

MODELLING AND ANALYSIS OF SEISMIC BASE-ISOLATED MASONRY HERITAGE STRUCTURES

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Abstract: In recent years seismic base isolation has become a widely used technology for seismic protection of buildings since it is very effective in minimizing structural damage during strong earthquakes. Even though the idea of using various kinds of isolation devices as an earthquake mitigation method has been conceived centuries ago, its actual implementation has a history of only about 30 years. The paper analyses the applicability of seismic base isolation made of elastomeric bearings to unreinforced masonry (URM) structures of heritage architecture. A new method for the mathematical modelling of base-isolated URM structures is presented. The proposed approach is applied to an existing three-storey masonry building. In order to signify the advantages of implementing a base isolation system as a retrofit technique for masonry heritage structures a case study comparison of results obtained by non-linear static analyses and the N2 method on fixed base and base-isolated variants is presented.

Introduction

Heritage structures include buildings and monuments with an inestimable cultural and historical value. By nature such structures should be altered as little as possible. That is why seismic retrofitting of heritage structures presents an extremely delicate field. On the one hand its aim is to increase the seismic resistance of the structure to a very small level of permissible risk, while on the other hand any remedial measures into such objects must be selected with care and should have minimum visual and other impacts that would degrade its historical significance. Conventional remedial actions for the seismic improvement of existing masonry buildings include the consolidation of the foundations and walls, establishing ties with sufficient stiffness between walls and between walls and floors, to achieve a global box-like behaviour of the structure, and the implementation of safeguarding measures to prevent potential collapse mechanisms.

In recent years the implementation of base isolation for the seismic mitigation of buildings has become a common alternative to conventional strengthening measures. In all over 16.000 structures have been protected in the world by seismic isolation, energy dissipation and other anti-seismic systems (Martelli et al., 2012). The majority of them are located in Japan, although they are more or less numerous in over 30 other countries. A base isolation system consists of a decoupling isolation layer between the building and the ground. The isolation layer is made of devices that support the building against static actions and, in case of seismic actions, implement a decoupling effect of the dynamic response of the building compared to the ground. A base-isolated building is characterized by a smaller acceleration response, leading to a smaller amount of forces of inertia, a smaller amount of interstorey drifts and internal forces in structural elements.

The paper deals with base isolation as a seismic restoration technique for masonry buildings of heritage architecture. A case study in which base isolation is implemented in a typical neo-renaissance masonry building is presented and discussed.

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Seismic base-isolation of masonry structures

In comparison with new buildings, the seismic base-isolation of existing buildings is tied to some specific features. Installation issues and accessibility to the equipment for inspection, maintenance or replacement present the most challenging factors for the installation of base-isolation in existing buildings. Masonry walls have to be decoupled from the foundation. This is done by using a diamond wire cutting saw or similar equipment (Bailey and Allen, 1988). Therefore to examine the quality of the walls rising from the foundations, to carry out any required consolidations and to define the most appropriate working demolition techniques, also with reference to the narrowness of the available spaces (Mezzi et al., 2011).

Another particular issue is that the isolators behave as concentrated supports under continuous masonry walls: the positioning distance influences the stresses in the walls, the total amount of devices and their layout in plan (Mezzi et al., 2011). When positioning the isolation devices in layout, care has to be taken in order to prevent global torsional effects. The problem of optimizing the placement of the centre of stiffness to prevent global torsion in the base-isolation layer has already been extensively studied by Kilar and Koren (2009a).

As of today, existing applications of base isolation to existing masonry buildings are scarce. These applications generally focus on heritage buildings with strong preservation needs. One of the best known examples is The Salt Lake City and County Building in Salt Lake City, Utah, USA (Bailey and Allen, 1998). The building is a five story masonry structure with a large central tower, built in 1891. The isolation system consists of 239 NRB (Natural Rubber Bearing) and 208 LRB (lead Rubber Bearings) isolators. Another example from the USA is The Hearst Memorial Mining Building in Berkeley, California (Davis and Robertson, 2000). The structure in question is a masonry building, built in 1907. It is located inside the Berkeley University campus that has been base isolated with 134 elastomeric devices. The largest of base-isolated masonry buildings are the headquarters of the New Zealand Parliament in Wellington, New Zealand (Poole and Clendon, 1992), which is a masonry building dating from the second decade of 1900. It has been base-isolated with 417 HDRB (High Damping Rubber Bearings) and LRB isolating devices.

