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Reliability of traditional timber-floor masonry buildings to seismic action

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Research Paper

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Reliability of traditional timber-floor masonry buildings to seismic action

A considerable number of timber floor masonry structures, typical for construction practices of the late 19th century, were damaged in the earthquake that struck Zagreb on 22 March 2020. Due to the lack of reliable earthquake analyses of this type of structures, it is difficult to estimate the level of risk such buildings are exposed to in the case of design earthquake events. The results of seismic analysis of one such structure are presented in the paper. The analysis is based on displacements and is known as PBD (Performance Based Design). The real necessity, efficiency, and methods for improving seismic response of this type of structures, are discussed.

Key words:
earthquake, masonry buildings, timber floors, PBD method

Prethodno priopćenje

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Pouzdanost tradicionalnih zidanih građevina s drvenim stropovima na potresno djelovanje

Nakon potresa koji je pogodio Zagreb 22. ožujka 2020. oštećene su brojne građevine od ziđa s drvenim stropovima, tipične za gradnju potkraj 19. stoljeća. Zbog nedostatka pouzdanih analiza takvih građevina na potresna djelovanja, teško je procijeniti razinu rizika kojima su one izložene za slučaj računske potresne situacije. U ovome radu su predstavljeni rezultati potresne analize ponašanja jedne takve građevine. Analiza je temeljena na pomacima, a poznata je kao Performance Based Design (PBD metoda). Razmatrana je stvarna potreba, učinkovitost i načini poboljšanja odziva ovoga tipa građevine na potresna djelovanja.

Ključne riječi:

Vorherige Mitteilung

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Zuverlässigkeit traditioneller Mauerwerksgebäude mit Holzdecken bei Erdbebeneinwirkung

Bei dem starken Erdbeben, das am 22. März 2020 Zagreb erschüttert hat, wurden zahlreiche Gebäude, typisch für die Bauart des späten 19. Jahrhunderts, beschädigt. Da aber verlässliche Erdbebenanalysen solcher Tragwerke fehlen, ist es schwierig das Risiko abzuschätzen, dem solche Bauten im Falle eines Bemessungsbebens ausgesetzt worden wären. In dieser Arbeit wurden Ergebnisse einer, auf Verformungen basierten Erdbebenanalyse vorgestellt und diskutiert. Die möglichen Ertüchtigungsmassnahmen, deren Notwendigkeit sowie Art deren Ausführung wurden aufgezeigt.

Schlüsselwörter:
Erdbeben, Mauerwerksgebäude, Holzdecken, PBD-Methode

1. Introduction

Earthquakes are exceptional events that take place in statistically accurately defined time intervals known as return periods. Basic tool engineers use in seismic analysis of structures is design response spectra, defined for the return period of 475 years, with the 10 percent probability that the design earthquake will be exceeded in fifty years.

A considerable number of structures belonging to the same architectural type were damaged in the earthquake that hit Zagreb on 22 March 2020. These buildings are four-storey structures with solid brick walls and wooden floor frames. The aim of this paper is to determine the reliability of this building archetype with regards to designed seismic actions, based on two mutually independent analyses: deterministic non-linear static analysis, and probabilistic incremental dynamic analysis, [1]. The resistance analysis to seismic action will be conducted for direction that coincides with the load bearing walls planes (in plane analysis), while the wall stability will be checked for the direction perpendicular to their planes (out of plane analysis), regardless of whether or not they participate in the transfer of vertical gravity loads. Nonlinear structural responses obtained by pushover analysis for both orthogonal directions will be used as input data for the incremental dynamic analysis (IDA) in the scope of which probabilistic methodology will be used to generate, based on the sum of all individual IDA curves, the 50% fractile for various levels of seismic intensity, as expressed through maximum acceleration of ground as a function of damage expressed through relative level's drift values. A sensitivity curve with the corresponding probabilities of exceedance will be derived below for typical limit states: unconditional serviceability, immediate occupation (IO), life safety limit state (LS), , collapse prevention (CP) and collapse, for ground accelerations generated by a particular level drift. Values defined in this way will be used to discuss the behaviour of structures, their safety, and possibilities for improving their response to design seismic action.

2. Calculation method based on displacements

U uvodu je spomenuto da će u ovome radu biti primijenjena As mentioned in the introduction, a nonlinear static pushover analysis will be performed as part of the performance based design. In the opinion of the authors of this paper, this is the most appropriate analysis, if not the only acceptable analysis, as it is highly applicable to the existing masonry structures. As the method based on forces, the so called response spectrum method, is widely applied in practice, it seems appropriate to look into this aspect more closely already at this initial stage of the analysis. By their nature, earthquake actions are waves characterised by content of frequency, displacements, velocities, and ground accelerations. In contact with a structure, seismic waves pass through foundations and apply seismic energy onto the structure, affecting and deforming it in the process. The consequences of deformations are shear forces that are generated in all elements of the structure. The value of an earthquake-caused forces and displacements - excited to oscilate in the structure - is a dynamic properties function while modes, and periods of oscillation, are function of the mass and stiffness of the structure. It is clear from the very nature of things that forces exceeding the limit resistance to shear and bending cannot occur in the structure. When shear force achieves the plastic value of resistance, this value can no longer increase while the displacement or deformation can increase. The earthquake-induced energy continues to be dissipated throughout the time in which the deformation of plasticised cross-section increases. Deformation or displacement is more appropriate than force as a design criteria to estimate seismic impact on structures. That is why ductility is defined through displacement: $\mu = \Delta_u/\Delta_y$. For each location, seismologists create response spectra and generate, for ground acceleration deduced from seismic hazard analysis, elastic response spectra for systems with one degree of freedom. The elastic response spectrum can be read as a requirement to be fulfilled by the structure so that it can present linear elastic behaviour in case of an earthquake event. This requirement is measured through spectral acceleration (force) and spectral displacement. Their relationship is simple: $S_e = \omega^2 \cdot S_d$, and so S_e can easily be determined from acceleration spectrum S_d. Displacement that "earthquake requires" from the structure is a function of the basic oscillation period. As such, it is defined from the very beginning. The structure must fulfil this requirement by "working" in either elastic or plastic response. Shear force, as a deduction from mass and acceleration, does not need to fulfil this requirement. As we have already seen, plastic resistance of a structure can be much lower than the required elastic bearing capacity. That is why, in case of seismic action, deformation capacity of a structure has to be compared related to seismic demand, rather than force. If is the (earthquake)required displacement lower than the deformation capacity of the structure, then seismic safety of the structure can be considered proven. This proof is not implemented in the methodology based on forces.

All these deficiencies and conceptual drawbacks of the force based method, which are not mentioned in this paper, are properly summarized in [2]. The readers who are not able to consult the book *Direct displacement based seismic design* are advised to watch the following videohttps://www.youtube.com/watch?v = MZUhSHmIUdI. In the first part, professor J.N. Priestley, one of greatest authorities for the field of earthquake engineering, discusses deficiencies and controversies related to FBD.

In seismic analysis of the existing buildings force based approach can figuratively be described in the following way: we do not know the real capacity of the building, but it would be desirable for it to behave according to the behaviour factor q. For instance, let us say q is equal two and, regardless of whether is an improvement it is necessary or not, structural strengthening, is recommended so that the building behaviour would really correspond to behaviour factor 2. In case this method is uncritically applied to the existing structures in general, it would result in disproportionately great – mostly unnecessary – investments and remedial works. That is why the use of this paradigm in the verification of existing structures would be inappropriate and should be critically examined.

The method based on forces would provide the result similar to the one based on displacements, if both of them would have the same starting point, which is the real response of the existing structure. This response must be determined analytically in the best possible and most realistic manner. This point must not be bypassed. In this case, it would be possible, even for FBD, to determine real and not idealised ductility and behaviour factors, and the results would be quite acceptable, regardless of the type of analysis used.

3. Seismic influence

The elastic response spectrum for the city of Zagreb, with ground acceleration of 2.55 m/s^2 and for soil type C, is shown in Figure 1 in the standardly applicable A-T (acceleration-period) format.

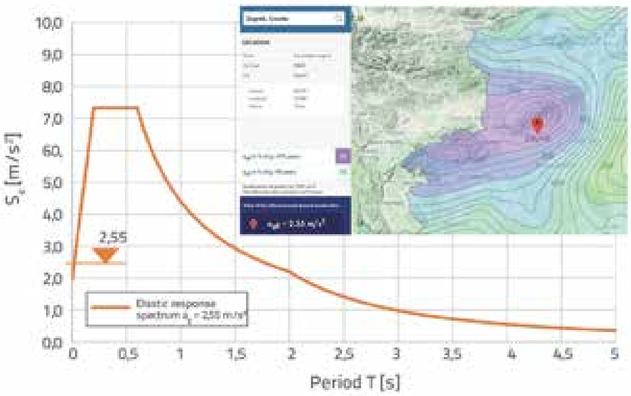


Figure 1. Elastic response spectrum for the city of Zagreb for ground acceleration of a_g = 2.55 m/s^2 and for the foundation soil type C in A-T (acceleration - period) format

Design ground accelerations can be obtained for every location in the Republic of Croatia at the official earthquake map of the Republic of Croatia, which is available at http://seizkarta.gfz.hr/karta.php.

According to available geotechnical reports, northern and southern parts of the city of Zagreb lie on foundation soil B and C. For that reason, the design earthquake layout will comprise both types of soil with the corresponding properties. A more appropriate way of presenting the response spectrum is given in Figure 2. This is the so called Yield Point Spectra [5] in ADRS (acceleration displacement response spectra) format which shows, in a single graph, the earthquake action (spectral displacement and spectral acceleration) and the load bearing capacity of the structure. Extended with equal ductility factor curves, this graph provides a clear preview of bearing capacity and deformation capability of the structure as related to demand elastic resistance and required displacement, represented with the intersection point between the elastic period and elastic response spectrum. The required nonlinear response (ductility demand) is read directly from the graph and is the same for both standardised forces and displacements. N2 method preview, also adopted in EC8, is given in Figure 2. This representation of the nonlinear response spectrum, for ductility factor μ, is the same as in YPS. Both nonlinear spectra (YPS and N2) provide an identical result, represented with the point of plastic contact (performance point) that lies directly on the nonlinear response spectrum curve defined in accordance with the N2 method. Reduction of the elastic response spectrum in the zone of constant accelerations, with periods $T_B < T < T_C$, where rule of similar displacement does not apply and is replaced with the equal energy rule, is defined according to the relation R – μ – T expression $R_u = (\mu - 1)\cdot(T/T_C) + 1$, proposed in the N2 methodology for definition of ultimate load bearing capacity, cf. [6]. Required displacement in the zone of constant accelerations is defined by expression: $S_d = \mu \cdot \Delta_y = (S_{de}/R_\mu) \cdot (1 + (R_\mu - 1) \cdot (T_C/T))$. Here, the coefficient of reduction of the elastic seismic force, R, also used in the original document, as adopted from American literature and used in European nomenclature, assumes the meaning of the behaviour factor q. The reduction factor for zones of constant velocities, with periods $T_C < T < T_D$, is equal to $R_\mu = \mu = q$.

The response spectrum in the zone of constant acceleration (acceleration sensitive region) for soil types B and C is limited by a relatively great range of periods varying from 0.2 to 0.5 or 0.6 [s]. Because of small mass of floors and great stiffness of masonry structures, it can be expected that basic-mode periods of masonry buildings will be in the zone with design accelerations on the elastic response spectrum level with the values:

$S_e = 2{,}5 \cdot a_g \cdot S\,\eta = 2{,}5 \cdot 2{,}55 \cdot 1{,}2 \cdot 1 = 7{,}65$ [m/s^2] for soil type B,

$S_e = 2{,}5 \cdot a_g \cdot S\,\eta = 2{,}5 \cdot 2{,}55 \cdot 1{,}15 = 7{,}33$ [m/s^2] for soil type C,

as graphically presented in Figure 2.

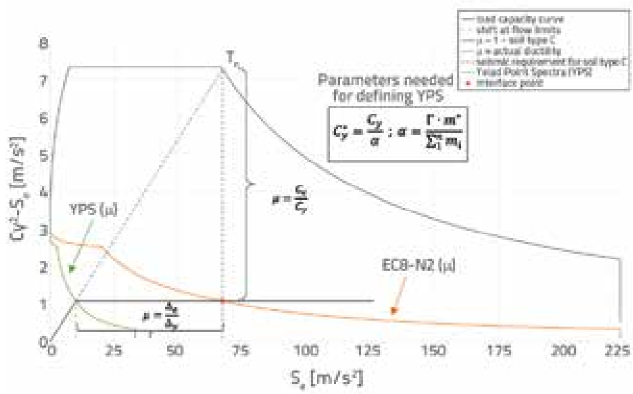


Figure 2. YPS and N2 response spectra for the city of Zagreb with ground accelerations of a_g = 2.55 m/s^2 and foundation soil type C in ADRS format for identical ductility factor

For illustration purposes, the graphical representation includes bilinear approximation of the design capacity of the structure (capacity curve).

4. Distribution of seismic forces along the height of the structure

4.1. Distribution of lateral forces according to first mode

Most of European countries require verification of design resistance to earthquake action in such a way that lateral load caused by earthquake is distributed affinely with the first mode of vibrations or proportionally to floor masses. Total earthquake force is defined by the following expression (1):

$$F_b = S_{d(T1)} \cdot \sum_j (G_k + \sum \psi_2 \cdot Q_k)_j \tag{1}$$

while redistribution of the total earthquake force F_b affinely with the first mode of vibrations is determined by the following expression (2):

$$F_i = \frac{z_i \cdot (G_k + \sum \psi_2 \cdot Q_k)_i}{\sum_j z_j \cdot (G_k + \sum \psi_2 \cdot Q_k)_j} \cdot F_b \tag{2}$$

where $z_{i,j}$ is the ceiling height measured from the plane of fixity.

4.2. Redistribution of lateral forces proportional to floor masses

The distance of the ceiling from the plane of fixity (z = 0) does not exert any influence on the redistribution of earthquake force proportional to floor masses. The redistribution of the total earthquake force F_b is determined by the following expression:

$$F_i = \frac{(G_k + \sum \psi_2 \cdot Q_k)_i}{\sum_j (G_k + \sum \psi_2 \cdot Q_k)_j} \cdot F_b \tag{3}$$

It should be noted that the way in which the transverse earthquake force is redistributed along the structure's height is of central significance for the procedures based on forces, i.e. to the so called force based design (FBD), while in the pushover method operated in the scope of the performance based design (PBD), this scheme is used during incremental application of forces to the system, with simultaneous determination of the corresponding deformations.

