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Post-earthquake structural assessment of the Parish house in Sela

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The 2020 earthquakes in Croatia highlighted the vulnerability of historical unreinforced masonry buildings. The Parish house next to St. Mary Magdalene church in Sela is one of the cultural heritage buildings that suffered significant damage. As these buildings hold important tangible and intangible values for the local population, their seismic assessment and subsequent retrofitting that respects the value of their historical components is crucial for the recovery of the areas affected by the earthquakes. This paper contributes to this topic by performing structural assessment both for in-plane and out-of-plane mechanisms. Analyses comprise both state-of-art numerical simulations for the in-plane behaviour and hand calculations used for both the verification of in-plane behaviour and the assessment of the out-of-plane behaviour. The paper concludes by proposing retrofitting ideas on a conceptual level.

Keywords: unreinforced masonry, post-earthquake assessment, pushover analysis, equivalent frame models, retrofitting
Ključne riječi: nearmirana zidana konstrukcija, poslijepotresna procjena, metoda postupnog guranja, metoda ekvivalentnih okvira, rekonstrukcija

INTRODUCTION

Earthquakes can cause unacceptable casualties and socio-economic damage when occurring in a region with vulnerable buildings such as unreinforced masonry buildings. This type of construction is widely spread over the world, even in countries with a high seismic activity, such as Croatia. On December 28th and 29th 2020, Croatia was struck by two major earthquakes of magnitude Mw = 5.2 and Mw = 6.4,[1] followed by multiple aftershocks. The region of Petrinja was particularly affected by these events and lots of buildings were severely damaged.[2] This was also the case with the parish house in Sela, constructed between 1759 and 1765,[3] before the introduction of any seismic code. This building is of special interest in the area because of its cultural heritage value. The parish house is the subject of this work (sl. 1). This paper carries out a seismic assessment of its structure and addresses some points and directions for further retrofitting interventions.

The parish house studied is a historical unreinforced masonry building. Its conservation, is of high concern for the society because of both tangible and intangible values. Therefore, to propose ideal retrofitting procedures for such a building and make it usable again, a seismic assessment is the first step. To do so, an adequate modelling of the structure of the masonry building is required. For this purpose, several methods are proposed in literature but some of them have a high computational cost. An alternative choice that offers good accuracy with relatively low computational cost is the

1 MIRANDA, EDUARDO et al., 2020, 206.
2 MARKUŠIĆ, SNJEŽANA et al., 2021, 1095; STEPINAC, MISLAV et al., 2021, 102-140.
3 According to the Registry of Cultural Property of the Republic of Croatia established by the Ministry of Culture and Media of the Republic of Croatia.

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Procjena otpornosti konstrukcije župnog dvora u Selima na djelovanje potresa

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Potresi koji su 2020. pogodili Hrvatsku istaknuli su ranjivost povijesnih zgrada s nearmiranom zidanom konstrukcijom. Župni dvor uz crkvu sv. Marije Magdalene u Selima jedna je od građevina kulturne baštine koje su pretrpjele znatna oštećenja. Budući da ove građevine imaju važne materijalne i nematerijalne vrijednosti za lokalno stanovništvo, njihova seizmička procjena i kasnija rekonstrukcija koja poštuje vrijednost njihovih povijesnih sastavnica, ključni su za oporavak područja pogođenih potresima. Ovaj rad predstavlja doprinos toj temi izradom procjene otpornosti konstrukcije koja uključuje mehanizme otkazivanja u ravnini i izvan ravnine. Analize obuhvaćaju i najsuvremenije numeričke simulacije za ponašanje u ravnini te ručne izračune koji se koriste i za provjeru ponašanja konstrukcije u ravnini i izvan ravnine. Rad završava idejnim prijedlogom rekonstrukcije.

Ključne riječi: nearmirana zidana konstrukcija, poslijepotresna procjena, metoda postupnog guranja, metoda ekvivalentnih okvira, rekonstrukcija

Keywords: unreinforced masonry, post-earthquake assessment, pushover analysis, equivalent frame models, retrofitting

UVOD

Potresi mogu uzrokovati neprihvatljive žrtve i društveno-ekonomske štete kada se dogode u regiji s ranjivim zgradama kao što su to zgrade s nearmiranom zidanom konstrukcijom. Taj način gradnje vrlo je raširen u svijetu, čak i u zemljama s visokom seizmičkom aktivnošću, poput Hrvatske. Hrvatsku su 28. i 29. prosinca 2020. godine pogodila dva velika potresa magnitude $Mw = 5.2$ i $Mw = 6.4$,[1] praćena višestrukim naknadnim potresima. Ovim događajima posebice je pogođeno petrinjsko područje, gdje su mnoge građevine teško oštećene.[2] To je bio slučaj i sa župnim dvorom u Selima, podignutom između 1759. i 1765. godine[3] prije uvođenja bilo kakvoga seizmičkog propisa. Ta građevina od posebnog je značaja za ovo područje zbog svojih vrijednosti kao kulturno dobro. Župni dvor, prikazan na slikovnom prilogu 1, predmet je ovoga rada, kojim je opisana procjena otpornosti konstrukcije na djelovanje potresa, a bavi se nekim točkama i smjerovima za daljnje zahvate rekonstrukcije.

Proučavani župni dvor je povijesna zgrada s nearmiranom zidanom konstrukcijom. Njezino je očuvanje, zbog materijalnih i nematerijalnih vrijednosti, velika briga za društvo. Stoga je procjena otpornosti konstrukcije na djelovanje potresa prvi korak u izradi prijedloga idejnih postupaka rekonstrukcije za takvu zgradu i njezinu ponovnu upotrebu. Za to je potrebno odgovarajuće modeliranje zidane konstrukcije zgrade. U tu svrhu, u literaturi je predloženo nekoliko metoda, ali za neke od njih je dugačko trajanje proračuna. Alternativni izbor koji nudi dobru razinu točnosti uz relativno kratko trajanje proračuna jest metoda

1 MIRANDA, EDUARDO et al., 2020., 206.
2 MARKUŠIĆ, SNJEŽANA et al., 2021., 1095.; STEPINAC, MISLAV et al., 2021., 102-140.
3 Prema Registru kulturnih dobara Republike Hrvatske koji vodi Ministarstvo kulture i medija Republike Hrvatske.

1 Parish house in Sela (photo: Thibaud Maillard, 2023)
Župni dvor u Selima (foto: Thibaud Maillard, 2023.)

Equivalent Frame Method (EFM) using macroelements.[4] In EFM, walls are divided into elements; piers and spandrels, linked by rigid node panels.[5] The hypotheses made for discretization the structure into an equivalent frame model play an important role to structural analysis results.[6]

The paper is structured in six sections. The section following this introduction gives more details on the case-study building. The next section describes the in-plane assessment using Tremuri software, verified by hand calculations, followed by the section describing the out-of-plane assessment using kinematic limit analysis. Then a conceptual overview of retrofitting methods on a conceptual level is shown in following section. The last section of the paper summarizes the findings and gives possible directions for improvement and further investigations needed to assess the building in the future.

BUILDING DESCRIPTION

The parish house is located in the vicinity of the town of Sisak in Croatia, approximately 50 km southeast of the capital Zagreb. In the following, geometry, floor and roof system, and material properties of the building are described.

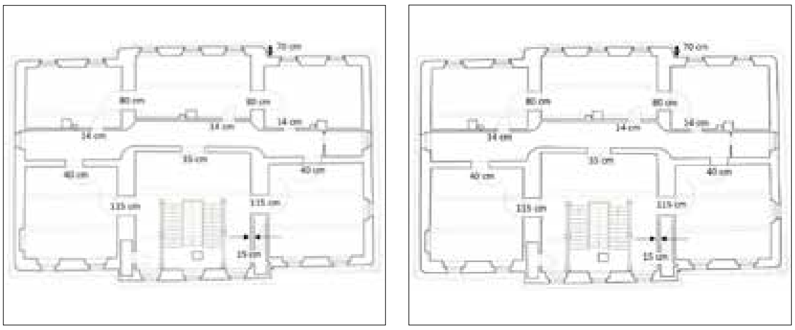
Geometry

The parish house has two storeys and an attic. The building is 23.6 m long and 13.9 to 15.3 m wide, depending on the position along the building's length. The ground floor plan is shown in Figure 2. The height measured from the foundation to the top of the vertical walls is 7.8 m, as shown in Figure 3. The storey height is 4.1 m and 3.7 m for the first and second floor, respectively. Walls are made of solid masonry bricks of dimensions 290x140x65 mm^3 and lime mortar. The external walls' thicknesses are estimated to be 700 mm,

Floor and roof system

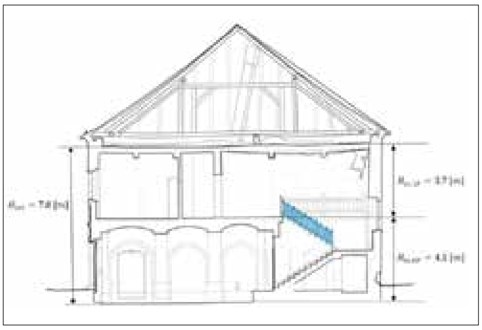
Regarding the floor composition, the ceiling of the ground floor level is mainly made of brick masonry vaults except for a small portion which is made of a concrete slab and concrete stairs. Two different typologies of vaults (cloister and barrel vaults) are observed depending on the room (Fig. 4). In ground floor rooms A, B, D and E, vaults are cloister vaults, symmetrical and estimated to be 550 mm thick. Room C has a 400 mm thick barrel vault from its north part to the stairs, while the remaining is made of a 200 mm concrete slab and concrete stairs (Fig. 5). The

4 QUAGLIARINI, ENRICO; MARACCHINI, GIANLUCA; CLEMENTI, FABIO, 2017, 166–182; VANIN, FRANCESCO, PENNA, ANDREA; BEYER, KATRIN, 2020 (a)
5 BRACHI, SERGIO et al., 2015, 3423-3448; VANIN, FRANCESCO, PENNA, ANDREA; BEYER, KATRIN, 2020 (b); DEGLI ABBATI, STEFANIA et al., 2021
6 QUAGLIARINI, ENRICO; MARACCHINI, GIANLUCA; CLEMENTI, FABIO, 2017, 166–182; PENNA, ANDREA; LAGOMARSINO, SERGIO; GALASCO, ALESSANDRO, 2014, 2



2 Ground floor level (a) and first floor level (b) plans of the Parish house in Sela (adapted from: VUČIĆ-ŠNEPERBERG, BORIS, 2021)

Tlocrt prizemlja (a) i prvog kata (b) Župnog dvora u Selima (prilagođeno prema: VUČIĆ-ŠNEPERBERG, BORIS, 2021.)



