cimellaro *et al*., 2010). In general t_p depends on the available technical and human resources, on the general preparedness of society, or on public policies, and may take different forms. It is clear that preparation time depends on the actual seismic intensity (a_g). It should be noted that the base-isolated structure remains elastic after the earthquake and its operational functionality is therefore not affected. From this viewpoint, for BI structural models only the recovery time t_p (representing the time needed to perform an overall inspection of the base isolation system) has been considered. In our study the assumed value of recovery time for BI models was taken equal to t_p=10 days (the same value for both considered ground motion intensities) and resulted in costs related to the interruption of the activity. For FB models t_p was assumed to be equal to 8 days for a design ground motion intensity of a_g =0.175 g, and equal to 12 days for a_g =0.25 g.

In order to have a better estimation of the costs, an estimate of the damage to the content (stored merchandise) have been assessed and taken into consideration. The costs of the damaged content (C_c) can be expressed by the following expression:

$$C_c = n_r \cdot C_p \tag{3}$$

where n_r is the number of damaged racks with the pallets of stored merchandise and C_p is the average cost of damaged content per pallet (fully loaded). In the presented study the C_p was assumed to have been equal to 4,000 EUR/pallet. The number of damaged racks was obtained by pushover analysis where the rack was considered as damaged if any plastic hinge was formed in any element of the rack. In all analysed cases the plastic hinges, if any, were formed at the bottom of rack columns (Fig. 3). In such a case it was considered that all stored merchandise on the pallets in the whole bottom storey had been damaged.

The total costs (C) can be further expressed as the sum of the above three costs:

$$\begin{aligned} C &= C_r + C_d + C_c \\ &= \sum_{i=1}^{n} m_i \cdot C_s \cdot \zeta_i + \left(t_p \cdot C_{rent} + \sum_{i=1}^{m} t_{r,i} \cdot C_{rent} \right) + n_r \cdot C_p \end{aligned} \tag{4}$$

Table 1. The geometrical data and masses for the different types of elements

Element (type)	Section	Section area A (cm^2)	Length L (cm)	Mass m (kg)	Repair time t_r (h)
Diagonal (E-1)	2 L 50/50/4	7.78	300	18.3	6.0
Diagonal (E-2)	2 L 50/50/4	7.78	410	25.0	6.0
Diagonal (E-3)	2 L 50/50/4	7.78	270	16.5	6.0
Diagonal (G)	2 L 90/90/9	31.00	1,550	377.2	15.0
Beam (F)	HEA 100	21.20	220	36.6	18.0
Column (A)	HEA 200	53.80	500	211.2	20.0

Table 2. The mass of the replaced part of a single rack (unified element type R)

Element (type)	Section	Section area A (cm^2)	Length L (cm)	No. of pcs.	Mass m (kg)	Repair time t_r (h)
Column (H)	Omega 100/120	12.06	500	12	568.0	40.0
Horizontal (J)	2 L 50/50/5.5	10.40	1,200	1	98.7	25.0
Diagonal (K)	C 50/30/3	3.89	125	42	160.3	15.0
Rack (R)					**Σ=827.0**	Σ=80.0

where n is the number of damaged elements in the whole structure.

In order to obtain an adequate cost analysis and comparison, the expected structural repair costs for each individual analysis case were expressed as a percentage of the costs of the newly erected initial structure. The price of the initial structure was estimated to have been equal to 850,000 EUR. The general steel price (C_s =1.70 EUR/kg), the cost of non-structural members, façade elements, as well as the cost of the RC foundation slab, were considered in the price estimation.

The geometrical data for the calculation of the masses for the different types of elements are listed in Table 1. It should be noted that the short diagonals of the rack structure (element type E) were, due to different lengths, further divided into three subgroups: E-1 (bottom storey of both the inner and the outer side), E-2 (the remaining storeys of the outer side) and E-3 (the remaining storeys of the inner side). The estimated repair time for each element is also given in Table 1.

The racks were treated somewhat differently. The results (Fig. 3) showed that when a single rack frame undergoes inelastic deformations the damage is spread over the entire bottom part (i.e. plastic hinges are formed in all the columns of the rack). Consequently it was considered that the whole bottom panel (up to a height of 5 m) of a damaged rack had to be replaced. The repair costs of the racks are therefore not calculated based on the individual structural elements, but each rack is considered as one unified structural element (in the further text denoted as type R). Definitions of the mass of the replaced parts of a single rack are presented, together with the estimated repair times, in Table 2.

Based on the defined input data the repair costs for the previously discussed analysis cases were calculated. The particular example of the analysed asymmetric model, with 10% eccentricity and a 70% occupancy level, for the case of a_g =0.25 g, is shown in Table 3. The calculated costs amount to 144,489 EUR, which is approximately 17% of the initial structural costs.