Equivalent frame modelling and seismic failure mode interaction of URM

In a seismic event, the damage to masonry structures usually includes in- and out-of-plane failures of structural elements, which can be taken into consideration only if a comprehensive 3D model of the whole building is used (Belmouden and Lestuzzi, 2009). Unreinforced masonry (URM) buildings are usually built of load-bearing masonry walls, arranged in different orthogonal planes and in most cases connected together by means of flexible diaphragms (timber floors). In general, masonry walls often incorporate irregularities in the form of an irregular layout of openings, so that special modelling considerations have to be taken into account (Parisi and Augenti, 2012). In the cases of irregular geometry a macro-element discretization of masonry elements is usually adopted, which is able to predict the seismic response with sufficient accuracy and with relatively low computational costs compared to those involved in the case of conventional nonlinear finite element modelling approaches (Calio et al., 2012).

The in-plane lateral resistance of URM buildings is provided by the piers and spandrels. In addition to the vertical force in the piers due to the dead loads, these resisting elements are under horizontal seismic actions at the base of the structure subjected to shear and bending. Depending on the width to height ratio of a masonry element and on the respective values of the normal force, bending moment and shear force, three failure mechanisms might be observed (Magenes and Della Fontana, 1998):

- Rocking failure mechanism in which the horizontal load produces tensile flexural cracking at the corners and the pier begins to behave as a rigid body rotating (rocking) about its toe.
- Diagonal cracking shear failure occurring with the formation of a diagonal crack that typically involves both the mortar joints and the masonry units. This failure mechanism is a

result of several interacting factors, where the heterogeneity of masonry plays a dominant role.

- Shear sliding failure along a bed joint at one end of the structural element. This mechanism takes place only in piers.

