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TIMBER-BASED SEISMIC RETROFIT TECHNIQUES FOR EXISTING MASONRY STRUCTURES IN NORTH EUROPE

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ABSTRACT: This paper presents the results of an application of seismic retrofitting techniques to an existing masonry structure in Oslo. Due to low seismicity, the Norwegian National annex of 1998-1 (EC8-1, Standard Norge, 2021) allows omission from seismic design of several structures. Furthermore, the EC8-3 Norwegian National annex (Standard Norge, 2013) states that, unless the probability of collapse in the case of an earthquake changes significantly as result of a general refurbishment of the structure, it is not necessary to perform seismic retrofitting of the building. However there are cases where, depending on the geographical site, intended use or changes to the lateral force resisting structure, it is necessary to perform seismic analysis and possibly strengthen the building's resistance to seismic forces. In this paper, a real case study is used to investigate how the effect of changes to shear walls due to a refurbishment intervention can reduce the capacity of a masonry building, and how two combined timber based retrofit systems can reduce the probability of collapse after the interventions: timber posts working as "strong-backs" with nailed OSB connected to the internal surface of the masonry walls; OSB panels applied to the timber floor.

KEYWORDS: Strengthening masonry structures; timber engineering; masonry shear walls; earthquake strengthening.

1 INTRODUCTION

The background of this study is the need to understand how seismic loads should be evaluated for changes such as renovation or change of use for existing masonry buildings, with a focus on a masonry type construction which is typical in Oslo. Oslo is located in an ancient deep rift zone corresponding to Oslofjorden, which is relatively active but with fairly low seismic loads [1]. The biggest recorded seismic event occurred in 1904, with the Ms 5.4 Oslofjorden earthquake. Masonry buildings constructed during the period 1850-1920 make up a significant part of the building stock in Oslo (Oslo Byleksikon [2], see Figure 1). The constructions are typically unreinforced masonry consisting of clay bricks and lime mortar. The floors are wooden beams.

Building standards for designing resistance to seismic forces were introduced with the Eurocodes in Norway in 2004. For certain buildings the codes require demonstration of capacity against earthquakes for both new buildings and changes in existing buildings. In cases where seismic design cannot be omitted according to EC8-1[3] , the construction must be demonstrated to be resistant to seismic forces.

EC8-1 has a set of criteria for when seismic evaluation is not necessary. The latest revision as of June 2021 has a new set of rules regarding omission of seismic evaluation of structures. According to the National annex of EC8-1, most structures in Oslo in seismic class II or lower, will no longer need to be evaluated for seismic loads. However, if ground conditions are unfavourable and the building project involves a change in use, masses and/or altering of stabilizing constructions, a seismic evaluation should be performed. EC8-3 guidelines are given for the design of existing structures. The National Annex, paragraph NA.2.1[4], states the following:

"*The reinforcement of existing structures that have not been exposed to earthquake damage can be limited to changes or additions that change the load effect and/or the carrying capacity in such a way that the probability of collapse during an earthquake increases significantly if the structure is not reinforced. In the assessment, NS-EN 1998-1 is used for calculation of load effect both before and after the change/addition (Standard Norge, 2013, page 2 in the National Supplement, authors' translation)*."

The standard does not provide provisions for what is considered a significant increase in the probability of collapse. However, RIF - The Norwegian Association of Consulting Engineering Firms has a published the paper "Dimensjonering for jordskjelv av eksisterende konstruksjoner" [5]). In this paper, a significant increase is defined as a 20% increase in the risk of collapse during an earthquake. The RIF guide reviews EC8-3 and EC8-1 for use in the design of existing buildings, and provides recommendations for calculation methods for demonstration of significant increase. However, the paper does not go into detail on the practical application for masonry buildings.