Table 3. Repair costs for the analysed case with e_{max} = 10% (a_g =0.25 g)

Element (type)	Mass/pc. (kg)	No. of damaged pcs.	z	C_r (EUR)
Diagonal (E-1)	18.3	12	2	2,554
Diagonal (E-2)	25.0	30	2	748
Diagonal (E-3)	16.5	34	2	1,906
Diagonal (G)	377.2	4	2	5,130
Beam (F)	36.6	15	2	1,867
Column (A)	211.2	4	4	5,744
Rack (R)	827.0	15	6	126,541
Total				Σ=144,489

In Figs. 4-6 the results for all the analysis cases are presented. Fig. 4 presents only the repair costs, Fig. 5 presents the repair costs associated with the downtime costs, whereas Fig. 6 presents the total costs including the costs of the damaged content. For comparison, the costs of seismic isolation are also indicated. Considering the required number of isolators and supports and the dimensions of the RC base grid, the total costs of seismic isolation amount to 56,400 EUR (approximately 6.6% of the initial structural cost). In case of analysing the total costs considering also the recovery time t_p (assumed equal to 10 days for BI structure) the total costs of seismic isolation amount to 80,400 EUR (approximately 9.5% of the initial structural costs).

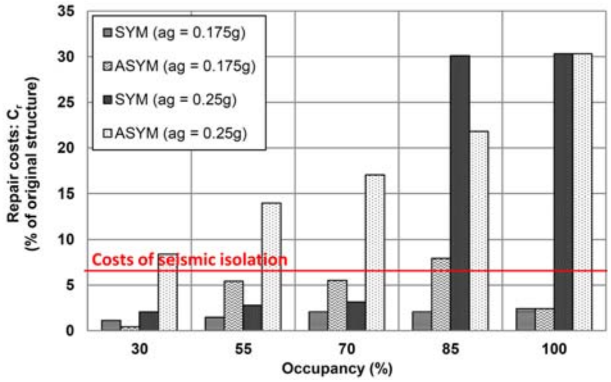

Figure 4. Seismic isolation costs versus structural repair costs for different occupancy levels and ground motion intensities.

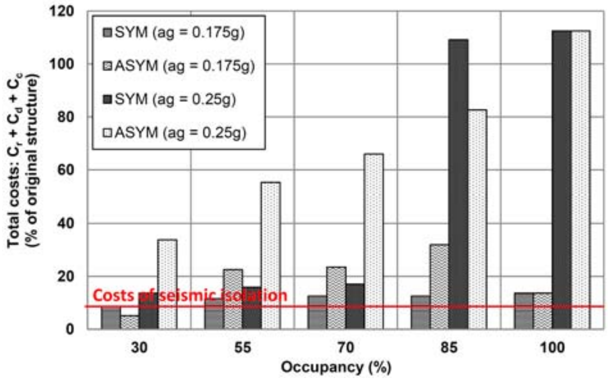

Figure 6. Seismic isolation costs versus total costs (structural repair, downtime and damaged content) for different occupancy levels and ground motion intensities.

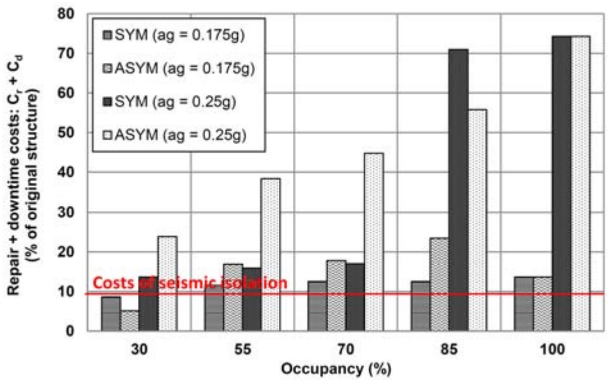

Figure 5. Seismic isolation costs versus structural repair costs and downtime costs for different occupancy levels and ground motion intensities.

When only the repair costs (Fig. 4) are taken into consideration it can be seen that the use of a base isolation system is not viable in all cases where the ground acceleration amounts to 0.175 g. The exceptions are some asymmetric cases with higher occupancy levels, where an unfavourable combination of eccentricity and occupancy might justify the use of base isolation in these particular cases. More meaningful is the use of base isolation in the case of the stronger ground motion intensity (a_g =0.25 g). In this case the isolation system for the symmetric structure is viable as soon as the occupancy level exceeds approx. 70%, when a large number of plastic hinges occur practically at the same time. For occupancy levels higher than approx. 85%, the repair costs rise up to 30% of the cost of the initial structure, which is already 5 times the cost of the isolation system.