3 Cross section of the Parish house in Sela (adapted from: VUČIĆ-ŠNEPERBERG, BORIS, 2021)

Presjek župnog dvora u Selima (prilagođeno prema: VUČIĆ-ŠNEPERBERG, BORIS, 2021.)

ekvivalentnih okvira (EFM) koja koristi makroelemente.[4] U EFM-u zidovi su podijeljeni na elemente; stupce i nadvoje, povezane krutim čvornim plohama.[5] Hipoteze izrađene za diskretizaciju konstrukcije u model ekvivalentnih okvira imaju važnu ulogu za rezultate analize konstrukcije.[6]

Rad je podijeljen u šest odjeljaka. U odjeljku nakon uvoda navodi se više pojedinosti o građevini kao studiji slučaja. Sljedećim odjeljkom iznosi se procjena djelovanja u ravnini pomoću softvera Tremuri, potvrđena ručnim izračunima, a slijedi odjeljak posvećen procjeni djelovanja izvan ravnine temeljem kinematičke granične analize. Potom je u sljedećem odjeljku prikazan konceptualni pregled metoda rekonstrukcije na razini koncepata. U posljednjem odjeljku ovoga rada iznose se sažetak zaključaka te moguće smjernice za poboljšanje i za daljnja istraživanja potrebna za buduće procjene građevine.

OPIS GRAĐEVINE

Župni dvor nalazi se u blizini grada Siska u Hrvatskoj, otprilike 50 km jugoistočno od glavnog grada Zagreba. U nastavku su opisani geometrija, sustav stropova i krova te svojstva materijala građevine.

Geometrija

Župni dvor ima dva kata i potkrovlje. Dužina građevine je 23,6 m, a širina od 13,9 do 15,3 m, ovisno o položaju uzduž duljine građevine. Tlocrt prizemlja prikazan je na slikovnom prilogu 2. Visina mjerena od temelja do vrha okomitih zidova je 7,8 m, kao što je prikazano na slikovnom prilogu 3. Visina prvog kata je 4,1 m a drugog kata 3,7 m. Zidovi su izrađeni od pune opeke dimenzija 290 x 140 x 65 mm^3 i vapnenog morta. Debljina vanjskih zidova procjenjuje se na 700 mm.

Sustav stropova i krova

S obzirom na njegov presjek, strop prizemlja uglavnom je izveden kao zidani svod osim u manjem dijelu koji je izveden od betonske ploče i betonskih stepenica. Uočene su dvije različite tipologije svodova (križni i bačvasti svodovi) ovisno o prostoriji (sl. 4).U prizemlju su prostorije A, B, D i E svođene križnim svodovima, simetričnima te procijenjene debljine od 550 mm. Prostorija C svođena je bačvastim svodom debljine 400 mm od njezina sjevernog dijela do stepenica, dok je preostali strop izveden kao betonska ploča debljine 200 mm i betonske stepenice (sl. 5). Strop prvog kata izrađen je od drvene građe s betonskim slojem debljine 70 mm iznad drvene konstrukcije, što je prikazano shemom presjeka prikazana poda (sl. 6). Pod nose grede od 180 x 200 mm (D x V) u poprečnom smjeru (y os prema slikovnom prilogu 4). Razmak između greda iznosi 1,2 m. Na njih su začavlana dva sloja dasaka debljine 30 mm i širine 200 mm – jedan sloj ispod, a drugi iznad drvenih greda. Grede jednostavno leže na zidovima, stoga se pretpostavlja samo veza trenja. Ispod toga su četiri glavne grede dimenzija 230 x 260 mm^2, raspoređene u uzdužnom smjeru zgrade (x os prema slikovnom prilogu 4) koje nose pod. Krovna konstrukcija također je od drvene građe. Čini se da krov funkcionira kao trokut sila. Drugim riječima, vertikalna opterećenja prenose se na rogove koji opterećenje prenose na horizontalne grede. Iz toga proizlazi da se na vanjske zidove ne prenose horizontalna opterećenja i stoga treba uzeti u obzir samo dodatna gravitacijska opterećenja krovne konstrukcije.

Svojstva materijala

Zbog razdoblja izgradnje, podaci o izgradnji zgrade nisu bili dostupni tijekom pripreme ovoga rada. Štoviše, još nisu provedena ispitivanja *in situ*. Stoga, za potrebe ovoga istraživanja, pretpostavljena su svojstva slična materijalima

4 QUAGLIARINI, ENRICO; MARACCHINI, GIANLUCA; CLEMENTI, FABIO, 2017., 166-182.; VANIN, FRANCESCO, PENNA, ANDREA; BEYER, KATRIN, 2020. (a)

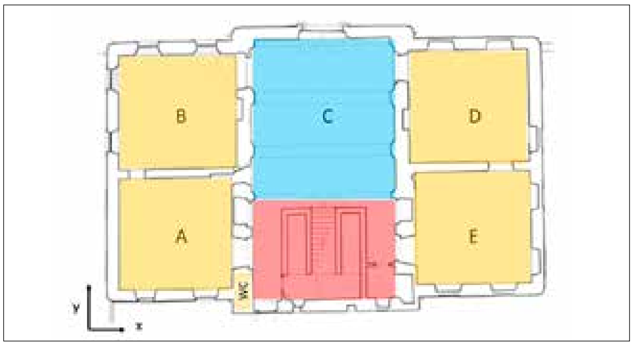
5 BRACHI, SERGIO et al., 2015., 3423-3448.; VANIN, FRANCESCO, PENNA, ANDREA; BEYER, KATRIN, 2020., 1365-1387. (b); DEGLI ABBATI, STEFANIA et al., 2021.

6 QUAGLIARINI, ENRICO; MARACCHINI, GIANLUCA; CLEMENTI, FABIO, 2017., 166–182. ; PENNA, ANDREA; LAGOMARSINO, SERGIO; GALASCO, ALESSANDRO, 2014., 2.

Table 1 Masonry properties

Masonry properties		Unit	Source
$E_{masonry}$	1000	MPa	MINISTERO DELLE INFRASTRUTTURE E DEI TRASPORTI, 2009
$G_{masonry}$	400	MPa	MINISTERO DELLE INFRASTRUTTURE E DEI TRASPORTI, 2009
f_m	6.5	MPa	MORANDI, PAOLO et al., 2018., 593-611
c	0.11	MPa	PENNA, ANDREA et al., 2016., 159-179.
μ	0.073	-	PENNA, ANDREA et al., 2016., 159-179.
$\gamma_{masonry}$	18	kN/m³	MINISTERO DELLE INFRASTRUTTURE E DEI TRASPORTI, 2009

ceiling of the first floor is made of timber with a 70 mm thick concrete layer above the wooden structure, as seen from the schematized composition of the floor (Fig. 6). The floor is supported by beams of 180 by 200 mm (L x H) in a transversal direction (y-axis according to Figure 4). The beams are spaced every 1.2 m. Two layers of 30 mm thick and 200 mm wide planks are nailed to them, one below and one above the wooden beams. The beams are simply lying on the walls and therefore, only a frictional connection is assumed. Below everything, four super beams with dimensions 230 x 260 mm^2 are distributed along the longitudinal direction of the building (x-axis according to Figure 4), supporting the floor. The roof structure is also made of timber. The roof seems to work as a "closed triangle". In other words, vertical loads are transferred to inclined beams, which transfer the load to horizontal ones. It results that no horizontal loads are applied on external walls and, therefore, only additional gravity loads from the roof structure are to be considered.



4 Plan view of the ground floor with its different floor typologies. In orange are cloister vaults, in blue is the barrel vault, and in red is the concrete portion with the concrete stairs. (adapted from: VUČIĆ-ŠNEPERBERG, BORIS, 2021)

Tlocrt prizemlja s različitim tipologijama stropova. Narančastom bojom označeni su križni svodovi, plavom bačvasti svod, a crvenom betonski dio s betonskim stepenicama. (prilagođeno prema: VUČIĆ-ŠNEPERBERG, BORIS, 2021.)

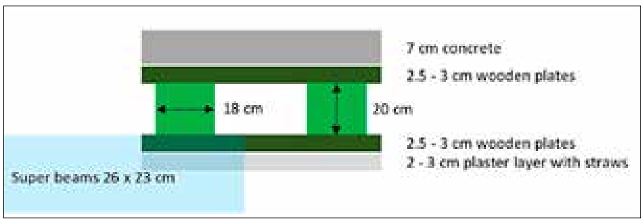
Material properties

Due to the period of construction, no data are available concerning the building construction during this work. Moreover, no *in situ* tests have been performed yet. Therefore, for the sake of this study, similar material properties found in the literature[7] and in a database[8] were assumed as representative of reality. A comparison to the Italian codes[9] is then done to ensure the estimations are coherent. The final set of material parameters is shown in Table 1 for masonry, Table 2 for timber, and Table 3 for concrete, together with the source references.

It is worth noting that when investigating the actual composition of the building, walls are not always homogeneous. Some differences in materials were observed, probably because the construction stretched over a long time

5 Cloister and barrel vaults found on the ground floor (photo: Thibaud Maillard, 2023)

Križni i bačvasti svodovi uočeni u prizemlju (foto: Thibaud Maillard, 2023.)



6 Sketch of the wooden floor composition (design: Thibaud Maillard, 2023)

Skica presjeka drvenog poda. Glavne grede 26 x 23 cm, 7 cm beton, 2,5 – 3 cm drvene daske, 2,5 – 3 cm drvene daske, 2 – 3 cm sloj žbuke sa slamom (crtež: Thibaud Maillard, 2023.)

7 PENNA, ANDREA et al., 2016, 159-179.; DAMJANOVIĆ, DOMAGOJ et al., 2021, 127-140.
8 MORANDI, PAOLO et al., 2018, 593-611.
9 MINISTERO DELLE INFRASTRUTTURE E DEI TRASPORTI, 2018

Tablica 1. Svojstva zidane konstrukcije

Svojstva zidane konstrukcije		Jedinica	Izvor
$E_{zidana\ konstrukcija}$	1000	MPa	MINISTERO DELLE INFRASTRUTTURE E DEI TRASPORTI, 2009.
$G_{zidana\ konstrukcija}$	400	MPa	MINISTERO DELLE INFRASTRUTTURE E DEI TRASPORTI, 2009.
f_m	6,5	MPa	MORANDI, PAOLO et al., 2018., 593-611.
c	0,11	MPa	PENNA, ANDREA et al., 2016., 159-179.
μ	0,073	-	PENNA, ANDREA et al., 2016., 159-179.
γ_{opeka}	18	kN/m³	MINISTERO DELLE INFRASTRUTTURE E DEI TRASPORTI, 2009.