In the design of structures to seismic action, relevant results are the ones that result in greater forces in the case of FBD, i.e. in smaller deformational capacities of the structure in the case of the PBD.

5. Typical building

5.1. Geometry of the building

Figure 3 shows a copy of the original plan view and cross-section of the building that will be analysed for seismic action in order to estimate reliability of buildings belonging to this structural type. It is a four-storey building with several residential units. The building is approximately 17 m in length and 10 m in width at the front side, and 17 m in width at the back side of the building. The height of the building from the floor slab to cornice is approximately 14 m, while an average floor height is 3.5 m. Despite the fact that the basement storey is partially buried, the plane at which the structure is fixed to the ground is at the level of foundations (cf. cross-section). Walls are massive and are made of solid bricks. Timber joists span the distance of approximately 5.5 m between the main walls, and the load carrying direction is symbolically presented in Figure 3. The main function of walls whose direction of spreading coincides with the direction of load bearing of timber joints is to close the space, without any significant function in the static sense.

Despite the fact that main walls at the basement level and ground floor level are 75 cm and 60 cm thick, respectively, the analysis was conducted for the thickness of the first storey and second storey walls, which is 45 cm, while secondary walls are 30 cm in thickness. The floor structure (cf. Figure 4) is a traditional structure composed of timber joists placed at approximately 65 cm intervals, loose fill or slag, plaster on reed bedding at the bottom, and floorboards and flooring on top side of the floor structure. Depending on the realisation method, the weight of the structure ranges from 2.0 to 2.5 kN/m^2. If partition walls are taken into account and distributed by square mater of floor structure space, then the design mass of the floor structure is approximately 300 kg/m^2.

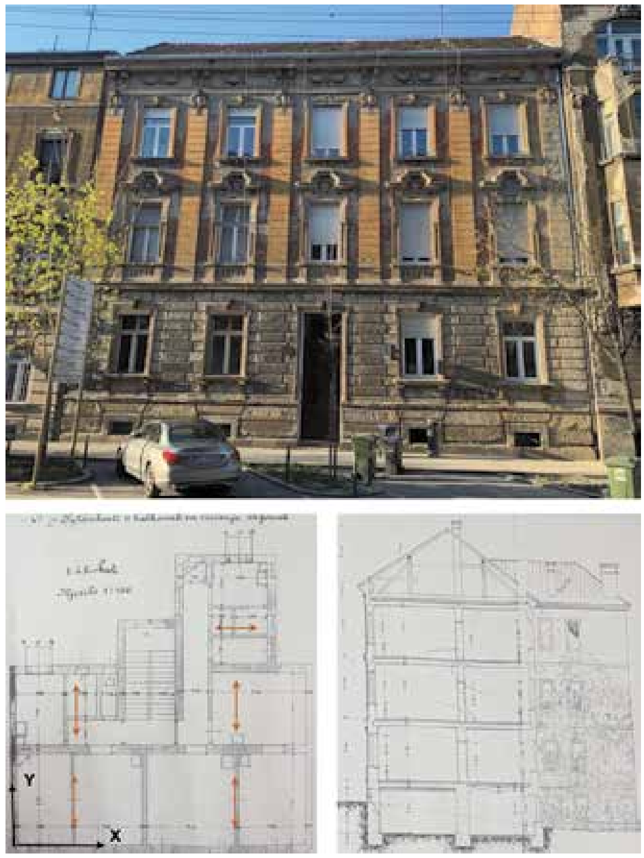


Figure 3. Copy of original floor plan and cross-section of the building (bottom), photograph of the building (top)

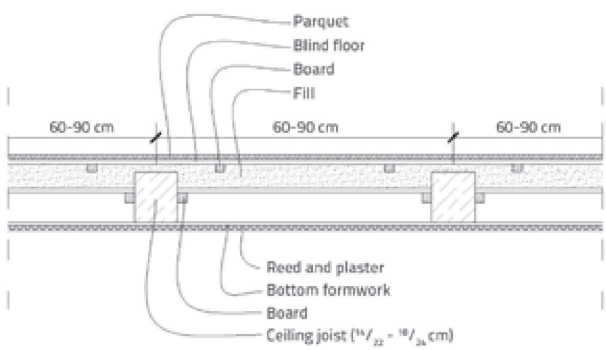


Figure 4. Floor structure of typical building

5.2. Mechanical properties of incorporated materials

The following mechanical properties of incorporated materials were adopted in calculation, Table 1.

Table 1. Mechanical properties of materials, typical values

Material	Elastic modulus E [N/mm²]	Shear modulus G [N/mm²]	Weight density [kN/m³]	Compressive strength f_k/f_m [N/mm²]	Shear strength f_{vk} [N/mm²]	Shear strength f_{vk0} [N/mm²]
Timber	9500	500	3.06	12	1.5	
Brick			18.0	7.0	1.2	0.29/0.15[*)]
Mortar				5.0[**)]	0.15	0.29/0.15[*)]
Wall	1000 f_k	0.4E_k		5.0[**)]	0.15	0.15

[*)] Values of shear resistance at joint when the wall is not subjected to vertical load should be determined by testing. As such data were unavailable, the authors decided to analyse the building for two shear stress values f_{vko} = 0.29 [N/mm²] (corresponds to a general case) and f_{vko} = 0.15 [N/mm²] (corresponds to an expected lower quality of mortar).
[**)] The compressive strength values presented in Table 1 should be determined in situ by testing, and standardised with the here adopted wall thickness of 450 mm.

6. Gravity load

6.1. Load

Radi preglednosti gravitacijsko je opterećenje predočeno u tablici 2.

Tablica 2. Gravitacijska opterećenja

Load	Nominal load [kN/m²]	Reduction factor ψ_2	$G_k + \psi_2 \cdot Q_k$ [kN/m²]
Self-weight	2.5	1.0	2.5
Floors	0.5	1.0	0.5
Occupancy load	2	0.3	0.6
Total			**3.6**

The proof of load bearing capacity for an accidental earthquake situation is determined for a simultaneous action of permanent loads (self-weight and floor weight) that are taken into account in the total nominal value, and the corresponding occupancy load (service load) is reduced by the reduction factor for quasi-static actions, which amounts to ψ_2 = 0,3 for residential buildings.

$$E_d = E(G_k + \psi_{2i} \cdot Q_{ki}) = E(G_k + 0{,}3 \cdot Q_{ki})$$

7. Modelling

7.1. General information on modelling using program package "3muri"

The building was analysed using the program package *3muri*, as adapted to spatial analysis of existing masonry buildings. The wall structure is described with in plane macro-elements, whose nonlinear response is calculated for each increment of lateral force. Three possible failure modes are analysed in this respect: horizontal and sawtooth sliding along the joint (shear failure), cracking of compression and tensile zones of wall elements exposed to bending (rocking mode), and tensile failure along the diagonal of masonry elements that assume lateral force at the top. At that, the reduction of stiffness and load bearing capacity caused by wall element cracking is continuously calculated. The

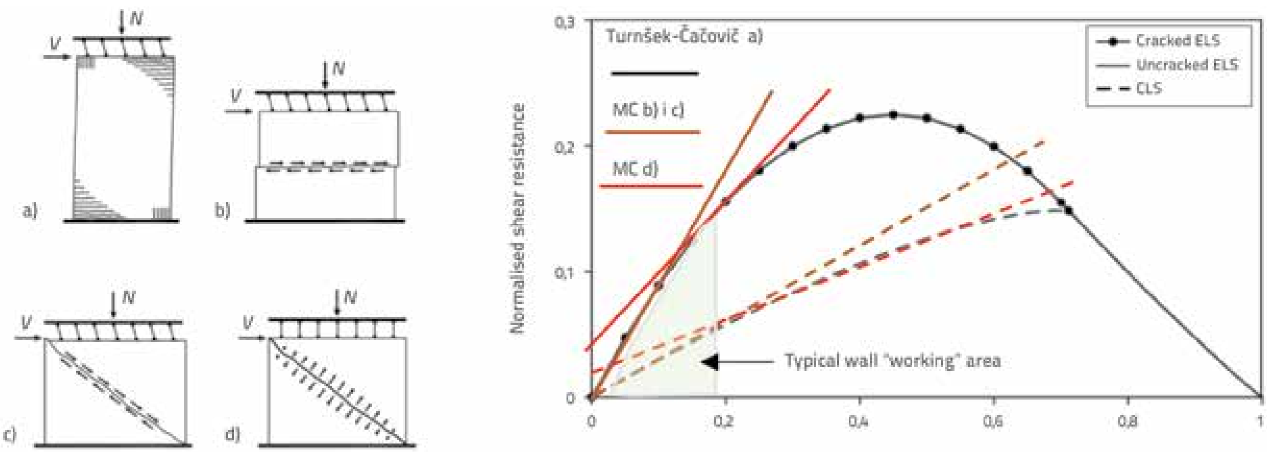


Figure 5. Constitutive models applied in program package 3muri: elastic limit of wall element (Turnšek-Čačović) a); Mohr-Coulomb failure condition b), c) and d). Solid line denotes ultimate limit state in the zone of elasticity, while dashed line denotes limit state of cracked wall element

three-dimensional structure model is represented with the replacement in-plane frames, which are most often placed in two orthogonal directions, although this is not a condition (cf. Figure 8). Floor structures covered in the analysis can be made of stone or brick, or they can be reinforced-concrete slabs, floors with timber joists or in combination with metal girders, the so called "hourdis" floors. Horizontal stiffness of timber joist floors and transfer of horizontal influences onto load bearing walls is taken into account automatically via axial stiffness of elements (timber joists and timber boards) which lie at the floor level. Thus, different stiffness values are taken into account in the redistribution of forces at walls depending on the direction of earthquake action. Additional information about the program package can be found in [7] or in [8, 9].

It should be noted that shear resistance, resistance to bending, and deformation capacity (capacity of an element to develop quasi-plastic deformations) of a wall element exposed to lateral forces depend on the level of wall exposure to longitudinal forces. Figure 6 shows the bending moment capacity as related to exposure to normal forces. If the standardised force is N/N_m = 0.1, the constitutive model results in relatively great displacements, which must be critically re-examined. On the other hand, the highest shear resistance could be activated for N/N_m = 0.5 with much lower deformation capacity. If the standardised force amounts to N/N_m = 0.75, a visible fall in limit bearing capacity and deformation capacity can be noticed when compared to other standardised N force values. Less than 50 % of compressive strength is used during the use of the building: this is on the one side due to high partial safety coefficients applied to wall as material (γ_M = 2.0) and, on the other side, to an increase in nominal load via partial safety factors γ_G = 1.35 and γ_Q = 1.5.

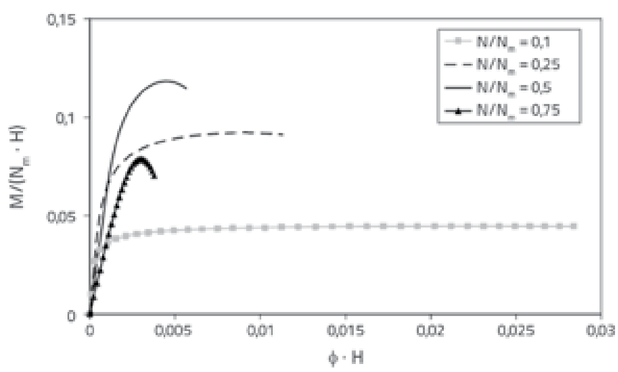


Figure 6. Dependence of bending moment (for Turnšek-Čačović constitutive model) on various levels of normal forces applied to masonry element, [9]

7.2. Deformation capacity of wall elements

The analysis of masonry structures is considered to be quite complex due to anisotropy of walls as composite materials, and due to interdependence of all geometrical values (for instance, in a general case, design moment of inertia is dependent on normal force acting on wall). In seismic analyses of walls, static pushover analysis has proven to have some advantages, important in practical applications, when compared to theoretically superior nonlinear dynamic analysis (time history analysis), provided that the bearing capacity of the structure can be approximated with sufficient accuracy via a bilinear relationship $F-\Delta$. Even if that is not the case, this relatively simple analysis can be extended to multimodal pushover analysis (cf. for instance [10]), which will certainly provide desired results.

The greatest comparative advantage of pushover analysis lies in its simplicity and transparency. While the task of determining total resistance to lateral forces (shear resistance) is relatively easy to solve, this has not been fully harmonised with regard to the methodology for defining limit displacement values, which are also related to different levels of building damage. Thus for instance the IBC (International Building Code) allows relative interstorey displacements of Δ_u = 0,4 % for structures capable of shear bearing, and Δ_u = 0,6 %, for structures capable of bending bearing. Almost the same value is recommended in EC8 (CEN,2005b), where it is expressed as follows:

$$\Delta_u = \begin{cases} 0{,}4\ \%, & \text{za posmik} \\ 0{,}8\ \% \times \dfrac{H_0}{B}, & \text{za savijanje} \end{cases} \tag{4}$$

Here, H_0 is the height at which moment is equal to zero, while B is the width of the wall element.

Swiss standard SIA 268:2017 (cf. [11]) offers somewhat more accurate values for the calculation of existing structures. Here the values are dependent on the stress mode and the level of normal force in the element. If the utilisation of walls with regard to normal force is ≤ 0.2, then the maximum relative interstorey displacement of walls is determined according to [11] in the following way:

- Δ_u = 0.004 for walls that are fixed at the bottom and top to reinforced concrete (floor) slabs, which corresponds to deformation capacity with regard to shear,
- Δ_u = 0.008 for walls that are not fixed at the top, which corresponds to the model of walls that are dominantly susceptible to bending
- $\Delta_u = 0{,}01 \cdot \left(1 - 0{,}9 \cdot \dfrac{N_d}{l_w \cdot t_w \cdot f_d}\right) \cdot \dfrac{h_v}{\min(h_f, l_w)} \cdot \sqrt{\dfrac{h_{ref}}{h_f}}$,

  for walls for which displacement is equally due to bending and shear deformations.

Where N_d-normal force, l_w- wall length, t_w- wall thickness, f_d – compressive strength of the wall, h_v – wall length in compression in M/N interaction (wall length that can transfer shear), h_{ref} – constant referenced to test wall height of 2.4 m, h_f – (free) height of the wall. Note: According to SIA 269-8, is the deformation capacity (*ultimate value*) of the walls. The design value and hence the comparative value is obtained in computation by dividing with the partial safety factor for walls.

By comparison of a great number of experimental results obtained at ETH Zürich [12], an empirical pattern was obtained for the determination of limit displacements as related to utilisation of walls in compression, which will be applied in this text.