Fig. 5 presents a somewhat more real case, since it takes into consideration also the downtime costs with assumed values of C_{rent} =100 EUR/hour (2,400 EUR/day) and t_p equal to 8 and 12 days for the design ground motion intensities a_g =0.175 g and a_g =0.25 g, respectively. The cost of recovery time t_p was added also to that of base isolation, since all recovery costs cannot be avoided by means of a base isolation system. In cases where the repair time (t_r) of the element is proportional to its mass (m), only a parallel shift of the curves shown in Fig. 4 is needed. As can be seen from Tables 1 and 2, in the present study this relationship was not assumed to be proportional. In comparison with the previously presented case (with consideration of the structural repair costs only) the use of a base isolation system is much more viable. From the results shown in Fig. 5 it can be seen that the isolation system is economically viable for normal occupancy levels (e.g. those greater than approx. 50%) for all symmetric as well asymmetric structural variants, and for both analysed seismic intensities. The only exceptions are a few (30%) occupied structures subjected to design ground motions (a_g =0.175 g). For greater analysed ground motion intensities the isolation system seems to be the only reasonable solution from the economic point of view. In cases of high occupancy levels and stronger ground motion intensities, the total costs significantly increase and can in extreme cases exceed the cost of original structure.

Figure 6 presents the most realistic case, since it takes into consideration also the damaged content costs with assumed pallet value of C_p =4,000 EUR/pallet. It can be seen that in this case the seismic isolation proves to be viable practically for all ground motion intensities and occupancies. In the cases of higher seismicity and fuller occupancy the total induced costs could exceed the costs of original structure, whereas the cost of base isolation remains at around 10% of initial building costs. It should be however pointed out, that the change of assumptions related to initial costs or durations of different recovery stages in risk assessment analysis might significantly influence the presented results, which should be taken into consideration only as a case study that should be in practice carefully adopted for each particular local situation and seismic event.

5. Conclusions

The paper makes an attempt to analyse the structural repair, downtime and content damage costs of several variants of symmetrically and asymmetrically occupied warehouse steel buildings. It was considered that in the cases when the repair costs, costs related to loss of function (i.e. downtime) and costs of the damaged content are smaller than the cost of base isolation, implementation of the latter is not economically viable. The repair costs were calculated based on the actual damage of the structural elements which was obtained using the N2 method. The downtime costs were calculated from loss of operation time, which was divided on the recovery part (expressing community resilience to an earthquake event) and the repair time of the selected element (expressing just the time for repairs, including the time needed to fabricate the steel profiles and transport them to the location concerned). The costs of the damaged content were assessed based on the actual number of damaged racks and the assumed average cost of damaged content per pallet. While certain downtime cost are inevitable, the cost of damaged content were considered only in the cases where the rack columns have been damaged. In the case of the evaluated case study, some price assumptions based on information available in Slovenia had to be made; this information was obtained from warehouse managers and from designers/specialists for steel structures. The duration of different recovery stages are the authors' assumptions and should be re-evaluated for each specific case. For our particular examined case the cost of seismic isolation system was estimated to only around 10% of the initial costs. It was further shown that a base isolation system is probably not economically feasible for smaller to moderate ground motion intensities, if only the pure repair costs, as shown for example in Fig. 4, are observed. However, if the downtime costs and damaged content costs are taken into consideration, it can be seen that the isolation system could be economically viable for both of the analysed seismic intensities (a_g =0.175 g and a_g = 0.25 g) for all normal occupancies, and for all symmetric as well as asymmetric structural variants. In the cases of high occupancies and stronger ground motion intensities the total costs increase significantly, and can in extreme cases exceed the costs of the original structure.

References

Affolter, C., Piskoty, G., Wullschleger, L., and Weisse, B. (2009). "Collapse of a high storage rack." *Engineering Failure Analysis*, 16(6), pp. 1846-1855.

Aguirre, C. (2005). "Seismic behavior of rack structures." *Journal of Constructional Steel Research*, 61(5), pp. 607-624.

Bruneau, M., Uang, C. M., and Whittaker, A. (1998). *Ductile design of steel structures.* McGraw-Hill, Boston.

CEN (2005a). *Eurocode 3: Design of steel structures - Part 1-1: General structural rules, EN 1993-1-1.* European Committee for Standardization, Brussels.

CEN (2005b). *Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, EN 1998-1.* European Committee for Standardization, Brussels.

Cimellaro, G. P., Reinhorn, A. M., and Bruneau, M. (2010). "Framework for analytical quantification of disaster resilience."*Engineering Structures*, 32(11), pp. 3639-3649.

CSI (2008). *SAP2000 (v12.0.1) - Linear and nonlinear static and dynamic analysis and design of three-dimensional structures.* Computer & Structures, Inc., Berkeley.

Erberik, M. A. (2008). "Fragility-based assessment of typical mid-rise and low-rise RC buildings in Turkey." *Engineering Structures,* 30(5), pp. 1360-1374.

