17th World Conference on Earthquake Engineering, 17WCEE
Sendai, Japan - September 13th to 18th 2020

SEISMIC ASSESSMENT OF AN EXISTING SWISS UNREINFORCED MASONRY BUILDING WITH FLEXIBLE FLOOR DIAPHRAGMS

S. Arslantürkoglu[(1)], N. Volken[(2)], N. Mojsilović [(3)], B. Stojadinović [(4)]

[(1)] *PhD Student, Institute of Structural Engineering, ETH Zurich, safak@ibk.baug.ethz.ch*
[(2)] *MS Student, ETH Zurich, volken_nicolas@hotmail.com*
[(3)] *Senior Scientist, Institute of Structural Engineering, ETH Zurich, mojsilovic@ibk.baug.ethz.ch*
[(4)] *Professor, Institute of Structural Engineering, ETH Zurich, stojadinovic@ibk.baug.ethz.ch*

Abstract

Switzerland is a country of low and moderate seismicity. Nonetheless, because unreinforced masonry (URM) residential buildings are a major portion of the built inventory, the seismic risk is not negligible. To meet a growing need for seismic evaluation of buildings, SIA 269/8, a Swiss code for seismic assessment of structures, was recently adopted. SIA 269/8 prescribes a risk-based seismic evaluation using the compliance factor concept, a factor that relates the seismic capacity of an existing building to the seismic capacity corresponding to the collapse safety requirement of a hypothetical (essentially) identical new structure. The goal is to achieve an acceptable low risk of casualties (between 10^{-5} and 10^{-6} individual annual casualty risk linked to compliance factor values between 25% and 100%). SIA 269/8 also defines a process on how to decide if a seismic retrofit is mandatory or not and how to select the retrofit measures that are commensurable to the actuarial value of the potentially saved lives.

This paper presents a case-study investigation of a typical existing Swiss URM building with flexible floor diaphragms that features an in-depth analysis of both local failures and global building behavior. Prior to undertaking a global nonlinear static analysis, local failure modes such as the out-of-plane wall failure mechanism, support and resistance of the floor diaphragms, and the load transfer from the floor diaphragm to walls were examined. The main emphasis was on the interaction between the out-of-plane responding walls and the flexible floor diaphragms. The force-based method based on rigid-body motions and the Paulay & Priestley approach were considered. Being known for its conservatism, the latter is still the most common approach among the Swiss engineering community. Equivalent frame approach was then used to model the building globally, followed by the global seismic performance assessment using the N2 method that compares the displacement demand to the displacement capacity of the entire structure.

The outcome of the SIA 269/8 evaluation procedure is that the governing compliance factor of 75% is dictated by the out-of-plane responding wall of the north façade, resulting in a corresponding cost limit of 7’500 USD. In other words, seismic retrofit is mandatory if the upgrade related cost does not exceed 7’500 USD. Following SIA 269/8, a seismic upgrade may be waived if the expected cost is higher than this threshold value as the achieved risk reduction is not reasonably justified.

The findings of this case study indicate that an implicit assumption of global structural integrity may give a false sense of safety for URM buildings with flexible diaphragms as local failure mechanisms often govern the structural performance. Another distinct finding to emerge from this study is the importance of updating of geometric and material parameters of the existing structure for the seismic capacity assessment using in-site investigations and laboratory tests to improve the often limited knowledge of the current state of the building and reduce uncertainties.

Keywords: equivalent frame modeling; out-of-plane failure; seismic assessment; unreinforced masonry

1. Introduction

Unreinforced masonry (URM) is among the most common structural types in the residential building inventory of Switzerland. In light of the damage observations from past earthquakes, the structural integrity of the URM buildings with flexible diaphragms became a core seismic assessment issue due to the lack of “box behavior” in such buildings. The term box behavior is generally understood to mean that the stiff floors can provide diaphragm action so that the load-bearing URM walls are subjected only to in-plane actions, and the behavior of the building can be evaluated using a global, building-level structural model.

A case study investigation of an existing URM building with flexible floor diaphragms was done to illustrate how to do a detailed seismic assessment. Local investigations of the structural members were conducted with an emphasis on the out-of-plane behavior of the URM walls. Two techniques were used to evaluate such out-of-plane behavior: firstly, the Paulay & Priestley method [1], to obtain the displacement capacity of the out-of-plane-loaded walls and their nonlinear behavior; and secondly, the Italian codified force-based procedure [2] based on rigid-body kinematics to evaluate wall stability. As for the global analysis, equivalent frame modeling was employed to obtain the building behavior through a nonlinear static pushover analysis.

With increasing requirements in Swiss seismic building codes in the last five decades, existing buildings have become more critical for seismic actions: it is common that they do not meet the seismic design criteria for new buildings. However, the cost of a seismic retrofit may be unreasonably high compared to the achieved level of seismic risk reduction. Following an evaluation of both local and global behavior of the case study building, a risk-based seismic assessment and cost-benefit considerations were performed as prescribed by the Swiss Structural Code SIA 269/8 [3].