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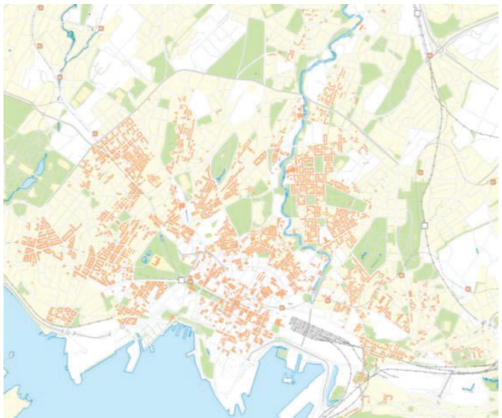

Figure 1: Murbyen Kristiania, construction before 1915 (reproduced from Kart: Byantikvaren, 2011 [6]*)*

In Oslo (and probably Norway in general), seismic retrofitting of masonry structures is uncommon, and the perception amongst most building owners and people in general is probably that earthquakes don't impose a real threat in Norway. The aim of the study is therefore to look at simple seismic retrofitting strategies that are cost efficient and can improve the seismic resistance of the building structure so that the structure can withstand the design seismic load. For an existing construction where the lateral resistance is below seismic design loads, and the probability of collapse increases significantly after the changes have been made to the construction, the aforementioned guidelines are relevant to decide the need for retrofitting.

A large part of the building stock in inner Oslo consists of older masonry buildings that require repurposing or renovation.

The purpose of this study is to propose a retrofit technique when changes in masses, stiffness and/or strength constitute a significant increase for risk of collapse under seismic influence.

2 DESCRIPTION OF THE CASE STUDY

The project is a case study where a sample building is modelled with different situations where the walls' geometry is altered, introducing openings in lateral force resisting elements of the building.

The building in the case study is a typical masonry structure built around 1890 in the Grünerløkka quarter in Oslo, located in a densely populated residential area with buildings from the same era and with approximately equivalent geometry and construction methods.

The structures have a fairly consistent building design, with a load-bearing structure with piers and spandrels of unreinforced masonry of solid bricks and with timber floors and roofs. The floor slabs consist of floor joists that are supported by the façade wall, the central wall, and the back wall.

Normally, the timber floors have a dimension of approximately 170mm x 220mm, spaced 700mm–900mm. Dried clay, often from the construction site was used between the floor beams to increase the capacities for insulation, fire-resistance and sound insulation (see for example Figure 2).

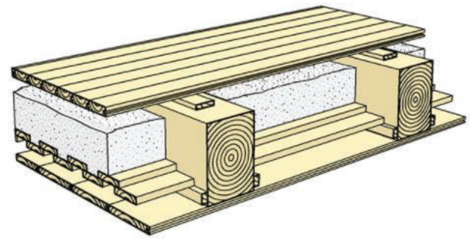

Figure 2: Typical floor construction (Sintef Byggforsk 2017, reproduced from [7]*)*

The building is relatively symmetrically designed, even if not completely regular on the ground floor. In the sample building, the structural design was changed in the 1960s by partially demolishing the load-bearing walls at the ground floor, to create more open spaces better suited for serving or retailing. All changes were in the main load-bearing walls in the X direction (façade wall, central wall, and back wall, see Figure 3). For the calculations, the existing/original situation is as it was before the renovation, while the new situation largely coincides with the actual changes in the load-bearing walls that were carried out in the 1960s.

The original (existing) and new situations for load-bearing walls in the X direction are shown in Figure 4. In the existing situation, the beams are dimensioned for the facade openings and the openings in the central wall. For changes in the new situation, both beams and columns are dimensioned.

The case study will be analysed both in the original situation and after the opening of the structure, in order to evaluate the effect of changes in terms of capacity of the building. The type of retrofitting techniques considered in this analysis will be the stiffening of timber floors and the application of timber strong backs with nailed OSB in the masonry walls in X direction.

A numerical model of the capacity of the structure is presented and the results are expressed in term of push-over curve, with respect to five design situations: original building; building after the opening; building with timber floors stiffened at the first storey; building with one wall retrofitted with timber strong-backs and OSB; building with timber floors stiffened at the first storey and one wall retrofitted with timber strong-backs and OSB.