Fajfar, P. (2000). "A nonlinear analysis method for performance-based seismic design." *Earthquake Spectra*, 16(3), pp. 573-592.

Fajfar, P., Marušić, D., and Peruš, I. (2005). "Torsional effects in the pushover-based seismic analysis of buildings." *Journal of Earthquake Engineering*, 9(6), pp. 831-854.

Fajfar, P. and Dolšek, M. (2012). "A practice-oriented estimation of the failure probability of building structures." *Earthquake Engineering and Structural Dynamics*, 41(3), pp. 531-547.

FEM (2005). *Recommendations for the design of static steel pallet racks under seismic conditions - prFEM 10.2.08.* European Racking Federation, Birmingham.

Filiatrault, A., Higgins, P. S., Wanitkorkul, A., Courtwright, J. A., and Michael, R. (2008). "Experimental seismic response of base isolated pallet-type steel storage racks." *Earthquake Spectra*, 24(3), pp. 617-639.

FIP Industriale (2012). *Catalogue on SI Elastomeric Isolators* (available online: http://www.fip-group.it/fip_ind_eng/prodotti.html).

Freitas, A. M. S., Souza, F. T., and Freitas, S. R. (2010). "Analysis and behavior of steel storage drive-in racks." *Thin-Walled Structures*, 48(2), pp. 110-117.

Goda, K., Lee, C. S., and Hong, H. P. (2010). "Lifecycle cost-benefit analysis of isolated buildings." *Structural Safety*, 32(1), pp. 52-63.

ICBO (1997). *Uniform Building Code.* International Conference of Building Officials, Whittier, California.

Kang, C.-K. and Choi, B.-J. (2011). "New approach to evaluate the response modification factors for steel moment resisting frames." *International Journal of Steel Structures*, 11(3), pp. 275-286.

Kilar, V. and Koren, D. (2009). "Seismic behaviour of asymmetric base isolated structures with various distributions of isolators." *Engineering Structures*, 31(4), pp. 910-921.

Kilar, V. and Koren, D. (2010). "Simplified inelastic seismic analysis of base-isolated structures using the N2 method." *Earthquake Engineering and Structural Dynamics*, 39(9), pp. 967-989.

Koren, D. and Kilar, V. (2011). "The applicability of the N2 method to the estimation of torsional effects in asymmetric base-isolated structures." *Earthquake*

Engineering and Structural Dynamics, 40(8), pp. 867-886.

Kilar, V., Petrovčič, S., Koren, D., and Šilih, S. (2011). "Seismic analysis of an asymmetric fixed base and base-isolated high-rack steel structure." *Engineering Structures*, 33(12), pp. 3471-3482.

Kim, H.-D. and Lee, M.-J. (2010). "Analytical investigation of the P-Ä effect of middle-rise unbraced steel frames." *International Journal of Steel Structures*, 10(3), pp. 221-232.

Krawinkler, H. (2011). "Challenges in improving earthquake resilience through performance based earthquake engineering." *Proc. Bled4 – International Workshop on Performance-Based Seismic Engineering Vision for an Earthquake Resilient Society,* Institute for Structural and Earthquake Engineering of the Department of Civil Engineering, University of Ljubljana, Slovenia.

Medina, R. A. and Krawinkler, H. (2005). "Evaluation of drift demands for the seismic performance assessment of frames." *Journal of Structural Engineering,* 131(7), pp. 1003-1013.

Mezzi, M., Comodini, F., and Rossi, L. (2011). "A base isolation option for the full seismic protection of an existing masonry school building." *Proc. 13th International Conf. on Civil, Structural Engineering Computing,* Civil-Comp Press, Stirlingshire, Scotland.

Petrovčič, S. and Kilar, V. (2012). "Effects of horizontal and vertical mass-asymmetric distributions on the seismic response of a high-rack steel structure." *Advances in Structural Engineering,* 15(11), pp. 1977-1988.

Rajeev, P. and Tesfamariam, S. (2012). "Seismic fragilities for reinforced concrete buildings with consideration of irregularities." *Structural Safety,* 39, pp. 1-13.

Rodgers, J. E. and Mahin, S. A. (2011). "Effects of connection deformation softening on behavior of steel moment frames subjected to earthquakes." *International Journal of Steel Structures*, 11(1), pp. 29-37.

Sideris, P., Filiatrault, A., Leclerc, M., and Tremblay, R. (2010). "Experimental investigation on the seismic behavior of palletized merchandise in steel storage racks." *Earthquake Spectra,* 26(1), pp. 209-233.

SEAONC (1986). *Tentative seismic isolation design requirements.* Structural Engineers Association of Northern California, San Francisco, California.

Taflanidis, A. A. and Beck, J. L. (2009). "Life-cycle cost optimal design of passive dissipative devices." *Structural Safety*, 31(6), pp. 508-522.