Table 3. Displacement at top of the building and corresponding damage levels according to [13]

Damage level	Spectral displacement	Assumed losses [%]	International nomenclature
No damage	$\Delta < 0{,}7\ \Delta_y$	0	
Minor damag	$0{,}7\ \Delta_y < \Delta < \Delta_y$	0-4	Fully operational (FO)
Moderate damage	$\Delta_y < \Delta < 2 \cdot \Delta_y$	4-20	Immediate occupancy (IO)
Great damage	$2\ \Delta_y < \Delta < 0{,}7\Delta_u$	20-50	Life safety limit state (LS)
Very great damage	$0{,}7\ \Delta_y < \Delta < \Delta_u$	50-100	Collapse prevention (CP)
Building collapse	$\Delta > \Delta_u$	100	

$\delta_u = \delta_0 (1 - (\sigma_n/f_{xd})$, where $\delta_0 = 0.008$, σ_n normal stress at joint, and wall strength perpendicular to joint. The analysis of utilisation of walls of the archetypal structure to compression strength during service live shows range from 0.025 to 0.1 for gable walls, while for main bearing walls range is from 0.04 to 0.17, for the fourth floor and ground floor, with the constant wall thickness of 450 mm. The limit value of the coefficient of deformation capacity of walls is obtained by adopting a reasonable value of $(\sigma_n/f_{xd}) = 0.15$, with the wall response for the combination of shear and bending modes amounts to $\delta_u = 0.0068$.
It can be seen from geometry of the archetypal building analysed in this text that, in the direction X, the building laterally behaves as a frame system susceptible to shear and bending, which is due to a great number of openings and short slender elements. Wall deformations in the direction Y, with the length to height ratio of , are laterally shear wall. The following limit displacement values are taken as parameters for comparison:

- $\Delta_u = (0.4/100) \cdot 14000 = 56$ mm, for elements in shear (spreading direction: Y)
- $\Delta_u = (0.68/100) \cdot 14000 = 95.2$ mm, for elements in bending and shear (spreading direction: X)

The following criterion will be appropriate for estimating damage and defining limit states *(performance levels)* during earthquake action [13], kako je prikazano u tablici 3.

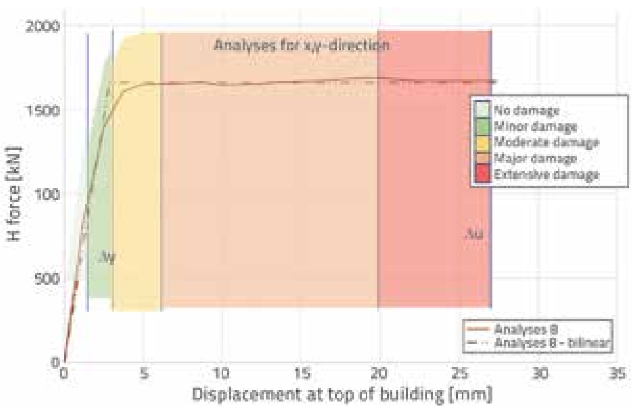


Figure 7. Qualitative presentation of damage due to displacement at the top of the building, according to [13]

7.3. Replacement frames

Figure 8 shows a plan view of in-line replacement frames for the analysed building as well as a three-dimensional view of the building. Design entities (frames) contain two-dimensional macro-elements of door lintels, columns and wall elements which withstand lateral displacements. However, the stress state of each element is defined separately in the displacement actuation process.

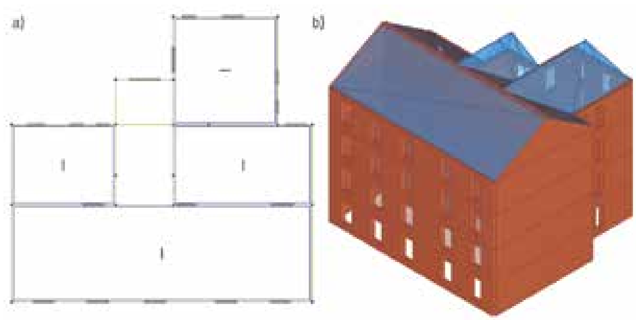


Figure 8. Plan view of replacement frames (left) and 3D model of the building (right)

7.4. Seismic analysis

In the global seismic analysis, it is usual to conduct a three-dimensional analysis of structures for two main orthogonal directions, which in general coincide with the (in plane) spreading of load bearing walls. In this case, global analysis is conducted for two types of soil (B and C), two different qualities of mortar (f_{vk0} = 0.15 i 0.29 N/mm^2), and two different distributions of lateral forces along the building's height (proportional to masses, and affine to the first mode). The corresponding analyses are shown in Table 4.
In addition to global analysis in the direction of load bearing walls, the analysis was also conducted perpendicular to the spreading of gable walls in order to determine failure mechanisms, which greatly depend on the way gable walls are connected with other parts of the structure by transverse walls and floor structures.

Table 4. Overview of global pushover analyses for prototype building using program package 3muri

Analysis	Direction	Distribution of lateral forces	Type of soil	Shear resistance for normal force N = 0 kN [N/mm²]	Ground acceleration [m/s²]
1	X +	Uniform	B | C	f_{vk0} = 0.29 i f_{vk0} = 0.15	a_g = 2.55
2	X +	1_mode			
3	X -	Uniform			
4	X -	1_mode			
5	Y +	Uniform			
6	Y+	1_mode			
7	Y-	Uniform			
8	Y-	1_mode			

8. Results

8.1. Global response of the building

8.1.1. Global analysis results for shear strength values at joint f_{vk0} = 0.29 N/mm²

Global analysis results for directions X and Y are presented in figures 9 and 10 for f_{vk0} = 0.29 N/mm². It can be concluded from bilinear approximation that analysis 1 is relevant for the direction X, despite the fact that analysis 2 results in lower limit bearing capacities of $E_{d,x,2}$ = 871.76 < $E_{d,x,1}$ = 1051.3 kN, but with greater deformation capacity $\Delta_{u,2}$ = 176,7 mm > $\Delta_{u,1}$ = 133.0 mm and so the criterion of lower deformation capacity is relevant for PBD. The analysis 5 with the distribution of lateral forces along the height of the building proportional to floor masses is relevant for the direction Y. When compared to analysis 6, analysis 5 is relevant for PBD despite lower limit bearing capacity $E_{d,y,6}$ = 1454.54 kN < $E_{d,y,5}$ = 1752.6 kN (see also Table 4), as it results in lower deformation capacity $\Delta_{u,6}$ = 27.1 mm > $\Delta_{u,5}$ = 18.2 mm. IIt can be seen from Figure 9 that horizontal resistance of the building suddenly decreases at the displacement of approximately 50 mm, which is the sign of an element's failure by either shear or bending. Failure mechanism at the moment of resistance loss and sudden increase in bearing capacity of the building in the scope of analysis 3 is shown – with all significant details – in Figure 11.

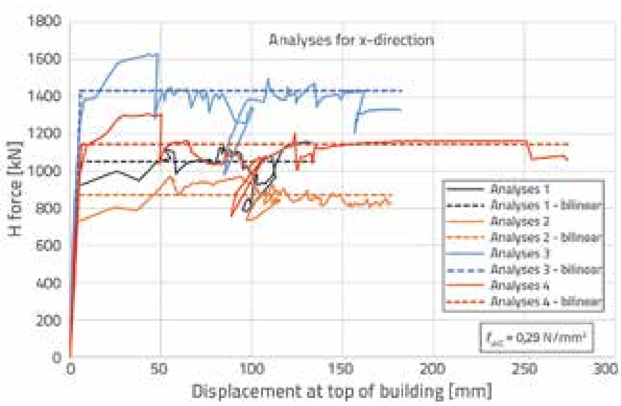


Figure 9. Results of analyses 1-4 and their bilinear approximations

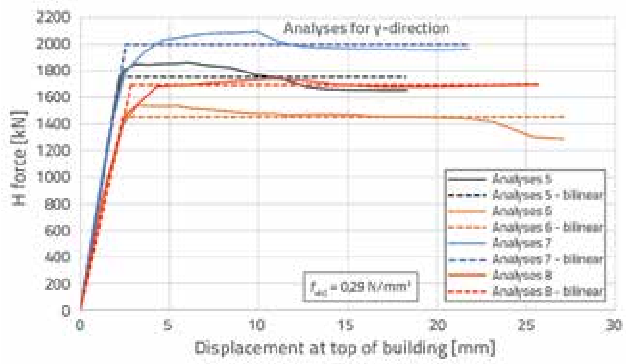


Figure 10. Results of analyses 5-8 and their bilinear approximations

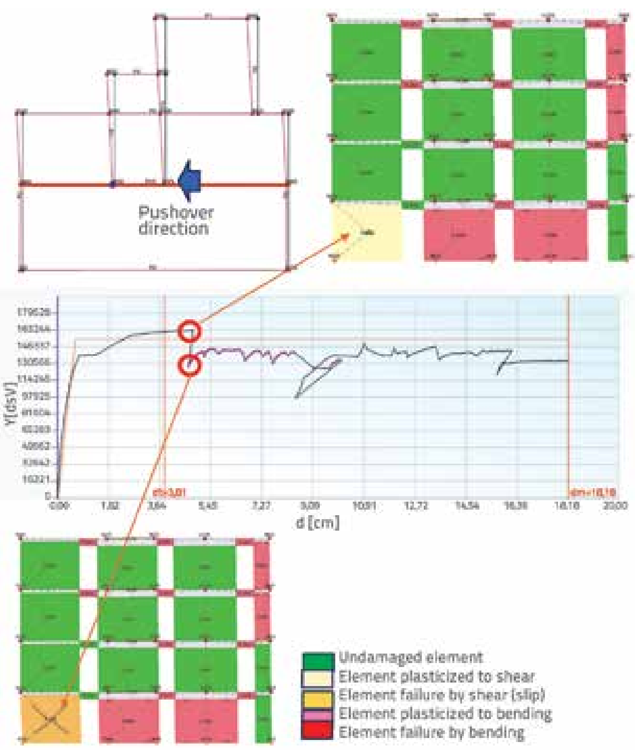


Figure 11. Failure mechanism of central wall in X-direction (analysis No. 3)

Table 5. Results of pushover analyses 1-8 with vibration periods and spectral values of force and displacement for minimum shear strength f_{vk0} = 0.29 [N/mm²]

Analysis	T [s]	Fy* [kN]	m* [kg]	Δy* [mm]	Δu* [mm]	Γ [-]	Δy [mm]	Δu [mm]	F [kN]
1	0.331	773	620511	3.45	97.79	1.36	4.7	133.0	1051.28
2	0.375	641	620511	3.68	129.95	1.36	5.0	176.7	871.76
3	0.317	1054	620511	4.33	133.82	1.36	5.9	182.0	1433.44
4	0.366	843	620511	4.62	201.49	1.36	6.3	274.0	1146.48
5	0.183	1270	686934	1.58	13.22	1.38	2.2	18.2	1752.6
6	0.21	1054	686934	1.71	19.66	1.38	2.4	27.1	1454.52
7	0.186	1449	686934	1.84	15.76	1.38	2.5	21.7	1999.62
8	0.213	1227	686934	2.06	18.61	1.38	2.8	25.7	1693.26

Table 6. Analysis results obtained by program 3muri, design values for behaviour q and ductility μ, and shear coefficient at the plane of fixity level, including shear coefficient needed for building YPS

Analysis	Soil type B				Soil type C			
	qu [-]	μ [-]	cy* [m/s²]	μ/qu [-]	qu [-]	μ [-]	cy* [m/s²]	μ/qu [-]
1	6.14	8.94	1.25	1.46	5.88	10.15	1.25	1.73
2	7.4	9.61	1.03	1.30	7.09	10.91	1.03	1.54
3	4.5	6.71	1.70	1.49	4.32	7.58	1.70	1.76
4	5.63	7.40	1.36	1.31	5.39	8.36	1.36	1.55
5	4.14	9.80	1.85	2.37	3.77	10.10	1.85	2.68
6	4.99	10.50	1.53	2.10	4.78	11.80	1.53	2.47
7	3.63	8.10	2.11	2.23	3.33	8.50	2.11	2.55
8	4.28	8.70	1.79	2.03	4.11	9.8	1.79	2.38

Table 7. Fulfilment factor for all individual analyses conducted with f_{vk0} = 0.29 [N/mm²]

Analysis	Direction	Required (elastic) displacement Δ_{el} [mm]		Deformation capacity Δ^*_u [mm]	Fulfilment factor α_i		Comment ! – condition not fulfilled
		B	C		B	C	
1	X	30.9	35.0	70.0	2.25	2.00	✓
2		35.4	40.2	70.0	1.97	1.73	✓
3		29.0	32.8	70.0	2.40	2.12	✓
4		34.2	38.6	70.0	2.0	1.80	✓
5	Y	16.6	18.0	13.22	**0.80**	**0.73**	**!**
6		19.2	22.0	19.66	1.02	**0.89**	✓ **!**
7		16.6	18.2	15.76	**0.95**	**0.86**	**!**
8		19.3	22.0	18.61	**0.96**	**0.85**	**!**

Even a rough comparison of building response for two main directions shows that deformation capacity of walls in the Y-direction (shear) is up to ten times smaller compared to the one in the X-direction (bending).

Bilinear approximations of bearing capacity in both directions are the so called pushover curves with displacements on top of the building. The limit capacity curve in ADRS format, shown for both directions in Figure 12, is obtained by dividing the maximum utilised resistance obtained by pushover analysis with the model mass m*, and by dividing the displacement on top of the building with the transformation factor 0,0068 H, thus translating MDOF into SDOF.

It can be seen from diagram presented in Figure 12 that the deformation capacity of the building in the X-direction is sufficient for both types of foundation soil, and this despite the fact that the design deformation capacity is limited to the value of (1/Γ) = (95.2/1.36) = 70 mm.