The push-over capacity curve has been derived considering the mean mechanical parameters reported in Table 1 based on common practice in Norway for week mortar brick type of masonry with an assumption for cohesion value (i.e. 0.4).

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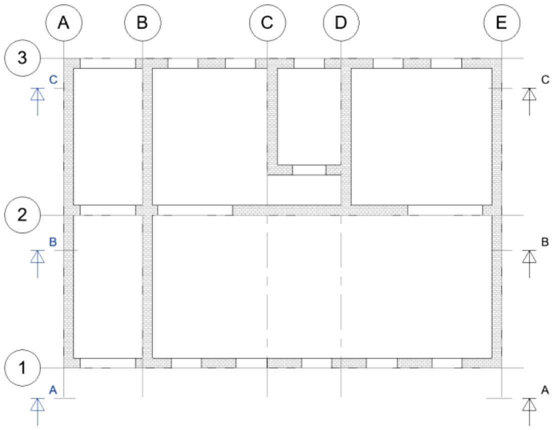

Figure 3: *Plan of structure, existing situation*

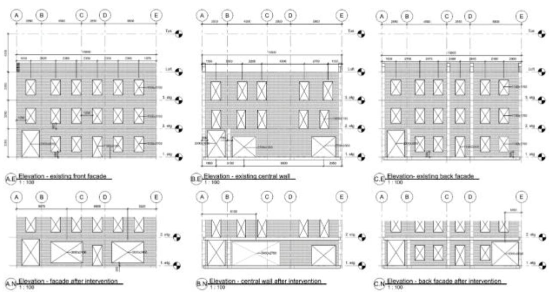

Figure 4: *Three load bearing walls before and after changes*

Table 1: Mechanical parameters adopted in the model

Parameter	Mean Value
Cohesion	0.1 MPa
Friction coefficient	0.4
Compression strength *fc*	2.1 MPa
Elastic modulus	1050 MPa

3 PROPOSED RETROFIT TECNIQUES

3.1 ENHANCEMENT OF THE MASONRY

The proposed retrofit system is based on the strengthening solutions proposed and tested by Dizhur et al. [8], which involves the use of vertical timber posts connected to masonry walls to improve their out-of-plane capacity. The solution herein reported has been further developed by Guerrini et al. [9] and Miglietta et al. [10] to also improve the in-plane strength and deformation capacity of piers and the connection between masonry and floor systems. In fact, the lack of connections between masonry and flooring is a significant weakness in existing URM buildings and is addressed by the proposed retrofit system.

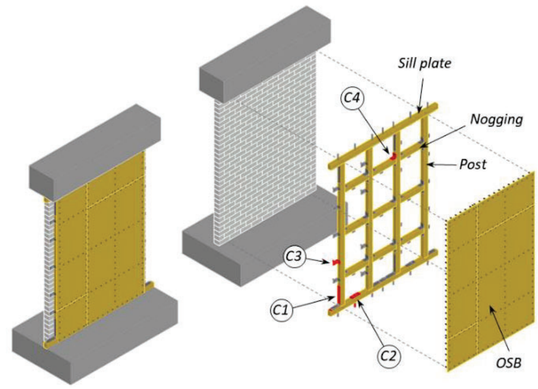

Figure 5: *Retrofit components and layout* [9]

The proposed retrofit system aims at improving the in-plane capacity of piers by adding horizontal timber members (nogging or blocking) between vertical posts to create a timber frame connected to the masonry. As shown in Figure 5, the frame is completed with top and bottom sill plates that connect to the floor and foundation. There are four different connection types: C1 (connections between posts and sill plates), C2 (anchorages between sill plates and floor/foundation), C3 (connections between timber frame and masonry wall), and C4 (connections between timber frame components).

The vertical posts improve the masonry wall's out-of-plane response by acting as strong-backs in flexure. The timber frame and OSB panels increase the pier's in-plane capacity by interacting with the masonry through connections C3. The posts and tie-down connections (C1) contribute to in-plane flexural strength, while the OSB layer and nailing contribute to in-plane shear strength.

