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Mw 6.4 Petrinja earthquake in Croatia: Main earthquake parameters, impact on buildings and recommendation for their structural strengthening

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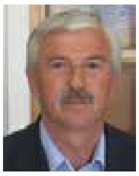

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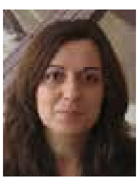

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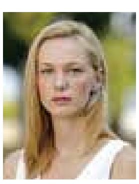

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Subject review

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Mw 6.4 Petrinja earthquake in Croatia: Main earthquake parameters, impact on buildings and recommendation for their structural strengthening

Strong Mw 6.4 Petrinja earthquake from 29.12.2020. took 7 lives and caused catastrophic damage in the Banovina area. The paper presents and analyses the most important earthquake parameters and highlights their importance in understanding the damage and demolition of buildings, as well as creating an optimal structure for their reconstruction. A contribution is made to the understanding of the complex mechanism of earthquake formation through the analysis of the stress-strain state in a rock mass during tectonic plate conflict. The causes of demolition and damage to buildings are explained by the combination of the properties of their structure, soil and the earthquake itself. Solutions for optimal structure of new buildings, as well as solutions for structural renovation of damaged buildings are proposed and described.

Key words:

Petrinja earthquake, main earthquake parameters, mechanism of earthquake formation, impact on buildings, reconstruction of buildings

Pregledni rad

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Potres Petrinja magnitude Mw 6.4: glavni parametri potresa, utjecaj na građevine i preporuka za njihovu obnovu

Razorni potres Petrinja (opći pojam za potrese na Banovini) od 29.12.2020. magnitude Mw 6.4 oduzeo je 7 ljudskih života i prouzročio katastrofalne štete na području Banovine. U radu su prikazani i analizirani najvažniji parametri potresa te naglašena njihova važnost u razumijevanju nastalih oštećenja i rušenja građevina te oblikovanju optimalne konstrukcije za njihovu obnovu. Dan je doprinos u razumijevanju složenog mehanizma nastanka potresa preko analize naponsko-deformacijskog stanja u stijenskom masivu tijekom međudjelovanja tektonskih ploča. Uzroci rušenja i oštećenja zgrada objašnjeni su kombiniranim utjecajem svojstava njihove konstrukcije, tla i samog potresa. Predložena su rješenja optimalne konstrukcije novih zgrada, kao i rješenja konstrukcijskog ojačanja oštećenih zgrada.

Ključne riječi:

potres Petrinja, glavni parametri potresa, mehanizam nastajanja potresa, utjecaj na građevine, obnova građevina

1. Introduction

Moderate earthquake of magnitude M_w 5.2 occurred on December 28, 2020 in the area of small Croatian towns Glina (12,000 inhabitants), Petrinja (25,000) and Sisak (50,000), located 60 km away from the capital city Zagreb. The main seismic event was preceded by a series of weaker earthquakes. The next day, on 29 December 2020, M_w 6.4 strong earthquake occurred in the same seismic area and was felt in the neighbouring countries and beyond. In the period between these two earthquakes, over a hundred of weaker earthquakes were felt in the area (Figure 1) [1]. The strong earthquake caused seven human casualties, and enormous material damage within the radius of over 50-60 km from the epicentre. Medium and weak earthquakes are still occurring in a wider epicentre area, and this seismic activity is not expected to stop any time soon.

A strong earthquake of M_w 5.8-6.0 was registered in 1909 in this seismic area, with the epicentre in Pokupsko [2]. Based on the analysis of this earthquake, the famous Croatian geophysicist Andrija Mohorovičić discovered a discontinuity between the Earth's crust and the mantle (the Mohorovičić discontinuity).

In order to determine, as reliably as possible, the causes of earthquake-provoked cave-in and damage of buildings, structural engineers should know at least the basic earthquake parameters, in addition to having proper knowledge of characteristics of structures and their interaction with soil. The most important parameters of M_w 6.4 Petrinja earthquake are therefore presented in Section 2. The stress-strain state of the rock mass before the collapse (generation of earthquake) due to tectonic plate interaction is also discussed. Earthquake effects on buildings in the area of Petrinja, Glina and Sisak are described in Section 3. Recommendations for the renovation and strengthening of the main and secondary structural elements of buildings are presented in Chapter 4. Finally, the main conclusions of the paper are presented in Section 5.

2. Some characteristics of the M_w 6.4 Petrinja earthquake

2.1. Basic seismotectonic characteristics of the earthquake epicentral area

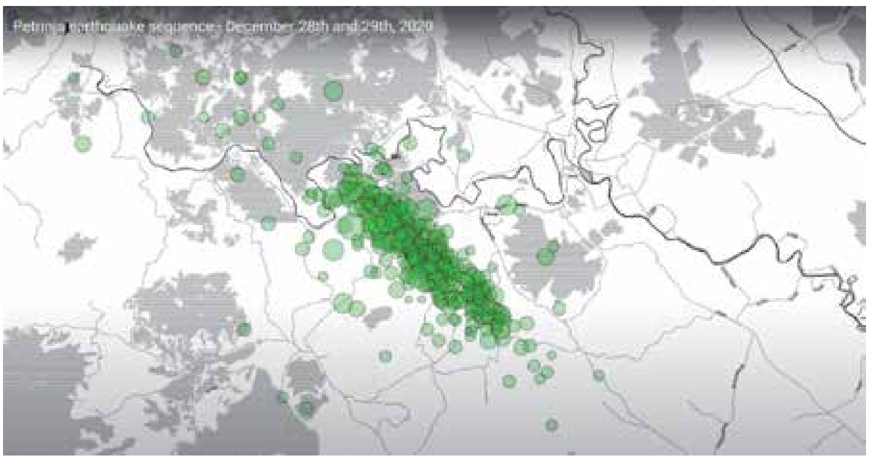


Figure 1. Earthquake epicentres in the area of Glina, Petrinja and Sisak in the period from 28 December 2020 to 29 December 2020 [1]

Figure 2. Major seismogenic faults in Croatia [3]

The tectonics of the Mediterranean area, located in a closed area between the African and Eurasian tectonic plates, is very complex and involves movement of multiple regional microplates and local small-scale plates [3]. Some of them sometimes move independently of Eurasian and African plates. The Adriatic microplate is a piece of the African plate, which also moves independently, pushing the mainland mountain range to the north. More details on this particular tectonics can be found in [4, 5]. The epicentral area of Petrinja earthquake follows the direction of the conflicting movement of the Dinarides to the north, and the opposite movement of the Pannonian microplate to the south. The fault is situated at the very border of the former Pannonian Sea. A more precise description of this issue can be found in [6]. According to the current knowledge [3], the main tectonic faults in Croatia are situated in the areas shown in Figure 2. The Petrinja earthquake occurred as a result of a shallow strike-slip faulting within the Eurasian plate, at the border with the opposing movement of the Dinaric and Pannonian microplates. The focal mechanism of the ground motions indicates that the rupture occurred at a nearly vertical fault, striking to the NE [6].

2.2. Basic parameters of 29 December 2020 earthquake

Main parameters of the M_w 6.4 Petrinja earthquake were estimated by several relevant institutions. According to the analysis of the United States Geological Survey (USGS) [7], the peak ground acceleration (PGA) in the earthquake epicentre was 114 % g and the peak ground velocity (PGV) 0.813 m/s, with the depth (hypocentre) at 13.5 km, which caused severe damage to buildings. The PGA was estimated to be at least 50 % g within a radius of 9 km from the epicentre (Figure 3a), the PGV was at least 0.2 m/s within a radius of 24 km (Figure 3b), and the PSA (peak spectral acceleration for T = 0.3 s; T = period of free oscillations of the single-degree-of-freedom model) amounted to at least 50 % g within a radius of 22 km (Figure 3c). Some other earthquake parameters are presented in Table 1, with an explanation according to Figure 4. The focal mechanism of the earthquake, as given in the USGS catalogue [7], is presented in Figure 5.

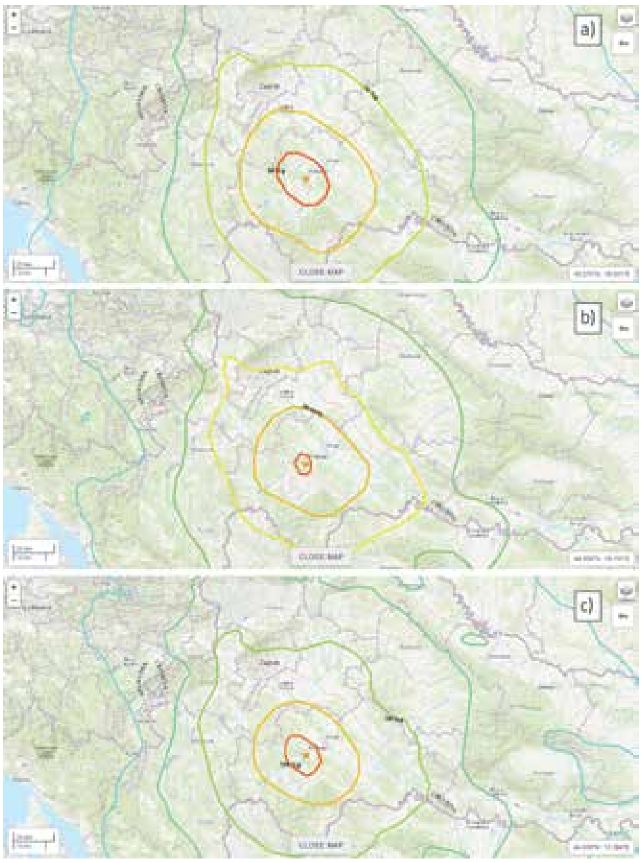


Figure 3. PGA, PGV and PSA parameters of the M_w 6.4 Petrinja earthquake [7]: a) Peak ground acceleration (PGA); b) Peak ground velocity (PGV); c) Peak spectral acceleration (PSA) for T = 0.3 s

The Petrinja earthquake was analysed by the Italian National Institute of Geophysics and Volcanology (INGV) using the SAR interferometry technique (InSAR) applied to satellite images obtained during the European Space Agency's (ESA) Sintel-1 mission conducted on 24 and 30 December 2020 [8]. The analysis of two successive images shows displacements on ground surface in the earthquake area, based on relative movement between faults during the seismic event. The result of this analysis is the map of the difference in the phase contribution between the two images – the rolled interferogram (Figure 6). It represents a displacement of approximately 2.8 cm along the satellites sightline. The earthquake epicentre is marked with yellow star [9]. The InSAR analysis was also used in [10].