Tablica 2. Svojstva drvene građe

Svojstva drvene građe		Jedinica	Izvor
$E_{drvo\_0°}$	1000	MPa	DOCUMENTATION TECHNIQUE LIGNUM, 2012.
$E_{drvo\_90°}$	600	MPa	DOCUMENTATION TECHNIQUE LIGNUM, 2012.
G_{drvo}	1000	MPa	DOCUMENTATION TECHNIQUE LIGNUM, 2012.
ρ	530	kg/m³	DOCUMENTATION TECHNIQUE LIGNUM, 2012.

Tablica 3. Svojstva betona

Svojstva betona		Jedinica	Izvor
E_{1_beton}	30	GPa	KALLIORAS, STYLIANOS; GRAZIOTTI, FRANCESCO; PENNA, ANDREA, 2019., 1521-52.
E_{2_beton}	30	GPa	KALLIORAS, STYLIANOS; GRAZIOTTI, FRANCESCO; PENNA, ANDREA, 2019., 1521-52.
G_{beton}	12,5	GPa	KALLIORAS, STYLIANOS; GRAZIOTTI, FRANCESCO; PENNA, ANDREA, 2019., 1521-52.
γ_{beton}	25	kN/m³	KALLIORAS, STYLIANOS; GRAZIOTTI, FRANCESCO; PENNA, ANDREA, 2019., 1521-52.

koji su pronađeni u literaturi[7] i u bazi podataka.[8] Potom je provedena usporedba s talijanskim propisima[9] kako bi se osigurala koherentnost procjena. Konačni skup parametara materijala prikazan je u tablici 1 za zidanu konstrukciju, tablici 2 za drvenu građu, i tablici 3 za beton, uz navođenje korištenih izvora.

Valja napomenuti da prilikom istraživanja stvarnog sastava građevine zidovi nisu uvijek homogeni. Uočene su neke razlike u materijalima, vjerojatno zato što je gradnja dugo trajala te su radnici koristili različite vrste opeke. Primjerice, kamen u zidovima prizemlja je različitih veličina i oblika. To može utjecati na otpornost zidova. Ipak, u ovom radu pretpostavlja se homogenost zidova, dok su gore opisana svojstva materijala pripisana stupcima i nadvojima bez ikakvog razlikovanja. Krovna konstrukcija i prvi kat izrađeni su od drvene građe. Prema opažanjima na licu mjesta, drvena građa može biti tvrdo drvo D30 ili meko drvo C24 za tanje elemente. Radi pojednostavljenja, pretpostavljeno je da su sve drvene grede ili daske izrađene od istog drva D30. Svojstva materijala za drvo D30 navedena u tablici 2 preuzeta su iz tablice za drvene konstrukcije.[10] Unatoč prisutnosti drva, ispravno je pretpostaviti da je prvi kat sastavljen samo od betona zbog velike krutosti koju donosi betonski sloj u usporedbi s drvenom konstrukcijom.

PROCJENA KONSTRUKCIJE U RAVNINI

Procjena ponašanja elemenata konstrukcije u svojoj ravnini provedena je numeričkim analizama, uz ručne izračune provedene u svrhu provjere. U nastavku su najprije opisane pretpostavke modeliranja, a zatim su prikazani rezultati uspoređeni s uočenim oštećenjima na župnom dvoru te potvrđeni ručnim izračunima.

7 PENNA, ANDREA et al., 2016., 159-179.; DAMJANOVIĆ, DOMAGOJ et al., 2021., 127-140.
8 MORANDI, PAOLO et al., 2018., 593-611.
9 MINISTERO DELLE INFRASTRUTTURE E DEI TRASPORTI, 2018.
10 DOCUMENTATION TECHNIQUE LIGNUM, 2012.

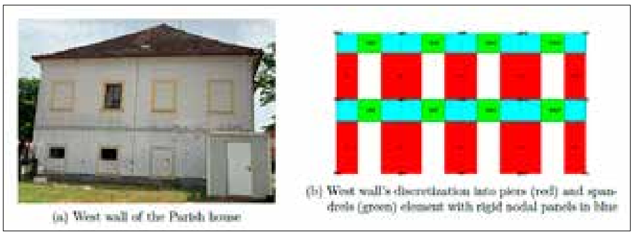
Table 2 Timber properties

Timber properties		Unit	Source
$E_{timber\_0°}$	1000	MPa	DOCUMENTATION TECHNIQUE LIGNUM, 2012
$E_{timber\_90°}$	600	MPa	DOCUMENTATION TECHNIQUE LIGNUM, 2012
G_{timber}	1000	MPa	DOCUMENTATION TECHNIQUE LIGNUM, 2012
ρ	530	kg/m^3	DOCUMENTATION TECHNIQUE LIGNUM, 2012

Table 3 Concrete properties

Concrete properties		Unit	Source
$E_{1\_concrete}$	30	GPa	KALLIORAS, STYLIANOS; GRAZIOTTI, FRANCESCO; PENNA, ANDREA, 2019, 1521-1552
$E_{2\_concrete}$	30	GPa	KALLIORAS, STYLIANOS; GRAZIOTTI, FRANCESCO; PENNA, ANDREA, 2019, 1521-1552
$G_{concrete}$	12.5	GPa	KALLIORAS, STYLIANOS; GRAZIOTTI, FRANCESCO; PENNA, ANDREA, 2019, 1521-1552
$\gamma_{concrete}$	25	kN/m^3	KALLIORAS, STYLIANOS; GRAZIOTTI, FRANCESCO; PENNA, ANDREA, 2019, 1521-1552

and the workers used different kinds of bricks. The ground floor has, for example, some stones in its walls, of different sizes and shapes. This might influence the resistance of the walls. Neverthless, walls were assumed as homogeneous in this work and the material properties described above were assigned to piers and spandrels without any distinction. The roof structure and the first floor are made of timber. According to observations on the site, the timber could be hard wood D30 or soft wood C24 for thinner elements. For simplification, it was assumed that every timber beam or planks were made of the same wood D30. The material properties of D30 wood found in Tabl 2 were taken from wood construction tables.[10] Despite the presence of wood, it is correct to assume the first floor as composed of only concrete because of the high stiffness brought by the concrete layer compared to the wooden structure.



7 Example of the discretization of a regular wall: (a) western façade of the Parish house; (b) west wall's discretization into piers (red) and spandrels (green) element with rigid nodal panels in blue (photo and drawing: Thibaud Maillard, 2023)

Primjer diskretizacije uobičajenog zida: (a) Zapadno pročelje župnog dvora; (b) Diskretizacija zapadnog pročelja u stupce (crveno) i nadvoje (zeleno) s krutim čvornim panelima u plavoj boji. (foto i crtež: Thibaud Maillard, 2023.)

IN-PLANE STRUCTURAL ASSESSMENT

In-plane behaviour was assessed by numerical analyses, with the hand calculations performed for verification purposes. In the following, first the modelling assumptions are described. Then results are reported, compared with observed damage on the parish house, and verified using hand calculations.

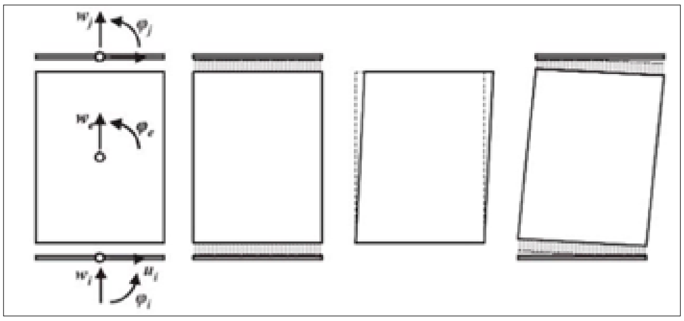
Modelling assumptions

The studied building has been modelled using an equivalent frame model. The building was defined by an assembly of structural elements called macroelements, which describe the behaviour of masonry structural members. To do so, two elements are defined: piers and spandrels.[11] Piers are vertical resisting elements carrying vertical and horizontal loads, whereas spandrels are horizontal elements found between two openings. In regular buildings, those elements are intuitively identified, however, when it comes to irregular structures with, for example, misaligned openings, the identification becomes more challenging and many assumptions should be done to give the building a more regular feature, as it has been done with the parish house. An example of the discretization of a regular wall is shown in Figure 7.

10 DOCUMENTATION TECHNIQUE LIGNUM, 2012

11 D'ALTRI, ANTONIO MARIA et al., 2020, 1153-1185

8 Examples of infilled masonry portions considered as openings in the EFM (photo and drawing: Thibaud Maillard, 2023)
Primjeri nadozidanih dijelova koji se smatraju otvorima u EFM-u (foto i oznake: Thibaud Maillard, 2023.)



9 Kinematic model of the macroelement (source: PENNA et al., 2014)
Kinematički model makroelementa (izvor: PENNA et al., 2014.)

Pretpostavke modeliranja

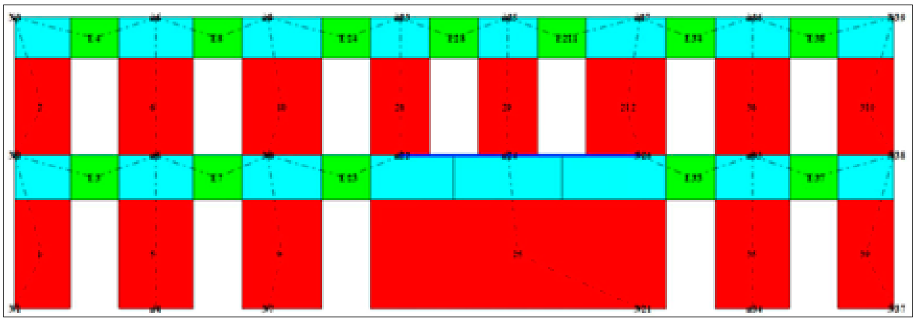
Proučavana zgrada je modelirana korištenjem modela ekvivalentnog okvira. Zgrada je definirana skupom elemenata konstrukcije koji se nazivaju makroelementi, a opisuju ponašanje elemenata zidane konstrukcije. Za to su definirana dva elementa: stupci i nadvoji.[11] Stupci su okomiti nosivi elementi koji nose okomita i vodoravna opterećenja, dok su nadvoji vodoravni elementi koji se nalaze između dva otvora. U zgradama pravilne konstrukcije ti se elementi utvrđuju intuitivno, međutim, kada se radi o nepravilnim konstrukcijama s, primjerice, nepravilno raspoređenim otvorima, identifikacija postaje zahtjevnija i potrebno je uvesti mnoge pretpostavke kako bi zgrada dobila pravilnije obilježje, kao što je to bilo učinjeno sa župnim dvorom. Primjer diskretizacije uobičajenog zida prikazan je na slikovnom prilogu 7.