The connection enhancement between masonry and floor offered by the retrofit system is critical in improving seismic performance as it promotes a global box-type response, preventing the onset of undesirable local mechanisms that can greatly reduce the building's seismic capacity. The system could be applied in one or both sides of the masonry, considering architectural limitations and required level of improvement.

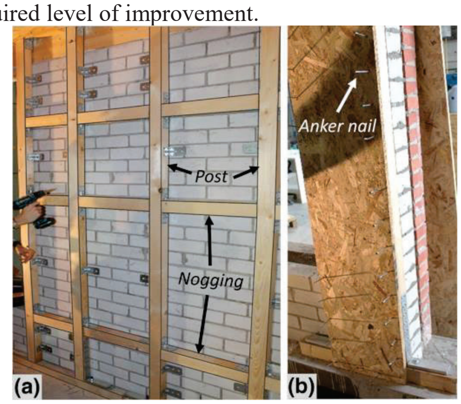

Figure 6: *Pictures of the retrofit system during the application on a building specimen tested on shake table: a) wall timber frame, b) nailing of a OSB for in-plane strengthening of longitudinal walls* [10]

This retrofit technique has been validated through components [9] and shake table tests on an entire URM building [10], as reported in **Error! Reference source not found.**. Analytical formulations to design this retrofit intervention are reported in Damiani et al. [11].

3.2 ENHANCEMENT OF THE FLOOR DIAPHRAGMS

Another structural element that may require to be stiffened and strengthened to transfer the shear between walls on different vertical planes and to assure a box-type response are the flexible timber diaphragms. Different strengthening techniques have been proposed in the past (see [12, 13]), but a typical in-plane floor reinforcement might use 18-mm thick OSB panels attached to the existing planks using anker nails and timber blocking beams inserted between joists. The blocking elements and joists have to be connected using steel angles and screws.

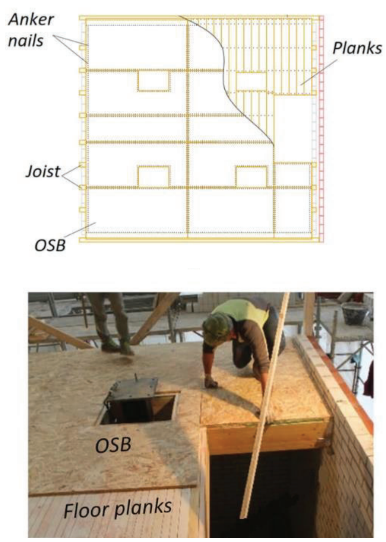

Figure 7: *OSB panels layout and picture of the diaphragm retrofit system during the application on a building specimen tested on shake table* [10].

The system could in principle be applied from underneath the diaphragm or on top of the diaphragm adapting the adequate detailing. Analytical formulations to design this retrofit intervention are reported in Damiani et al. [11]

4 MODELING OF THE RETROFIT INTERVENTIONS

4.1 STRUCTURAL ANALYSIS SOFTWARE

The commercial software selected for the analyses is 3Muri. It is based on the macro-element approach (with elastoplastic pier and spandrel behaviour) and allows execution of nonlinear static (i.e. pushover) and analyses of entire masonry buildings, by means of an equivalent-frame idealisation of the structure (Lagomarsino et al. [12]. This analyses allow to estimate the global capacity and behaviour of the structure.

The nonlinear static analyses were carried out with two different force distributions (e.g. mass proportional and first mode distributions), indicated by several codes (e.g. EN 1998-1, 2004, Eurocode [3]), to account for the dynamic response in the different phases of damage evolution.

The force distribution is applied along two orthogonal directions (both positive and negative), also taking into account accidental eccentricities introduced by the codes to account for uncertainties in the location of masses. Since the building originally has flexible timber floors, the position of the control node is critical. For this reason, one of the nodes belonging to the central wall at the top level was selected, as it was considered representative for the portion of the building with a higher displacement.