2. Risk-based seismic assessment according to SIA 269/8

This section gives an overview of the seismic assessment procedure for existing building structures in Switzerland, according to SIA 269/8 [3]. The assessment procedure starts with the inspection of the structure and gathering data to assess the state of the structure, followed by modeling and seismic response analysis of the structure and an examination of the construction details. The compliance factor is computed and used to evaluate the casualty risk posed by the building to its occupants. In the last step, risk-proportionate recommendations are made to improve the seismic performance of the investigated structure.

2.1 Mathematical expression of the seismic safety

In an effort to express the seismic safety of an existing structure, many researchers have addressed the cost efficiency of a seismic upgrade by paying particular attention to the cost of a seismic upgrade and the achieved level of risk reduction [4], [5]. Their findings formed the foundation of the Swiss code SIA 269/8 [3] for the evaluation of the seismic safety of existing structures using the compliance factor concept. The compliance factor compares the relevant capacity A_R (local or global, force- or displacement-based) of the existing structure to the capacity $A_{d,act}$ of a hypothetical essentially identical structure that satisfies the safety requirements for newly designed code-compliant structures. In this matter, the compliance factor α_{eff} describes to what extent an existing building satisfies the corresponding SIA codes for new structures. The concept of compliance factor in its generic form is defined in Eq. (1). In the Swiss engineering practice, it is common to use nonlinear static pushover analysis to obtain the displacement capacity d_u of a structure before a global or partial collapse is triggered and compare it to the displacement demand d derived from the corresponding SIA 261 [6] design spectrum. A displacement-based compliance factor is given in Eq. (2).

$$\alpha_{eff} = \frac{A_R}{A_{d,act}} \tag{1}$$

$$\alpha_{eff} = \frac{d_u}{d} \tag{2}$$

2.2 Commensurability of the interventions

Threshold values for the compliance factors are defined to ensure a certain level of life safety based on the importance class and the remaining service life of a structure (Fig. 1). If the computed compliance factor is lower than the threshold value α_{min}, implementation of the seismic retrofit measures is compulsory at any cost, as the earthquake-related risk is considered too high. For structures in the SIA 261 [6] importance class I, for example, a minimum compliance factor α_{min} of 0.25 refers to a maximum acceptable annual individual mortality risk of 10^{-5}. For compliance factors greater than α_{min} and lower than 1.0, retrofit decisions are made based on the commensurability of the seismic retrofit, according to Fig. 1. Existing structures that satisfy the SIA code requirements for new structures have a compliance factor greater than 1.0, corresponding to annual individual mortality risk of 10^{-6}.

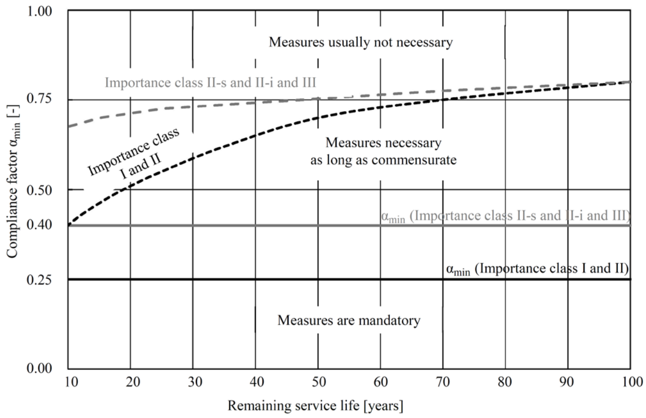

Fig. 1 – Compliance factor ranges delineating mandatory, commensurate and optional seismic retrofits, as a function of the remaining service life of a structure [3]

Commensurability of a seismic retrofit is established through a cost-benefit analysis, where the monetary value of the achieved annual risk reduction ΔRP_M is compared to the annualized costs of the retrofit measures SC_M, computed over the expected remaining service life of the structure. If this ratio, the efficiency EF_M of the seismic upgrade, is greater than 1.0, seismic retrofit measures are said to be commensurate, and hence have to be taken. Otherwise, seismic retrofit measures are optional. The monetary value of the achieved risk reduction is computed by considering the actuarial value of protected lives (assuming average building occupancy), as well as the value of protected property or preserved function. As the considerable portion of the existing Swiss building stock has a compliance factor lower than 1.0, the risk-based cost-benefit analysis offers a rational basis for decisions on financing, designing and implementing seismic retrofits.

3. Case Study Building

The 4-story URM case study building investigated in [7] is located in canton Aargau in Switzerland (Seismic zone I, soil class E) and was built in the 1930s (Fig. 2, Fig. 3). Two identical mirror-image buildings share a fire protection wall along the north-south axis. Only one half of the building with a footprint of 26 m by 11 m is hence assessed. Following an investigation in 2004, two basement walls were strengthened, and dovetailed steel sheets were added on existing timber joists.