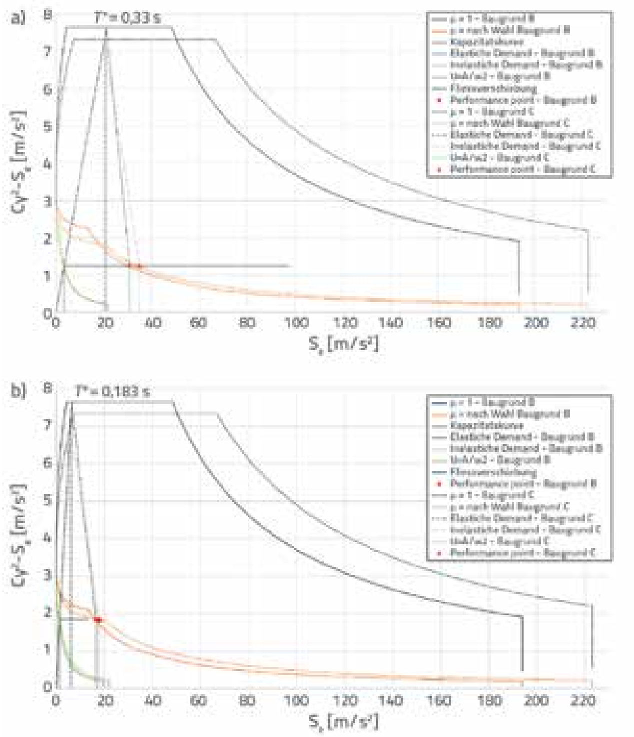


Figure 12. Results of the analysis No. 1 for X-direction (up) and analysis No. 5 for Y-direction (down) in ADRS format for the analysed building on soil type B and C, with wall mortar quality f_{vk0} = 0.29 N/mm²

For the Y-direction, the factor of fulfilment of the required deformation capacity is less than 1 for both soil types, which means that the condition is not fulfilled (cf. Table 7).

8.1.2. Global analysis results for shear strength value at joint f_{vk0} = 0,15 N/mm²

Global analysis results for directions X and Y are presented in figures 13 and 14 for f_{vk0} = 0.15 N/mm². It can be concluded from bilinear approximation that analysis 3 with smallest deformation capacity of $\Delta_{u,3}$ = 47.0 mm is relevant for the X-direction. The analysis 5 with $\Delta_{u,5}$ = 17.9 mm is relevant for the Y-direction. It should be noted that both values are much lower than comparative limit values defined in Section 7. Analysis results are presented collectively in tables 7 and 8.

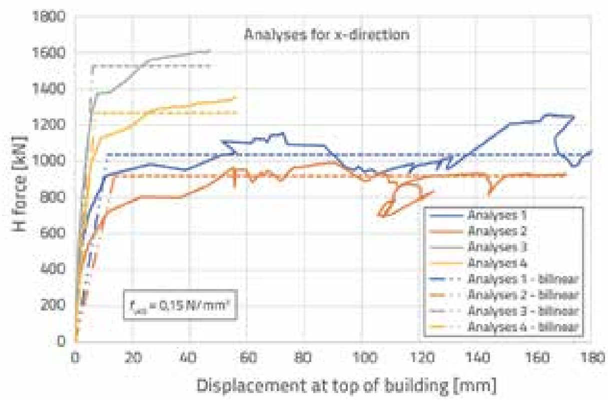


Figure 13. Pushover curves for X-direction

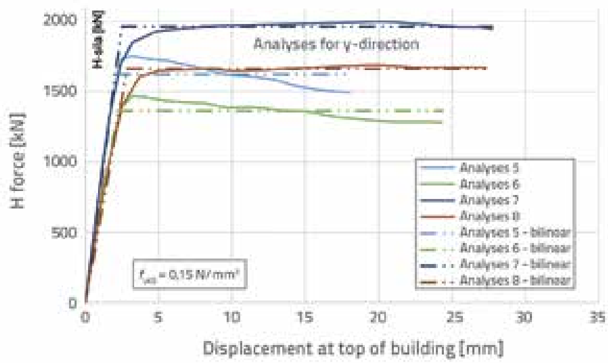


Figure 14. Pushover curve for Y-direction

It can be seen from diagram shown in Figure 15 that the building's deformation capacity in the X-direction is sufficient for both types of foundation soil despite the fact that the design deformation capacity is limited to the value of 0,0068 H (1/Γ) = (95.2/1.36) = 70 mm. As to the Y-direction, the factor of fulfilment of the required deformation capacity is lower than 1 for both soil types, which means that the condition is not fulfilled (Table 10).

Tablica 8. Rezultati analiza postupnoga guranja 1 do 8 s periodima vibracija i spektralnim vrijednostima sila i pomaka za minimalnu vrijednost posmične čvrstoće f_{vk0} = 0.15 [N/mm²]

Analysis	T [s]	Fy* [kN]	m* [kg]	Δy* [mm]	Δu* [mm]	Γ [-]	Δy [mm]	Δu [mm]	F [kN]
1	0.52	761	620511	8.35	189.5	1.36	11.4	257.7	1034.96
2	0.61	675	620511	10.08	126.15	1.36	13.7	171.6	918
3	0.32	1123	620511	4.55	34.57	1.36	6.2	47.0	1527.28
4	0.37	931	620511	5.15	41.17	1.36	7.0	56.0	1266.16
5	0.18	1176	686934	1.46	13	1.38	2.0	17.9	1622.88
6	0.21	987	686934	1.59	17.63	1.38	2.2	24.3	1362.06
7	0.19	1422	686934	1.79	20.07	1.38	2.5	27.7	1962.36
8	0.22	1204	686934	2.06	19.76	1.38	2.8	27.3	1661.52

Table 8. Results obtained in program 3muri, design values for behaviour factor q, ductility μ and shear coefficient at the plane of fixity level

Analysis	Soil type B				Soil type C			
	qu [-]	μ [-]	cy* [m/s²]	μ/qu [-]	qu [-]	μ [-]	cy* [m/s²]	μ/qu [-]
1	6.01	6.01	1.23	1.00	5.97	6.76	1.23	1.13
2	5.81	5.81	1.09	1.00	6.69	6.69	1.09	1.00
3	4.23	6.31	1.81	1.49	4.05	7.13	1.81	1.76
4	5.1	6.65	1.50	1.30	4.88	7.50	1.50	1.54
5	4.47	10.30	1.71	2.30	4.07	11.00	1.71	2.70
6	5.32	11.40	1.44	2.14	5.1	13.00	1.44	2.55
7	3.7	8.40	2.07	2.27	3.38	8.70	2.07	2.57
8	4.36	8.80	1.75	2.02	4.18	10.2	1.75	2.44

Table 10. Fulfilment factor for all individual analyses conducted with f_{vk0} = 0,15 [N/mm²]

Analysis	Direction	Required (elastic) displacement Δ_{el} [mm]		Deformation capacity Δ^*_u [mm]	Fulfilment factor α_i		Comment ! – condition not fulfilled
		B	C		B	C	
1	X	50.2	56.4	70.0	1.39	1.24	✓
2	X	58.6	67.4	70.0	1.19	1.04	✓
3	X	28.7	32.5	34.57	1.20	1.06	✓
4	X	34.2	38.6	41.17	1.20	1.07	✓
5	Y	16.6	18.0	13.0	**0.78**	**0.72**	**!**
6	Y	19.2	22.0	17.63	**0.92**	**0.80**	**!**
7	Y	16.6	18.1	20.07	1.21	1.10	✓
8	Y	19.5	22.2	19.76	1.01	**0.89**	✓ **!**

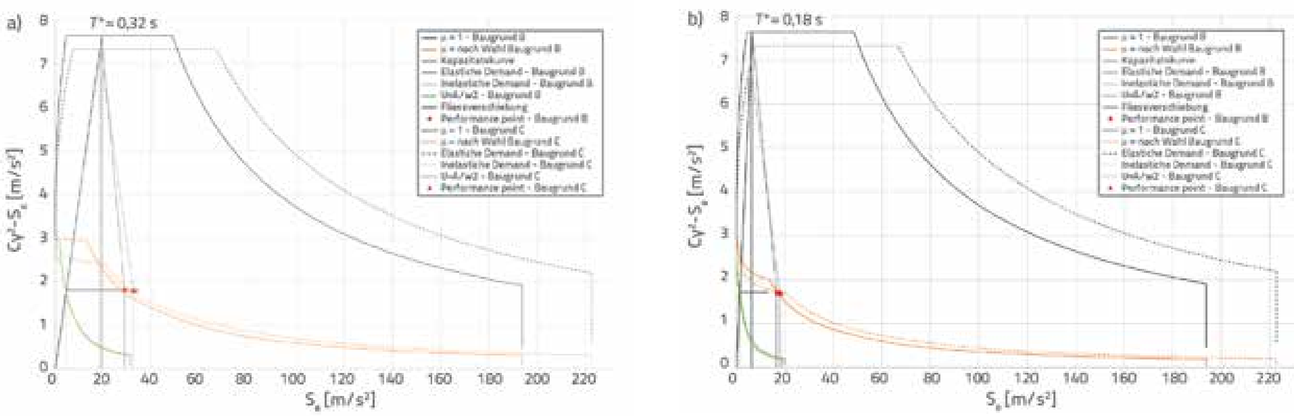


Figure 15. Results of analysis No..3 for X-direction (up) and analysis No. 5 for Y-direction (down) in ADRS format for the analysed building on soil type B and C and for mortar quality f_{vko} = 0,15 N/mm²

Results presented in Section 8 show that the building analysed as a compact entity can withstand with a great level of probability an earthquake of designed intensity for the X-direction, although it would suffer considerable damage during such seismic event. Thus for instance buildings built with mortar exhibiting a nominal shear strength of 40 $\cdot\Gamma$ = 54.4 mm = oko 57 % $\cdot\Delta_u < 0{,}7 \cdot\Delta_u$ = 0.7 95.2 = 66.5 mm, with design deformations at the top of the building of about, would suffer considerable damage according to Table 3 and Figure 7, while a structure built using mortar exhibiting a nominal shear strength of 0.15 N/mm², with design deformations at the top of the building of about 67.4 $\cdot\Gamma$ = oko 96 % $\cdot\Delta_u \approx \Delta_u$ would suffer very great damage. However, for the designed seismic situation, the building would not collapse although in the sense of earthquake engineering it is situated in the critical area between *live safety* and *collapse prevention limit state*, depending on mortar quality.

According to tables 6 and 9, the seismic demand fulfilment factor for Y-direction lies at the level of 0.72 and 1.0 and, at that, the building made of a better quality mortar has a smaller deformation capacity to shear and is on an average stiffer in response to seismic action compared to a building made of weaker mortar. Using the methodology similar to that used for X-direction, we would have to say that in this case the measures ensuring better structural response to shear must be planned, or otherwise the building would fail or collapse in the case of a design earthquake. The question of whether repair or improvement of response to seismic action is obligatory is a matter for scientific discussion, and the answer will have to be provided by professional community, taking into account various culturological and social implications. At this point, it should be noted that such improvement obligation is not planned in the Swiss standard [3, 11], for buildings whose remaining technical and economic service life is 30-40 years, for fulfilment factors of $\alpha_i > 0.7$. This procedure can be interpreted as an artificial reduction of design seismicity of a microlocation from about 2.55 to 2.0 m/s^2 or, alternatively, reduction of return period from, for instance, 475 to 325 years, which is explained by the reduced probability of exceedance of a design seismic event in the remaining (technical) service life of the building. However, in case the decision is made to use method for improvement of structural response to design seismic action, then the fulfilment factor of 1.0 should be aimed at $\alpha_i \geq 1.0$.

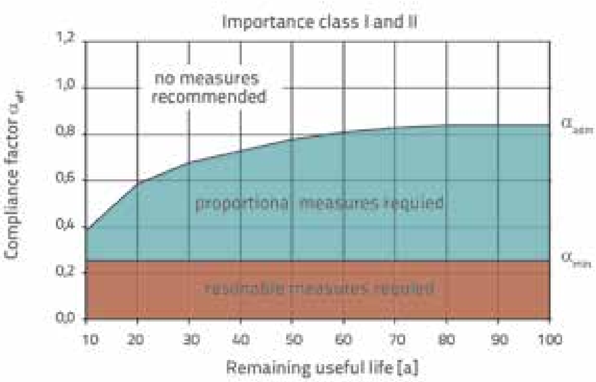


Figure 16. Recommended compliance factors as related to remaining useful life of buildings, according to SIA 269/8 [11]

9. Incremental dynamic analysis

Seismic analyses based on forces are most often applied in practical situations. Here we will not examine in great detail the unreliability and deficiencies of this methodology, especially when attempts are made to represent nonlinear response through linear models such as the one used in the response spectrum analysis [14]. However, it is quite probable that the results presented in previous section do not correspond to experience with FBD as obtained by intuitive expectations such as those suggested by calculations based on forces. In addition, the earlier presented methodology has some major limitations. For instance, the nonlinear response obtained by pushover analysis results in displacements which take into account only the first mode, but are generally independent on the time. In other words, the methodology is inaccurate if applied on buildings in which higher modes have a considerable influence on the total response of buildings.

In order to overcome deficiencies of PBD and gain a better insight into the response of structures to earthquake actions, the results obtained by using the deterministic methodology of performance based design, as presented in Section 8, will be checked using an independent methodology, i.e. the incremental dynamic analysis (IDA), with a pronouncedly probabilistic background [15, 16].

In simpler terms, the IDA is used as a means to determine the fragility function of structures that are exposed to seismic action. It is based on an advanced numerical model of structures subjected to nonlinear dynamic analysis. The seismic input information is represented with a set of several tens of earthquake records that incrementally scan amplitudes thus simulating an increase in seismic intensity. Nonlinear response of a building is determined for each level of seismic intensity, and limit intensity (intensity measurement IM) is sought – in this case of spectral acceleration - for which a predefined limit state will be achieved, in this case life safety (LS) and/or collapse prevention (CP) limit state. As a rule, fragility function has a lognormal distribution. For its derivation, it is necessary to define limit displacement values (the so called engineering demand parameters) for design limit states (IO, LS, CL) so as to determine the intensity that signalises the exceedance of the defined limit state (for instance IO) or failure (for instance CP).

The fragility function is expressed via the Gaussian function of normal distribution, which is fully defined by the mean value of logarithm and by the logarithmic standard distribution $\pm\sigma$. As the normal distribution function is a function with the standard deviation of , the integral of the area below the normal distribution function is 68% of the total 100% probability, which means that the values of the integral of the normal distribution function will not be covered by this integral in the fractile range lower than 16% and greater than 84%. The probability that the limit state will be exceeded is expressed by the equation (5).

$$P\left[IM_f^{LS} \leq im\right] = \phi\left[\frac{\ln(im) - \eta}{\beta}\right] \quad (5)$$

It is known as IM-formulation, which is favourable particularly in cases when the focus of interest is on the limit state of global failure of the building (CPLS).