The proposed analyses are valid for evaluating the overall behaviour of the structure. Local analyses, such as the evaluation of out-of-plane mechanisms of masonry or the actual effectiveness of the steel beam inserted in place of the removed portion of masonry, should be evaluated with specific methods or software (these calculations are beyond the scope of this paper).

4.2 DEFINITION OF THE INTERVENTIONS

The building work carried out on the ground floor, such as the widening of openings, and in particular the central wall, worsens the seismic capacity both in terms of maximum shear at the base and displacement. Structural retrofit solutions must therefore be aimed at restoring the original structure's resistance and deformation capacity. In order to minimize the cost of the intervention, especially in terms of invasiveness for residents on the upper floors, it may be possible to intervene locally only on the ground floor.

The best solution from a seismic performance point of view would be to retrofit all the diaphragms and all the masonry piers and spandrels. Considering that this solution may be too expensive and also takes up valuable space, three more cost-effective solutions are investigated in this paper: (i) the first is to retrofit only underneath the first diaphragm above ground floor to redistribute the shear between walls and give the structure a more box-like behaviour; (ii) the second is to retrofit the main pier in the central wall on both faces, as the length of this structural element has been heavily reduced, and (iii) the third solution is to retrofit both the floor and the wall pier. A scheme of the retrofit solutions can be seen in Figure 8.

Regarding retrofit of the masonry (ii, iii), increasing the shear and deformation capacity on the shortened pier appears to be the only solution that maximizes the cost/benefit ratio. As shown in Figure 9 in section number 5 in the original structure (at the top), the pier remained undamaged (green) until the structure's ultimate capacity was reached, while after enlargement of the openings, (central figure), the remaining pier was the first to reach ultimate deformation (orange). Retrofitting it by increasing both its strength and deformation capacities

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allows the entire structure to have beneficial effect both in terms of base shear and displacement capacity.

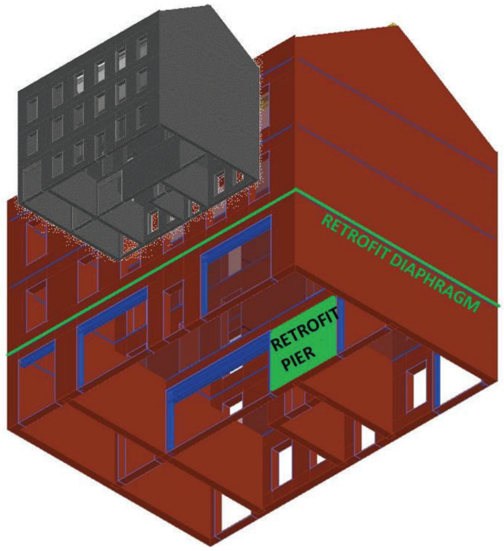

Figure 8: *Original and enlarged openings structures with scheme of the proposed structural interventions* [9]

4.3 MODELLING OF THE RETROFIT INTERVENTIONS

Detailed analytical formulations to design retrofit intervention are reported in Damiani et al. [13]. A detailed modelling of the same type of retrofit is also possible as reported in Damiani et al. [11]. For the preliminary study proposed in this paper, the retrofit was not designed in detail. The increase in shear strength and diaphragm stiffness was based on test results, and can therefore be considered plausible, but valid only for the specific details reported in Miglietta et al. [10]. The strength of the retrofitted masonry and the stiffness of the diaphragm may be increased or decreased depending on the nail spacing and/or on the thickness of the OSB or other detailing.

In particular, to model the presence of the retrofit for the masonry, the shear strength of masonry was increased by approximately 10 kN/m for the pier as experimentally assessed when retrofitting one side of the wall. As the pier is 3.6 meters in length, a retrofitting on both faces adds up to a shear strength increase of approximately 72 kN (about 1.5 times the strength of the non-retrofitted pier).

The ultimate shear deformation capacity of the retrofitted pier was increased from 0.5% to 0.8%. The latter value is conservative, as demonstrated by the physical testing described in [9]. For comparison 0.8% is the deformation capacity prescribed by Eurocode for reinforced masonry.