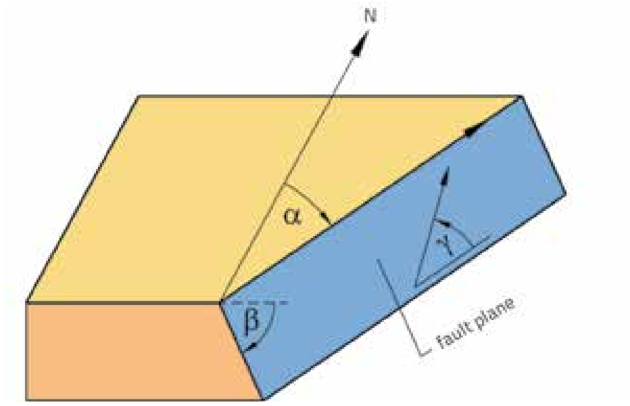


Figure 4. Fault position parameters [7]

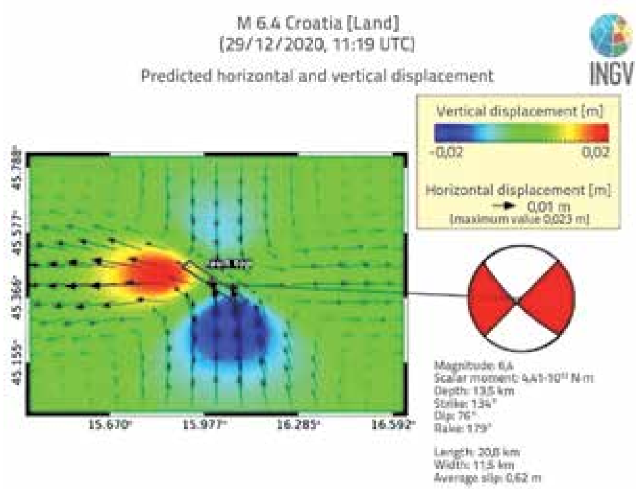


Figure 5. Focal mechanism of the M_w 6.4 Petrinja earthquake [7]

Table 1. Moment tensor [7]

Magnitude	6,36 Mw		
Depth	13,5 km		
Half duration	3,93 s		
Strike (α)	134⁰		
Dip (β)	76⁰		
Slip (γ)	179⁰		
Moment:			
Axis	M [Nm]	Pluge	Azimut
T	$4{,}444*10^{18}$	11⁰	90⁰
N	$-0{,}067*10^{18}$	76⁰	228⁰
P	$-4{,}377*10^{18}$	9⁰	358⁰

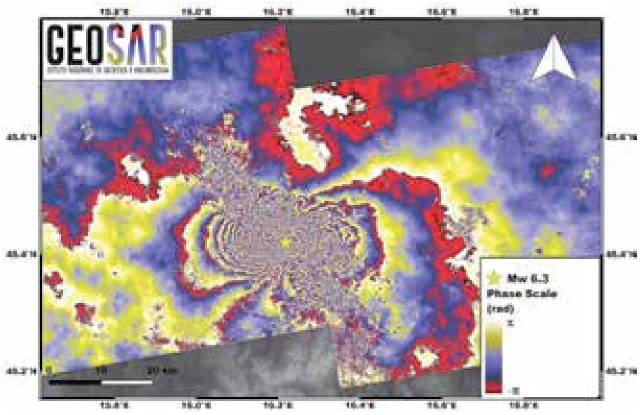


Figure 6. Rolled interferogram relating to the 29 December 2020 earthquake in Croatia [8]

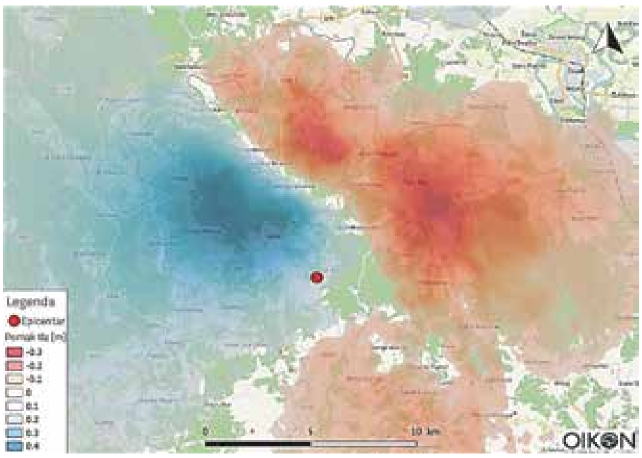


Figure 7. Map of vertical displacements along satellite sightlines on 29 December 2020 [9]

The transform of the interferogram into the surface displacement map is presented in Figure 7 [9]. It can be seen that the maximum displacement amounts to approximately 30 cm in the western part of epicentre, and to approximately -20 cm in the eastern part. That indicates that the surface failure was induced by a transcurrent fault mechanism (strike-slip), oriented approximately in the SE-NW direction. Such an opinion is not shared in [4, 10]. The interferometric data were used to identify fault parameters and slip distribution on the fault plane. An almost vertical fault, oriented SE-NW (β = 134°), was determined (β = 84°). The slip peak of approximately 3.5 km, located at the depth of approximately 4 km, was identified (Figure 8). It was established that the failure mechanism was approximately purely transcurrent (γ = 179°).

According to the analysis of the Department of Geophysics, Faculty of Science, University of Zagreb (PMF) [11], the earthquake magnitude was M_L 6.2, with the epicenter at about 4.5 km southwest of Petrinja (45.4002 N, 16.2187 E), at a depth of 11.5 km. The intensity of the earthquake was VIII-IX according to the EMS scale. The earthquake processing was performed based on the records of six accelerographs in Zagreb. Table 2 shows the accelerogram data obtained at four seismic stations from lower attitudes (similar to the earthquake epicentre), while the other two are not considered here (their altitude was up to 993 m and they deviate significantly from measured quantities due to site topography and local soil properties). According to PMF data, the average distance of the considered accelerographs from the epicentre of the earthquake is about 50 km. The average PGA values in Z, N, E directions were 0.046 g, 0.11 g, and 0.10 g, respectively. The average resultant PGA was approximately 0.16 g, which almost coincides with the PGA predicted by the USGS [7]. The average peak ground displacement (PGD) values in the Z, N, E directions were 0.76 cm, 2.06 cm, and 3.14 cm, respectively. The average resultant PGD was approximately 3.8 cm. Based on the above, it can be stated that the earthquake in the Zagreb area can be characterized as light-moderate, which is confirmed by the damage to city buildings.

The position of the epicentre of the devastating Petrinja earthquake differs depending on the source of information. Figure 9 shows the assumed earthquake epicentres according to USGS [7], INGV [8], and PMF [11], respectively. According to [7], the epicentre was about 3 km S-W of Petrinja (Križ Hrastovački) and, according to [8], it was about 3 km N-E of Petrinja (Nova Drenčina). According to [11], the epicentre was located about 4.5 km S-W of Petrinja (Strašnik). According to the level of damage to buildings in the vicinity of Petrinja, it is assumed that the earthquake epicentre position was determined the most accurately by PMF.

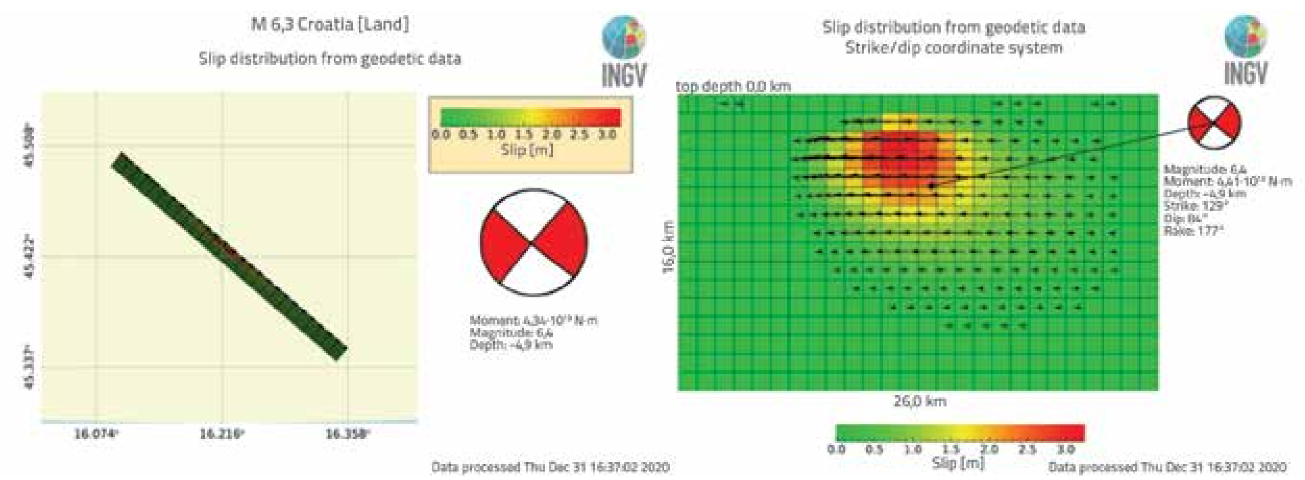


Figure 8. Fault position (left) and distribution of slip displacements on the fault plane (right) – by Lab GeoSAR-INGV [8]

Table 2. Measured data of accelerographs in Zagreb [11]

Seismic station	Distance to epicentre [km]	Altitude [m]	PGA [m/s²]				PGD [cm]			
			Z	N	E	R	Z	N	E	R
QARH	52.75	122.0	0.455	0.934	0.800	1.31	0.86	2.77	4.21	5.1
QUHS	50.78	179.0	0.427	1.243	0.958	1.62	0.86	2.30	2.87	3.8
QZAG	48.53	115.0	0.574	0.937	1.065	1.55	0.80	1.80	2.95	3.5
QGAJ	45.46	100.0	0.370	1.125	1.276	1.75	0.50	1.37	2.51	2.9
average	49.40	129.0	0.460	1.070	1.020	1.56	0.76	2.06	3.14	3.8

$R = (Z^2 + N^2 + E^2)^{1/2}$; Z = vertical, N = north, E = east

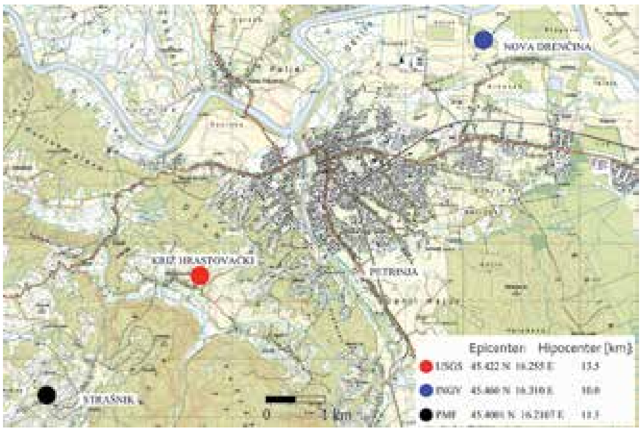


Figure 9. Some predicted epicentres of M_w 6.4 Petrinja earthquake

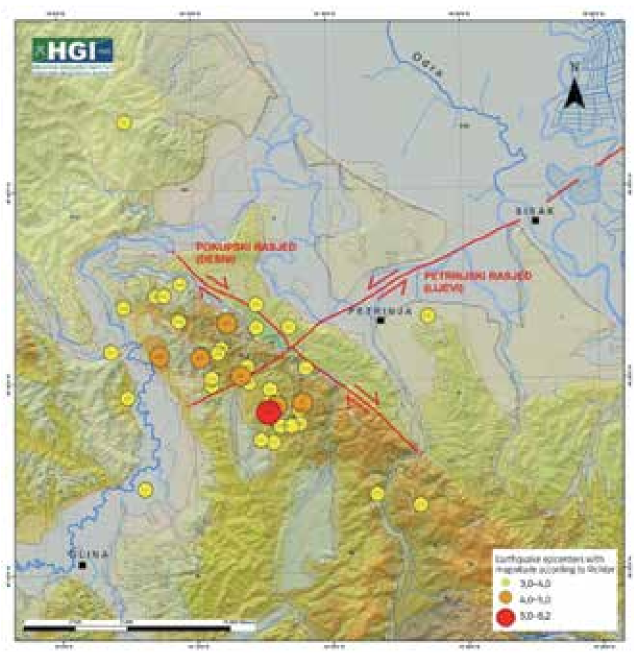


Figure 10. Main faults in Sisak-Petrinja-Glina area, with epicentres and magnitudes of strong earthquakes [12]

The wider epicentral area of the earthquake is located in the winding course of the Kupa River and its numerous tributary streams, in the zone with thick layers of soft and moist sand and silt prone to liquefaction. According to [7], a significant impact of liquefaction can be predicted in an area of about 400 km².