Fig. 2 – Southern façade of the case study building

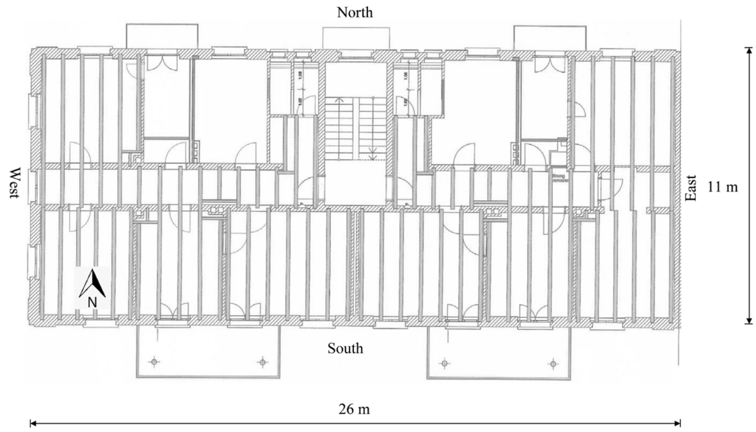

Fig. 3 – Plan view of a typical floor of the case study building, showing the floor timber beams

Both masonry façade and inner walls are consisted of clay bricks of 400 mm and 250 mm, respectively, whose material characteristics are given in Table 1. Building floors are timber beams topped by nailed wood flooring and are considered to be flexible. Data gathering and updates concerning the current state of the building have been conducted based on the existing plans and investigation reports from the last 15 years, as well as from the authors' experience.

Table 1 – Material parameters

	Clay brick masonry		
Specific weight	γ_d	13.0	kN/m^3
Compression strength	f_{xk}	3.0	N/mm^2
Young's modulus	E_{xk}	2100	N/mm^2
Young's modulus (cracked)	$E_{xk,eff}$	630	N/mm^2
Shear modulus	G_{xk}	840	N/mm^2
Shear modulus (cracked)	$G_{xk,eff}$	252	N/mm^2
Shear strength	τ_0	0.06	N/mm^2

3.1 Local analysis

The expected global behavior of a building can only be achieved if the integrity of the building is ensured. Thus, connections between the members play a crucial role in the building response to seismic excitation, both in terms of ensuring the load transfer assumed in the global building model and in terms of preventing localized partial collapses. For URM buildings with flexible diaphragms, it is known that local failure modes often govern the structural response. A discussion of the structural integrity of the ceiling support falls outside the scope of this paper. The interested reader is referred to the published master thesis [7].

3.1.1 In-plane stiffness of the floor diaphragms

Transfer of the seismically induced forces from the floors to the walls, and hence the building response, is strongly dependent on the in-plane stiffness of the floor diaphragms [8]. The built-in floor consists of 150 mm wide and 200 mm high timber beams placed at 700 mm on-center, with floorboards on top nailed perpendicular to these beams. As it was not possible to measure the spacing of these nails in-situ, a rather conservative value of 100 mm nail spacing is assumed for further analysis. Technical reports indicate that retrofit measures to strengthen the floor unit were adopted in the form of dovetailed steel sheeting of the diaphragm joints. Structural detailing and the exact extent of this measure is, however, not documented. Shear resistance of the floor has hence been computed without the consideration of the dovetailed steel sheeting. The story weight of the area between the southern façade walls and the corridor walls is 2.6 kN/m^2 · 26 m · 4.5 m = 304 kN. In the plateau domain of the elastic response spectrum, the equivalent lateral story force amounts to S_e(T = 0.33 s) = 0.214 g · 304 kN = 65 kN. This force will be shared between the southern façade walls and the corridor walls so that each wall assembly is subjected to 32.5 kN lateral force on each story level. Finally, one can obtain the equivalent lateral force per running meter with E_d = 32.5 kN / 26 m = 1.25 kN/m and compare it to the shear resistance R_d of the wooden floor according to [9], giving a diaphragm shear force transfer compliance factor:

$$\alpha_{eff} = \frac{R_d}{E_d} = \frac{4.1 \text{ kN/m}}{1.25 \text{ kN/m}} = 3.28 \tag{3}$$

3.1.2 Load transfer from the floor to the walls

Timber beams of the floor are oriented in the building's transverse direction and placed loosely on top of the masonry walls that are perpendicular to the direction of the beams. Anchorage elements to allow a distribution of the seismically induced internal forces or to restrain the out-of-plane loaded walls could not be detected.