Concerning the diaphragm, the shear stiffness (expressed in terms of shear modulus times the thickness of the floor $G \cdot t$) was increased from 400 N/mm, a value assumed realistic for a flexible timber diaphragm with nailed planks, to 6300 N/mm for the diaphragm retrofitted with OSB, a value that can be considered plausible and valid for the specific detailing reported in Miglietta et al. [10]

5 RESULTS OF THE ANALYSES

As previously reported, the analyses were performed in all directions and for both force distributions (i.e. proportional to the first mode of vibration and proportional to the masses). For the sake of simplicity, only the results of the most critical analyses are reported here: parallel to the enlarged openings, with forces proportional to the masses and applying forces from left to right with no eccentricity.

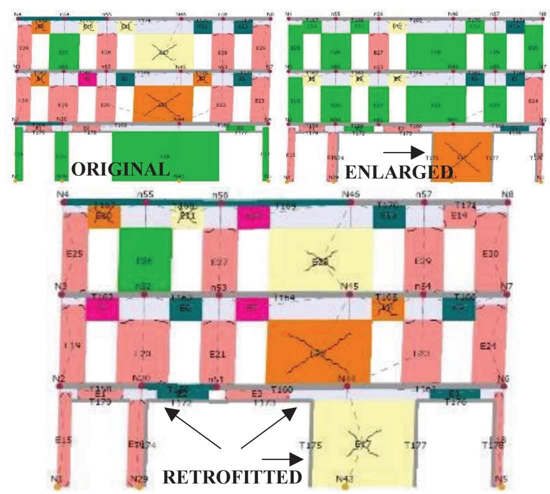

Figure 9: *Visualization of deformed shape and damage at ULS of the central wall for original (top left), enlarged openings w/o retrofit (top right) and retrofitted masonry and diaphragm (iii) (bottom) buildings.*

Figure 9 shows the deformed shape and damage at the ultimate limit state of the central wall (wall number 2 as per Figure 3) for the original (top left), enlarged openings without retrofit (top right), and retrofitted masonry and diaphragm (iii) (bottom). Analysing the top part of the figure, it is evident that the enlargement of the opening significantly affects the structure's behaviour. In the original building (top), the near-collapse condition is reached due to failures at the second storey, while the construction with enlarged openings collapses due to premature failure of the shortened pier at the ground floor, with walls in the upper floors almost undamaged. These results clearly suggest that a retrofit intervention should be focused on an enhancement of that particular portion of the building, both in terms of strength and deformation capacity.

The lower part of Figure 9 shows how the application of the retrofit to the masonry and diaphragm allows for more diffused damage through the storeys. In particular, the diffusion of damage throughout the storeys of the retrofitted structure is similar to the one obtained with the original one. This allows to avoid the soft-storey

mechanism of the building with enlarged openings increasing the displacement capacity of the building.
Figure 10 shows the deformed shape at ULS of the first diaphragm for enlarged openings without retrofit (left) and retrofitted masonry and diaphragm (iii) (right).

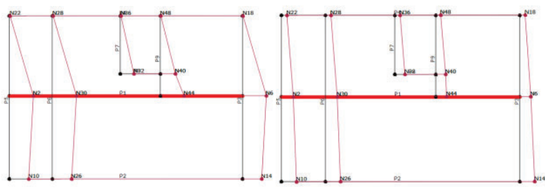

Figure 10: *Visualization of deformed shape at ULS of first diaphragm for enlarged openings w/o retrofit (left) and retrofitted masonry and diaphragm (iii) (right) buildings.*

The result of the analysis is shown in Figure 11. Firstly, comparing the original situation (black line) with the enlarged openings (red line) it is possible to observe a significant reduction both in terms of strength and displacement capacity (-16% both in terms of a_y* and d_u* according the bilinearization proposed by EC8).
Figure 11 reports the capacity (i.e. pushover) curves associated with the different models in terms of equivalent SDOF acceleration and displacement.