Posebnost župnog dvora jest da su dijelovi mnogih zidova ispunjeni opekom, poput starih prozora, ili da su nadvoji ispod prozora uvijek tanji od okolnih zidova ili loše ispunjeni opekom (sl. 8). Ti nadozidani dijelovi smatrani su otvorima sukladno raspravi autora ovog rada i prema relevantnim izvorima.[12]

Softver korišten u ovom radu bio je Tremuri Ricerca 2.0, a razvio ga je Lagomarsino et al.[13] na Sveučilištu u Genovi. Zidovi zidane konstrukcije župnog dvora modelirani su makroelementom autora Penna et al.[14] Taj model može opisati dva glavna mehanizma loma u ravnini: načine oštećenja uslijed savijanja-ljuljanja i posmika. Međutim, ne uzima u obzir ponašanje zidova izvan ravnine. Elementi su jednodimenzionalni i definirani s dva čvora na svojim krajevima, što nam je omogućilo da opišemo cikličko ponašanje zidanog panela u njegovoj ravnini. Dva dodatna unutarnja stupnja slobode dala su mogućnost opisivanja međuovisnog uzdužnog ponašanja i ponašanja kod savijanja, ali i interakcije između oštećenja uslijed smicanja i savijanja. Elementi su stoga definirani s ukupno osam stupnjeva slobode (sl. 9). Makroelement je podijeljen u tri dijela: središnji panel koji bilježi samo deformacije uslijed smicanja te dva sučelja zanemarive debljine s vanjskim stupnjevima slobodewith possible applications in nonlinear static and dynamic analysis of masonry structures. The model, starting from a previously developed macroelement model, has been refined in the representation of flexural–rocking and shear damage modes, and it is capable of fairly simulating the experimental response of cyclic tests performed on masonry piers. By means of two internal degrees of freedom, the two-node macroelement permits to represent the coupling of axial and flexural response as well as the interaction of shear and flexural damage. Copyright © 2013 John Wiley & Sons, Ltd.","container-title":"Earthquake Engineering & Structural Dynamics","DOI":"10.1002/eqe.2335","ISSN":"00988847","issue":"2","language":"en","page":"159-179","source":"Crossref","title":"A nonlinear macroelement model for the seismic analysis of masonry buildings: NONLINEAR MACROELEMENT MODEL FOR SEISMIC ANALYSIS OF MASONRY BUILDINGS","title-short":"A nonlinear macroelement model for the seismic analysis of masonry buildings","volume":"43","author":[{"family":"Penna","given":"Andrea"},{"family":"Lagomarsino","given":"Sergio"},{"family":"Galasco","given":"Alessandro"}],"issued":{"date-parts":[["2014",2]]}}}],"schema":"https://github.com/citation-style-language/schema/raw/master/csl-citation.json"} .[15]

Drveni stropovi, betonske ploče i svodovi modelirani su pomoću ortotropnih elastičnih membranskih elemenata. Parametri krutosti elemenata izračunati su prema

11 D'ALTRI, ANTONIO MARIA et al., 2020., 1153-1185.
12 QUAGLIARINI, ENRICO; MARACCHINI, GIANLUCA; CLEMENTI, FABIO, 2017., 166–182.
13 LAGOMARSINO, SERGIO et al., 2013., 1787-1799.
14 PENNA, ANDREA; LAGOMARSINO, SERGIO; GALASCO, ALESSANDRO, 2014., 2.
15 PENNA, ANDREA; LAGOMARSINO, SERGIO; GALASCO, ALESSANDRO, 2014., 2.



10 South façade wall with its large pier number 25 influencing the failure mode in the x-direction (drawing: Thibaud Maillard, 2023)

Zid južnog pročelja s velikim stupcem broj 25 koji utječe na način sloma u smjeru x (crtež: Thibaud Maillard, 2023.)

A particularity of the parish house is that many walls' portions are filled up with masonry such as old windows, or the spandrels below the windows that are always thinner than the surrounding walls or badly filled up with masonry (Fig. 8). Those infilled masonry portions were considered openings as discussed within the team of authors and suggested by relevant literature.[12]

The software used in this work was Tremuri Ricerca 2.0 developed by Lagomarsino et al.[13] at the University of Genoa. The masonry walls of the parish house were modelled with the macroelement developed by Penna et al.[14] The model can describe the two main in-plane failure mechanisms; flexural-rocking and shear damage modes. However, it does not account for the out-of-plane behaviour of the walls. Elements are one-dimensional and defined by two nodes at their extremities, which allowed us to describe the in-plane cyclic behaviour of a masonry panel. Two additional internal degrees of freedom gave the possibility to describe the coupled axial-flexural behaviour but also the interaction between shear and flexural damage. The elements are therefore defined by a total of eight degrees of freedom (Fig. 9). The macroelement is divided into three parts; a central panel which captures only shear deformations and two interfaces of negligible thickness with the external degrees of freedom.[15]

Timber floors, concrete slabs, and vaults were modelled using orthotropic elastic membrane elements. Stiffness parameters of the elements were calculated according to Brignola et al.[16] and based on timber properties from Table 2 for timber floors,[17] for vaults, and based on Table 3 for concrete floors.

Analysis results

A pushover analysis with an inverse triangular load pattern was performed. The results obtained by this type of analysis were more conservative than the one obtained with a uniform load pattern. Applying this load pattern made results more comparable to hand calculations later on, which are even more conservative. The results obtained are presented and discussed hereafter. The displacement demand of the building was then computed based on the N2 method.

The results obtained from the pushover analysis on the whole building are presented in Table 4 for solicitations in the positive axis. The lateral resistance of the building appears to be higher in the y direction than in the x direction, and the same can be observed for the displacement capacity. However, the stiffness is larger for the x direction. The same behaviour was observed for solicitations in the negative direction.

The behaviour seems plausible regarding the geometric characteristics of the walls spanning in the x and y directions. When looking at the y direction, two particularly thick walls are present and can explain the bigger lateral resistance computed in this direction. On the other hand, despite the large pier 25 belonging to the south façade wall (spanning in the x direction) shown in Figure 10, all the other walls are quite slender. This difference can explain the higher resistance in the y direction.

Regarding the displacement capacity, because of the very large pier 25 (Fig. 10), the longitudinal direction is controlled by the shear failure of this element, which has a lower drift limit than in the case of flexural failure. The drift limits applied were 0.4 % for shear and 0.8 % for flexural failure.[18] In the y direction, the building fails in flexure, explaining why the displacement capacity is higher than in the x direction.

The difference in the failure mode explains also the stiffness obtained in each direction. Because of the squat pier element 25, stiffness in this direction is higher.

Comparison with displacement demand

Seismic properties of the ground are necessary to calculate the elastic acceleration and displacement spectrums. The PGA was provided by the Department of Geophysics and

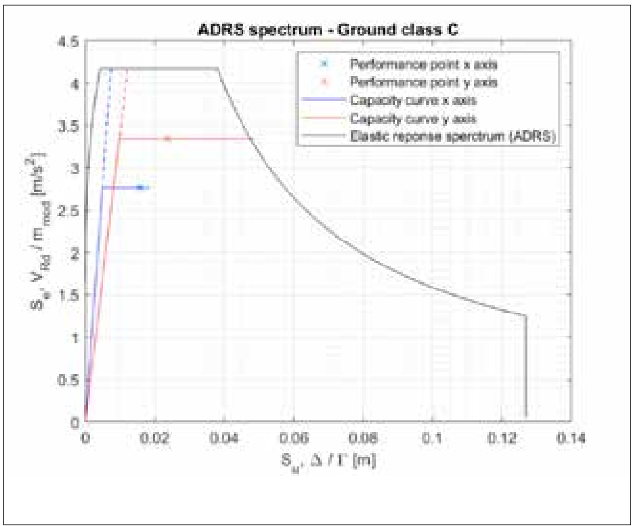
12 QUAGLIARINI, ENRICO; MARACCHINI, GIANLUCA; CLEMENTI, FABIO, 2017, 166–182
13 LAGOMARSINO, SERGIO et al., 2013, 1787-1799
14 PENNA, ANDREA; LAGOMARSINO, SERGIO; GALASCO, ALESSANDRO, 2014, 2
15 PENNA, ANDREA; LAGOMARSINO, SERGIO; GALASCO, ALESSANDRO, 2014, 2
16 BRIGNOLA, ANNA et al., 2008, 52-73; BRIGNOLA, ANNA; PAMPANIN, STEFANO; PODESTÀ, STEFANO, 2012, 1687-1709
17 CATTARI, SERENA; RESEMINI, SONIA; LAGOMARSINO, SERGIO, 2008, 537-44
18 ECS, 2017

Tablica 4. Rezultati metode postupnog guranja dobiveni za cijelu zgradu za pobude u pozitivnom smjeru osi

Osi pobude	V_{Rd} [kN]	Kapacitet pomaka [mm]	Krutost [kN/mm]
x	2817	24,5	310,6
y	3301	66	174,1

Tablica 5. Seizmički podaci za područje interesa, Sela, i tlo koje pripada klasi C

Klasa tla	S [-]	T_B [s]	T_C [s]	T_D [s]	a_{gR} [g]
Klasa C	1,15	0,2	0,6	2	0,148



11 Capacity curves for longitudinal (x axis) and transversal (y axis) directions for the whole building and performance points (chart: Thibaud Maillard, 2023)

Krivulje kapaciteta za uzdužne (x os) i poprečne (y os) smjerove za cijelu zgradu i granične točke (grafikon: Thibaud Maillard, 2023.)

Brignola et al.,[16] a na temelju svojstava drva iz tablice 2 za drvene stropove[17] za svodove te, na temelju tablice 3, za betonske podove.

Rezultati analize

Provedena je metoda postupnog guranja s inverznim trokutastim uzorkom opterećenja. Rezultati dobiveni ovom vrstom analize bili su konzervativniji od rezultata dobivenih s uzorkom uniformnog opterećenja. Primjenom uzorka tog opterećenja kasnije su rezultati bili usporedivi s ručnim izračunima, koji su još konzervativniji. Dobiveni rezultati prikazani su i raspravljeni u nastavku. Zahtijevani pomak zgrade tada je izračunat na temelju metode N2.

Rezultati dobiveni metodom postupnog guranja na cijeloj zgradi prikazani su u tablici 4 za pobude u pozitivnom smjeru osi. Čini se da je bočna otpornost zgrade veća u smjeru y nego u smjeru x, a isto se može uočiti i za kapacitet pomaka. Međutim, krutost je veća za smjer x. Isto ponašanje uočeno je za pobude u negativnom smjeru.