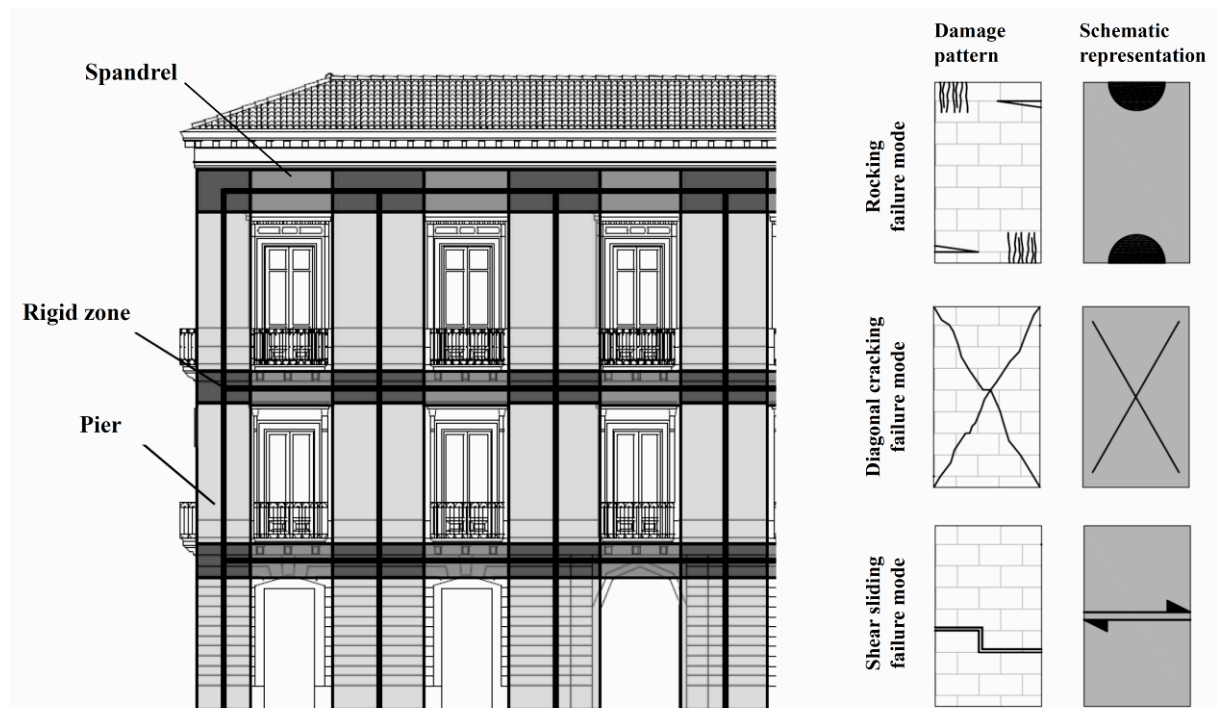


Figure 1. Equivalent frame representation of a three-story masonry wall - left (Liberatore et al., 1999) and in-plane masonry damage patterns under seismic loading - right (Petrovčič and Kilar, 2013)

In order to accommodate different failure modes in a simple and, in comparison with a comprehensive nonlinear shell finite-element model, computationally less demanding procedure, the authors of the paper have proposed a new technique for the modelling and analysis of regular URM structures (Petrovčič and Kilar, 2013). This technique is based on the equivalent frame approach, and incorporates linear beam elements and the plastic hinge concept. The complex seismic failure mechanism of masonry piers is expressed by a single failure mode interaction surface (an “FMI surface”), taking into account the influence of variation in the pier’s vertical loading, and its bending moment distribution. The ultimate lateral strength of a masonry element is expressed as a section which cuts through the FMI surface. A single failure mode interaction plastic hinge (an “FMI hinge”) for each masonry frame element is introduced by combining specific failure modes, taking into account their minimum envelope. The left side of figure 2 presents an example of an FMI surface which shows the ultimate shear strength of a masonry pier (V_U) for a varying axial force N , slenderness λ and corresponding failure mode. The expressions describing their mathematical formulations are readily available in European building codes and elsewhere in the literature.

On the right side of Figure 2 the normalized axial force–shear force (N – V) relation of a pier with slenderness equal to $\lambda = 0.5$ is presented, taking into account all three possible failure mechanisms. The presented relation is obtained by performing a section cut through the FMI surface for the given λ . Within the scope of an equivalent frame methodology with lumped plasticity, a single N – V failure mode interaction plastic hinge can thus be used instead of combining several types of plastic hinges in one element. The combined plastic hinge includes the interaction between the flexural, shear and diagonal failure modes with respect to the vertical (axial) load in a masonry pier.

This means that the proposed graphical representation of the FMI surface presents the main behavioural characteristics for the whole assemblage of masonry piers of an examined building. This feature can have many benefits in various aspects. For example, it can be used to determine possible damage patterns in piers before conducting the actual analysis. The accuracy of the proposed technique was confirmed by Petrovčič and Kilar (2013) by means of a comparative analysis of an URM wall assemblage. In the paper at hand this modelling technique is applied for the analysis of a base-isolated URM building.