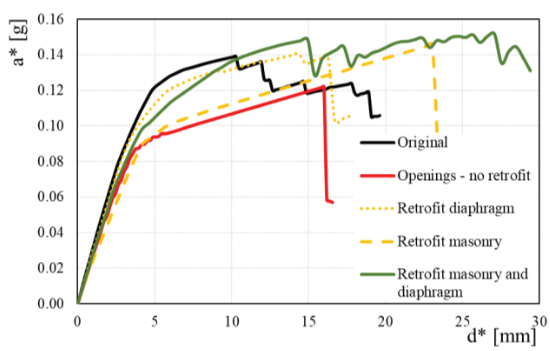

Figure 11: *Capacity curves associated with the different models in terms of equivalent SDOF acceleration and displacement.*

From the figure it is also evident that applying a retrofit intervention on the diaphragm alone (i) is beneficial for the overall strength, nearly restoring the capacity of the original situation. This is due to the inertia force primarily acting on the central wall with flexible diaphragm, while with the strengthened diaphragm loads are transferred also to the other walls, making use of their flexural/shear capacity. On the other hand, it is evident that this intervention does not restore the full displacement capacity of the original building. A representation of the difference between the behaviour of the original and the retrofitted diaphragm can be seen in Figure 10. Retrofitting only the central pier (ii) gives a different behaviour: this limited intervention is not able to restore the full strength of the building, but the displacement capacity is higher than the original situation (+21%); as visible in Figure 9, this is due to the combined effect of increase of displacement capacity of the ground floor pier and the increase of its strength, allowing for a better spread of damage throughout the storeys (i.e. avoiding the soft-story mechanism happening in the structure with enlarged opening but without retrofit interventions). Considering the positive effects of (i) and (ii), the best solution appears to be a combined retrofit of masonry and diaphragm. In this case (green line), both the strength and the displacement capacities were enhanced (+7% and +50% respectively from the original configuration).
Table 2 reports the SDOF parameters according EC8 [3]which summarize the results graphically represented in terms of capacity curves in Figure 11.

Table 2: *SDOF parameters associated with the different models according to the bilinearization proposed by EC8.*

	a_y* [g]	d_u* [mm]	Γ [-]
Original	0.13	19	1.31
Opening-no retrofit	0.11	16	1.28
(i) Retrofit diaphragm	0.13	16	1.26
(ii) Retrofit masonry	0.13	23	1.28
(iii) Retrofit masonry and diaphragm	0.14	29	1.29

6 CONCLUSIONS

Norway can be seen as a low to intermediate seismicity area, with a low seismic hazard and a low exposure level except the area around Oslo and on the west coast. The original motivation of this study is the need to understand how seismic capacity should be evaluated for changes such as renovation or change of use for existing masonry buildings, with a focus on a masonry type construction which is typical in Oslo. For that purpose, a case study of an existing brick masonry building has been analysed before and after a typical refurbishment intervention which introduces some opening in the lateral resisting masonry walls. The different design situations were investigated by means of a global analysis of the building through an equivalent frame macro-element model, using the commercial software 3Muri specific for masonry structures. The results of the analysis for the different cases were reported in terms of capacity curves.
The results showed that the intervention produced a significant weakening of the lateral capacity of the case study (around 16% both in terms of acceleration and displacement), even if, due to the low seismic input of Oslo area (PGA of 0.03g), the verification for seismic resistance could be still considered satisfied after the structural intervention. However, since these types of constructions are typical also of other area in Norway (and more generally in Northern Europe) where the conditions could be more unfavourable (higher seismic hazard, ground amplification), two retrofitting techniques, based on timber elements reinforcing the building structural elements, were investigated to reduce the probability of collapse after the weakening intervention: timber strong-backs with nailed OSB applied to both surface of a central masonry pier; OSB panels applied to the first floor level.

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The results of the analysis showed both retrofitting solutions effectively improve the global performance of the building, and that a combined retrofit of floors and diaphragms produces the best performance both in terms of capacity and displacement.

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