According to geological analyses of the Croatian Geological Institute (HGI) [12], the earthquake activated a system of faults in the underground of the wider area of Sisak, Petrinja and Glina. The earthquake was generated at the intersection of longitudinal and transverse faults at the NE edge of the Dinarides (Figure 10). Both fault systems consist of multiple fault-slip faults. One is better known as the right Pokupsko fault, and the other is the lesser known left Petrinja fault. This is due to the compression stress in the N-S direction at the contact surface of the tectonic plates. Both faults are shown in the Croatian geological map [13]. Considering the demolition and severe damage of houses in the vicinity of Glina, the left Petrinja fault probably extends to Glina [4, 10]. Various surface manifestations of this event were registered in the wider epicentral area along the fault lines, such as open cracks and paraclases, fluid spills, sand volcanos due to liquefaction, deformation of soil surface and linear infrastructure facilities, etc. [12].

Unfortunately, there were no accelerographs near the earthquake epicentre that would record duration and other important parameters of soil acceleration. According to numerous camera recordings, the total duration of the earthquake near the epicentre was approximately 10 s, with the dominant ground displacements lasting probably less than approximately 8 s. According to the USGS [7], the so-called half-duration of the earthquake was 3.93 s. It was therefore the so-called impact earthquake, with high peak ground acceleration (PGA) and a very small predominant period of oscillation. These earthquake types are particularly unfavourable for rigid structures, which are dominant – mostly as low-rise buildings - in the wider epicentre area. Spreading from the hypocentre through different layers of soil, the amplitude of the earthquake was damped and the predominant period of oscillation increased. Thus, in Zagreb, which is also located on thick deposits of soft soil, the average PGA dropped to 1.56 m/s² (Table 2), and the total duration of the earthquake increased to about 20 - 30 s.

2.3. Stress-strain analysis of rock failure in focal zone of the earthquake

The content of this subsection is not related to the Petrinja earthquake itself but is generally applicable, i.e., an attempt is made to explain the occurrence and spread of earthquakes in the rock mass due to movement of tectonic plates, by considering the mechanism of its failure resulting from exceedance of strength, when the released deformation energy is largely converted into kinetic energy. Statements made in this subsection mostly reflect the opinion of the authors.

The lithosphere, i.e., the Earth's rock crust and the upper mantle, is not a continuous homogeneous and isotropic structure, but is divided into several large, dozens of smaller, and many small tectonic units, formed of diverse occurrences of rock mass. Due to complex millennial formation and constant changes, numerous discontinuities, caverns, fissures, broken zones, faults and numerous other effects and anomalies, are present in the rock mass, resulting in pronounced anisotropy and reduced strength in certain zones and directions.

In general, the bulk density and strength of the rock mass increase with the depth below the earth's surface. For the basic geostatic/gravitational state, the rock mass is in stable equilibrium, with varying sizes of normal and shear stresses at each point in space. The increase in compressive and shear strength of the rock with the depth is due to the influence of the lateral compressive stress, i.e., to the prevention of lateral deformation that causes the spatial stress state. The strength of a rock in triaxial pressure can be several times higher compared to its uniaxial compressive strength. The shear strength of the rock increases rapidly with an increase in compressive stress perpendicular to the shear surface (up to the level close to compressive failure). The tensile strength of a homogeneous rock is quite high and for some types of rocks it is approximately one-tenth to one-fifth of the compressive strength. However, due to numerous discontinuities/ruptures and anomalies in the rock mass, the tensile strength is almost negligible. In general, the compressive strength of a rock mass is significantly higher than the shear strength, while the tensile strength is negligible.

The change of the basic equilibrium geostatic state in the rock mass is influenced by numerous factors, which change the initial stress-strain state. Thus, for example, the movement of tectonic plates over the "plastic" Earth's mantle affects in various ways the previous stress-strain state of the rock mass. Namely, this movement generates additional forces that increase the previous normal and shear stresses in the rock mass. Where and when the failure will occur in the affected area will depend on the total level of stress/strain, strength and other parameters of the rock mass. If a rupture occurs, a huge amount of accumulated internal strain energy is immediately released, i.e., the accumulated potential energy is instantly converted into the mechanical kinetic energy and into the thermal and sound energy. The released kinetic energy is manifested in a strong vibration of the rock mass in the focal space, and in propagation of oscillatory waves/displacements in all directions from the focus (hypocentre). This natural phenomenon is called an earthquake.

Failure of rock mass during an earthquake most often occurs due to exceedance of shear strength, which was in fact the case with the 29 December 2020 Petrinja earthquake. The reason for this is the significantly lower shear strength of the rock mass compared to the compressive strength. A simplified schematic presentation of possible stress state in the fault plane of the rock mass due to some conflicting motion of tectonic plates is presented in Figure 11. The shear stress τ occurs in the assumed fault plane, and the compressive stress σ occurs perpendicular to this plane.

The schematic relation of the rock mass shear strength in the fault plane τ_m to the lateral compressive stress σ is shown in Figure 12a. The peak shear strength (point B) is at the normal compressive stress σ_B. Rock fracture due to compressive stress (point D) occurs by the exceedance of the compressive strength of rock perpendicular to the fault f_m. The τ - γ relation (shown as an idealized elastoplastic relation), where τ is the shear stress and γ is the shear strain, also depends on the level of lateral compressive stress, which affects magnitude of the initial shear modulus and the shear deformation at fracture. The area below the τ - γ diagram in Figure 12b represents the shear strain energy per unit volume. For the same shear strength (points A and C on the curve in Figure 12a), the limit shear strain is much higher at higher normal stress, i.e., the accumulated shear strain energy is much higher (greater release of strain energy - stronger earthquake).

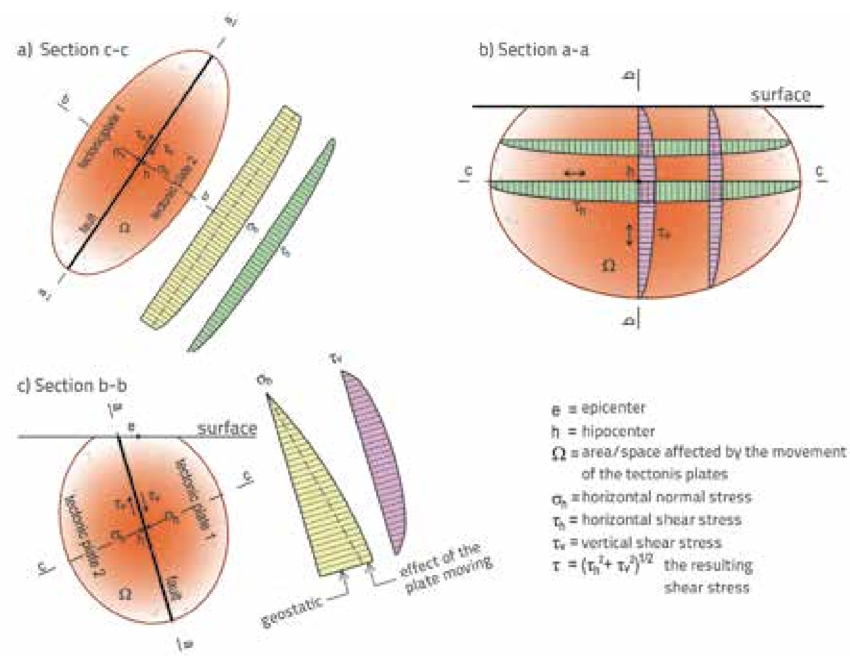
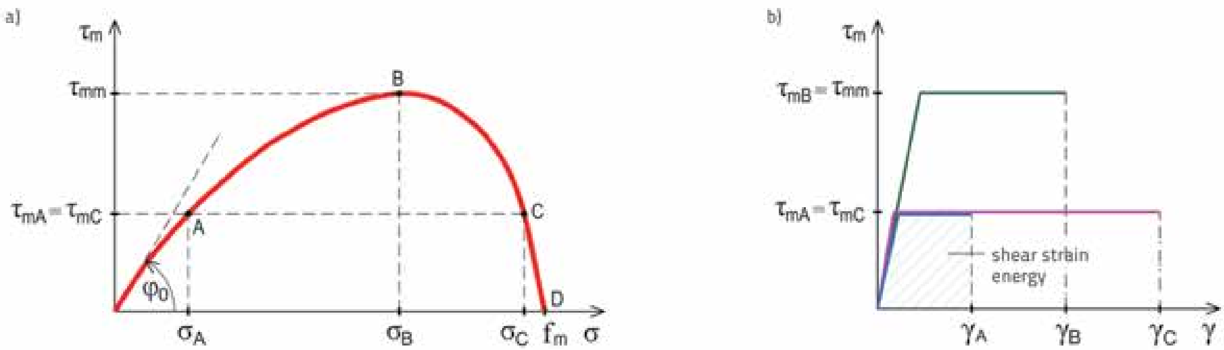


Figure 11. Simplified schematic representation of rock mass stress state in fault plane affected by conflicting motion of tectonic plates



Slika 12. a) Shematska ovisnost posmične čvrstoće stijenskog masiva u ravnini rasjeda od poprečnog tlačnog naprezanja; b) Idealizirana veza posmično naprezanje - posmična deformacija

The plan view of a part of two tectonic plates with opposite longitudinal displacement at contact is shown in Figure 13. There is a mutual hooking and encroachment of the rock mass at the contact plane, with generation of significant shear stresses. Assuming equal rock strength and equal shear displacements in Focal zone 1 and Focal zone 2, if the shear strength of the rock mass is exceeded, shear failure will first occur between points A and B in Focal zone 1 because the shear capacity here is significantly lower than between points C and D in Focal zone 2. Therefore, it can be assumed that an earthquake will occur first in Focal zone 1 if the rock mass shear capacity between points A and B is exceeded. Then the stresses in Focal zone 2 will increase immediately. Due to the continued movement of the tectonic plates, there will be a further increase in the previous stresses in Focal zone 2. If the shear capacity of the rock mass is exhausted in the coming time, there will also be an earthquake in Focal zone 2. It will be far stronger than in Focal zone 1 due to the accumulated higher strain energy in that space. Such a sequence can explain the resulting series of earthquakes in the local tectonic zone over a longer time period. Seismic activity in the local seismic area will calm down in the long run when the stress level in the rock mass is sufficiently reduced in relation to its bearing capacity, i.e., when a long-term stable stress balance is established.