Ponašanje se čini vjerojatnim s obzirom na geometrijske karakteristike zidova koji se protežu u smjerovima x i y. Kada se razmatra smjer y, prisutna su dva posebno debela zida i mogu objasniti veću bočnu otpornost izračunatu u ovom smjeru. S druge strane, usprkos velikom stupcu 25 koji pripada zidu južnog pročelja (proteže se u smjeru x), prikazanom na slikovnom prilogu 10, svi ostali zidovi su dosta vitki. Ova razlika može objasniti veću otpornost u smjeru y.

Što se tiče kapaciteta pomaka, zbog vrlo velikog stupca 25 (sl. 10), uzdužni smjer je kontroliran posmičnim lomom toga elementa, koji ima nižu granicu pomaka nego kod loma savijanjem. Primijenjene granice pomaka bile su 0,4 % visine elementa za posmični lom i 0,8 % za lom na savijanje.[18] U smjeru y, u građevini nastupa lom savijanjem, što objašnjava zašto je kapacitet pomaka veći nego u smjeru x.

Razlika u načinu loma također objašnjava krutost dobivenu u svakom smjeru. Zbog elementa stupca 25, krutost u ovom smjeru je veća.

Usporedba sa zahtijevanim pomakom

Seizmička svojstva tla neophodna su za izračun spektara elastičnog ubrzanja i pomaka. Vršno ubrzanje tla (PGA) je osigurao Zavod za geofiziku i znanost Zagreb. Za područje Sela, vršno ubrzanje tla je stoga agr = 0,148g za povratno razdoblje od 475 godina. Štoviše, prema hipotezi jedne od autorica Martine Vujasinović, temeljem iskustva u regiji, odabrana je klasa tla C. Vrijednosti koje pripadaju ovoj klasi tla nalaze se u EC8 (2017) i, prema nacionalnom prilogu za Hrvatsku (HRN EN 1998-1:2011/NA:2011), korišten je tip 1. Vrijednosti su prikazane u tablici 5.

Seizmički zahtijevani pomak izračunat je na temelju karakteristika tla i vršnog ubrzanja tla na lokaciji župnog dvora. Da bi se to postiglo, zgrada je pojednostavljena iz MDOF sustava u SDOF sustav, a njezina krivulja kapaciteta prikazana je u formatu ADRS (Spektar

16 BRIGNOLA, ANNA et al., 2018., 52-73.; BRIGNOLA, ANNA; PAMPANIN, STEFANO; PODESTÀ, STEFANO, 2012., 1687-1709.
17 CATTARI, SERENA; RESEMINI, SONIA; LAGOMARSINO, SERGIO, 2008., 537-544.

18 ECS, 2017.

Table 4 Results of the pushover analysis obtained for the whole building for positive axis solicitations

Axis of solicitation	V_{Rd} [kN]	Displacement capacity [mm]	Stiffness [kN/mm]
x	2817	24.5	310.6
y	3301	66	174.1

Table 5 Seismic data for the area of interest, Sela, and ground belonging to class C

Ground class	S [-]	T_B [s]	T_C [s]	T_D [s]	a_{gR} [g]
Class C	1.15	0.2	0.6	2	0.148

Science Zagreb.[19] For the area of Sela, the peak ground acceleration is therefore a_{gR} = 0.148g for a return period of 475 years. Moreover, according to the hypotheses put forth by one of the authors, Martina Vujasinović, based on her experience in the region, the soil class C was chosen. The values belonging to this ground class are found in EC8 (2017)[20] and according to the national annex for Croatia (HRN EN 1998-1:2011/NA:2011),[21], type 1 was be used. The values are presented in Table 5.

The seismic displacement demand was calculated based on the soil characteristics and the PGA at the location of the parish house. To do so, the building was simplified from an MDOF system into an SDOF system and its capacity curve was presented in a ADRS (Acceleration Displacement Response Spectra) format with the response spectra of the soil. This graph allowed us to compare and obtain the displacement demand of the structure. The seismic data of soil class C and the PGA in Sela are presented in Table 5. The N2 method proposed by EC8 (2017)[22] was used. The first step of this method consists in transforming the MDOF system into an equivalent SDOF system. This transformation allowed us to determine the displacement capacity of the SDOF system. Then, the second step was the calculation of the displacement demand of this equivalent systema discrete element model of a vertically spanning wall is built and validated against experimental results from static and dynamic test conditions. The model is then analysed for a large range of wall configurations. For each configuration, a static pushover analysis and a series of incremental dynamic analyses are run, the latter permitting to determine the capacity of the wall under dynamic loading. The accuracy of the assessment methods in predicting the acceleration at which the walls collapse is evaluated. It is found that the displacement-based method is more accurate, robust, and safe than the force-based method. The comparison also shows that for walls characterised by a relatively high ratio of axial load to Euler's critical load, both assessment methods lead to an overestimation of the wall capacity. As a remedy, a modification to the methods based on a recently developed mechanical model is put forward and tested. For the force-based method, it is additionally suggested to set for walls with relatively high overburden ratios the behaviour factor equal to 1. To ensure reproducibility of this study, all input and output files of the numerical simulations are made publicly available.","container-title":"Earthquake Engineering & Structural Dynamics","DOI":"10.1002/eqe.3144","ISSN":"00988847","issue":"4","journalAbbreviation":"Earthquake Engng Struct Dyn","language":"en","page":"454-475","source":"DOI.org (Crossref.[23]

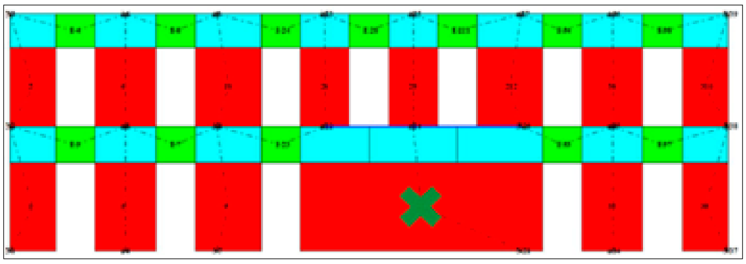
The calculation of the displacement demand showed that for both directions the parish house should be able to resist the seismic event. The compliance factors for the x and y directions are, respectively, α_x = 1.18 and α_y = 2.07. The transversal direction has a higher compliance factor, which can be explained by the fact the dominant failure in this direction is flexural and thus, the structure has a higher displacement capacity, as shown in Table 4. The results can be illustrated in an ADRS format graph (Fig. 11). The dashed lines are the prolongations of the elastic part of the capacity curves and its intersection with the response spectrum shows that the period of the SDOF is indeed on the plateau and thus the equal displacement rule could not be applied. The performance points are indicated with crosses.

Comparison with observed damage

This chapter compares, by a visual inspection, the results predicted by the analysis in Tremuri to the real damage observed directly on the parish house. Regarding the damage predicted by Tremuri in the x direction, it seems that in the simulations the model experienced less damage than in reality. As shown in Figure 12, illustrating the modelled south facade of the parish house with crosses indicating which element has failed in shear (green cross) and flexure (blue cross), only element 25 is failing according to Tremuri. Whereas in reality, almost all the spandrels have failed. All the deformations in the x direction were captured by

19 HERAK, MARIJAN et al., 2011
20 ECS, 2017
21 HRN, 2011
22 ECS, 2017

23 GODIO, MICHELE; BEYER, KATRIN, 2019, 454-475



12 Prediction given by Tremuri for the south facade. The green cross indicates that element 25 failed in shear (drawing: Thibaud Maillard, 2023)

Predviđanje iz Tremurija za južno pročelje. Zeleni križić označava da lom elementa 25 na posmik (crtež: Thibaud Maillard, 2023.)

13 Identification of the cracks belonging to in-plane failure modes for the south façade (photo and drawing: Thibaud Maillard, 2023)

Identifikacija pukotina koje pripadaju načinima loma u ravnini za južno pročelje (foto i crtež: Thibaud Maillard, 2023.)

odgovora u formatu akceleracije pomaka) sa spektrom odgovora tla. Ovaj nam je graf omogućio usporedbu i pribavljanje zahtijevanog pomaka konstrukcije. Seizmički podaci o tlu klase C i vršnom ubrzanju tla u Selima prikazani su u tablici 5. Korištena je metoda N2 koju je predložio EC8 (2017). U ovoj se metodi prvi korak sastoji se u transformaciji MDOF sustava u ekvivalentni SDOF sustav. Ta transformacija omogućila nam je određivanje kapaciteta pomaka SDOF sustava. Zatim je drugi korak bio izračun zahtijevanog pomaka ovog ekvivalentnog sustava.

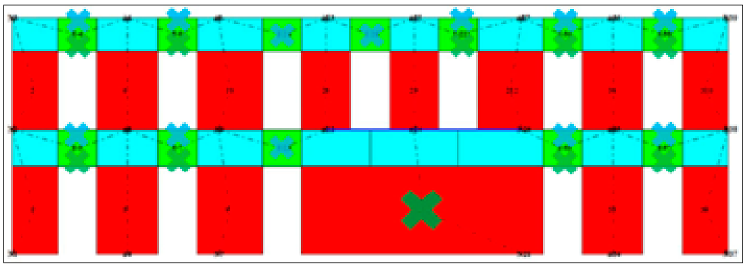
Proračun zahtijevanog pomaka pokazao je da bi župni dvor trebao biti otporan na seizmički događaj u oba smjera. Indeksi sigurnosti za x i y smjerove iznose $\alpha x = 1{,}18$ i $\alpha g = 2{,}07$. Poprečni smjer ima veći indeks sigurnosti, što se može objasniti činjenicom da je u ovom smjeru dominantan lom savijanjem te stoga konstrukcija ima veću sposobnost pomaka, kao što je prikazano u tablici 4. Rezultati se mogu ilustrirati u grafu ADRS formata (sl. 11). Isprekidane linije su produžeci elastičnog dijela krivulja kapaciteta, a njihovo sjecište s odgovarajućim spektrom odgovora pokazuje da je period SDOF-a doista na platou i stoga se ne može primijeniti pravilo jednakih pomaka. Granične točke označene su križićima.