Depending on the direction of the seismic excitation, induced compression or tension forces will be transferred from the timber beams to the transversely loaded walls. The only resistance against the pulling-out of the timber beams from their wall supports between the timber beams and the masonry walls is the friction resistance R_d:

$$R_d = N_d \cdot \mu = g_d \cdot \frac{L}{2} \cdot \mu = 2.6 \frac{\text{kN}}{\text{m}^2} \cdot \frac{4.5 \text{ m}}{2} \cdot 0.6 = 3.5 \frac{\text{kN}}{\text{m}} \tag{4}$$

The ratio of the friction resistance to the equivalent lateral force E_d (per running meter) yields the compliance factor of the load transfer from the floor diaphragm to the walls:

$$\alpha_{eff} = \frac{R_d}{E_d} = \frac{3.5 \text{ kN/m}}{1.25 \text{ kN/m}} = 2.80 \tag{5}$$

3.1.3 Out-of-plane resistance of the walls

The assessment of the out-of-plane behavior of URM walls constitutes one of the most challenging tasks in the seismic assessment. To date, various methods have been developed for the performance evaluation of the out-of-plane loaded masonry walls, but their implementation is included in only a few codes worldwide [10]. For the case study building, the out-of-plane assessment was carried out for the west and the north façade wall (Fig. 4) using the Italian codified force-based approach [2] and the Paulay & Priestley method. Although the latter method is known for its sensitivity to the material characteristics estimates, it is still the most common method among the Swiss engineering community and, hence, part of this case study investigation. The major challenge of the out-of-plane assessment is the identification of the critical collapse mechanism. Six different mechanisms, enumerated in Fig. 4, were investigated in this study.

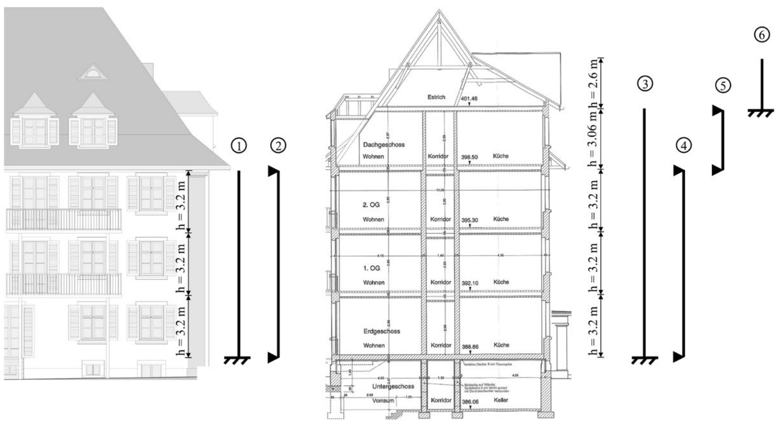

Fig. 4 – Investigated out-of-plane mechanisms; west façade (left) and the north façade (right)

3.1.3.1 Paulay & Priestley method

Paulay & Priestley method [1] was used to obtain the displacement capacity of transversely loaded walls and their nonlinear behavior. In the following, the north façade wall with a thickness t_w is modeled as a simply supported system under the action of its self-weight W_d, vertical loads P_i that represent the loads transferred from the floors to the wall as well as the additional vertical load P_d due to the self-weight of the roof and the wall on top of the third floor (Fig. 5).

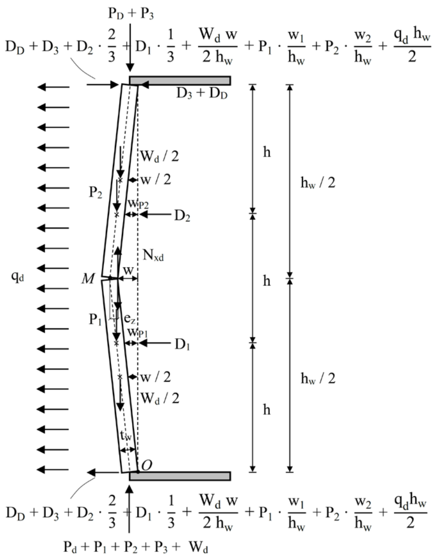

Fig. 5 – Simply supported idealization of the north façade wall (mechanism 4)

As the live load acting on the diaphragm and the self-weight of the wooden floor are assumed not to change at story levels, vertical loads P_i that are transferred from the timber beams of the floor to the north façade wall are constant and can be computed using the floor seismic load g_d (self-weight and 30% of live load) from Eq. (4): $P = g_d \cdot L / 2 = 2.6 \text{ kN/m}^2 \cdot 4.5 \text{ m} / 2 = 5.85 \text{ kN}$. The additional load on top of the wall in the third floor is defined as P_D separately to account the self-weight of the roof structure and the self-weight of the wall in the roof: $P_D = 4 \text{ kN} + (13 \text{ kN/m}^3 \cdot 1.0 \text{ m} \cdot 3.06 \text{ m} \cdot 0.30 \text{ m}) = 15.93 \text{ kN}$. Horizontal inertia forces generated by the timber beam depend on the floor acceleration and can be computed as follows [8]:

$$D_i = \frac{a}{g} \cdot 2P = \frac{2 \cdot q_d \cdot P_i \cdot h_w}{W_d} \tag{6}$$