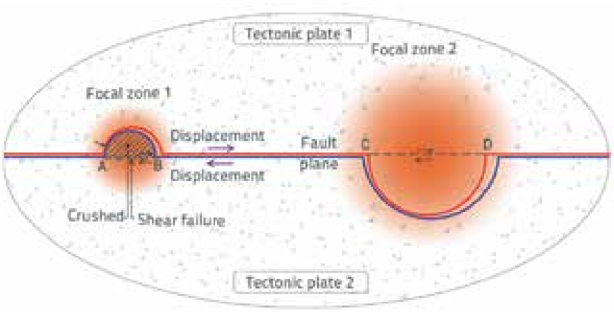


Figure 13. Simplified plan view of successive earthquake occurrence at opposite displacements of tectonic plates along fault (2D view)

An earthquake can also occur away from the contact plane of tectonic plates, even when they are in frontal pressure conflict (Figure 14) when there is an anomaly in the rock mass in that area, i.e. when there is a rock mass discontinuity or a zone of reduced strength. In effect, it is then that the tangential component F_t of the frontal force F within the discontinuity can cause rock mass failure and initiate an earthquake.

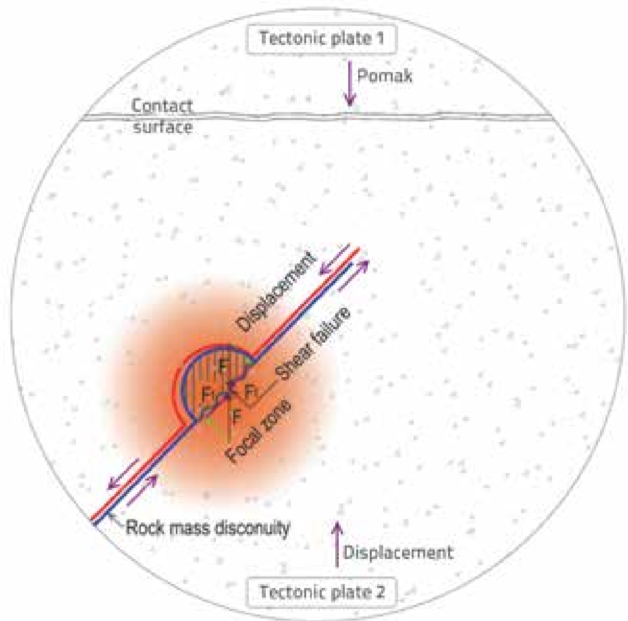


Figure 14. Simplified plan view of earthquake occurrence away from contact of tectonic plates

Figure 15 shows a simplified plan view of a frontal collision of two tectonic plates where, due to anomalies in the rock massif and exceedance of its bearing capacity, faults can occur in both tectonic plates, and even in different directions. Due to the concentration of shear-tensile-compressive stresses lateral to the main fault, and the exceedance of bearing capacity, minor earthquakes (local faults) occur regularly on both sides of the main fault. These minor earthquakes, as well as smaller earthquakes in the direction of the main fault, usually occur in smaller numbers and for a shorter time before the main fault, and for a longer time and in greater numbers after it. It is a part of the often-longer time process of balancing the disturbed stress-strain stress state in a certain area of rock mass due to moving and conflict of tectonic plates. Theoretically, it is unquestionable that the redistribution of stress due to earthquake (especially stronger one) affects the movement of the soil and changes the existing stress state in the wider epicentral area, which can affect the occurrence of new earthquakes in a shorter or longer period away from epicentre of the previous earthquake. Unfortunately, it is only a matter of time before this natural phenomenon will occur again at the same or a nearby locality.

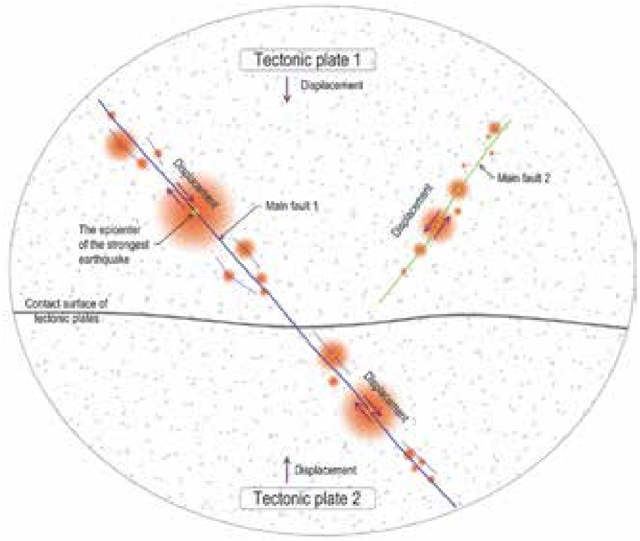


Figure 15. Simplified plan view of the area of frontal pressure conflict of two tectonic plates and possible formation of faults along the zones of exceeded bearing capacity of the rock mass

Figure 16 shows possible formation of faults and earthquakes due to collision of two tectonic plates at an oblique contact.

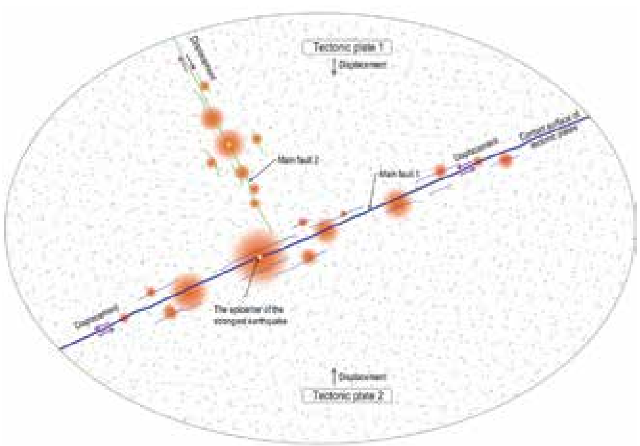


Figure 16. Simplified plan view of earthquake formation at collision of two tectonic plates at oblique contact

The amount of energy released during an earthquake, which corresponds to the released strain energy of rock mass during failure, depends on the depth of the focal zone, the level of exceedance of strength/strain, type of failure, soil volume affected by fracture, and other parameters. As a rule, earthquakes with shallow hypocentres are of smaller acceleration amplitude due to lower gravitational stresses and total stresses during soil failure, and due to smaller soil volume affected by earthquake. Earthquakes with a deeper hypocentre are generally stronger for opposite reasons. It should be highlighted that the impact of earthquakes on people and buildings decreases with the distance from the hypocentre, as well as from the epicentre, because earthquake waves (ground movements) are increasingly damped (especially when passing through softer soils and discontinuities). In practical terms, it is important to know how the earthquake manifests itself on the surface of the Earth. In the light of the above, it can be stated that very strong earthquakes with a very deep hypocentre can be less devastating to people and buildings than weaker earthquakes with a shallower hypocentre.

3. Impact of Petrinja earthquake on buildings

In addition to the loss of seven lives and dozens of injured, thousands of peoples were left homeless. The greatest levels of destruction were registered in the area of Petrinja, Glina and Sisak. Damage to approximately 50,000 buildings has been reported so far. A lot of damage also occurred in the vicinity of Zagreb and Karlovac, as well as in Bosnia and Herzegovina and Slovenia. Figure 17 shows a satellite image of damaged buildings in the area of Petrinja, Glina and Sisak [8]. The actual extent of damage is much greater because the damage inside the buildings, and other damage that could not be registered by satellite, is not included.

As Glina, Petrinja and Sisak are close to the epicentre of the earthquake, and the hypocentre is at a depth of about 11.5 km [11], the impact of the vertical component of the earthquake acceleration/force in this area was significant. Figure 18 schematically shows a vertical section through the earth's crust in the direction of Glina-Petrinja-Sisak, with a simplified representation of geological characteristics of the rock mass according to [13]. This figure shows only radial P-waves of the earthquake, i.e., the radial accelerations a. It should be noted that horizontal S-waves appear on all discontinuity planes, including the ground surface. Their influence on the amplitude of the horizontal soil acceleration at the ground surface is generally dominant, especially for more distant locations and soft soils. The agglomerations in question dominantly lie on deep soft and moist soil deposits, in the area traversed by numerous rivers and their tributaries. Therefore, the impact of additional/transferred acceleration and liquefaction on the ground surface was very significant.

In the narrower epicentral area, the calculated acceleration of ground during the earthquake amounted to approximately 0.5 g [7], which provoked seismic forces of approximately 50 % of the weight. Compressive failure of the material of weaker compressive strength (soil under foundations, masonry in walls and columns, etc.) occurred at the moment of the earthquake-generated significant increase in gravitational acceleration (compressive stress). In locations with highly saturated sandy soils, water and sand erupted on the soil surface.

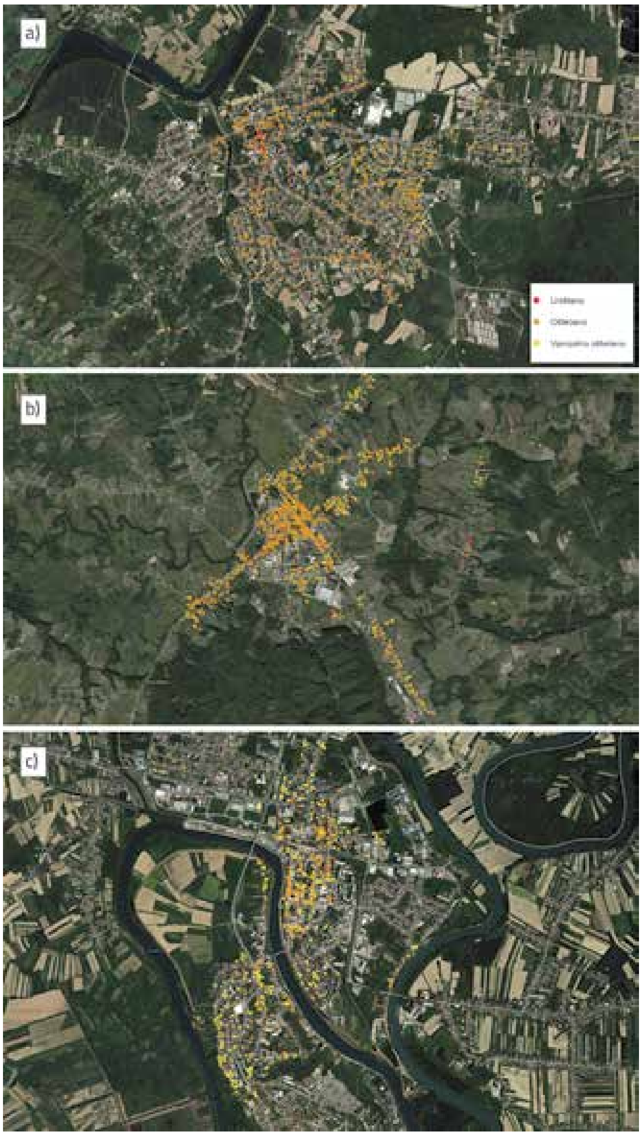


Figure 17. Satellite image of damage to buildings in the area of Petrinja, Glina, and Sisak[14]: a) Petrinja; b) Glina; c) Sisak

At the moment of significant reduction of vertical compressive stress by earthquake, a horizontal shear failure of the material (soil below foundation, masonry) occurred due to the simultaneous effect of horizontal earthquake accelerations (shear forces) and reduced material shear strength. According to [7] (Figure 3c), PSA values were up to 100 % g for T = 0.3 s in the narrow epicentral region of the earthquake. As the buildings in the area under study are predominantly low with a much smaller basic oscillation period, their PSA values are even higher. The effect of shear seismic forces is particularly pronounced in masonry buildings with large openings and especially at the building top where the shear strength of the material is lower due to lower vertical compressive stress. In relation to the total number of buildings, most buildings suffered the heaviest damage in Petrinja, and then in Glina and Sisak. The level of damage to buildings depends on a number of factors, such as:

- Earthquake characteristics (strength, hypocentre, PGA, duration, spectral displacement, velocity and acceleration values, predominant acceleration period, etc.),
- Properties of soil through which seismic waves propagate, including those at the site of the building,
- Characteristics of the building (location in relation to the epicentre, dynamic characteristics - stiffness and mass, type of structure and material, foundations, capacity to dissipate seismic energy, etc.).