Usporedba s uočenim oštećenjima

U ovom poglavlju se vizualnim pregledom uspoređuju rezultati predviđeni analizom u softveru Tremuri sa stvarnom štetom uočenom izvidom na župnom dvoru. Što se tiče štete predviđene u softveru Tremuri u smjeru x, čini se da je u simulacijama model doživio manju štetu nego u stvarnosti. Kao što je prikazano slikovnim prilogom 12 – koji ilustrira modelirano južno pročelje župnog dvora, dok križići označavaju elemente na kojima je došlo do loma uslijed posmika (zeleni križić) i savijanja (plavi križić) – prema softveru Tremuri, do loma dolazi samo na elementu 25. Međutim, u stvarnosti je gotovo na svim nadvojima došlo do loma. Sve deformacije u smjeru x obuhvaćene su elementom 25 u modelu softvera Tremuri. Jedno objašnjenje te razlike moglo bi potjecati iz spojeva prvog kata. U softveru Tremuri su spojevi između zida i ploče kruti i ne pružaju korisniku mogućnost da ih prilagodi stvarnijoj situaciji. Međutim, u župnom dvoru su spojevi slabi i svode se samo na trenje greda o zidove. To znatno utječe na ponašanje zgrade koja će težiti ka kutijastom ponašanju u softveru, s važnom preraspodjelom sila te s ograničenjem izduženja elemenata nadvoja. Naprotiv, u stvarnosti, riječ je samo u vezama trenja, a to ne može jamčiti kutijasto ponašanje, što rezultira većim oštećenjima kakvo se vidi u župnom dvoru. Na slikovnom prilogu 13 oštećenja u ravnini istaknuta su narančastom ili sivom bojom, ovisno o smjeru pobuda. Pukotine koje nisu obojene vjerojatno pripadaju mehanizmu izvan ravnine i o tome će biti riječi u sljedećem odjeljku. Uočeno je da su pukotine lokalizirane uglavnom u nadvojima. Još jedan izvor razlike je prisutnost mehanizma izvan ravnine u stvarnoj zgradi, dok Tremuri ne uzima u obzir lomove izvan ravnine.

Kako bismo istražili scenarij fleksibilnih dijafragmi, proveli smo metodu postupnog guranja svakog zida kao samostalnog elementa. Prigodom provedbe metode postupnog guranja samo na jednom zidu nisu uzeti u obzir utjecaji stropova. Rezultati analize za zid južnog pročelja prikazani su slikovnim prilogom 14. Križići u plavoj boji označavaju elemente kod kojih dolazi do loma savijanjem, a u zelenoj boji elemente s posmičnim lomom. Usporedba je pokazala da je takva analiza dobro predvidjela lom nadvoja, tj. da je on nalik ponašanju u stvarnosti, barem za neizbačene dijelove pročelja. Kao što je gore navedeno, pukotine izbačenog dijela vjerojatno odgovaraju mehanizmu izvan ravnine.

Kada se pogledaju rezultati dobiveni za cijelu zgradu kod pobude u smjeru y, istočno i zapadno pročelje čine se u dobroj korelaciji s onim što je predviđeno s Tremurijem. Isto se može zaključiti o dobroj softverskoj procjeni loma nadvoja. Slikovni prilog 15 ilustrira pukotine koje odgovaraju



14 Prediction given by Tremuri for the south facade. Blue crosses indicate elements that have failed in flexure, green crosses indicate elements that have failed in shear. (drawing: Thibaud Maillard, 2023)

Predviđanje iz Tremurija za južno pročelje. Križići plave boje označavaju elemente kod kojih dolazi do loma savijanjem, a križići zelene boje elemente s posmičnim lomom. (crtež: Thibaud Maillard, 2023.)

15 Identification of the cracks belonging to in-plane failure modes for the east façade (photo and drawing: Thibaud Maillard, 2023)

Identifikacija pukotina koje pripadaju načinima loma u ravnini za istočno pročelje (foto i crtež: Thibaud Maillard, 2023.)

element 25 in the Tremuri model. One explanation of the difference might come from the connections of the first floor. In Tremuri, the wall-slab connections are rigid and no possibility is offered to the user to adapt it to a more realistic one. Meanwhile in the parish house, the connections are weak and based only on the friction of the beams with the walls. This impact significantly the behaviour of the building which will tend towards a box behaviour in the software, with important redistribution of forces, and with the restriction of the elongation of the spandrels elements. On the contrary, in reality, the connections are only frictional ones and this can not guarantee a box behaviour, resulting in more damage as seen in the parish house. In Figure 13, the in-plane damage are highlighted in orange or grey depending on the direction of the solicitations. The cracks not coloured are likely belonging to an out-of-plane mechanism and this will be discussed in the next section. It is observed that cracks are localized mainly in spandrels. Another source of difference is the presence of an out-of-plane mechanism in the real building, while Tremuri does not account for out-of-plane failures.

In order to investigate the scenario of non-rigid diaphragms, we performed the pushover analysis of each wall as an independent element. When running a pushover analysis on only one wall, no influences of the floors were accounted for. The results of the analysis for the south façade wall are shown in Figure 14. Crosses in blue indicate elements failing in flexure and green is for shear failure. The comparison showed that such analysis predicted well the failure of the spandrels similar to reality, at least for the not extruded parts of the facade. As mentioned above, the cracks of the extruded part correspond probably to an out-of-plane mechanism.

When looking at the results obtained for the whole building when solicited in the y direction, the eastern and western facades seem well correlated to what is predicted with Tremuri. The same conclusions can be drawn with regard to the spandrel failure being well estimated by the software. Figure 15 illustrates the cracks corresponding to in-plane failure modes, whereas Figure 16 shows the model of the eastern facade with, again, green and blue crosses indicating shear or flexural failures of the elements. In this facade also, the cracks not coloured in orange or grey belong to an out-of-plane mechanism.

Verification and comparison with hand calculations

This section presents hand calculations done for the parish house to verify the in-plane analysis results obtained with Tremuri. The method used hereafter for the in-plane verification follows the procedure proposed by Lestuzzi and Badoux,[24] which is based on the technical book SIA 2018[25] from the Swiss codes.

The method for estimating in-plane capacity consists of a displacement-based analysis. The principle of such analysis is the construction of a capacity curve for each stabilizing element. This curve is assumed bilinear and can be defined by three points: lateral resistance V_{Rd}, yield displacement Δ_y and the displacement capacity Δ_u. This method uses the hypothesis of a diaphragm effect, which allows us to obtain the capacity curve of the whole building by summing up the capacity curves of each element. The displacement capacity of the building can then be determined using the criteria of the displacement obtained when the lateral force drops to 80% of the maximal lateral load. Seismic safety is calculated afterwards by comparing the displacement capacity of the whole building to the displacement demand given by the appropriate response spectra. This comparison can be shown graphically in an Acceleration Displacement Response Spectrum (ADRS) graph.[26] Another assumption made in this method is the concentration of the plastic deformations at the ground floor level. Also, the frame effect is considered with the value of the height of zero moment H_0.

24 LESTUZZI, PIERINO; BADOUX, MARC, 2013
25 WENK, THOMAS, 2005
26 LESTUZZI, PIERINO; BADOUX, MARC, 2013

Tablica 6. Rezultati ručnih izračuna i simulacije Tremuri za pobude u pozitivnom smjeru osi. Pretpostavlja se da je visina nultog momenta jednaka visini kata.

	Ručni izračuni		Tremuri		Varijacija u [%]	
Smjer	V_{Rd}[kN]	Δ_u [mm]	V_{Rd}[kN]	Δ_u [mm]	V_{Rd}[kN]	Δ_u [mm]
x	1556	18,24	2817	24,5	81	34
y	1503	39,65	3301	66	120	66

Tablica 7. Rezultati ručnih izračuna i simulacije Tremuri za pobude u pozitivnom smjeru osi. Vrijednosti visine nultog momenta i osnog opterećenja prilagođene su vrijednostima iz simulacija Tremuri.

	Ručni izračuni		Tremuri		Varijacija u [%]	
Smjer	V_{Rd}[kN]	Δ_u [mm]	V_{Rd}[kN]	Δ_u [mm]	V_{Rd}[kN]	Δ_u [mm]
x	3220	19,1	2817	24,5	-13	28
y	3679	30,75	3301	66	-10	115

načinima loma u ravnini, dok slikovni prilog 16 prikazuje model istočnog pročelja na kojem su opet zelenim i plavim križićima označeni lomovi elemenata posmikom ili savijanjem. I na tom pročelju pukotine koje nisu obojene u narančasto ili sivo pripadaju mehanizmu izvan ravnine.

Provjera i usporedba s ručnim izračunima

U ovom odjeljku prikazani su ručni izračuni provedeni za župni dvor radi potvrde rezultata analize djelovanja u ravnini dobivenih s Tremurijem. Metoda koja se u nastavku koristi za provjeru u ravnini, slijedi postupak koji predlažu Lestuzzi i Badoux,[19] a temelji se na tehničkoj knjizi SIA 2018[20] iz švicarskih propisa.

Metoda za procjenu kapaciteta u ravnini obuhvaća analizu temeljenu na pomaku. Načelo takve analize je konstrukcija krivulje kapaciteta za svaki stabilizirajući element. Pretpostavlja se da je ova krivulja bilinearna i može se definirati s tri točke: bočna otpornost V_{Rd}, pomak tečenja Δ_g i kapacitet pomaka Δ_u. Ova metoda koristi hipotezu o efektu dijafragme, što nam omogućava dobivanje krivulje kapaciteta cijele zgrade zbrajanjem krivulja kapaciteta svakog elementa. Kapacitet pomaka zgrade tada se može odrediti korištenjem kriterija pomaka dobivenog kada bočna sila padne na 80% najvećeg bočnog opterećenja. Seizmička sigurnost izračunava se nakon toga usporedbom kapaciteta pomaka cijele zgrade sa zahtijevanim pomakom koji je određen odgovarajućim spektrima odgovora. Ta se usporedba može prikazati grafički u grafu spektra odgovora na ubrzanje i pomak (ADRS).[21] Druga pretpostavka u ovoj metodi je koncentracija plastičnih deformacija na razini prizemlja. Također, razmatra se učinak okvira s vrijednošću visine nultog momenta H_0.

Tablica 6 sažima rezultate dvaju pristupa i njihove međusobne varijacije (uzimajući kao referencu postotak vrijednosti ručnog izračuna) za slučaj simulacija s Tremurijem izvedenih u pozitivnoj x i y osi zgrade. Visina nultog momenta u ručnim izračunima pretpostavljena je jednaka visini kata $H_0 = h_{sv} = 4,1$ m za prvi kat.