9.1. Selection of mechanical model of structures for IDA analysis

The structure of a typical masonry building, with floor structures made of wooden joists, was analysed in the preceding section for two shear strengths of mortar, and for two types of foundation soil. As the IDA analysis results are not dependent on the type

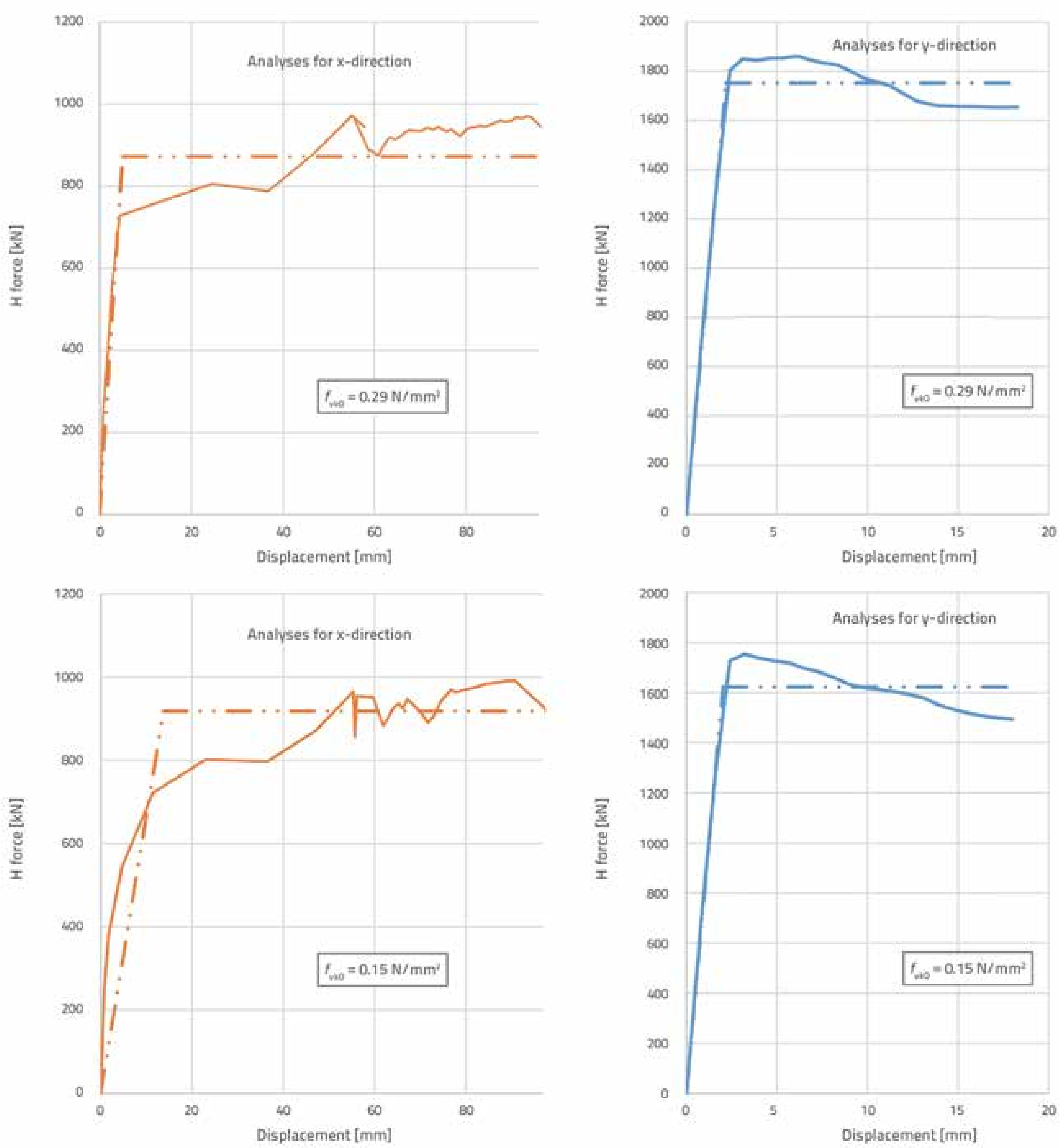


Figure 17. Curves obtained by pushover method for directions X and Y, and used as input for IDA analysis

of soil, a pushover curve appropriate for further analysis will be selected based on the results presented in tables 7 and 10, separately for each direction. For the structure with the mortar shear strength of f_{vk0} = 0.29 N/mm^2 and f_{vk0} = 0.15 N/mm^2 this involves analysis No. 2 for X-direction and analysis No. 5 for Y-direction. Pushover curves are shown in Figure 17.

The IDA was conducted for X-direction with affine distribution of lateral forces in the first mode, while the analysis for Y-direction was conducted for lateral forces distributed in proportion with floor structure masses.

9.2. IDA analysis results

The IDA analysis results will be presented separately by direction, and will contain 16 %, 50% and 84 % fractals in IDA curves, graphical view of limit states defined directly at the PO (performance objectives) curve, and table of results of spectral values that symbolise achievement or exceedance of the desired behaviour (limit states) of the building.

9.2.1. Results for X-direction

It can be seen from results shown in figures 18-22 that the limit state LS and CP will be achieved for the spectral acceleration of 0,6016 (g) = 5.94 m/s^2 i.e. 0.7454 (g) = 7,31 m/s^2 (figures 20 and 22).

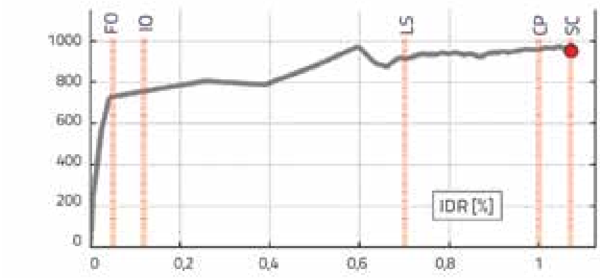


Figure 18. Definition of limit states on PO curve for X-direction

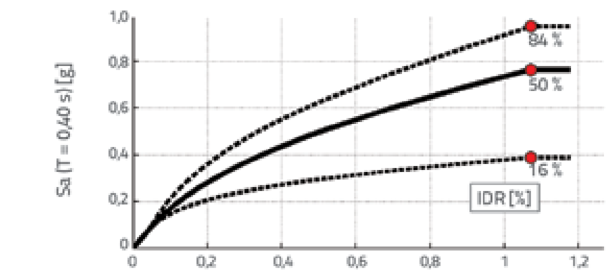


Figure 19. Fractile of IDA curves in the format of spectral acceleration as a function of period T = 0.4s and displacement at the top of the building in % as defined in Figure 18.

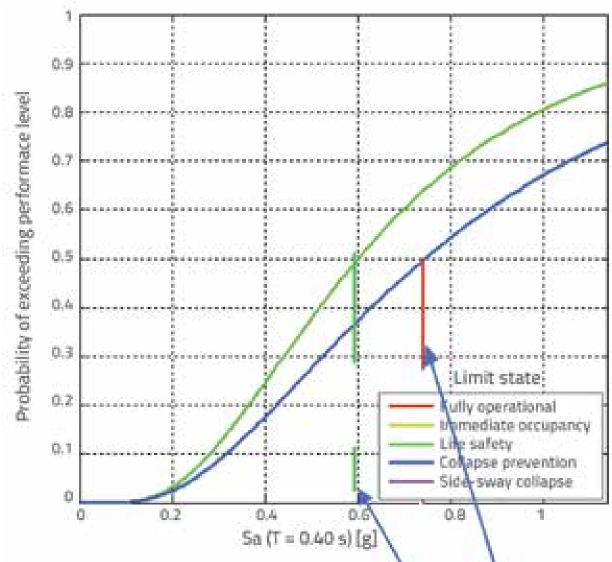


Figure 20. Fragility functions for two limit states. LS and CP

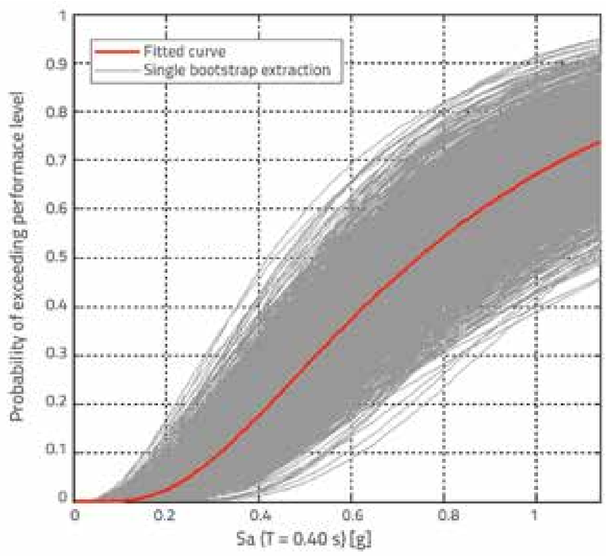
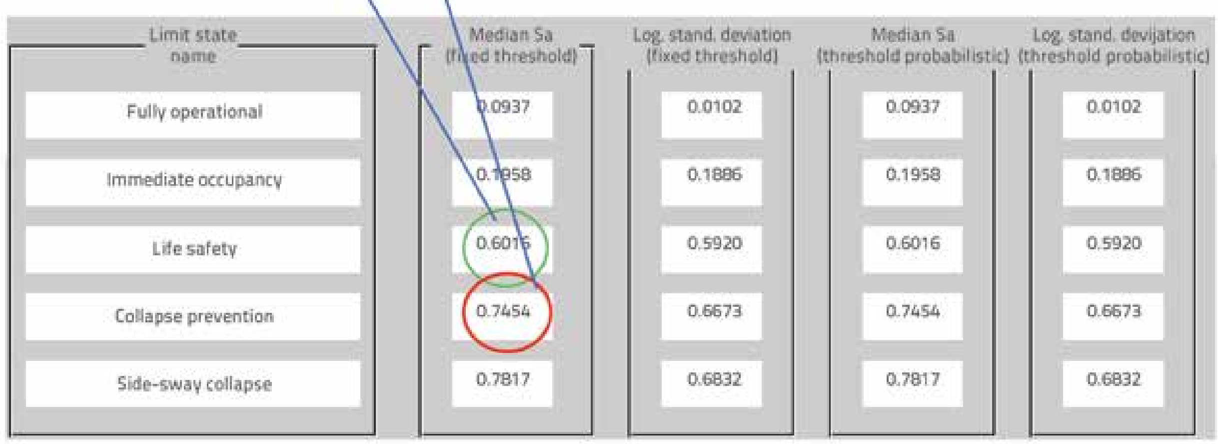


Figure 21. Visualisation of deviation of results for all seismic records as related to mean value



Limit state name	Median Sa (fixed threshold)	Log. stand. deviation (fixed threshold)	Median Sa (threshold probabilistic)	Log. stand. devijation (threshold probabilistic)
Fully operational	0.0937	0.0102	0.0937	0.0102
Immediate occupancy	0.1958	0.1886	0.1958	0.1886
Life safety	0.6016	0.5920	0.6016	0.5920
Collapse prevention	0.7454	0.6673	0.7454	0.6673
Side-sway collapse	0.7817	0.6832	0.7817	0.6832

Figure 22. Spectral accelerations for defined limit states

It can clearly be seen in Figure 23 that IDA defines the value of spectral acceleration of type C soil that is similar to the value prescribed by computation in the standard, and so the factor of fulfilment of computational demand is approximately 1. For the type B soil, the computation requirement has not been fulfilled and, with the value of 0.96, it is somewhat below the required value.

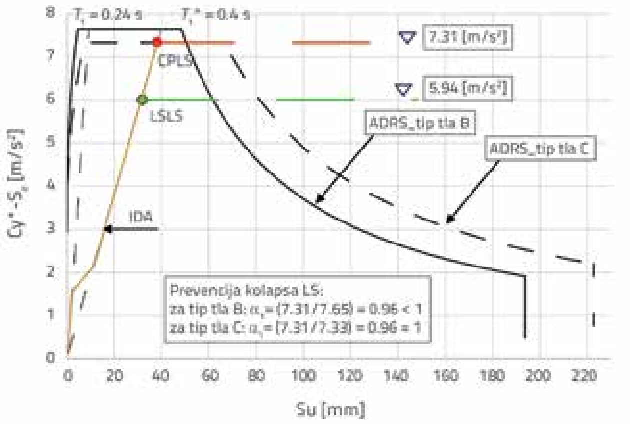


Figure 23. IDA curve with points of achievement of design limit states (LS and CP) and fulfilment factor obtained by IDA analysis for X-direction in ADRS format

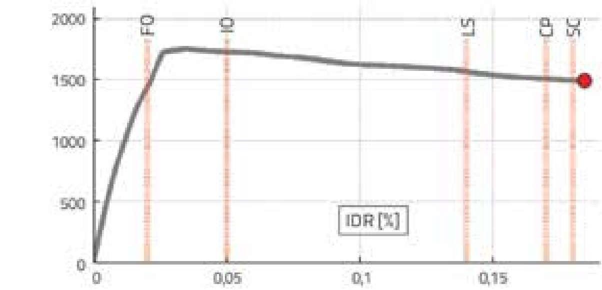


Figure 24. Definition of limit states on PO curve for Y-direction

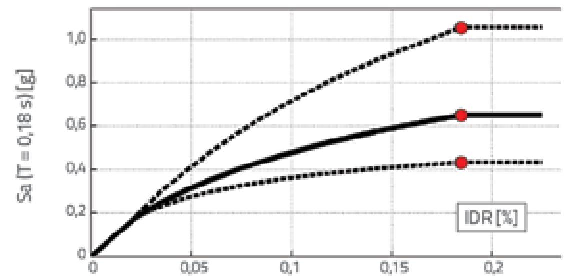


Figure 25. Fractile of IDA curves in the format of spectral acceleration as a function of period T = 0.18s and displacement at the top of the building in % as defined in Figure 24.