The heaviest seismic damage in Petrinja, especially in its historic centre, is explained by the following facts:

- Small distance from the epicentre of the earthquake (about 4 km) - large earthquake forces
- Location on thick layers of soft soil
- In addition to the effect of the vertical component of acceleration, the effect of its horizontal component was also significant
- Significant influence of additional seismic forces due to thick deposits of saturated soil
- Significant proportion of buildings from the period of the Austro-Hungarian monarchy, which is not favourable for seismic areas (massive masonry walls made of small bricks, flexible floor structures, mostly high floor heights and high steep roofs, often insufficient quality of bricks and mortar, inadequate interconnection of floor structures and walls, inadequate wall-to-wall connection, high unsupported gable walls, numerous larger openings in external walls, often inadequate wooden roof structures without stiffeners in the plane of the roof, and horizontal pressure on the facade walls for vertical loads only, high massive chimneys, numerous massive decorative elements on the cornice and along the facade height, etc.).

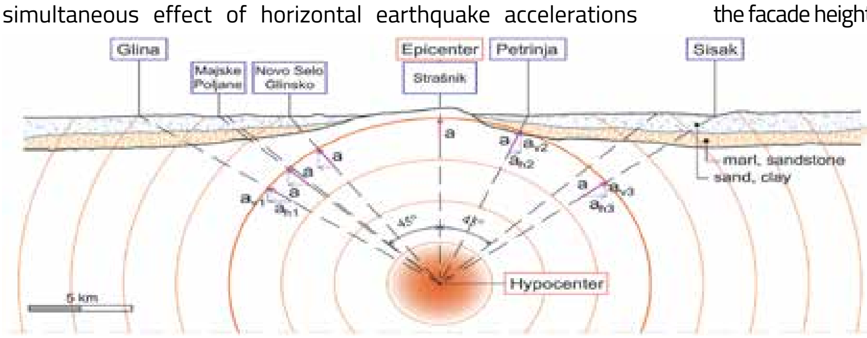


Figure 18. Schematic vertical section through the Earth's crust in the direction of Glina-Petrinja-Sisak

Flexible gable walls and wooden roof structures of many buildings either caved-in or were severely damaged (see some examples in Figure 19). At that, these gable walls were often free-standing and of great height, and the horizontal pressure of rafters for permanent load was often directly transmitted via the wooden overhang laterally to the plane of the facade walls (not to the connecting beams). An example where an inadequate wooden roof structure caused horizontal lateral displacement of the wall is shown in Figure 19e.

Characteristic damage to masonry walls with openings is visible in buildings shown in Figure 20, as well as in some buildings in Figure 19. Typical cross cracks in the columns between the openings in the wall, as well as in lintels, are due to the influence of bending and shear arising from earthquake action, i.e., they are due to the exceedance of principal tensile stresses (especially in the horizontal direction, where compressive stresses are minimal). High vertical seismic acceleration results in: (i) a decrease in vertical compressive stress and exceedance of shear strength of the material when force is opposite to gravity and (ii) an increase in compressive stress and exceedance of compressive strength of the material when force acts in the direction of gravity.

A significant embankment subsidence was registered along the left bank of the river Petrinjčica, near the bathing facility. It is due to the influence of seismic acceleration, especially its vertical component, on the thick layer of saturated soft soil, and to soil liquefaction (Figure 21a). Traces of horizontal displacement of more than 10 cm are visible in the asphalt surfacing on top of the embankment. Such ground displacements are also visible next to a nearby two-storey bathing facility situated along the Kupa River (Figure 21 b), which suffered severe damage (cracks) at the base and on top of reinforced concrete pillars of the first floor. In addition to large seismic forces, the unfavourable flexible structure of the building at the ground floor also contributed to this damage. Traces of eruption of sand on the ground surface due to liquefaction are visible along the bathing facility.

A slightly smaller damage to buildings in Glina and Sisak, compared to Petrinja, is explained by the fact that they are farther away from the epicentre, and that they were exposed to a lower seismic acceleration value, especially its vertical component (i.e. greater impact of the horizontal component of acceleration was registered). The average height and weight of buildings in Glina is

Figure 19. Buildings in Petrinja with severe damage to gable walls and wooden roof

Figure 20. Buildings in Petrinja with typical damage to load-bearing façade walls

Figure 21. a) Subsidence of earth embankment at the approach to the bathing facility and (b) bathing facility in Petrinja along the Kupa River (b)

slightly lower, while these values are probably slightly higher in Sisak. Furthermore, the buildings in Sisak are on an average newer and probably of a somewhat higher quality. Housing construction in Petrinja, Glina, and Sisak is of urban type, dominantly involving row buildings. In general terms, the building damage pattern in these cities is quite similar. In addition to numerous severely damaged and collapsed buildings in Petrinja, Glina, and Sisak, there are also many severely damaged and collapsed family houses and smaller farm buildings in villages within a radius of approximately 25 km from the epicentre of the earthquake. These are predominantly two-storey buildings with masonry walls that are relatively stiff in both directions but, unfortunately, they are in most cases inadequately built (especially farm buildings). Such poor quality of construction reflects the low standard of living of local habitants, who mainly make their living from agriculture and cattle breeding.

The indicator of earthquake strength in the affected area is most clearly manifested in the level of damage to new two-storey residential houses built after the Homeland War. They are made of small, lesser quality blocks of baked clay, bound with low quality lime-cement mortar with little cement. As a rule, the ground floor of such houses is raised above the ground. It is assumed that the houses are founded on thin concrete slabs (below which is a lightly compacted riprap), or on strip foundations. There are no vertical tie beams at the ground floor walls, while there are only few of such columns on the first floor and at gables. In some demolished buildings, such columns were devoid of reinforcement. Floor structures are probably made of semi-prefabricated elements or as thin reinforced concrete slabs. Unplastered external walls reveal that horizontal tie beams are of small height or are simply not present (the surface is covered with thermal insulation). The roof structure is wooden. Damage to houses in the village of Strašnik, which is situated in the epicentre of the earthquake [11], is lower than in nearby villages in the narrower epicentral area of the earthquake. This is explained by the fact that this village is located on a small hill, probably with a thin layer of soft surface soil above the rock mass. As the previously described typical houses have a lot of walls in both directions, and the seismic acceleration was predominantly vertical (Figure 18), they mostly did not suffer severe damage. The greatest damage was registered at roofs and slender gable walls. Mostly older buildings, with inadequate load-bearing systems and poor quality of construction, were demolished in the earthquake.

In the village of Novo Selo Glinsko, located about 7 km SW of the earthquake epicentre, more severe damage is noticeable on the previously described typical buildings, compared to Strašnik. This is explained by the fact that this village is located on thick layers of soft soil and that the influence of the horizontal component of seismic acceleration was quite significant. Namely, in most of the damaged buildings, the entire walls or their edges were shifted to the SW, i.e. in the direction of the shock wave from the earthquake epicentre. This caused numerous vertical cracks in the walls, resulting from the action of large impact transverse forces. Large seismic forces at this site can clearly be seen in two overturned small stone monuments, whose dimensions in the direction of the overturning was only slightly smaller than their height (Figure 22 shows an overturned chapel).

Figure 22. Overturned chapel in Novo Selo Glinsko

The monuments were also overturned in a SW direction, towards the earthquake epicentre (due to opposite direction of the inertia force of the monuments as related to the acting seismic force). Some characteristic damage to the buildings in this village is shown in Figure 23.

Figure 23. Characteristic damage to family houses in Novo Selo Glinsko

The most severe damage to family houses in the villages around the earthquake epicentre area probably occurred in the village of Majske Poljane, where six people lost their lives. The village is located about 11 km SW of the earthquake epicentre, and it lies on a predominantly flat terrain with possibly thicker deposits of soft water-saturated soil. The strongest damage, but not the greatest seismic forces, is explained by the fact that in this area family houses are of the worst quality, and the load-bearing structures are in many cases improvised only (especially those for farm buildings). Based on such quality of the buildings, it can be concluded that local residents are economically deprived. It is suspected that, in addition to poor structure and inadequate construction, the foundation conditions, and possibly soil liquefaction, also greatly contributed to the collapse of houses in which human lives were lost. One of the rarer cases of family houses collapsing due to flexibility at the ground floor level can be seen in Figure 25, which shows a two-storey building in the village of Prekopa, located about 9 km SW of the earthquake epicentre, along the Glina - Petrinja road. Due to low stiffness of the ground floor, stronger seismic forces impacted the upper stiffer floor and caused collapse of the elements of the lower floor, with translation of the upper floor by approximately 1.0 - 1.5 m in the SW direction (earthquake impact direction). The large eccentricity of the stiffness centre on the ground floor as related to the mass centre also provoked horizontal rotation of the upper floor (which was damaged, but preserved integrity) in relation to the ground floor plan. Some other factors also contributed to the collapse: poor quality of masonry, lack of tie beams/columns, poor construction, etc.

Figure 24. Photos of some severely damaged and demolished houses in Majske Poljane

Several churches and other sacral buildings were also severely damaged or destroyed in the area affected by the earthquake. Some of them are shown in Figure 26. Bearing structures of most of them are even weaker than those of the previously described residential houses. In fact, they are usually much higher, more massive, subject to higher seismic forces, characterised by larger spans/measurements and more complex structure and, in some cases, they have larger unfavourable openings.

Figure 25. A family house in Prekopa was demolished due to flexible ground floor and force of the earthquake

Figure 26a shows the severely damaged St. Mary Magdalene Church in the village of Sela near Sisak, located about 13 km NE of the earthquake epicentre. The church has numerous vertical cracks in masonry walls due to excessive horizontal tensile stress in a poor-quality brick wall, as well as oblique cracks in columns and lintels along openings due to shear. Numerous cracks were registered in the massive, large-span dome. The mentioned dome generated large seismic forces on the lower structure and severely damaged it. St. Blaise and Benedict Church in Novo Selo Glinsko suffered extensive damage and its bell tower collapsed (Figure 26b). Causes of this damage and direction of fall of the bell tower are analogous to those previously mentioned for family houses in this village. Some additional disadvantages for this church should be mentioned, i.e., its elevated location (higher seismic forces) and inappropriate wooden, concrete and masonry structure.

Figure 26c shows the ruined bell tower and the southern gable wall of Church of The Assumption of the Blessed Virgin Mary in the village of Gora, located 3 km SW of the earthquake epicentre. The ruined parts of the church are externally made of stone blocks, with a poor-quality filling of coarse-grained crushed stone and concrete. The collapsed cantilever gable wall was not adequately connected at a height of about 6 m with the wooden roof, which did not cave-in and appears to be structurally sound. The bell tower also collapsed in the southwest direction from the epicentre of the earthquake.