Equilibrium of the vertical forces in the upper half of the wall gives the axial force N_{xd} in the wall:

$$N_{xd} = \frac{W_d}{2} + 2P + P_D = \frac{(13 \text{ kN/m}^3 \cdot 1 \text{ m} \cdot 9.6 \text{ m} \cdot 0.4 \text{ m})}{2} + 2 \cdot 5.85 \text{ kN} + 15.93 \text{ kN} = 52.6 \text{ kN} \tag{7}$$

A moment equilibrium around the base corner point *O* to compute the horizontal component at the supports is followed by a moment equilibrium at the mid-height *M*, leading to the response acceleration a of the wall:

$$a = \frac{q_d}{m} = \frac{N_{xd} \cdot e_z - w \cdot \left(\frac{W_d}{2} + 2 \cdot P + P_D\right)}{mh_w^2 \cdot \left(\frac{1}{8} + \frac{2 \cdot P}{3 \cdot W_d}\right)} \tag{8}$$

Paulay & Priestley method defines the load-deflection relationships for the transversely loaded unreinforced masonry wall for four different states to assess the energy requirements at the failure (Fig. 6).

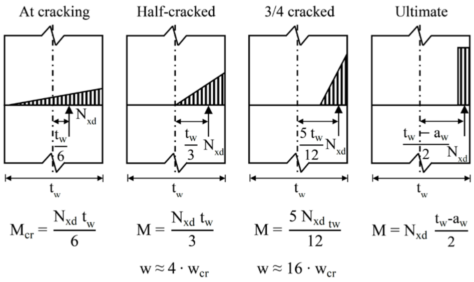

Fig. 6 – Moments and displacements at the center of the transversely loaded wall [1]

For a simply supported system at cracking, the distributed lateral load $q_{d,cr}$ to cause the bending moment at the transversely loaded wall, the central lateral displacement w_{cr} and the response acceleration a_{cr} are given as follows:

$$q_{d,\,cr} = \frac{8 \cdot M_{cr}}{h_w^2} = \frac{8 \cdot N_{xd} \cdot t_w}{6 \cdot h_w^2} = \frac{8 \cdot 52.59 \text{ kNm} \cdot 0.4 \text{ m}}{6 \cdot (9.6 \text{ m})^2} = 0.304 \text{ kN/m} \quad (9)$$

$$w_{cr} = \frac{5 \cdot q_{d,\,cr} \cdot h_w^4}{384 \cdot E \cdot I} = \frac{5 \cdot 0.305 \text{ kN/m} \cdot (9.6 \text{ m})^4 \cdot 10^3}{384 \cdot 630\,000 \text{ N/m}^2 \cdot \left(1 \text{ m} \cdot \frac{(0.4 \text{ m})^3}{12}\right)} = 10.0 \text{ mm} \quad (10)$$

$$a_{cr} = \frac{52.59 \text{ kN} \cdot \frac{400 \text{ mm}}{6} - 10 \text{ mm} \cdot \left(\frac{49.92 \text{ kN}}{2} + 2 \cdot 5.85 \text{ kN} + 15.93 \text{ kN}\right)}{530 \frac{\text{kg}}{\text{m}^2} \cdot (9.6 \text{ m})^2 \cdot \left(\frac{1}{8} + \frac{2 \cdot 5.85 \text{ kN}}{3 \cdot 49.92 \text{ kN}}\right)} = 0.300 \ \frac{\text{m}}{\text{s}^2} \quad (11)$$

Table 2 summarizes the load-deflection relationships in different stress conditions as the crack propagates along the wall section:

Table 2 – Load-deflection relationships of the mechanism 4 from Fig. 6)

State	**M [kNm]**	**q [kN/m]**	**w [mm]**	**a [m/s²]**
At cracking	3.51	0.304	10.0	0.300
Half-cracked	7.01	0.609	40.0	0.494
¾ cracked	8.77	0.761	160.0	0.034
Ultimate	9.43	0.819	179.4	0.000

An equivalent linear elastic response acceleration a_e is obtained by equating the area A_2 with the area A_1 under the nonlinear acceleration-displacement curve of the investigated mechanism (Fig. 7).

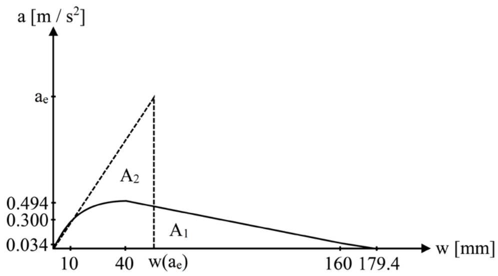

Fig. 7 – Equal energy principle for equivalent elastic stiffness

Equivalent linear elastic response acceleration a_e is then reduced by a safety factor γ, as suggested in [8]:

$$a_{ed} = \frac{a_e}{\gamma} = \frac{\sqrt{2 \cdot (a_{cr} / w_{cr}) \cdot A_1}}{\gamma} = \frac{\sqrt{2 \cdot (0.300 / 10) \cdot 45.51}}{2.0} = 0.826 \text{ m/s}^2 \quad (12)$$

Amplification of the floor accelerations during the response of the floor is not expected, as the natural period of the flexible floor is assumed to be significantly longer than that of the transversely loaded wall [1], [8]. Assuming a linear first mode shape, the response accelerations at ground level, at the effective center of seismic response h_E, and the highest level of the mechanism are computed in Fig. 8.