Općenito, Tremuri predviđa veće vrijednosti za bočnu otpornost i kapacitet pomaka u usporedbi s ručnim izračunima. To je bilo očekivano jer su ručni izračuni pretjerano konzervativni. Kada se istražuje porijeklo razlika, prvo objašnjenje je razlika u visini nultog momenta (H_0). U ručnim izračunima visina je prvo postavljena na visinu kata. Međutim, prigodom izračuna visine nultog momenta danog u Tremuriju, koja se dobiva dijeljenjem momenta s horizontalnom silom u bazi svakog stupca, jasno je da ta pretpostavka nije točna i da je samo aproksimacija. Štoviše, vrijednost H_0 nije konstantna i mijenja se tijekom analize. Ta se varijacija objašnjava preraspodjelom sila između stupaca zbog nadvoja i ploče. Zanimljiv pristup je procjena njezina utjecaja na rezultate ako se visina nultog momenta prigodom ručnog izračuna zamijeni onom dobivenom Tremurijem. Ta izmjena dovodi do točnijih rezultata za bočnu otpornost. Drugi parametar koji je utjecao na razliku između dvaju modela, varijacija je osnog opterećenja. Prigodom promatranja vrijednosti osnog opterećenja u svakom stupcu, ona se također mijenjala tijekom ispitivanja i nije bila konstantna kao što je navedeno u ručnim izračunima. I ovdje je to uslijed preraspodjele sila zahvaljujući nadvojima i pločama. Ako se osno opterećenje uneseno u ručne izračune zamijeni onim danim u Tremuriju, procjena bočne otpornosti je još točnija. Rezultati dobiveni s osnim opterećenjem i visinom nultog momenta koje je izračunao Tremuri prikazani su u tablici 7. Također je uočeno kako je tendencija da je bočna sila u smjeru y veća nego u smjeru x, prema Tremuriju, sada također ispravno predviđena u ručnim izračunima.

S druge strane, može se primijetiti da je velika razlika u kapacitetu pomaka između numeričkih i ručnih izračuna za smjer y. Kada se promatra način loma u smjeru y,

19 LESTUZZI, PIERINO; BADOUX, MARC, 2013.
20 WENK, THOMAS, 2005.
21 LESTUZZI, PIERINO; BADOUX, MARC, 2013.

Table 6 Results of hand calculations and Tremuri simulation for positive axis solicitations. The height of zero moment is assumed equal to the storey height.

	Hand calculations		Tremuri		Variation in [%]	
Direction	V_{Rd}[kN]	Δ_u [mm]	V_{Rd}[kN]	Δ_u [mm]	V_{Rd}[kN]	Δ_u [mm]
x	1556	18.24	2817	24.5	81	34
y	1503	39.65	3301	66	120	66

Table 7 Results of hand calculations and Tremuri simulation for positive axis solicitations. Height of zero moment and axial load values were adapted to the values obtained by Tremuri.

	Hand calculations		Tremuri		Variation in [%]	
Direction	V_{Rd}[kN]	Δ_u [mm]	V_{Rd}[kN]	Δ_u [mm]	V_{Rd}[kN]	Δ_u [mm]
x	3220	19.1	2817	24.5	-13	28
y	3679	30.75	3301	66	-10	115

Table 6 sums up the results of the two approaches and variation between them (taking as reference the percentage of the values of the hand calculation) for the case of Tremuri simulations performed in the positive x and y axis of the building. The height of zero moment in the hand calculations was assumed to be equal to the storey height $H_0 = h_{st} = 4.1$ m for the first storey.

Generally, Tremuri predicts higher values for the lateral resistance and displacement capacity compared to the hand calculations. This was expected as hand calculations tend to be overly conservative. When investigating where the differences come from, the first explanation is the difference in the height of zero moment (H_0). In the hand calculations, the height was first set to the storey height. However, when calculating the height of zero moment given in Tremuri, which is obtained by dividing the base moment by the base shear for each pier, it is clear that this assumption is not exact and it is only an approximation. Moreover, the value of H_0 is not constant and varies during the analysis. This variation is explained by the redistribution of the forces between the piers due to the spandrels and the slab. An interesting approach is to evaluate the influence it has on the results when the height of zero moment in the hand calculations is replaced by the one obtained by Tremuri. This modification leads to more accurate results for the lateral resistance. Another parameter that played a role in the difference between the two models is the variation of the axial load. When looking into the values of the axial load in every pier, it is also varies during the test and is not constant as stated in the hand calculations. Again, this is due to forces redistribution thanks to spandrels and slabs. If the axial load inserted in the hand calculations is replaced by the one given in Tremuri, the estimation of the lateral resistance is even more accurate. The results obtained with the axial load and the height of zero moment computed by Tremuri are shown in Table 7. It is also observed that the tendency that the lateral force in the y direction is greater than in the x direction, which is given by Tremuri, is now correctly predicted in the hand calculations as well.

On the other hand, it can be observed that the difference of the displacement capacity between numerical and hand calculations is high for the y direction. When looking at the failure mode in the y direction, the dominant failure was a flexural mode. On the other hand, the failure mode is shear dominant in the x direction, because of the large pier in the south facade (Fig. 12). The assumption made for hand calculations regarding the deformed shape is that plastic deformations are concentrated only at the ground floor level and there are only shear or translational and not rotational deformations at the storey level, whereas in Tremuri, rotation of the floors is accounted.[27] This assumption implies that displacement capacity is better approximated for failure governed by shear rather than flexure.

OUT-OF-PLANE STRUCTURAL ASSESSMENT

Masonry structures are also prone to out-of-plane failure modes due to a lack of effective floor-to-wall, roof-to-wall, and wall-to-wall connections. It is therefore crucial to verify the out-of-plane resistance of masonry walls, especially since Tremuri does not allow to evaluate this behaviour. The method described here is based on the Italian code and is recalled by Godio and Beyer.[28] The code proposes displacement-based and force-based methods for the life-safety limit state and damage limit state, except for the displacement-based method which is not suitable for the damage limit state.

Results for the out-of-plane mechanisms assessment are presented facade by facade in the following. Both observed and plausible mechanisms were studied. The mechanism under investigation is first drawn on the facade and a sketch is presented to illustrate the way it deforms. Then, the results for force-based and displacement-based methods are

27 BEYER, KATRIN, et al., 2015
28 GODIO, MICHELE; BEYER, KATRIN, 2019, 454-475

Tablica 8. Rezultati temeljeni na primjeni sila i na primjeni pomaka za procjenu rotacijskog mehanizma južnog pročelja izvan ravnine. Ocjenjuju se granično stanje oštećenja (q = 1) i granično stanje sigurnosti ljudskih života (q = 2).

Južno pročelje	Metoda temeljena na primjeni sila	Metoda temeljena na pomaku
Granično stanje oštećenja	0,58	-
Granično stanje sigurnosti ljudskih života	1,15	7,88

Tablica 9. Rezultati temeljeni na primjeni sila i na primjeni pomaka za procjenu mehanizma prevrtanja južnog pročelja izvan ravnine. Ocjenjuje se granično stanje oštećenja (q = 1) i granično stanje sigurnosti ljudskih života (q = 2).

Južno pročelje	Metoda temeljena na primjeni sila	Metoda temeljena na pomaku
Granično stanje oštećenja	0,96	-
Granično stanje sigurnosti za život	1,32	3,35

dominantan lom je bio uslijed savijanja. S druge strane, dominantna način loma je uslijed horizontalnog pomaka u smjeru x, zbog velikog stupca na južnom pročelju (sl. 12). Pretpostavka za ručne izračune u odnosu na deformirani oblik jest da su plastične deformacije koncentrirane samo na razini prizemlja te da postoje samo posmične ili translacijske, a ne rotacijske deformacije na razini kata, dok se u Tremuriju uzima u obzir rotacija katova.[22] Ta pretpostavka implicira da je kapacitet pomaka bolje aproksimiran za lom uslijed posmika nego uslijed savijanja.

PROCJENA KONSTRUKCIJE IZVAN RAVNINE

Zidane konstrukcije također su sklone načinima loma izvan ravnine uslijed nedostatka učinkovitih veza pod-zid, krov-zid i zid-zid. Stoga je ključno provjeriti otpornost zidane konstrukcije na djelovanja izvan ravnine, posebice zato što Tremuri ne omogućava procjenu toga ponašanja. Ovdje opisana metoda temelji se na talijanskim propisima, a na nju upućuju autori Godio i Beyer.[23] Propisi predlažu metode temeljene na pomaku i silama za granično stanje sigurnosti ljudskih života i granično stanje oštećenja, osim metode temeljene na pomaku koja nije prikladna za granično stanje oštećenja.

Rezultati za procjenu mehanizama izvan ravnine prikazani su u nastavku za svako pročelje. Proučavani su i promatrani i mogući mehanizmi. Ispitivani mehanizam najprije je nacrtan na pročelju, a prikazana je skica koja ilustrira način njegove deformacije. Zatim su prikazani rezultati za metode na temelju sile i na temelju pomaka. Glavno zapažanje pri usporedbi tih dviju metoda jest da pristup temeljen na sili uvijek daje manje indekse sigurnosti od pristupa temeljenog na pomaku. Valja primijetiti da je i pristup temeljen na pomaku prikladan samo za granično stanje sigurnosti ljudskih života.[24]

Južno pročelje

Proučavani mehanizam na južnom pročelju ilustriran je slikovnim prilogom 17, a odgovara prevrtanju pročelja s dijagonalnim pukotinama.[25] Određena su dva parametra, a i b, te varirana kako bi se pronašla minimalna vrijednost indeksa sigurnosti. Taj mehanizam odgovara promatranom mehanizmu u stvarnoj zgradi. Čini se da pukotine doista koincidiraju s prevrtanjem pročelja s dijagonalnim pukotinama. Ovoj vrsti mehanizma može se pristupiti na dva različita načina: bilo pretpostavljajući da dva makroelementa rotiraju u odnosu na jedan okomiti i dva dijagonalna zgloba, ili da se elementi prevrću u odnosu na dva dijagonalna zgloba, s tim da se najniži kut smatra točkom rotacije. U nastavku je procjena dviju konfiguracija.

Rezultat dobiven za mehanizam koji uzima u obzir rotaciju dvaju makroelemenata prikazan je u tablici 8. Valja primijetiti da je u ovom slučaju potrebno prvo izračunati otpor okomito postavljenih zidova. To se može izračunati kako su opisali Beolchini et al.[26] Čini se da se kod simetričnog mehanizma pojavljuje vrijednost za dva parametra čiji je indeks sigurnosti minimalan kada je kut $\psi = 1$. To znači kada je a = b = 5 [m]. Prilikom provjere graničnog stanja sigurnosti ljudskih života (q = 2) dovelo je do indeksa sigurnosti od $\alpha = 1{,}15$ za pristup koji se temelji na sili i $\alpha = 7{,}88$ za pristup koji se temelji na pomaku. Za granično stanje oštećenja (q = 1) je indeks sigurnosti $\alpha = 0{,}57$, metoda koja se temelji na pomaku nije prikladna za ovo granično stanje.[27] Vrijednost manja od 1 za granično stanje oštećenja (PGA=0.072 g) ukazuje na to da se mehanizam trebao početi razvijati, što potvrđuje i promatranje pukotina na zgradi.