Soil liquefaction generated during this earthquake greatly influenced formation of over a hundred sinkholes in soil within the narrower epicentral area of the earthquake, mostly in the village of Mečečani located about 22 km SE of the epicentre. They are large, reaching up to 15 m in diameter and depth, and some have appeared next to houses (Figure 27). Their formation is due to soil displacement (vibration) caused by earthquake in areas of intensely saturated water in the drainage basin of nearby rivers and their tributaries, where deep layers of loose sandy soil are likely to be found under a thin layer of humus. In fact, over time, and especially during an earthquake impact, loose soil is lowered and compacted, and the

Figure 26. Some of the churches heavily damaged in the earthquake: a) St. Mary Magdalene Church in Sela; b) SS. Blaise and Benedict Church in Novo Selo Glinsko; c) Church of the Assumption of the Blessed Virgin Mary in Gora

surface layer is deeply deteriorated due to exceedance of its very low shear strength at the failure plane. Sinkholes, as expected, are approximately circular in plan and almost vertical. The formation of sinkholes is probably greatly influenced by the change in the level of water in soil, and its vertical and horizontal movement, which leads to washout and removal of small sand particles and to deterioration of larger and heavier soil particles.

Figure 27. One of many sinkholes in Mečečani [16]

Zidane zgrade s krutim armiranobetonskim međukatnim The appearance of sinkholes next to houses can be explained by additional pressure on soil surface generated by weight of such houses, which increases soil subsidence. Clearly, the appearance of such sinkholes under houses is extremely dangerous. This issue is also discussed in [15].

Masonry buildings with rigid reinforced concrete floor structures and correct vertical and horizontal tie beams suffered no or just some slight damage. Modern reinforced concrete buildings, and especially those designed in accordance with current regulations, were virtually unharmed, although the magnitude of the earthquake in the area was higher than previously predicted. This confirms the reliability of current regulations for the design and calculation of buildings in seismically active areas. However, damage to some newer buildings which will require higher repair costs, and which were designed in the spirit of the current concept that they can be damaged but must not collapse at the adopted design acceleration, suggests that is would be wise to consider and adopt an even more conservative design concept, with a lower level of building damage during an earthquake event.

4. Guidelines for reconstruction of earthquake-demolished and damaged buildings

4.1. New buildings

All earthquake-damaged and severely damaged buildings, as well as those whose reconstruction would not be rational, should be removed and new ones should be built in accordance with current regulations, norms and rules of the profession for construction in seismically active areas. In doing so, all acquired experience on the consequences of this earthquake should be taken into account, as well as local specifics in the wider affected area (geological and geotechnical soil characteristics, foundation conditions, traditional construction practices, etc.).

This is an opportunity to increase awareness of citizens and participants in construction about the need to build sufficiently seismically resistant structures during future construction activities in Croatia, and to ensure proper seismic reinforcement of a large number of existing structures that do not meet seismic resistance criteria, with the aim of avoiding the loss of human lives and enormous material damage during a stronger earthquake. The consequences of such a stronger earthquake - with even more destructive ones likely to hit many parts of Croatia - should resonate strongly in the minds of citizens and result in a firm realization that provisional and often illegal and unprofessional construction of houses, family farms in particular, can bring about catastrophic consequences.

This is also an opportunity for some structural designers to pay more attention to crucial issues related to actual safety of buildings and to levels of possible damage in a strong earthquake (e.g., proper foundation work solutions, correct design of load-bearing structure, adequate solution of details, appropriate selection and quality of materials, correct calculation of building safety based on actual displacements and stress/strain during earthquake, suitable selection of the so-called structural behaviour factors, dealing with safety issues for the so-called non-structural elements, etc.).

Unfortunately, oftentimes we witness formalistic and insufficiently reliable approaches, where an emphasis is placed on calculation of required reinforcement and finding proof of displacement and stresses in load-bearing elements, using often inadequate design models. We should always be aware that our design solutions have a decisive impact on seismic and general safety of the building, which is in turn also dependant on the quality of performance, i.e., on the integrity of the contractor, which is sometimes lacking.

It is also necessary to raise the level of expertise and responsibility of technical supervision teams during construction, and to legally introduce mandatory occasional designer's supervision during construction of buildings. Unfortunately, current legislation allows the designer not to have to participate in the preparation of the implementation design, nor in the construction of the building.

It is a big mistake that suits individuals, but is detrimental to the society. It is necessary for the designer to participate from the idea to the end of realization of each building, which was a good practice until recently. Everyone should be aware that severe consequences of devastating earthquakes are not only a problem of those directly affected, but of the whole society, which suffers enormous losses as a result.

As buildings in earthquake-affected areas are largely located on thicker compressible soil of low bearing capacity and high deformability, with significant amplifications of earthquake displacement/acceleration, and due to structural and other characteristics of mostly low and stiff buildings, it is desirable to provide a foundation by laying a concrete slab over the entire surface of the building. Since most buildings have a small number of floors and closely spaced load-bearing walls, the thickness of such slabs can be small (20 - 30 cm), and the same applies to the thickness of reinforcing steel (Figure 23). Stiff above-ground reinforced concrete walls and thinner reinforced concrete floor slab on a stone base should be constructed above the foundation slab to the ground floor level, and in accordance with the level at which the foundation should be performed (mandatory control by structural designer, geotechnical engineer and supervising engineer). Such a spatially rigid reinforced concrete structure will reduce stresses in the ground, as well as mitigate the total and relative settlement of the soil, which would increase the global safety of the building with regard to landslides. In localities with better foundation soil, and where it is technically more favourable and rational, strip foundations should be used with the highest possible bending stiffness - height (cross-sectional shape should be:). These strip foundations should be connected along the entire perimeter of the building.

4.1.1. Load-bearing vertical structure

As buildings in earthquake epicentral area are mostly low, and especially due to the speed of construction, hiring more potential contractors and people, lower technological requirements, etc., bounded masonry walls should be a favourable solution for vertical load-bearing structure in existing conditions. The walls should be properly distributed in the floor plan of the building in both directions (the area of the walls in each direction and in each floor should be at least 3 % of the gross floor area), and these walls should measure at least 25 cm in thickness. It is desirable that the walls of one direction be connected as much as possible at the ends with the walls of the other (perpendicular) direction, and that the centre of stiffness be as close as possible to the centre of mass of the building.

Vertical tie beams should be spaced up to 4 m apart and around larger openings, reinforced with at least 4Ø14 mm longitudinally and Ø 8 mm transversely, at a distance of up to 25 cm. Horizontal tie beams at the level of floor structures should be reinforced with at least 4Ø12 mm longitudinally, and transversely as vertical tie beams. On sloping gable walls, tie beams should be reinforced as vertical tie beams. The reinforcement of ring beams should be shaped as for concrete frames, so that their contribution to the load-bearing capacity of the wall can be as large as possible (tie beams are basically soft concrete frames characterized by domination of longitudinal forces, but also by smaller bending moments).

In the design of masonry walls/buildings for earthquake action, the actual geometry of walls with openings should be included in the spatial model (Figure 28a). Non-structural walls should be included in the stiffness model, but not in the load-bearing capacity (if they significantly contribute to the overall stiffness of the building), because this will result in more realistic (higher) seismic forces in the structure. All tensile forces in the concrete structure should be assumed by reinforcement (tie beams), and in walls by both-sided thin stainless steel meshes forming part of plaster. Given their low cost and simplicity of construction, it is recommended to use thin meshes of polypropylene, glass, basalt or similar fibres as intensively as possible, as this will significantly increase the load-bearing capacity of unreinforced masonry and increase its ductility while reducing propagation of cracks. This is particularly useful in the lintel above the opening and the parapet wall above it (where significant vertical shear stress and horizontal tensile stress occur), and in the columns between the openings (where significant horizontal shear stress and vertical tensile stress occur). The meshes should be load-bearing in both directions, and they should overlap by at least 20 cm, and be transversely wrapped around the openings (Figure 28b).

The coefficient of behaviour (reduction of pre-failure forces in relation to forces according to the theory of elasticity) ranging from 1.5 to no more than 2.0 should be applied in the earthquake-resistant design of masonry buildings according to prevailing regulations [17, 18]. In fact, there is no sense in "calculated prevention" of building collapse as this would result in severe damage making the building unfit for reconstruction (which is the result of excessive coefficient of

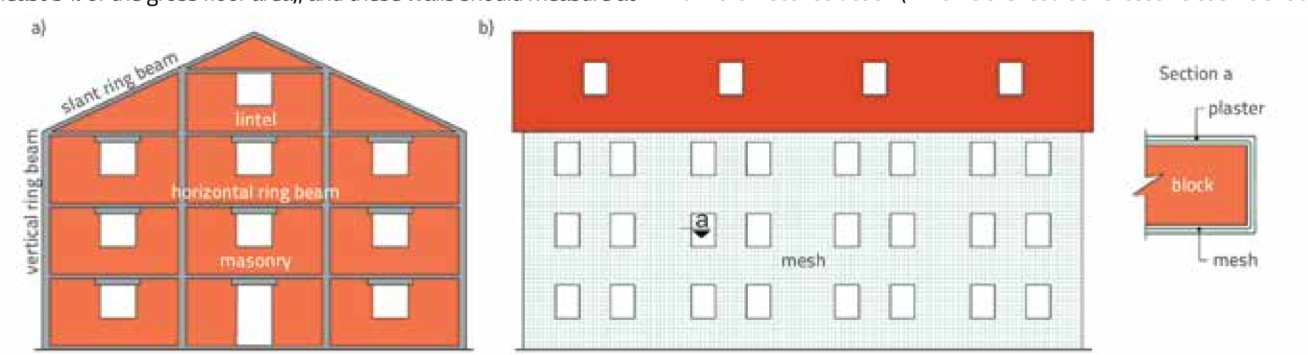


Figure 28. Some highly perforated masonry walls: a) Tie beams and lintels; b) Reinforcement of masonry with thin mesh

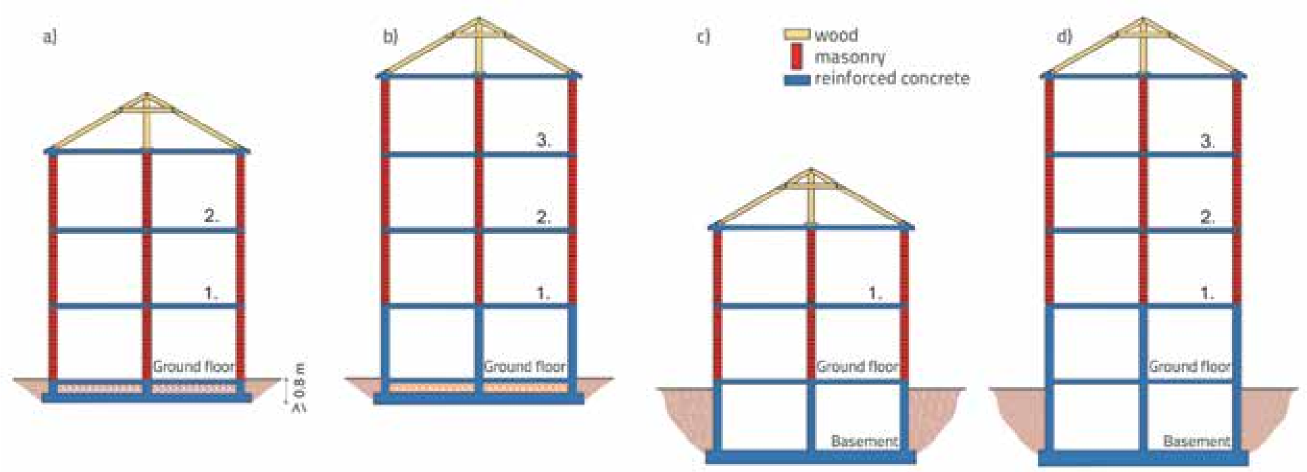


Figure 29. Schematic view of optimum structure of low-rise buildings

behaviour). In this regard, the basic task of structural designers is not to "blindly" apply provisions contained in regulations and standards, in order to "save" on construction costs, which would often have an adverse effect in a strong earthquake, but rather to design rational, safe and durable structures.