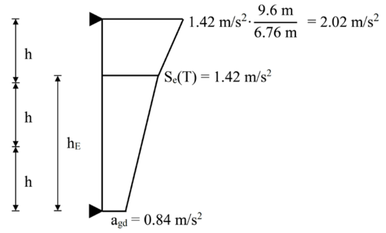

Fig. 8 – Variation of response acceleration with the height of the mechanism

The response of the transversely loaded wall is assumed to be constant over the height of the wall and corresponds to the average of the input accelerations at ground level and at the highest level:

$$a = \frac{a_{top} + a_{bottom}}{2} = \frac{0.84\ \text{m/s}^2 + 2.02\ \text{m/s}^2}{2} = 1.43\ \text{m/s}^2 \tag{13}$$

The ratio of the equivalent linear elastic response spectrum to the elastic design acceleration from SIA 261 [6] yields the compliance factor for the mechanism:

$$a = \frac{a_{ed}}{a} = \frac{0.826\ \text{m/s}^2}{1.43\ \text{m/s}^2} = 0.58 \tag{14}$$

3.1.3.2 Italian codified procedure

Italian codified procedure [2] offers force- and displacement-based assessment methods for masonry walls subjected to out-of-plane excitation by means of rigid-body kinematics, assuming that the disaggregation of the wall is prevented and the monolithic nature of the wall is guaranteed, and the partial collapse initiates due to the loss of the equilibrium between the wall sections. In this paper, only the force-based procedure is presented. In the following, the out-of-plane assessment of the free-standing north façade wall (Fig. 9) is conducted, while the remaining series of collapse mechanisms were evaluated in [7].

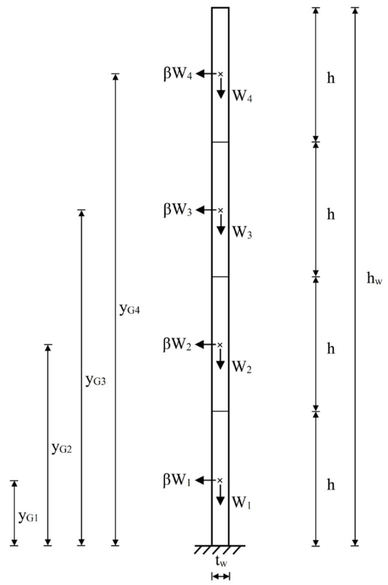

Fig. 9 – Free-standing cantilever idealization of the north façade wall (mechanism 3)

Moment equilibrium about the base corner parapet yields:

$$\beta = \frac{\sum_{i=1}^{n} W_i \cdot \frac{t_w}{2}}{\sum_{i=1}^{n} W_i \cdot y_{Gi}} = \frac{4 \cdot 134.9 \frac{kN}{m} \cdot \frac{0.4\ m}{2}}{134.9 \frac{kN}{m} \cdot (1.55\ m + 4.75\ m + 7.95\ m + 11.15\ m)} = 0.032 \quad (15)$$

Where W_i is the self-weight, t_w thickness, y_{Gi} distance from the ground level to the centroid of the parapet wall macroblocks, and β the load multiplier for which a collapse mechanism is triggered. In the next step, the acceleration a_0^* at the initial kinematic mechanism (acceleration capacity) is calculated:

$$a_0^* = \frac{\beta \cdot g}{e^*} = \frac{0.032 \cdot 9.81\ m/s^2}{1} = 0.314\ m/s^2 \quad (16)$$

g is the gravity acceleration and e^* is the mass participation factor of the first mode referring to the parapet wall involved in the mechanism. At a specific height, the amplification of the ground motion needs to be considered. For the assumed mechanism, however, spectral acceleration corresponds to the peak ground acceleration of the elastic demand spectrum at ground level, multiplied by the soil coefficient S:

$$S_e(T=0) = a_{gd} \cdot S = 0.6\ m/s^2 \cdot 1.4 = 0.84\ m/s^2 \quad (17)$$

Spectral acceleration S_e is reduced by the behavior factor q; that is q = 1 for damage limit state and q = 2 for life-safety limit state [2]:

$$a_d^* = \frac{S_e(T=0)}{q} = \frac{0.84\ m/s^2}{2} = 0.42\ m/s^2 \quad (18)$$

The compliance factor is characterized by the ratio of the triggering acceleration a_0^* (capacity) to the spectral acceleration a_d^* (demand):

$$\alpha_{eff} = \frac{a_0^*}{a_d^*} = \frac{0.314\ m/s^2}{0.42\ m/s^2} = 0.75 \quad (19)$$

3.2 Global analysis

Equivalent frame modeling represents a viable method for the performance-based global analysis of URM buildings with a reasonable computational effort [11]. A 3D equivalent frame model was generated in 3Muri to obtain the global behavior of the building through a nonlinear static pushover analysis (Fig. 10).