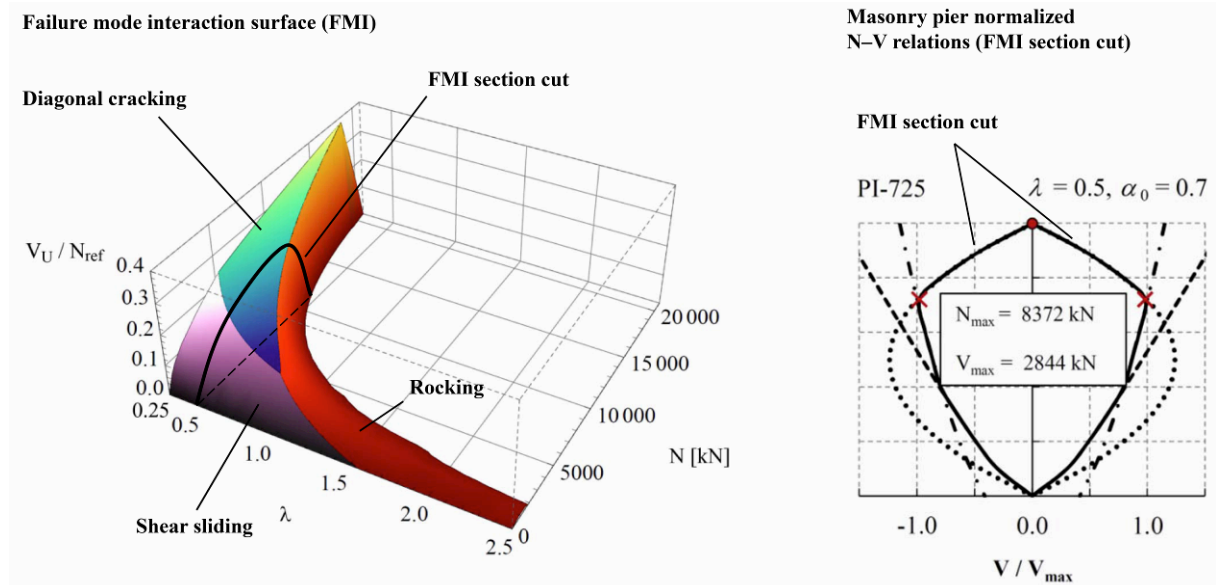


Figure 2. An example of a failure mode interaction surface of a masonry pier (left) and normalized pier N-V relations (right).

Examined masonry building

In the conducted case study a base-isolated masonry building typical for the European neo-renaissance era was analysed. The building is a three storey unreinforced masonry structure with floor plan dimensions 24.6m \times 12.9m (in directions X and Y, respectively) and a total height of 15.8m. The examined masonry structure is schematically presented in Figure 3a. Detailed information regarding its geometry, mechanical parameters and modelling are presented in the paper by Petrovčič and Kilar, 2013. The structure can be divided into four different types of planar wall assemblies, i.e. Wx-1, Wx-2, Wy-1 and Wy-2. Each planar wall of the structure has been modelled based on the equivalent frame approach, using piers, spandrels and rigid zones as indicated in Figure 1.

A non-linear equivalent frame mathematical model of the structure was created in the structural analysis program SAP2000 (CSI, 2011). Inelastic characteristics of masonry have been modelled by using the FMI plastic hinges presented in the previous section. The storey masses of the fixed-base structure (without base isolation) amount to $m_{\text{base}} = 878$ tons, $m_1 = 748$ tons and $m_2 = 499$ tons, whereas its fundamental periods of vibration equal to $T_X = 0.39\text{s}$ (in the X-direction) and $T_Y = 0.29\text{s}$ (in the Y-direction).