Flexible floors with reduced stiffness of vertical structures, especially at the ground floor level (which is often the case today), should not be used as such practice will significantly reduce seismic resistance of the building. Structural designers are often convinced that their "accurate calculations" will provide to the building the required mechanical resistance and safety for all intended loads, which sometimes proves to be incorrect in a strong earthquake. Therefore, more attention should be paid to creating a correct concept of the structure, solving details, analysing its actual stress-strain behaviour and the possibility of redistribution of internal forces during an earthquake. Certainly, adequate computational proof of the building's safety for the designed conditions is necessary.

Low-rise masonry walls, and especially those with openings, usually have a lower shear capacity when compared to their bending capacity. Therefore, it is necessary to control resistance of walls to shear. It is good that the shear load-bearing capacity of the masonry wall does not include a significant contribution of vertical tie beams, which is on the side of greater safety. Unfortunately, checking the walls for earthquake action perpendicular to their surface is often omitted in practice. This must be carried out with adequate computational seismic force (acceleration), and the required load-bearing capacity of the wall must be ensured by means of tie beams.

Design solutions should aim to minimize displacement of masonry buildings, and hence also displacement of poorly resistant partition walls and other non-structural elements, because this would inevitably lead to greater cracks and damage in structural and non-structural elements (especially at joints). In this regard, there is no need to save on the cross-sectional area of the walls in relation to the floor area of the building, the distance and dimensions of the ring beams, the stiffness of foundations, etc. The thermal insulation of a building should be solved independently from its structure, i.e., subsequently on external surfaces of the walls, so that the load-bearing masonry structure can be solved in the best possible way.

Boundary masonry walls, as the only load-bearing vertical elements in the areas of high seismicity, are suitable for buildings of up to three storeys in height (Figure 29a). If the building has four floors, reinforced concrete walls should be designed on the ground floor (especially in the case of highly perforated walls). This will not only increase the stiffness and safety of the building on the ground floor, but will also ensure lower displacement of the entire building and better redistribution of stress in the ground below foundations (Figure 29b).

If the building has a fully or partially buried basement, the basement structure should also be made of reinforced concrete (Figure 25c). Otherwise, masonry walls would require dense vertical tie beams due to lateral thrust of the soil, high stress concentrations would occur in the foundation soil, and the building would become less durable. If the building has a basement and four floors above it, reinforced concrete walls should be designed in the basement and the ground floor (Figure 29d). The quality of masonry is greatly dependant on the quality of construction work. Therefore, it is necessary to engage a reliable contractor, and to provide for a high quality and continuous technical supervision during construction of masonry buildings.

In the case of high-rise buildings with larger spans, with more complex structure, and with many openings in walls, reinforced-concrete walls should be selected as an optimum vertical structure. Compared to bounded masonry walls, reinforced concrete walls offer similar level of design safety but enable better connection with floor structures, a more favourable fracturing in tensile zones, better redistribution of internal forces during a seismic event, more uniform transfer of load to foundations, etc. Owing to previously mentioned benefits, a global conclusion is that such reinforced-concrete walls are technically more favourable. However, due to their previously mentioned good properties, bounded masonry walls can be a favourable solution for low-rise buildings in existing conditions, i.e., they can in such cases be as favourable as reinforced concrete walls.

4.1.2. Floor structure

An optimum floor structure can be a monolithic reinforced concrete slab at least 14 (15) cm in thickness, with adequate sound insulation. It is desirable that the slab is cross-reinforced, with continuous lower reinforcement (adequate overlaps above internal supports should ensure takeover of possible tensile forces resulting from seismic action), and continuous upper reinforcement in the upper slab of basements and floors in case of buildings with more floors and irregular layout of load-bearing walls. Slabs should be well connected to exterior walls with reinforcement, especially with masonry walls. This concrete slab has a high stiffness in its plane, and it efficiently "distributes" horizontal seismic forces onto vertical load-bearing walls in accordance with their levels of stiffness/deformability. The fact is that the concrete slab, compared to the lighter semi-prefabricated structure, has a greater mass and contributes to an increase in seismic forces. However, the higher weight of the concrete slab contributes to higher compressive stress in the wall, which increases the shear bearing capacity of the wall. This is especially important in low-rise stiff buildings where the shear impact during an earthquake is regularly greater than the bending impact. Due to the small vertical compressive stress and low shear strength at the top of the masonry walls, horizontal cracks from horizontal forces often occur during an earthquake at the contact between the floor structure and the wall. On the other hand, the increased weight of the floor structure contributes to the reduction of tensile reinforcement in walls and columns, and to the safety of foundations with regard to uplift and sliding. It should also be noted that other permanent loads above the slab and the weight of the walls, regularly exceed the weight of the concrete slab.

In general, it is good to take into account the relationship between vertical and horizontal internal forces in walls and columns, and the way in which it affects their reinforcement and load-bearing capacity. High bending moment and low longitudinal compressive force (high eccentricity of compressive force) leads to higher tensile reinforcement from bending, reduced shear capacity, and the risk of uplift and loss of stability of foundations. High compressive force for the same bending moment and transverse force, leads to lower tensile reinforcement, higher shear capacity, and lower foundation rotation. Large longitudinal compressive force leading to exceedance of compressive strength or loss of stability of the element is unfavourable. In brief, high compressive force is not always unfavourable, but can also be favourable (especially for light roof structures due to the possibility of lifting in the case of earthquake/wind action).

A rational solution for masonry buildings can also involve the use of a light semi-prefabricated floor structure, or other similar structure with comparable properties. In such floor structures, it is important that the monolithic concrete slab above the prefabricated block infill be at least 5 cm thick and continuously cross-reinforced with welded steel mesh, with 10 x 10 cm openings, with an area of at least 1.66 cm^2/m (Q-166), and with no less than 30 cm overlaps in both directions. It is also very important to make sure that the floor structure is well connected to external walls by reinforcement. Objectively, a reinforced concrete slab is a better solution than a semi-prefabricated floor structure.

4.1.3. Roof structure

A good quality wooden structure of sloping roofs is highly recommended because of the simplicity and speed of construction, technical acceptability, rationality, and preservation of traditional construction practices. It should be based on a properly prepared design (with drawings, details and proof of bearing capacity) which is unfortunately not often the case. Timber should be sufficiently dry, of good quality and protected from insects, rot, and moisture, at the joints with concrete and masonry. Natural timber should be preferred for small and medium spans, while glulam beams should be used for larger spans. The roof structure of the building should be included in the global spatial model of the building for the analysis of all relevant loads, including earthquake.

Wooden roof structure should be firmly connected to gable walls and concrete slab below it. It is well known, as confirmed by numerous cases of roof collapse during this earthquake, that large earthquake forces are generated at the top of buildings. Therefore, this part of the building structure should be adequately designed and calculated, in order to minimise damage during a strong earthquake.

The basic load-bearing system of the roof structure should be designed in such a way that horizontal forces are not transferred from the roof to the vertical load-bearing elements of the building, i.e., so that horizontal forces are transferred to the reinforced-concrete slab (as shown in Figure 30). It should also be emphasized that the roof is often light and that it can be lifted during an earthquake (especially in an epicentral area with a large vertical acceleration component) and that it can even "slide" when horizontal components of the earthquake acceleration act simultaneously.

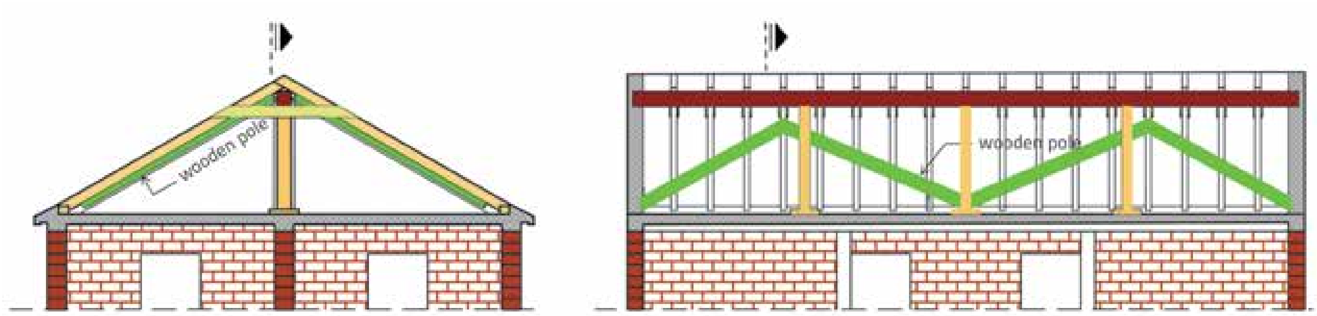


Figure 30. Wooden roof with diagonal longitudinal stiffening in the roof plane (inclined struts)

Therefore, the jamb wall should be well anchored with screws to the concrete base, and the connections between rafters and jamb wall, between individual rafters, and with other beams, should be made using screws and dowels. It is also important that the wooden roof structure be well anchored to the sloping gable walls (sloping tie beams), so that together they should form a spatial load-bearing structure resistant to vertical and horizontal loads. In this context, it is important to note that the wooden roof structure should be stiff in its plane so that it can transfer all seismic forces in that direction, both from its own mass and from the mass of gable walls. To put it simply, in case of an earthquake, the roof should carry a weak masonry gable wall, and not vice versa. To ensure this, it is best to provide above rafters a rigid tongue and groove board lining, fastened with screws (possibly nails) to the rafters. It also serves for placing the roof waterproofing and contributes to thermal insulation of the building. If it is not architecturally provided, or if it is not present on the existing roof that needs to be strengthened, inclined struts made of thick planks should be realized in the planes of the roof to provide for its longitudinal stiffening. Bolts should be used to connect these struts to the rafters (Figure 30).
From the aesthetic standpoint, wooden roof structures are especially pleasing in residential attics. In such cases, It is necessary to plan and realise aesthetically suitable wooden connections. Wall cantilevers (vertical cantilevers) reaching to jamb walls should be realised out of reinforced concrete, or in combinations of dense vertical tie beams with horizontal tie beam on the top.
If better sound insulation and a generally more durable solution is desired, the roof structure can be made as a sloping concrete slab, or as a semi-prefabricated structure, based on adequate design and construction.