Fig. 10 – Case study building (left) and its three-dimensional global model (right)

As SIA 269/8 [3] does not define damage limit states for the evaluation of building performance, the capacity curve of the building is truncated to the displacement that leads the first vertical element to fail, corresponding to the maximum admissible displacement d_u before a partial collapse mechanism is triggered. As for the demand displacement d according to the SIA 261, N2 method [12] was employed to combine the capacity curve of the investigated building with the elastic design spectrum. Displacement capacity of the east façade wall dictates the global compliance factor:

$$\alpha_{eff} = \frac{d_u}{d} = \frac{17.5\ \text{mm}}{20.8\ \text{mm}} = 0.84 \tag{20}$$

3.3 Compliance factor of the case-study investigation

The compliance factors of the investigated mechanisms are given in Table 3. The controlling global compliance factor is 0.84. The controlling local compliance factor was selected from those computed using the Italian codified procedure. The P&P method was found to produce widely varying compliance factors that are very sensitive to the material characteristics in Table 1: thus, the computed compliance factors using the P&P method were disregarded. Nevertheless, both methods identified that Mechanism 3 for the North wall are critical.

Table 3 – Overview of the compliance factors

Local compliance factors		
In-plane stiffness of the floor	3.28	
Load transfer from the floor to the walls	2.80	
Out-of-plane mechanisms	Paulay & Priestley	Italian procedure (Force-based)
Mechanism 1 (Cantilever, west)	0.27	0.97
Mechanism 2 (Simply supported, west)	0.85	1.91
Mechanism 3 (Cantilever, north)	0.05	**0.75**
Mechanism 4 (Simply supported, north)	0.58	3.76
Mechanism 5 (Simply supported, attic, north)	6.53	13.39
Mechanism 6 (Cantilever, gable wall)	2.47	-
Global compliance factors		
Longitudinal direction	1.32	
Transverse direction	**0.84**	

3.4 Commensurability of possible upgrade measures

To facilitate an easier understanding, monetary units have been harmonized by taking US dollars as the main reference with an exchange rate of 1 Swiss franc (CHF) to 1 US dollar (USD).

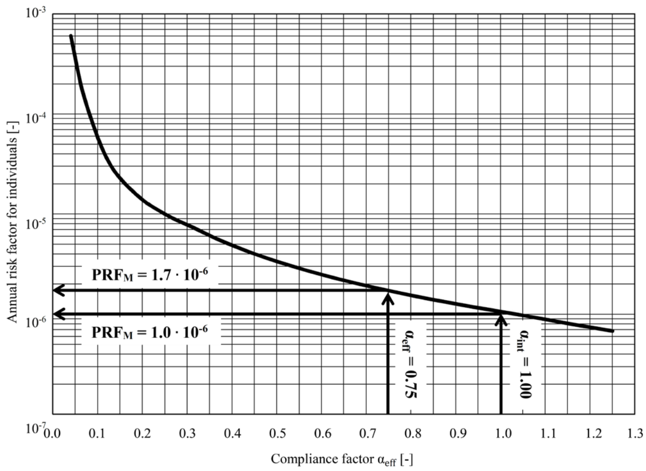

Fig. 11 – Reduction in annual risk factor for individuals due to the retrofit [3]

Achieved annual risk reduction ΔPRF_M due to the retrofit is the difference between the risk factors corresponding to the compliance factors before and after the seismic retrofit (Fig. 11):

$$\Delta PRF_M = PRF_M(\alpha_{eff}) - PRF_M(\alpha_{int}) = 1.7 \cdot 10^{-6} - 1.0 \cdot 10^{-6} = 0.7 \cdot 10^{-6} \quad (21)$$

By considering the statistical value of the occupancy rate PB, the actuarial value of the potentially saved lives GK, and the annual risk reduction ΔPRF_M, reduction of the annual risk for individuals ΔRP_M can be computed:

$$PB = \frac{1}{8736 \text{ hours/year}} \cdot 68 \text{ rooms} \cdot 0.4 \frac{\text{person}}{\text{room}} \cdot 24\text{hours} \cdot 7 \text{ days} \cdot 52 \text{ weeks} = 27.2 \text{ person} \quad (22)$$

$$\Delta RP_M = \Delta PRF_M \cdot PB \cdot GK = 0.7 \cdot 10^{-6} \cdot 27.2 \text{ person} \cdot 10'000'000 \text{ USD} = 190 \text{ USD/year} \quad (23)$$

An annual discount interest rate of 2% is used to quantify the long-term benefits of the mitigation with respect to the remaining service life d_r of the building (assumed to be 80 years, based on typical cases from practice in Switzerland) to compute the annual discount rate DF:

$$DF = \frac{i_d \cdot (1 + i_d)^{d_r}}{(1 + i_d)^{d_r} - 1} = \frac{0.02 \cdot (1 + 0.02)^{80}}{(1 + 0.02)^{80} - 1} = 0.02516 \quad (24)$$