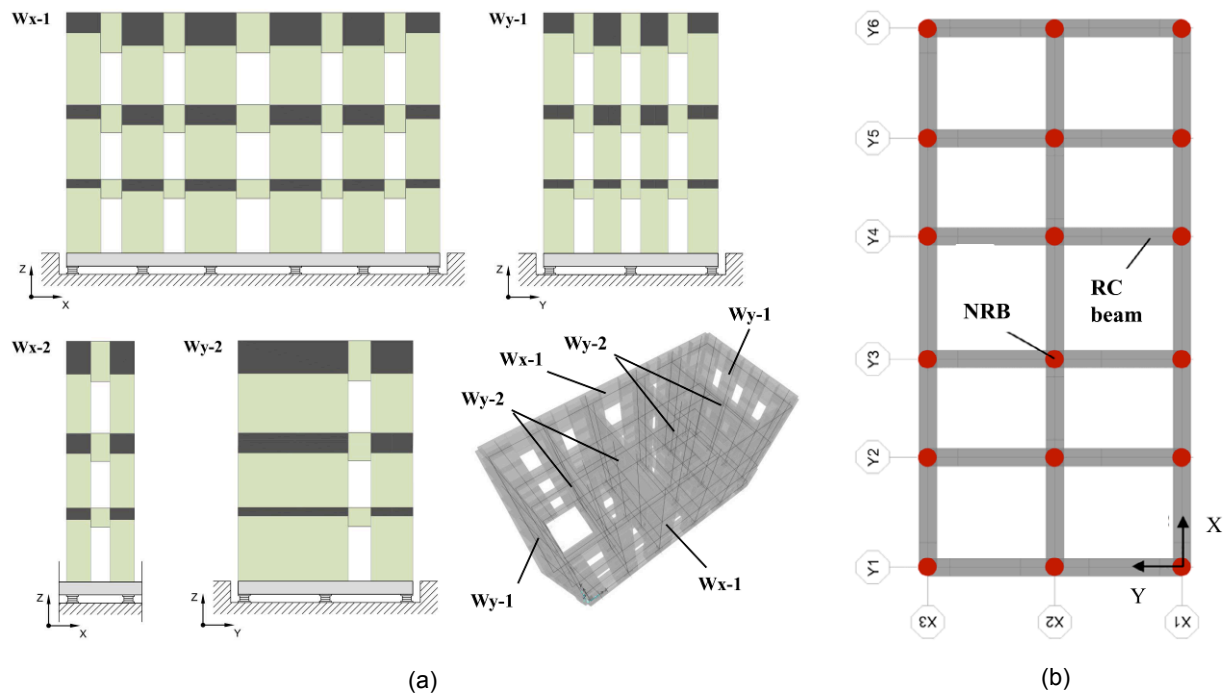


Figure 3. Examined masonry building (a) and the distribution of seismic isolators (b).

Seismic performance of the initial structure

For the selection of a proper base isolation system, a preliminary analysis of the fixed-base variant of structure was conducted in order to obtain its behaviour factor (q). In all conducted analyses presented in the paper only the X-direction of seismic loading was considered, since this direction is more critical for the structure at hand. Figure 4a presents the result of a non-linear static (pushover) analysis indicating the base shear vs. the top displacement relationships of the structure (capacity curve). The following seismic parameters have been considered: Eurocode 8 sub-soil class C and $\xi = 10\%$ of critical damping.

Three discrete limit states in accordance with Eurocode 8-3 (CEN, 2005) were observed, i.e. damage limitation (DL), indicating no or slight damage, significant damage (SD) and near collapse (NC). The limit states of the examined structure are indicated by dashed lines in Figure 4a. The global capacity NC limit state is defined in terms of the roof ultimate displacement. The ultimate displacement capacity is taken as the roof displacement at which total lateral resistance (base shear force) has dropped below 80% of the peak resistance of the structure. The SD limit state has been taken equal to $\frac{3}{4}$ of the NC displacement, while the DL limit state of the structure is by definition determined as the yield point (yield force and yield displacement) of the idealized elasto-perfectly plastic force – displacement relationship of the equivalent Single-Degree-of-Freedom (SDOF) system.

The displacement a structure will obtain when subjected to a seismic event, the so called “seismic demand”, has been determined by using the N2 method (Fajfar, 2000). The seismic demands for the DL and SD limit states are also presented in Figure 4a as two discrete points on the pushover curve. The seismic demand points correspond to a 95-year return period and $a_g = 0.15g$ for DL and to a 475-year seismic return period and $a_g = 0.25g$ for SD. It can be seen that the seismic demand displacements (points) exceed the capacity limits (dashed lines), meaning that the structure’s seismic performance is inadequate and that the implementation of seismic base-isolation or some other conventional strengthening technique is necessary.

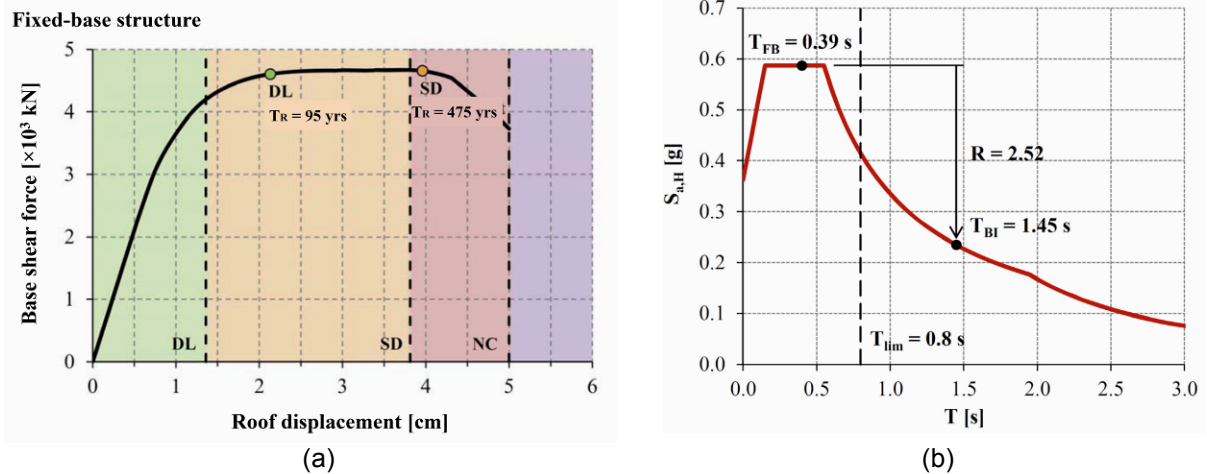


Figure 4. Pushover curve of the analysed building with corresponding limit states and seismic demand displacements (a) and the seismic force reduction factor R (b)