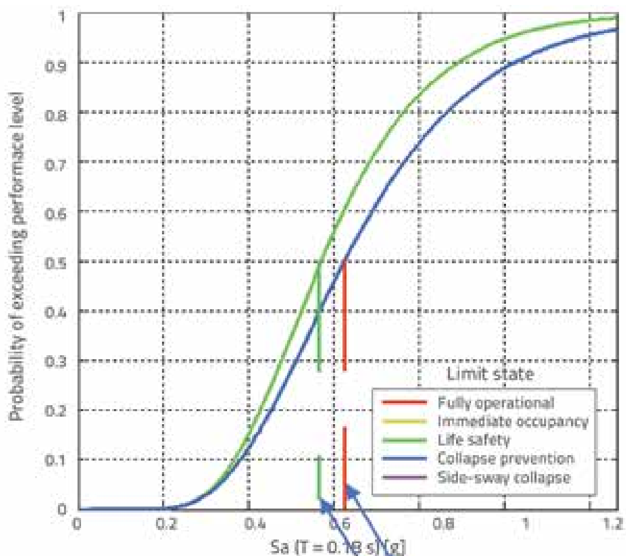


Figure 26. Fragility functions for two limit states. LS and CP

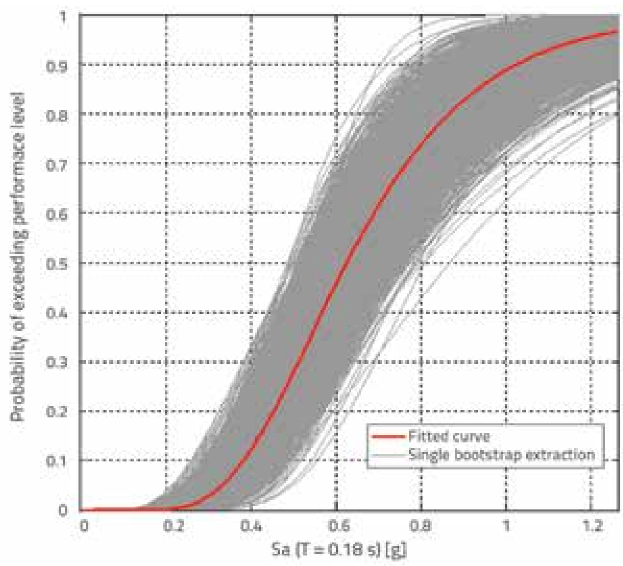


Figure 27. Visualisation of deviation of results for all seismic records as related to mean value

Limit state name	Median Sa (fixed threshold)	Log. stand. deviation (fixed threshold)	Median Sa (threshold probabilistic)	Log. stand. deviation (threshold probabilistic)
Fully operational	0.1606	0.0000	0.1606	0.0000
Immediate occupancy	0.3161	0.1352	0.3161	0.1352
Life safety	0.5712	0.3469	0.5712	0.3469
Collapse prevention	0.6255	0.3864	0.6255	0.3864
Kolaps bočnog njihanja (dinamička nestabilnost) Side-sway collapse	0.6437	0.3985	0.6437	0.3985

Figure 28. Spectral accelerations for defined limit states

9.2.2. Results for Y-direction

It can be seen from results shown in figures 23-27 that the limit states LS and CP will be achieved for spectral acceleration of 0.5712 (g) = 5.6 m/s^2 i.e. 0.6255 (g) = 6.13 m/s^2 (Figures 26 and 28).
It can be seen in Figure 29 that a lower value of spectral acceleration is calculated by IDA for both soil types (B and C), which would lead to the exceedance of limit state CPLS compared to design accelerations specified in the standard. The factor of fulfilment of design requirement for CP varies approximately from 0.80 to 0.84 and is thus below the required value of 1 (Figure 29).

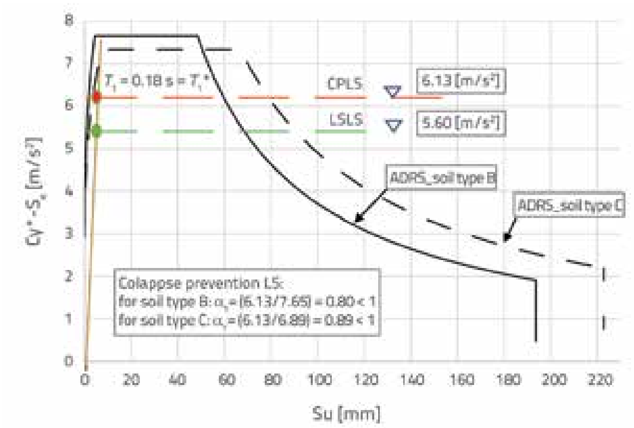


Figure 29. Fulfilment factor obtained by IDA analysis for Y-direction

The analyses show that the PBD and IDA result in similar conclusions: the structure of a typical building exposed to design earthquake would most probably withstand the earthquake in X-direction, where it assumes both bending and shear actions, while it would most probably fail in Y-direction, where it dominantly assumes shear actions. In this context, the term "collapse" implies opening of cracks of the order of magnitude of 5-10 mm, which in normal conditions should not lead to collapse of parts of a building, provided that all parts of the building are interconnected and that there are no independent elements that are fully detached from the main structure. The factor of fulfilment for buildings with the remaining service life of less than 40 years is relatively high and amounts to little less than 1.
Spectral values of ground acceleration at limit values of LS and CP, as obtained by IDA analysis, are presented in Table 11, where they are reduced to ground accelerations obtained by the analysis of seismic hazard.

Table 11. Ground accelerations at which limit state (performance limit state) of LS and CP is obtained

Direction	Type of soil	LS	CP
		a_g [m/s^2]	a_g [m/s^2]
X	B	1.98	2.43
	C	2.06	2.54
Y	B	1.87	2.04
	C	1.95	2.13

Depending on the agreement between the designer and investor (owner of the building) as to the desired level of protection, the values specified in Table 11 can be used as orientation for making decision on repair works. In the case under study, even if a higher level of protection is adopted by opting for LSLS as the desired limit state for the building, it can be concluded that here the analysed archetypal buildings, if located in areas with lower ground acceleration intensities of about 1.85 m/s^2, are not in danger and that they do not require improvement.
In addition to the analysis of walls in the direction of their spreading, it is also necessary to check the structure in the direction perpendicular to the spreading of walls. The so called *out of plane* analysis will be conducted in the following section.

10. Analysis of local mechanisms (*out of plane analysis*)

As a rule, all load bearing and non-load-bearing walls, connected with floor slabs that form a more or less rigid horizontal diaphragm, are capable of distributing seismic load onto the walls of the corresponding floor. At that, the distribution of forces onto walls is primarily dependent on the stiffness of the floor structure but also on the flexibility of walls themselves.
Here we will analyse wall behaviour in response to seismic action perpendicular to the direction of their spreading, considering also possible interaction between walls (link with connecting walls on the same floor that lie in perpendicular direction) and height wise interaction of connections with floor slabs and roof areas. Limit supporting conditions that could realistically occur in practical situations will be varied for all floors and two sides of the building (main facade and gable walls), the aim being to determine critical accelerations perpendicular to the wall that lead to wall failure by bending.

10.1. Analytical model

The analytical model for determination of critical lateral acceleration is based on the model presented in Figure 30.

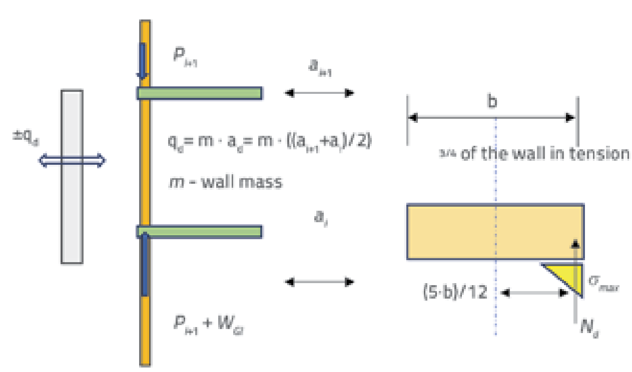


Figure 30. Wall model for determining critical horizontal acceleration, left, and distribution of stress along wall width, right

The critical acceleration value that causes wall collapse perpendicular to its plane is obtained by harmonisation of bending moment that acts on the wall as a consequence of horizontal acceleration of wall mass on the corresponding floor "i", with the limit moment of resistance resulting from the action of vertical forces in the wall, for critical value of wall width in tension of 3/4.

10.2. Models for support of external walls onto connecting elements

Non-linked and linked gable wall with perpendicular walls are shown in Figure 31 while than non-linked and linked facade walls with perpendicular connecting walls are shown in Figure 32. Results presented in Section 10.3.2 clearly show that in the case of the building analysed in this paper, the greatest threat comes from unsupported gable walls. The task is to verify their connection with other walls of the building and with floor structures. If it is concluded that that links and connections are insufficient, it would be necessary to conduct remedial work. As these are structural deficiencies, the improvement work should be relatively inexpensive.

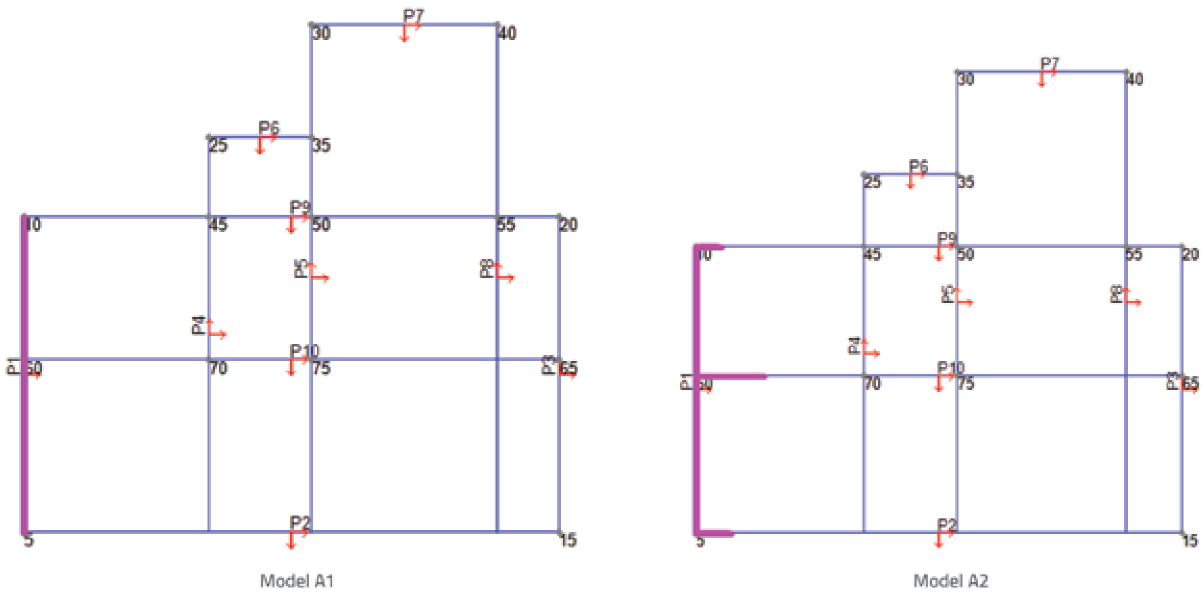


Figure 31. Gable wall non-linked (left) and linked with perpendicular walls (right)

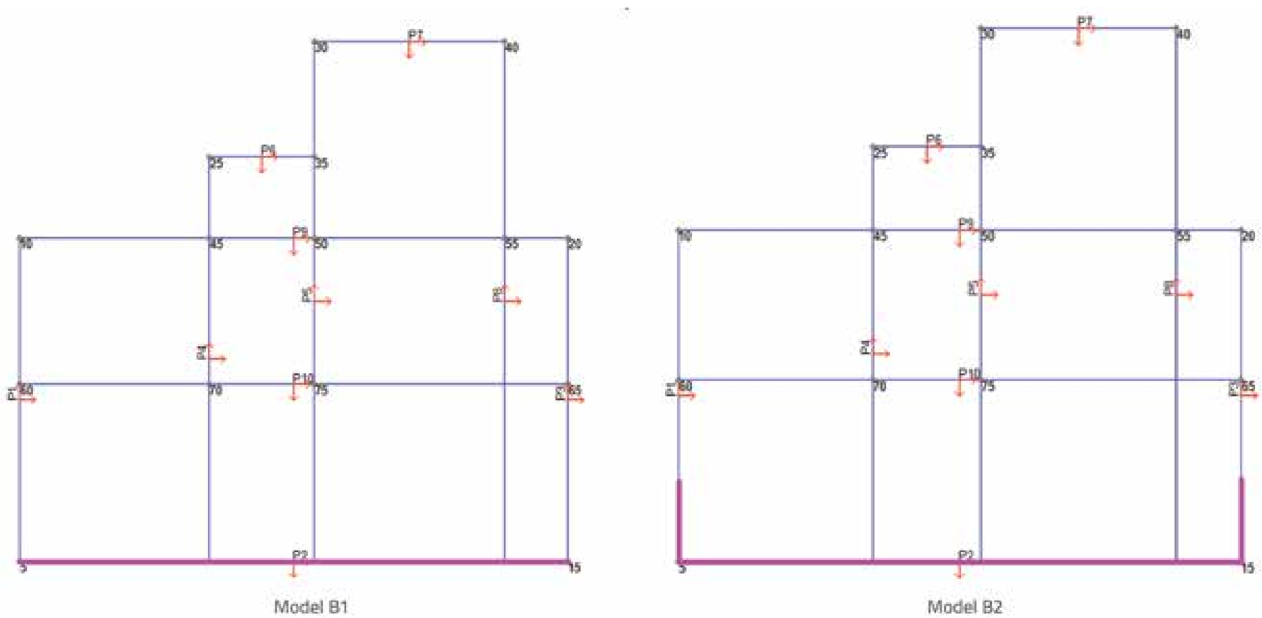
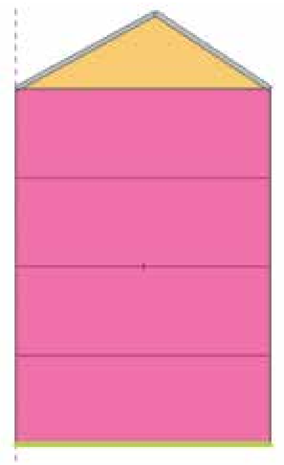
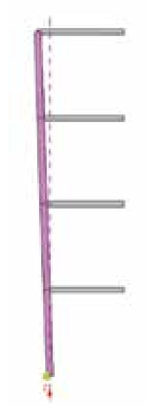
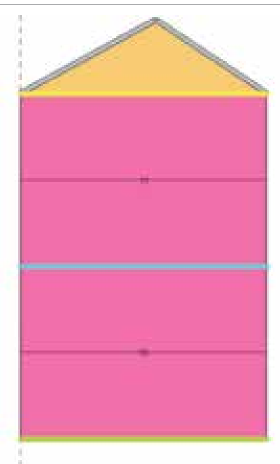
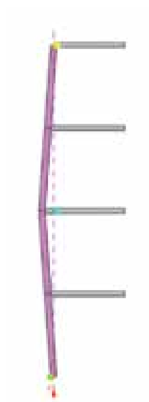
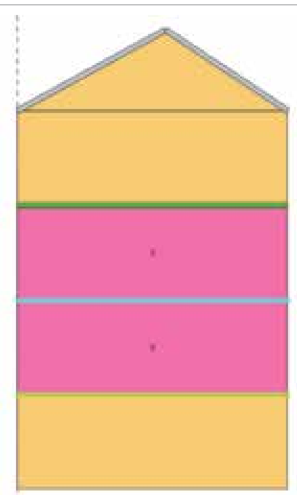
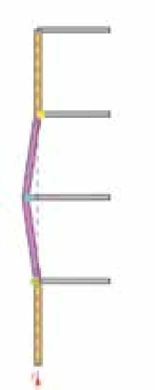
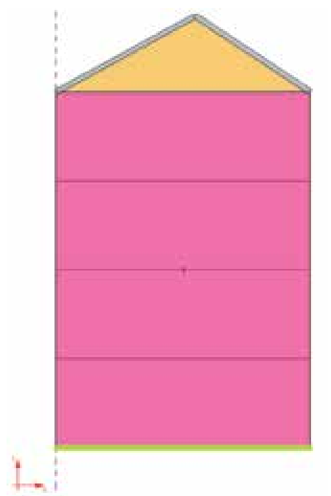
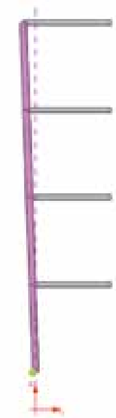
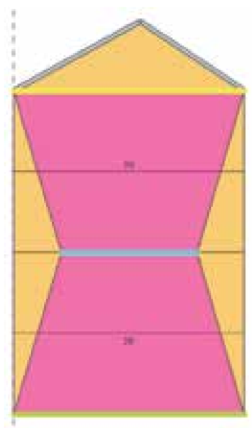
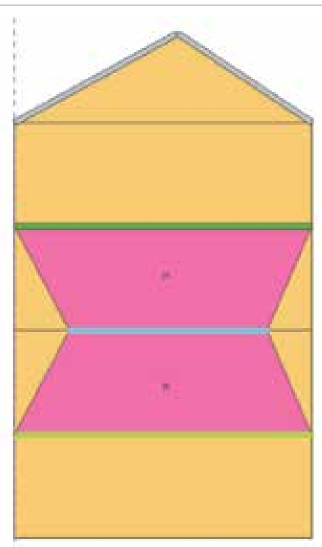
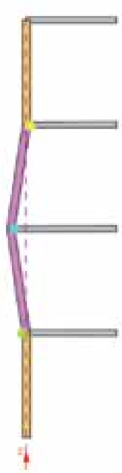