As the extent of possible interventions and their cost were not known by the time of the assessment, commensurate cost limit for the seismic interventions that satisfy the efficiency criteria $EF_M = 1.0$ is obtained:

$$\frac{\Delta RP_M}{DF} = \frac{190 \text{ USD / year}}{0.02516} = 7'500 \text{ USD} \quad (25)$$

This means seismic retrofit is mandatory only if the cost for the intervention does not exceed 7’500 USD, even though the critical compliance factor is less than 1. Considering the level of the achieved risk reduction, intervention costs higher than this threshold value cannot be reasonably justified. Hence, a retrofit can be waived in accordance with the commensurability evaluation method prescribed in SIA 269/8 .

4. Conclusion

This paper has given an account of the seismic assessment of an existing URM building with flexible floor diaphragms followed by a cost-benefit consideration whether a retrofit is necessary to reduce the earthquake-related risk or if the current state of the building can be accepted as the corresponding risk is too low according to the surrent Swiss SIA 269/8 code for seismic retrofit of structures.

The main goal of the current study was to determine whether the local behavior will dictate the final decision regarding the seismic retrofit. The evidence from this study indicates so. Namely, an implicit assumption of structural integrity might lead to an overestimation of a building’s displacement capacity and could result in a misguided retrofits that address deficiencies in the global behavior of a building. Local behavior, specifically, out-of-plane failure of URM walls, usually represents the weakest link and dictates the degree of code compliance for URM buildings with flexible floor diaphragms. Therefore, local behavior has to be examined carefully before planning structural intervention measures. Assessment of the out-of-plane failure showed that the outcome of the Paulay & Priestley approach is very sensitive to the assumed Young’s modulus, representing the main drawback of this method as the material parameters in existing buildings are subject to a significant level of uncertainty. Common in both the Paulay & Priestley method and the Italian codified procedure, the major challenge lies in the definition of the expected failure mechanism, as its identification is left to the analyst.

The cost of a seismic upgrade may be unreasonably high for the examined existing building. Comensurabilty considerations prescribed in SIA 269/8 offer one rational basis for making seismic upgrade decisions and avoiding costly retrofits without considerable, i.e. commensurable, risk reductions.

5. References

[1] Paulay T, Priestley M J N (1992): Seismic Design of Reinforced Concrete and Masonry Buildings. *John Wiley & Sons, Inc.*

[2] CNTC2009 (2009): Circolare 02.02.2009, n. 617 Istruzioni per l'applicazione delle "Nuove norme tecniche per le costruzioni" di cui al decreto ministeriale 14.01.2008. Gazzetta Ufficiale n. 47 del 26.02.2009 – Supplemento Ordinario n. 27, C8A.4.2. *Building code.*

[3] Schweizerischer Ingenieur- und Architektenverein (Swiss Society of Engineers and Architects) (2017): Erhaltung von Tragwerken – Erdbeben (Maintenance of structures – earthquakes). *Building code SIA 269/8*, Zurich, Switzerland

[4] Wenk T (2005): SIA 2018 – Überprüfung bestehender Gebäude bezüglich Erdbeben. *D-A-CH Tagung*, Köln, Germany

[5] Vogel T, Kölz E. (2005): Evaluation of existing buildings with respect to earthquake risks. *Ninth International Conference on Structural Safety and Reliability (ICOSSAR '05)*

[6] Schweizerischer Ingenieur- und Architektenverein (Swiss Society of Engineers and Architects) (2014): Einwirkungen auf Tragwerke (Actions on structures). *Building code SIA 261*, Zurich, Switzerland

[7] Volken N (2019): Evaluation of an Existing Masonry Building with Flexible Diaphragms in Switzerland. *Master thesis*, Zurich, Switzerland, https://ethz.ch/content/dam/ethz/special-interest/baug/ibk/structural-masonry-dam/education/MS_Thesis_Volken.pdf

[8] Schweizerischer Ingenieur- und Architektenverein (Swiss Society of Engineers and Architects) (2010): Beurteilung von Mauerwerksgebäuden bezüglich Erdbeben (Seismic assessment of masonry buildings). *Technical documentation SIA D 0237*, Zurich, Switzerland

[9] International Conference of Building Officials (1997): Uniform Building Code, Volume 2, Chapter 16, Table 23-II-I-1. *Building Code,* USA

[10] Sorrentino L, D'Ayala D, de Felice G, Griffith M C, Lagomarsino S, Magenes G (2016): Review of Out-of-Plane Seismic Assessment Techniques Applied to Existing Masonry Buildings. *International Journal of Architectural Heritage*

[11] Lagomarsino S, Penna S, Galasco A, Cattari S (2013): TREMURI Program: An equivalent frame model for the nonlinear seismic analysis of masonry buildings. *Engineering Structures*.

[12] Fajfar P (2000): A Nonlinear Analysis Method for Performance Based Seismic Design. *Earthquake Spectra*.