Implementation of a base isolation (BI) system

In the first step a reduction factor of seismic forces (R) was selected. The reduction factor indicates the level of reduction of seismic response spectrum forces due to the increase in the fundamental period of vibration of the BI system, abbreviated as T_{BI} (see Figure 4b). Care has to be taken when selecting R . If R is too large, then the lateral displacements of the base isolation layer could in turn present potential problems, e.g. for internal pipe installations. In the presented example a seismic force reduction factor equal to the maximum behaviour factor of the building (q) was selected as the input parameter for the design of rubber bearings ($R = q$). This means that the selected base isolation system will have for a 475-year seismic event the seismic demand equal to the yield point of the idealized elasto-perfectly plastic structural model.

Since the energy dissipation capacity of the structure can be expressed with the global ductility factor $\mu_u = d_{SD}/d_{DL}$, the structural behaviour factor q based on the equal displacement rule can be determined as follows (Tomaževič et al., 2004):

$$q = \sqrt{2\mu_u - 1} \quad (1)$$

In the case at hand the global DL and NC limit states are for the X-direction of seismic loading equal to $d_{DL} = 1.4$ cm and $d_{NC} = 5$ cm (see Figure 4a). The corresponding global ductility factor and behaviour factor equal to $\mu_u = 3.7$ and $q = 2.52$, respectively.

The mechanical properties of the bearings have been determined based on the selected seismic force reduction factor $R = q = 2.52$. The corresponding fundamental period of the BI system equals to $T_{BI} = 1.45$ s (see Figure 3b). The required stiffness of the BI system equals to:

$$k_{BI} = 4\pi^2 \frac{m_{tot}}{T_{BI}^2}, \quad (2)$$

where m_{tot} is the total mass of the structure with base isolation for the seismic load case. For the analysed building the required stiffness of the BI system equals to $k_{BI,req} = 47\,524$ kN/m.

A base isolation system consisting of natural rubber bearings (NRB) has been selected and implemented in the analysed mathematical model for the examined building. A total of 18 NRB have inserted at the foundation level and positioned in an orthogonal grid as presented in Figure 3.

The selected NRBs have a diameter of 60 cm and a height of 15 cm (height of rubber equal to 8 cm). They are made of soft rubber and have a horizontal stiffness of 2830 kN/m each, with damping equal to $\xi = 10\%$ of critical damping. The total horizontal stiffness of the base isolation system equals to $k_{BI} = 50\,940$ kN/m. The maximum horizontal displacement of the

selected NRBs is equal to 15 cm, which is about 200% of the height of the rubber. To ensure a uniform distribution of stresses onto the base isolation system RC beams has also been designed and implemented in the model.

Comparison of seismic performance of the fixed base and base-isolated variants

The seismic performance of two variants has been analysed: (i) a fixed-base (FB) variant without base isolation and (ii) a base-isolated (BI) variant. In both cases a pushover analysis has been carried out to the formation of the plastic mechanism in order to determine the inelastic capacity of the structural system. The N2 method has been used for the calculation of the seismic demand displacement (see previous chapters for details).

Figure 5a presents the capacity curves for both variants. In the figure the base shear force has been plotted with respect to the absolute and relative roof displacement. The absolute roof displacement is defined as the lateral displacement of the roof with respect to the ground, while in the case of the relative displacement the relative roof displacement is plotted with respect to base storey of the structure above the BI layer. The effect of inserting BI at the ground level can be seen from Figure 5a, where the absolute displacement of the BI variant is around 4-times larger than of the FB variant. This difference is due to the small lateral stiffness of the isolation layer. When comparing the relative displacement of the BI variant with the absolute displacement of the FB variant it can be seen that capacity curves of the structure have a similar shape. The BI variant has a slightly larger maximum base shear level due to the different lateral force distributions used in the N2 method. In the case of the BI a large lateral force has to be induced in the base level of the base-isolated structure in order to account for higher modes of vibration (for more details see for example the paper by Kilar and Koren, 2009b).