4.1.4. Non-structural building elements

Unfortunately, the safety of non-structural elements, such as partition walls, chimneys, decorative and other elements on roofs or facades, the collapse of which during an earthquake could result in death or injuries, is not normally considered in the design of load-bearing structures of buildings. In the light of experience with non-structural elements gained during the Petrinja and Zagreb earthquakes, this practice in the design and construction of non-structural elements of buildings should be changed without delay.
Partition walls must be checked for the action of seismic forces perpendicular to their plane, in a manner analogous to structural walls. It is desirable to ensure that partition walls be supported by side walls, rather than by floor structures only. Construction of high partition walls with unsupported ends should be avoided. Cantilever partition walls should not be used. If necessary, lateral stability of such walls should be ensured by separate vertical or oblique elements. It is especially important that the designed solutions be correctly implemented in construction work.
New chimneys should be made of lighter and stronger materials, with adequate thermal and mechanical protection, so as to generate lower seismic forces and to be much more resistant to them. Chimneys must be fixed/firmly connected to the roof structure, so that there is no unfavourable cantilever support system from the attic floor to the chimney top. In fact, in such cases, it would be almost impossible to ensure chimney stability, even in a weak earthquake. Inadequate support of chimneys in the plane of the roof is the most important reason for their collapse during an earthquake.
Every roof tile should be properly fastened with a screw or nail to a solid wooden batten. A fall of roofing tile can be fatal for someone situated close to such a building, even in the case of a weaker earthquake. Indeed, as previously stated for the entire roof, an earthquake can cause a roof tile to rise and, even with a small horizontal seismic acceleration, the tile can slip and fall off the roof, possibly causing bodily harm.
Decorative and other non-structural elements should not be used on the facades and roofs of buildings. If such elements must be placed, it is necessary to provide for their stability and for their safe connection to a load-bearing structural element of the building.

4.1.5. Other buildings

New smaller houses in rural areas can be built using dry, good-quality natural wood. Their advantages (lower weight - lower seismic forces, smaller foundations, easier and faster construction, lower short-term price, comfortable interior and exterior, etc.) and disadvantages (low durability, high maintenance costs, higher long-term price, poorer sound insulation, higher fire risk, lower functionality and quality of housing, etc.), compared to classic concrete and masonry houses, should be highlighted.
Smaller prefabricated reinforced-concrete family houses can be built in rural areas but, in such cases, it would be wise to insist on good-quality (durable) joints of prefabricated elements. In general, they should have the quality and other positive properties that are comparable to those of monolithic concrete houses. Prefabricated construction of larger (high-rise) residential buildings is not acceptable and has been abandoned long time ago. Prefabricated construction based on concrete and steel is preferred for commercial buildings.

4.2. Strengthening of existing buildings

4.2.1. Buildings not belonging to protected architectural heritage

It would be extremely important that severe consequences of recent earthquakes are translated, in the minds of individuals and all citizens of Croatia, into the need to increase the existing level of seismic safety for many existing buildings located in zones of high seismic activity, or at least for those buildings that are considered to be of a wider social interest (hospitals, schools, important economic and cultural facilities, bridges, etc.), because such and even more devastating earthquakes are possible and probable not only in the area of Banovina and Zagreb, but along the entire coastal area of Croatia. Such an initiative should be organized and systematically implemented at the national level, just like any other strategic long-term project of national significance.

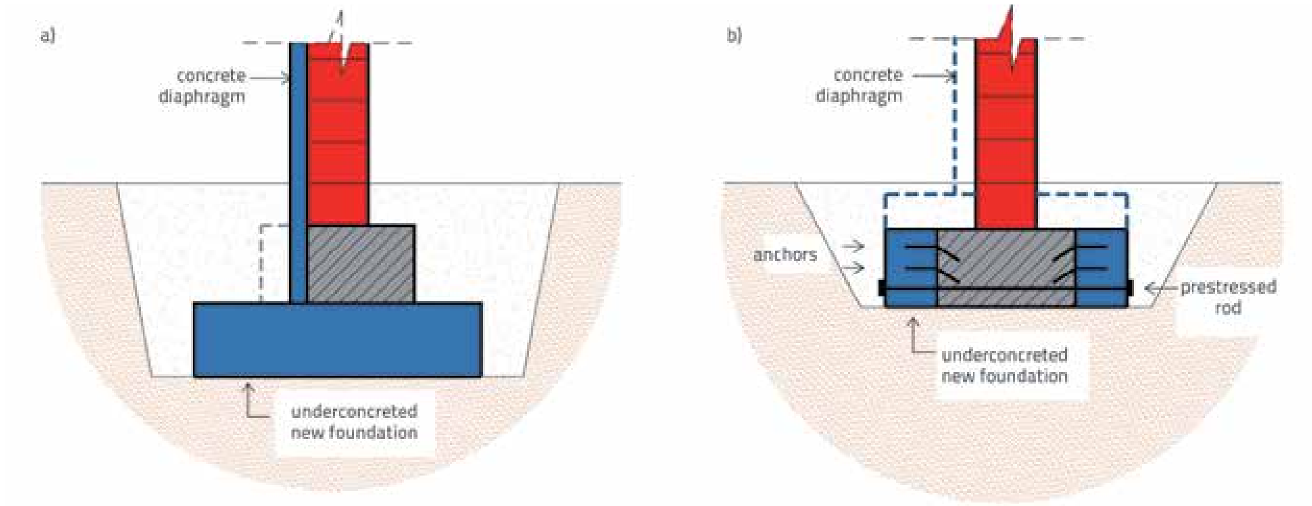


Figure 31. Possible solutions to increase load-bearing capacity of existing foundations

With a certain amount of state subsidy, a private owner of a seismically unsafe building could be willing to invest into seismic strengthening of such building in order to increase its market value. Interventions on the building can be gradual and long-term, depending on financial capabilities of the company or private investor, and on the desired level of seismic safety (which can be prescribed). A document on the level of seismic safety of each building should be one of important parameters for defining its market value.

According to the current legislation [19], it is not necessary to increase the original mechanical resistance and safety of existing buildings. Such intervention is however required in the case of reconstruction, when the existing safety is significantly affected, i.e., the existing and possibly new part of the building is in such cases required to have mechanical resistance and safety in compliance with applicable regulations. Therefore, the level of increase of the existing safety of a building is currently dependant on the decision of its owner, although it would be good that such decision is made by the state for public buildings. It would be desirable to strengthen all public buildings to such levels that they have the same design safety as new buildings.

A major step forward in improving the existing seismic safety of many buildings and reducing the danger to human health and life during an earthquake would be achieved by making interventions that do not necessitate extensive funding, i.e., by removing heavier elements hanging on the walls and by shortening or laterally supporting high heavy cabinets inside the building, by stiffening the roof structure, removing unnecessary elements from the roof and facade, stiffening gable walls and ensuring their better connection with the roof, etc.. In the current conditions, there is no reason to use special materials and technologies for the construction and strengthening of structures. In other words, it would be amply sufficient to use common widely applied building materials and technologies. The load-bearing capacity of existing foundations in deformable soil could probably be best increased by their adequate local extension, i.e., by adding a new shallow reinforced concrete foundation under the existing one (Figure 31a), or by lateral strengthening using concrete with a prestressed rod (Figure 31b), in successive construction. In the case of good-quality existing foundations in gravelly soil, a good option may be to strengthen the soil beneath the foundation by jet grouting.

The method of strengthening existing walls depends on the desired level of increase in their load-bearing capacity, on their plan-view distribution, and on the magnitude of the load/force they transmit. In general terms, local strengthening of existing walls on facades and internal staircases with a thin reinforced concrete diaphragm is advantageous, as it does not have to take up the living space. In order to connect the diaphragm and the existing masonry wall, it is necessary to cut into this wall recesses spaced at about 1.5 x 1.5 m intervals, while traditional reinforcing-steel anchors should be used for connection with the existing concrete wall/element. If it is convenient and possible, it is preferable to add new concrete walls. If a smaller increase in the existing load-bearing capacity of masonry walls is required, the problem can be solved more easily by strengthening them with thicker stainless meshes (made of carbon, basalt, etc.), exhibiting much higher load-bearing capacity than the previously mentioned thin meshes, to be realised as part of double-sided plaster. Such an approach is often used for strengthening masonry walls of protected historic buildings.

A huge improvement in the seismic safety of buildings with flexible floor structures involves their replacement with stiff structures. Traditional concrete slabs, thin concrete slabs coupled with wooden or steel beams, and other semi-prefabricated systems with thin cross-reinforced concrete slabs, can be used for this purpose. At that, it is important to ensure continuity of the floor structure within the plan of each floor, as well as its good connection with external walls.

When strengthening complex and important public buildings and infrastructure facilities, each structure should be carefully analysed and optimum seismic strengthening should be applied, with the design safety level similar to that used for new buildings.

4.2.2. Buildings belonging to protected architectural heritage

These buildings should be strengthened and renovated in accordance with the decision of the competent state institution, which should define the level and acceptable manner of their strengthening. Only the entities that have an appropriate permission from the Ministry of Culture to work on cultural property can participate in the design and realisation of such renovation activities. The development and implementation of such projects is an everyday activity and should not pose any problem.

4.3. Procedures for the design and realisation of new buildings and for renovation of existing buildings in earthquake-stricken areas

For all facilities owned or financed by the state, the development and implementation of reconstruction/strengthening projects should be conducted transparently and legally, according to the Public Procurement Act, without favouring any individual or company. The process of renovation of privately owned buildings can be conducted through direct award of works, without public bidding, according to the decision of the investor.

5. Conclusion

The most important conclusions about the issues treated in this paper are outlined below. In addition to devastating effects of the earthquake, the severe damage and demolition of buildings in the earthquake-affected area was also impacted by:
- Inadequate aseismic structure of buildings dating back to the period of the Austro-Hungarian monarchy
- Inadequate construction of family houses after the Homeland War
- Largely makeshift construction of farm outbuildings and, in some cases, of family houses in rural areas
- Additional increase of seismic forces acting on buildings due to mostly thick deposits of soft soil, and impact of soil liquefaction at localities where soil is saturated with water.

At least the most important earthquake parameters must be known as a precondition for reliable valorisation of the consequences of earthquakes on buildings, and for finding an optimum concept of reconstruction of their structures. Earthquake occurrence in a rock mass due to tectonic plate conflict can be explained by failure mechanism due to exceedance of ultimate strength (strain), when the strain energy released in the rock mass is abruptly converted into the kinetic earthquake energy. The causes of damage and demolition of each building can be explained by careful analysis of the interactive impact between the earthquake, soil and the building structure. When constructing new buildings in the area under study, it is desirable to dominantly use proven good-quality reinforced concrete and bounded masonry structures, designed in accordance with applicable regulations and rules of the profession, with a lower (conservative) factor of earthquake behaviour. Low-rise prefabricated wooden and concrete houses can be used in rural and possibly also in urban areas. The reconstruction of each severely damaged building is a specific undertaking, which must be handled solely by experienced structural engineers and contractors.

The design and realization of construction and reconstruction of each public facility should be carried out based on an open procedure in accordance with the Public Procurement Act. It would be extremely important and useful to translate heavy consequences of this earthquake, deeply engrained in the mind of each individual and the society as a whole, into a firm decision to increase seismic safety of numerous buildings in the Republic of Croatia up to an agreed level, which is currently such that many of the buildings could be fully destroyed even in a moderate-level earthquake.

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