Figure 32. Facade walls non-linked (left) and linked with perpendicular connecting walls (right)

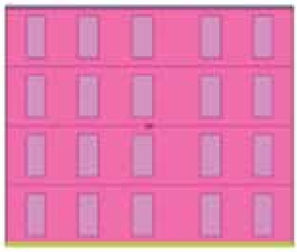
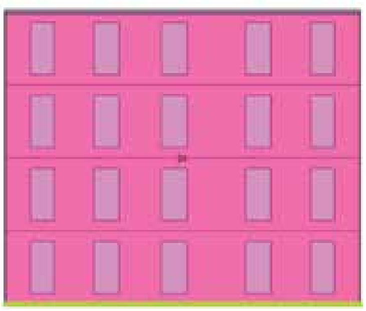
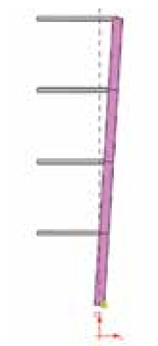
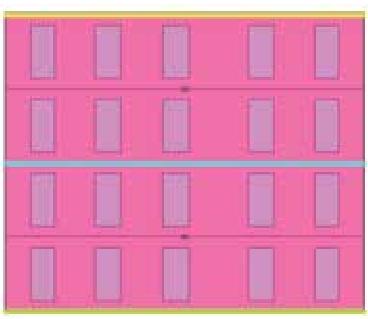
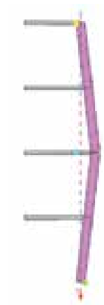
Tablica 12a. Analysis of walls perpendicular to their plane

Model	View of the wall with rotation hinge	Configuration along the height	Critical value of horizontal acceleration	Activated acceleration [m/s^2]	Fulfilment factor α_i
A1			0.27	2.70	0.1
			Gable **walls not linked** with floor slabs and walls from perpendicular direction **along the entire height** represent, during seismic action and even when it is much lower that than the design action, an acute threat to the life of persons that are in their vicinity at the moment the seismic action occurs. Rotation point is the foundation joint (joint at the level of foundations). *Improvement aimed at preventing overturning is obligatory.*		
A1			1.33	2.55	0.52
			Gable **walls not linked** with floor slabs and walls from perpendicular direction **along the entire height** represent, during seismic action and even when it is much lower than the design action, an acute threat to the life of persons that are in their vicinity at the moment the seismic action occurs. Rotation point is at the half of the building's height. *Improvement aimed at preventing formation of "protrusion" is obligatory..*		
A1			2.14	2.55	0.84
			Gable **walls not linked** with floor slabs and walls from perpendicular direction, spreading as an entity *along the height of two neighbouring floors*, represent in the case of seismic action a considerable danger to the life of persons located in their vicinity at the moment the seismic action occurs. Rotation point is at the level of floor structure. *Improvement aimed at preventing formation of "protrusion" at level of floor structure or floor structures is highly recommended.*		

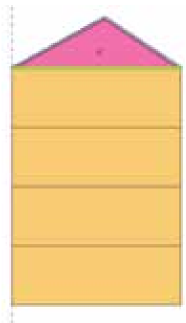
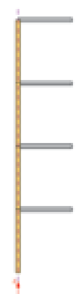
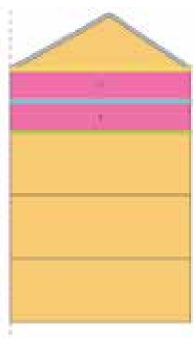
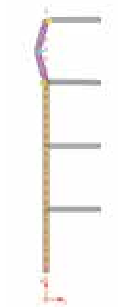
Tablica 12b. Analysis of walls perpendicular to their plane

Model	View of the wall with rotation hinge	Configuration along the height	Critical value of horizontal acceleration	Activated acceleration [m/s^2]	Fulfilment factor α_i
A2			0.59	2.56	0.23
			Despite being linked with walls from vertical direction, **if they are not linked with floor structures** along the entire height, **gable walls** constitute, in the case of an earthquake action - and even when such action is much weaker than the design action, an acute threat to life of persons that are in their vicinity at the moment the seismic action occurs. The rotation point is the foundation joint (joint at the level of foundations). *The improvement aimed at preventing overturning is obligatory..*		
A2			1.58	2.55	0.62
			Despite being linked with walls from vertical direction, **if they are not linked with floor structures** along the **entire height**, gable walls constitute, in the case of an earthquake action – even if it is much weaker compared to design action, an acute threat to life of persons that are in their vicinity at the moment the seismic action occurs. The rotation point is at the floor structure above the second floor. *The improvement aimed at preventing protrusions is obligatory.*		
A2			2.78	2.55	1.09
			Gable walls linked with walls from the vertical direction, if they are **linked at least with every other floor structure**, do not constitute a threat in case of a seismic action. *The fulfilment factor exceeds the required one.*		

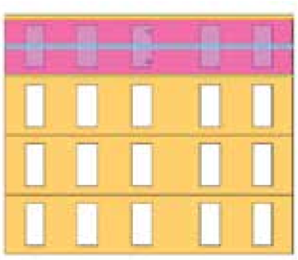
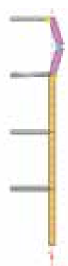
Tablica 12c. Analysis of walls perpendicular to their plane

Model	View of the wall with rotation hinge	Configuration along the height	Critical value of horizontal acceleration	Activated acceleration $[m/s^2]$	Fulfilment factor α_i
B1			1.29	2.52	0.19
			It is difficult to imagine that the facade walls could be unsupported, but it is not unrealistic to expect that the connection between joists and walls does not comply with good detailing practices. That is why it should be checked **how joists are inserted into walls** and how the connection was realised. In case of poor realisation, i.e. small length of joist leaning on walls, the here presented model of facade wall unconnected with other walls has a fulfilment factor that would require obligatory repair.		
B1			1.63	2.54	0.64
			See comment for the previous case.		
B1			4.07	2.54	1.6
			Facade wall insufficiently linked with the floor structure at "only one level", spreading over two floors, is not at risk of failure vertical to the wall plane *Fulfilment factor exceeds the required one.*		

Tablica 12d. Analysis of walls perpendicular to their plane

Model	View of the wall with rotation hinge	Configuration along the height	Critical value of horizontal acceleration	Activated acceleration [m/s²]	Fulfilment factor α_i
A1			1.29	2.52	0.51
			Gable **walls in loft ("swallows")** not linked with roofing along the entire height represent, during seismic action and even when such action is much weaker than the design action, an acute threat to the life of persons that are in their vicinity at the moment the seismic action occurs. Rotation point is at the height of the last floor structure. *Improvement aimed at preventing overturning is obligatory.*		
A1			2.23	2.56	0.87
			Gable walls of the fourth floor linked with floor slabs but not linked with walls in the orthogonal direction constitute, in case of a seismic event, a certain threat to life of persons that are in their vicinity at the moment the seismic action occurs. The rotation point is at the centre of the wall at the fourth floor. *The improvement aimed at preventing loss of local stability is highly recommended.*		

Tablica 12e. Analysis of walls perpendicular to their plane

Model	View of the wall with rotation hinge	Configuration along the height	Critical value of horizontal acceleration	Activated acceleration [m/s²]	Fulfilment factor α_i
B1			9.9	2.55	3.88
			Facade wall at the fourth floor, well connected with floor structures, exposed to seismic action with failure line at the mid-height, is not at risk in the case of a design seismic action. *Fulfilment factor exceeds the required one.*		

11. Strategies for improving response of masonry buildings to seismic action

The term "reinforcing" is currently used to denote improvement of structural response to earthquake action. It probably originates from inadequate translation of the English term *retrofitting* or German term *Ertüchtigung*. If the term were introduced in a semantically correct manner, then a more appropriate English word would be *strengthening*, and German one *Verstärkung*. The term "improvement" is consistently used instead of the term *strengthening* in this paper. This detail might seem irrelevant, but it targets the very essence of dynamic response and is a significant feature of the earthquake engineering philosophy. Strengthening would imply that there is only one strategy for improving response of existing buildings to earthquake action. However, it is generally not correct. Figure 33, [3] shows three completely different strategies for changing the building's response to dynamic excitation. Which strategy will in fact be used will depend on the problem that is being solved, but also on the building's position in space and on its treatment. For

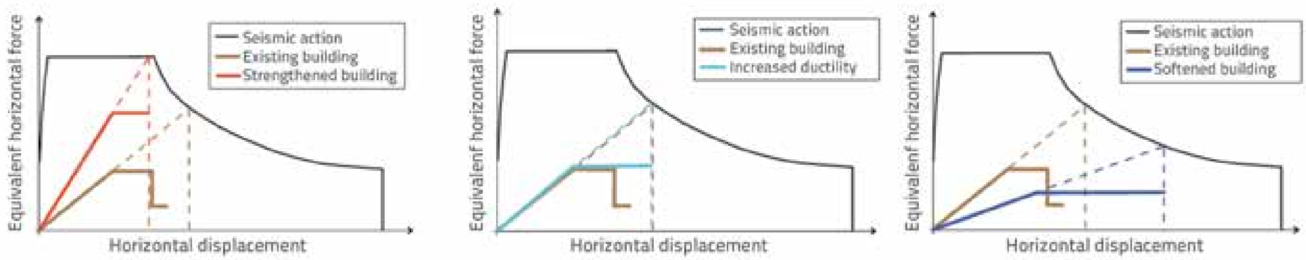


Figure 33. Three different strategies for improving structural response to seismic action: strengthening by increasing stiffness and bearing capacity (left), increase in ductility and deformation capacity (centre), and weakening (softening) of the structure e.g. by installing seismic isolators

instance, it is very likely that a zero category building protected as monument will not be repaired by "strengthening". Several examples have recently been realised worldwide that point to the efficiency of incorporation of, for instance, seismic isolators under foundations of masonry buildings. Thus, a structural "softening" method has been chosen as a means to improve response to seismic action.