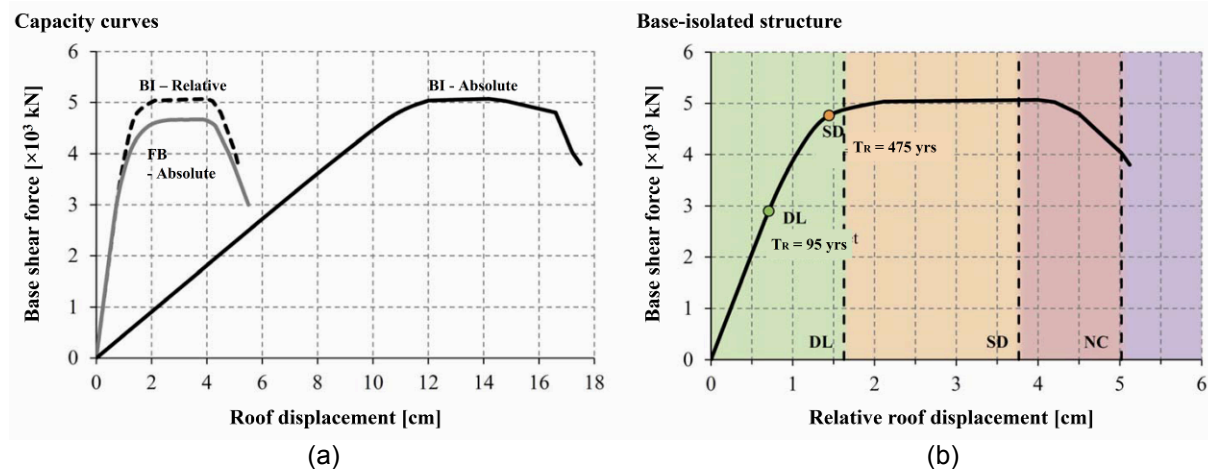


Figure 5. Comparison of pushover curves of both examined variants (a) and the pushover curve for the base-isolated variant with corresponding limit states and seismic demand displacements

The results of the pushover analysis of the FB case have already been presented (see Figure 4a). Since the seismic demand displacements for the DL and SD limit states exceed the capacity limits of the structure (indicated in the figure with dashed lines) the structure's seismic performance is inadequate. The calculated seismic demand displacement for the DL limit state at the roof level equals to 2.1 cm, while the capacity limit for this displacement at DL is at 1.4 cm. This means that the structure would suffer severe damage in the case of DL limit state (corresponding to a 95-year seismic return period) in which only mild structural damage is allowed by code provisions. In the case of the SD limit state, corresponding to a 475-year seismic return period) the structure would be close to collapsing.

In Figure 5b the capacity curve and limit states of the BI variant are presented. The target displacements for the DL and SD have also been indicated in the figure. It can be seen from the figure that the seismic demand displacements for both limit states are smaller than

capacity limits of the structure. The structure is lightly damaged even in the case of a 475-year seismic event.

The comparison of damage patterns for both variants is presented in Figure 6. In the figure the state of damage of specific structural elements is presented together with the activated failure mechanism. It can be seen from the figure that for the selected base isolation system the damage is significantly reduced in comparison with the fixed base building. Minor damage is observed only in a few piers in the upper storeys. The FB variant on the other hand, obtains a significant amount of damage. Most have reach their SD capacity level, while some have even exceeded the NC limit state, indicating their collapse. Damage is also observed in spandrels in the first two storeys.

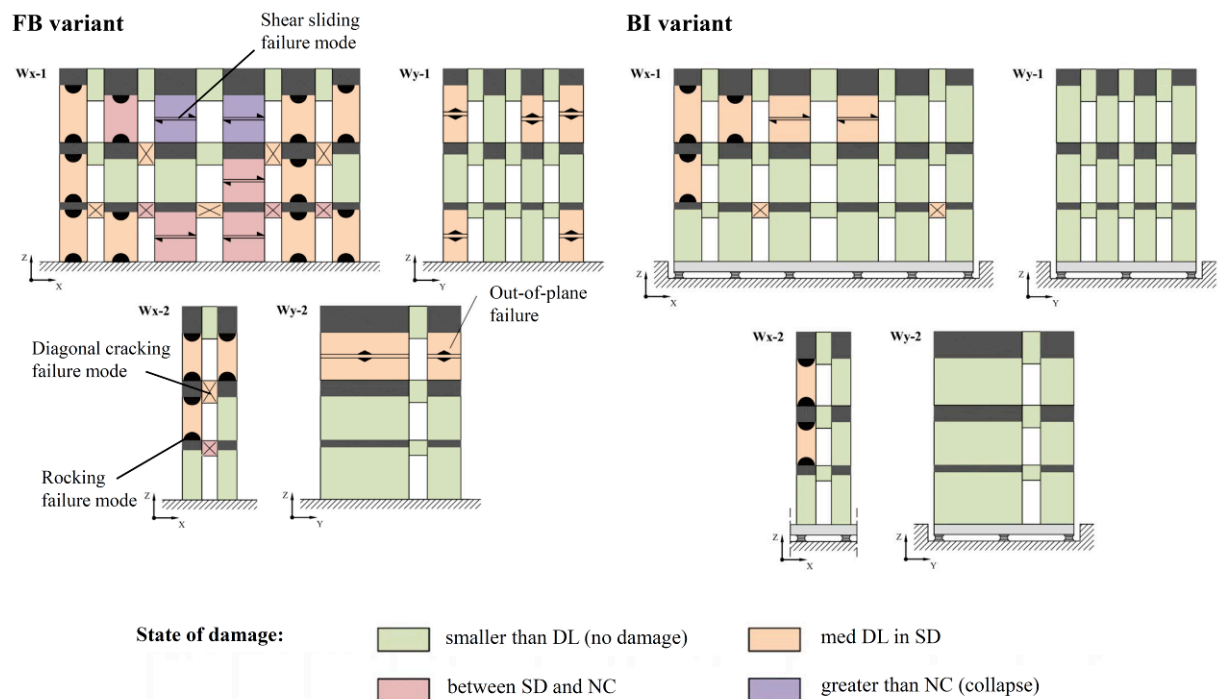


Figure 6. Comparison of sustained damage for the seismic loading corresponding to a 475-year seismic return period ($a_g = 0.25g$)

Conclusion

The use of base isolation can present an effective strengthening technique for increasing seismic resistance of architectural heritage buildings. Installation issues and accessibility to the equipment for inspection, maintenance or replacement at the moment limit the practical usability of base isolation only to extremely important buildings where additional costs of such measures can be justified.

In the first part of the paper a new modelling methodology for URM was presented, where complex seismic failure mechanisms of URM piers have been expressed by a single failure mode interaction (FMI) surface showing the relations between the pier's ultimate lateral strength V_u with respect to its axial force variation N , and its geometry expressed by the slenderness ratio λ . In the continuation of the paper it has been shown how the FMI surface can be used for mathematical modelling purposes and how a pier's lateral strength in terms of the N - V interaction can be expressed as a section cut through the FMI surface, considering the pier's λ . Using the SAP2000 computer program, a refined modelling approach for the inelastic behaviour of masonry elements has been defined.

The last part of the paper presents a case study in which a base isolation system has been utilized as a seismic restoration technique for a historical masonry building. The analysed structure has been modelled by using a macro-element frame model with the FMI surface to model the nonlinear response of masonry elements under seismic loading. In this approach a

single combined plastic hinge that includes the interaction between the flexural, shear and diagonal failure modes with respect to the vertical (axial) load in a masonry element was used. It can be also seen that the usage of proposed FMI surface enables a better insight in the damage and failure mode of a masonry units which enables a designer to tune the response of a building exactly to the wishes and needs of conservation plans provided by architects and conservation experts.

Two variants of the building have been studied. First the initial, fixed base variant was studied, which does not have base isolation or any other conventional strengthening technique applied. The second variant was base-isolated with natural rubber bearings. The results of a comparative pushover analysis have shown us that a base isolation system can dramatically decrease the damage caused by a seismic event on such structures. Furthermore, the installation of a base isolation layer requires minimum interventions in the original structure, since such a system is usually installed at the foundation level.

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