11.1. Repair of gable walls at roofing level (structural repair)

The entire analysis presented in this text shows that gable walls are the riskiest element with a pronounced risk of overturning in the case of seismic action perpendicular to their plane,

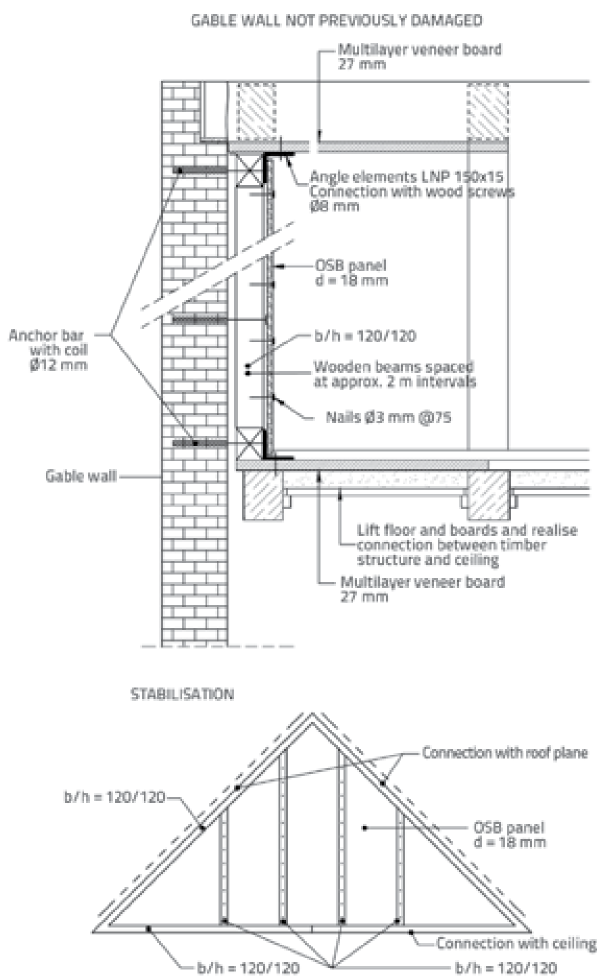


Figure 34. Proposal for stabilising gable wall that has not suffered damage in previous earthquakes, but reveals deficiencies with regard to connection with the floor structure and roof area

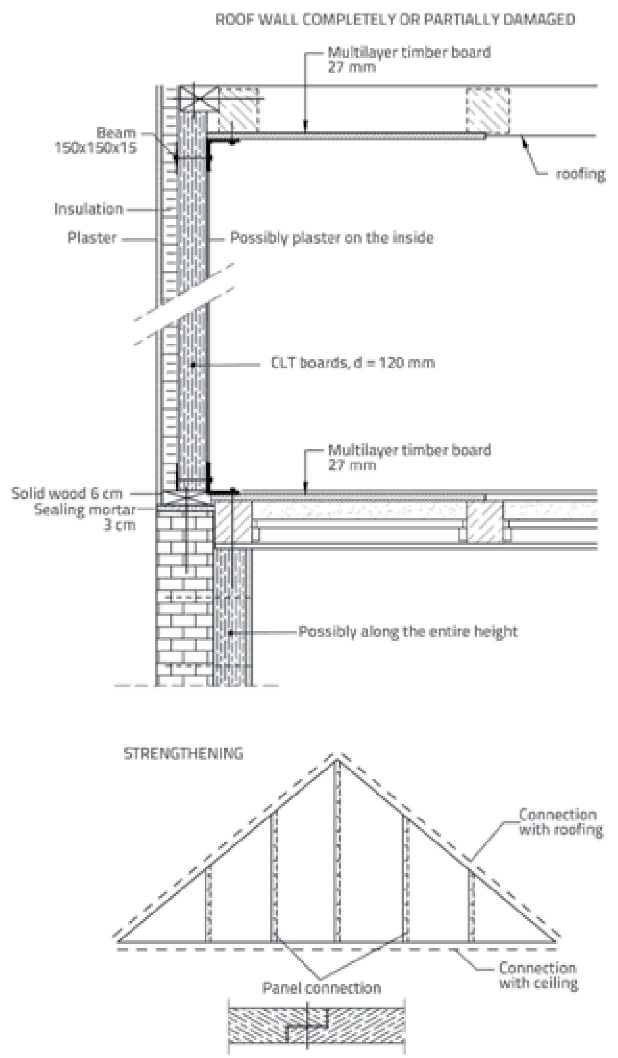


Figure 35. Proposal for improving gable wall with CLT-panels for walls that have been partly or fully damaged in previous earthquakes and must be removed

depending on the way the walls are connected to the rest of the building. Therefore, the possibility of repairing this kind of failure will first be considered below.

Repair measures involving the use of timber are presented in figures 34 and 35. This material is used because structural deficiencies can easily be corrected in timber, installation is performed using dry process, and the use is made of analogous materials that are already present in the structure. The objective of this intervention was to connect an unstable part with a stable part of the structure. If neither floor structure nor roof areas are stable in the sense that they represent some sort of a diaphragm, they must also be stiffened, for instance, by bracings or strengthened by subsequent installation of timber panels.

11.2. Remedy by strengthening, by increasing resistance to shear

An increase in shear resistance is probably the most logical first idea when considering improvement of a building's resistance to earthquake action. Such an increase in shear resistance is aimed at strengthening the structure, without increasing its stiffness. This most often involves introduction of new materials that eliminate deficiencies in shear resistance of walls in an additive manner, by increasing the existing shear resistance of walls through shear resistance of added elements. An increase in shear resistance is obtained by:

- shotcreting, i.e. by applying a shotcrete layer that is reinforced and connected with the existing wall by drilling and grouting of shear connectors
- adding CLT timber panels or other industrial panels, such as Kerto veneer panels
- subsequent application of FRP strips
- introducing energy into the system by wall stressing.

Shotcreting is a demanding process for which an appropriate machinery must be installed. It introduces moisture into the system, which is related to the rebound of shotcrete from the surface that is being improved. Its advantage is that shear resistance can be increased practically at will by adding reinforcement. Its disadvantage lies in practical application as a lot of space is needed for this relatively untidy (dirty) procedure, not very adequate in densely inhabited zones. From the static standpoint, its disadvantage is the transfer of forces into the foundation soil if the building is not partly buried into the ground and if does not have a basement.

Adding CLT panels is a more appropriate than shotcreting method. It is a completely dry process, all parts can be prepared in the workshop and connected at the place of assembly. CLT panels are used to connect elements made of the same material, timber to timber, and the extension in horizontal or vertical direction is relatively simple. Remedial work can be conducted from the inside (which is recommended) but also from the outside. An advantage of this type of remedial work is that it does not change the structure's dynamic response to excitation as materials such as panels of cross-glued boards or veneer have the deformation capacity similar to that of the walls. When this methodology is applied, there is no problem with introduction of eccentric force into the panel despite a single-layer application, as all seismic forces are assumed by timber panel, while walls only "lean" onto that panel. The deficiency of this remedial method is similar to that described for shotcrete, i.e. force transfer into the soil if the building is not fixed at the basement floor. In such case the transfer of vertical and horizontal forces to foundation soil must be realised by introducing new elements into the system, such as tensile elements (steel plates) and micropiles.

Subsequent application of FRP strips is much simpler compared to the first two methods involving increase in shear resistance, primarily as it is reduced to smaller areas. As a rule, the wall is prepared, smoothed down, cleaned by eliminating any traces of grease or impurities and, in the next step, epoxy glue is applied and carbon FRP strips 1.2 to 1.4 mm in thickness are placed. It can be stated from experience that this method for improving response, despite seeming quite simple, also implies several difficulties. The greatest one, in case the remedial work is carried out from inside, is the vertical and diagonal continuous guidance of FRP strip through the ceiling. If FRP strips are applied from one side only, the problem is also the eccentricity in the application of force to FRP strips. Just like in other methods, the problem arises with application of forces into foundation soil, when the lowest floor is not buried into ground.

The application of energy into the system by tensioning also has a clear physical background. As we have seen in the example analysed in this paper, the utilisation of walls with respect to normal forces is $(N_d/N_{Rd}) \approx 0.10$ for gable walls and for main load bearing walls. If at the level of utilisation of the wall compressive strength we start from $(f_k/\gamma_M) = (5.0/2.0) = 2.5$ N/mm² then stresses of aproximately 0.25 N/mm² occur at the joints of secondary walls, while stresses of 0.425 N/mm². occur on joints of main load bearing walls. This state of stress is represented with Mohr's circles in Figure 36. When the utilisation of walls in compression is increased to $(N_d/N_{Rd}) \approx 0.5$, then a maximum resistance to shear

Table 12. Design of prestressing in case normal force is introduced into the system

Wall	Stresses before strengthening [N/mm²]	Stresses after strengthening [N/mm²]	Difference [N/mm²]	Wall thickness [mm]	Force required for m' of wall [kN]	Bar selected (f_y/f_{tk})=(950/1050) [N/mm²] Ø [mm]/@cm	Final prestressing force [kN]
Gable	0.25	1.25	1.0	300	300	2 · Ø 26.5/270	2 · 405 = 810
Main wall	0.425	1.25	0.825	450	370	2 · Ø 32/320	2 · 591 = 1182

is obtained (Figure 6), without significant reduction in deformation capacity, while stresses at joint would then amount to σ_1 = 1.25 N/mm^2. It can be deduced from Mohr's circle that shear stress at joint of approximately τ_a = 0.54 N/mm would thus be activated, which would increase bearing capacity of walls to shear by more than three times. To introduce the stress of approximately σ_1 = 1.25 N/mm^2 into the system, it would be necessary to introduce the prestressing force as shown in Table 12. Swiss Gewi tendons are dimensioned in Table 12. The result are tendons measuring 26.5 and 32 mm in diameter, spaced at 270 and 320 cm intervals for gable walls and main load bearing walls, respectively. The effects of this intervention are schematically shown in Figure 37. The diagram showing strengthening by tendons and micropiles is given in Figure 38. Note: Micropiles shown in the diagram are needed in cases when tensile force must be introduced into the foundation soil, i.e. in case of elements assuming the bending load, but also in case of those assuming shear, if greater parts of the wall are exposed to bending moment due to lateral tensile forces.

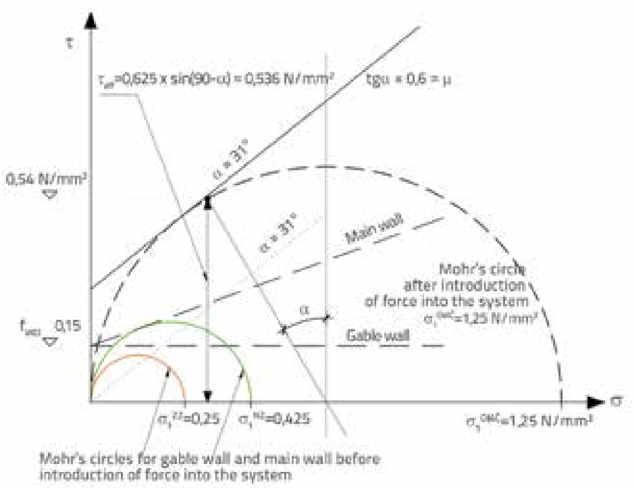


Figure 36. Stress at the joint of gable wall and load bearing wall before introduction of additional force into the system, represented by Mohr's circles, and stress after an increase in stress at the joint to approximately 50% of limit wall capacity = approximately 1.25

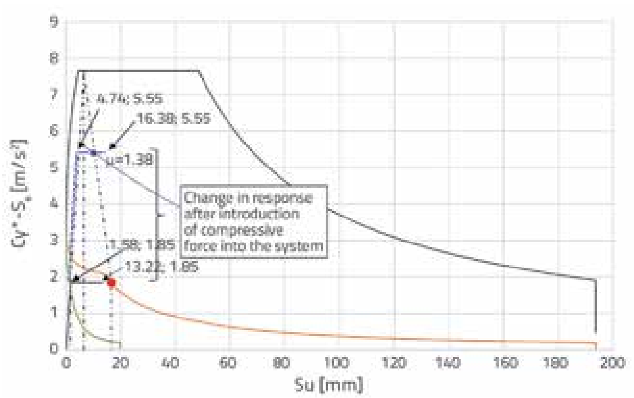


Figure 37. Example of change in response in case of design analysis No. 5 in Y-direction by introduction of prestressing force into the system

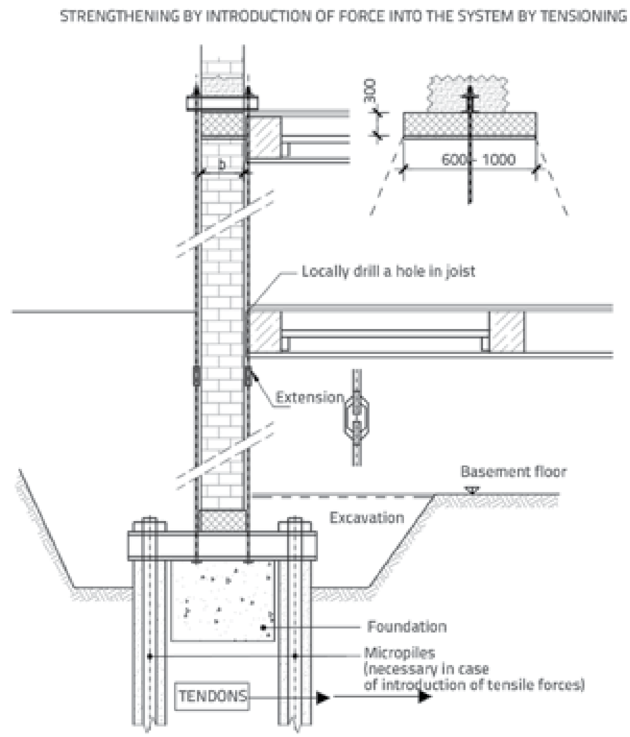


Figure 38. Example of introduction of force into the system by tendons and micropiles

11.3. Repair by increase in ductility

In the response mechanism, the repair by an increase in ductility can be regarded as some kind of serial relationship in which the first link in the chain are the walls and if this first link fails due to shear and/or bending, then seismic forces are assumed by the second element with a markedly different response, with longer period, with smaller induced forces, and with much greater deformation capacity. In other words, here we have a sort of "overlapping" in the sense of behaviour of two systems with different responses, i.e. one is stiff, with small displacement, and the other is flexible and would be activated only after the first stiff element (walls) fails. In this context, several possibilities can be considered:

- introduction of reinforced-concrete collector elements (tie beams) consisting of vertical and horizontal elements
- introduction of metal collector elements (tie beams) consisting of vertical and horizontal elements
- incorporation of vertical steel truss elements which, depending on their dimensions, assume shear or bending load
- incorporation of steel frames that dominantly assume bending load.

Although these repair models are theoretically possible, due to their scope and intensity of operations, they would be difficult to realize if buildings are inhabited.

11.4. Repair by structural "softening"

Sometimes it is appropriate to "soften" the structure by installing isolators to reduce the transfer of energy from soil to structure. For that purpose, entire structures can subsequently be placed, using highly demanding methods, onto seismic isolators in the form of vulcanised reinforced neoprene bearings that are often called rubber bearing isolators.

One of examples is the Los Angeles City Hall building built in 1928. This 32-storey building extending 138 m in height was renovated from 1998 to 2001 using base isolators. It would be option for permanent protection of zero category buildings significant for Croatian identity, such as Zagreb Cathedral, this would certainly be one of the methods worth considering.

In addition to masonry buildings, improvement of seismic response by means of seismic isolators could be appropriate in the case of facilities presenting high risk to environment such as liquefied gas tanks or highly significant infrastructure facilities such as some notable bridges for instance.

12. Conclusion

We hope that the analysis of a typical masonry building, as provided in this paper, has offered a detailed insight into the response of masonry structures, and that it has somehow demystified the level of threat imposed on such buildings by seismic action. It would be interesting to inspect the building itself, register the damage, and make qualitative comparison with results presented in this analytical study. In any case, the analysed building has greater deficiencies in the direction perpendicular to the spreading of main load bearing walls, and so the remedial activities are oriented toward connecting non-bearing walls with the remainder of the building, and can therefore be regarded as a building monolithization of a sort. Methods for changing structural response to seismic action are taxatively listed, and some of them are broadly outlined. The objective was to present this analysis to professional community, as soon as possible after the earthquake, as a contribution to the discussion about the direction in which renewal or repair activities should be focused. That is why this study should by no means be considered final or complete: it in fact represents a first step in opening the discussion about this theme, highly significant to the civil engineering and engineering profession in general.

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