Za mehanizam koji uzima u obzir prevrtanje elemenata, rezultati su prikazani u tablici 9. Ponovno nisu ispunjeni uvjeti za granično stanje oštećenja (PGA=0.072 g).

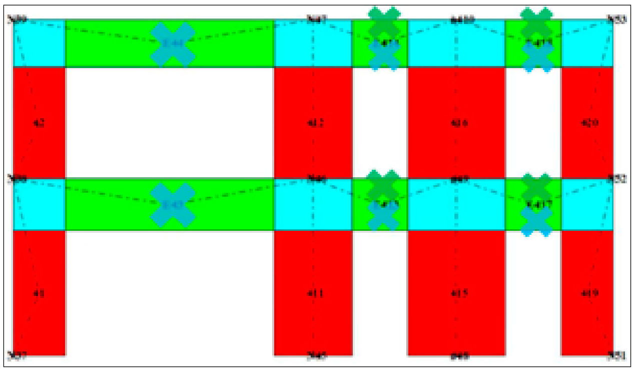
22 BEYER, KATRIN, et al., 2015.
23 GODIO, MICHELE; BEYER, KATRIN, 2019., 454-475.
24 GODIO, MICHELE; BEYER, KATRIN, 2019., 454-475.
25 D'AYALA, DINA, 2013., 334-365.
26 BEOLCHINI, GIOVANNI C.; MILANO, LUCIA; ANTONACCI, ELENA, 2005., 2.
27 GODIO, MICHELE; BEYER, KATRIN, 2019., 454-475.

Table 8 Force-based and displacement-based results for the out-of-plane evaluation of the southern facade rotational mechanism. Damage limit state (q = 1) and life-safety limit state (q = 2) are evaluated.

South façade	Force-based method	Displacement-based method
Damage limit state	0.58	-
Life-safety limit state	1.15	7.88

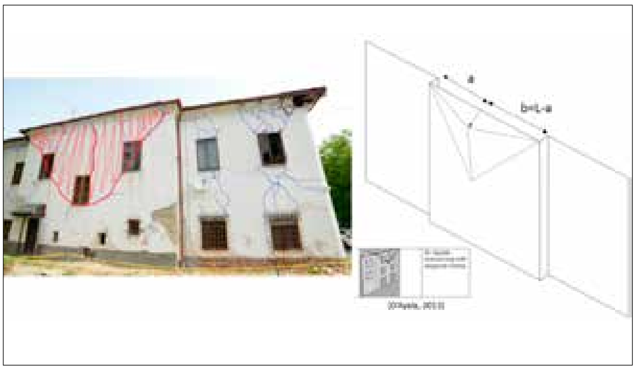
Table 9 Force-based and displacement-based results for the out-of-plane evaluation of the southern facade overturning mechanism. Damage limit state (q = 1) and life-safety limit state (q = 2) are evaluated.

South façade	Force-based method	Displacement-based method
Damage limit state	0.96	-
Life-safety limit state	1.32	3.35



16 Prediction given by Tremuri for the east facade. Crosses indicate elements that have failed (drawing: Thibaud Maillard, 2023)

Predviđanje iz Tremurija za istočno pročelje. Križići označavaju elemente u kojima je došlo do loma (crtež: Thibaud Maillard, 2023.)



17 Out-of-plane mechanism in the south façade (photo and drawing: Thibaud Maillard, 2023)

Mehanizam izvan ravnine na južnom pročelju (foto i crtež: Thibaud Maillard, 2023.)

shown. The main observation when comparing the two methods is that the force-based approach always gives smaller compliance factors than the displacement-based one. Notice also that the displacement-based approach is only suitable for life-safety limit state.[29]

South façade

The mechanism studied in the south facade is illustrated in Figure 17, and it corresponds to a facade overturning with diagonal cracks.[30] Two parameters, a and b, have been set and varied to find the minimum value of the compliance factor. This mechanism is one that can be observed in the real building. The cracks seem indeed to coincide with a facade overturning with diagonal cracks. This kind of mechanism can be approached by two different ways; either assuming that the two macroelements are rotating with respect to one vertical and two diagonal hinges, or that the elements are overturning with respect to the two diagonal hinges with the lowest corner considered as the rotation point. The two configurations are evaluated hereafter.

The result obtained for the mechanism considering the rotation of the two macroelements is presented in Table 8. Notice that in this case there is the need to compute first the horizontal restrains coming from the orthogonal walls. This can be calculated as described by Beolchini et al.[31] The value for the two parameters for which the compliance factor is minimal when the angle $\psi = 1$ appears to happen when the mechanism is symmetrical. That is to say when a = b = 5 [m]. When verifying the life-safety limit state (q = 2), it led to a compliance factor of $\alpha = 1.15$ for the force-based approach and $\alpha = 7.88$ for the displacement-based one. For a damage limit state (q = 1), the compliance factor is $\alpha = 0.57$ and the displacement-based method is not suitable for this limit state.[32] A value lower than 1 for the damage limit state (PGA=0.072 g) indicates that the mechanism should have started to develop, which is confirmed by the observation of the cracks in the house.

For the mechanism considering the overturning of the elements, the results are shown in Table 9. The conditions were again not satisfied for the damage limit state (PGA=0.072 g).

29 GODIO, MICHELE; BEYER, KATRIN, 2019, 454-475
30 D'AYALA, DINA, 2013, 334-365
31 BEOLCHINI, GIOVANNI C.; MILANO, LUCIA; ANTONACCI, ELENA, 2005
32 GODIO, MICHELE; BEYER, KATRIN, 2019, 454-475

Tablica 10. Rezultati temeljeni na primjeni sila i na primjeni pomaka za procjenu rotacijskog mehanizma istočnog pročelja izvan ravnine. Ocjenjuje se granično stanje oštećenja (q = 1) i granično stanje sigurnosti života(q = 2).

Istočno pročelje	Metoda temeljena na primjeni sila	Metoda temeljena na pomaku
Granično stanje oštećenja	0,62	-
Granično stanje sigurnosti za život	1,23	5,19

Tablica 11. Rezultati temeljeni na primjeni sila i temeljeni na primjeni pomaka za procjenu rotacijskog mehanizma istočnog pročelja izvan ravnine. Ocjenjuje se granično stanje oštećenja (q = 1) i granično stanje sigurnosti života (q = 2).

Istočno pročelje	Metoda temeljena na primjeni sila	Metoda temeljena na pomaku
Granično stanje oštećenja	2,15	-
Granično stanje sigurnosti za život	4,29	4,75

Tablica 12. Rezultati temeljeni na primjeni sila i na primjeni pomaka za procjenu istočnog pročelja izvan ravnine. Ocjenjuju se konfiguracije s beskonačnom i konačnom tlačnom čvrstoćom.

Istočno pročelje	Metoda temeljena na primjeni sila	Metoda temeljena na pomaku
Beskonačna tlačna čvrstoća (q=1)	0,99	-
Konačna tlačna čvrstoća (q=1)	0,94	-
Beskonačna tlačna čvrstoća (q=2)	1,99	3,03
Konačna tlačna čvrstoća (q=2)	1,87	2,79

Istočno pročelje

Proučeni mehanizam na južnom pročelju ilustriran je slikovnim prilogom 18. Prema D'Ayala,[28] odgovara prevrtanju pročelja s dijagonalnim pukotinama. Određena su dva parametra, a i b, te varirana kako bi se pronašla minimalna vrijednost indeksa sigurnosti. Taj je mehanizam promatran i u zgradi. Čini se da pukotine doista koincidiraju s prevrtanjem pročelja s dijagonalnim pukotinama. I ovdje se problemu može pristupiti na dva različita načina: kao rotacija makroelemenata u odnosu na okomite i dijagonalne zglobove te kao prevrtanje elemenata izvan ravnine. Dvije konfiguracije su provjerene u nastavku, kao što je učinjeno s južnom fasadom.

U pogledu rotacije makroblokova, rezultati se nalaze u tablici 10. Kada se navodi da je a = 3 [m], b = 9 [m] i c = 1,9 [m], analiza je dala najniže vrijednosti indeksa sigurnosti. Za takvu geometriju procjena je ukazala na to da se mehanizam trebao dogoditi, kao što se vidi s indeksom sigurnosti graničnog stanja oštećenja manjim od 1, što potvrđuju uočene pukotine na zgradi. Zanimljivo je istaknuti da je mehanizam nastao upravo na spoju s drvenim nadvojima prvog kata.

Mehanizam preokretanja s istom geometrijom koja je prethodno pronađena daje indekse sigurnosti u tablici 11. S obzirom na veće indekse, ovaj se mehanizam čini manje vjerojatnim od prethodnog s rotacijom makroblokova.

Drugi mehanizam istočnog pročelja, koji nije uočen u zgradi ali se čini vjerojatnim jer su nadvoji često slabi elementi (sl. 19). Budući da su veze na razini prvog kata frikcijske, sa samo tri drvene grede, vjerojatnost pomaka izvan ravnine je velika. Zbog toga je ovaj mehanizam moguć. Za ovaj slučaj procijenjene su dvije konfiguracije. Prva pretpostavlja beskonačnu tlačnu čvrstoću ziđa, dok drugi slučaj pokazuje utjecaj ako se pretpostavlja konačna tlačna čvrstoća ziđa.

Rezultati pokazuju da su indeksi sigurnosti manji ako se u obzir uzima konačna tlačna čvrstoća. Ova pretpostavka doista mijenja krak sile jer se rotacija ne događa u jednoj točki na kutu elementa, već cijelom duljinom tlačnog djelovanja izračunatog ekvilibrijem. Osim toga, pri procjeni mehanizma za granično stanje oštećenja, indeks sigurnosti ukazuje na to da je mehanizam trebao biti iniciran.

Sjeverno pročelje

Proučeni mehanizam sjevernog pročelja ilustriran je slikovnim prilogom 20. Prema D'Ayala,[29] odgovara prevrtanju pročelja s vertikalnim pukotinama. Proučavane su dvije konfiguracije: sa i bez čeličnih zatega. Taj je mehanizam uočen na zgradi zbog okomitih pukotina vidljivih u unutrašnjosti zgrade točno na mjestu spoja izbočenog dijela. Čini se da okomite pukotine na unutarnjim zidovima doista ukazuju na prevrtanje fasade. Rezultati u tablici 13 pokazuju da, kao što se i očekivalo, čelične zatege snažno utječu na indeks sigurnosti jer prisutnost čeličnih zatega dodaje dodatnu stabilizirajuću silu statičkom sustavu.

28 D'AYALA, DINA, 2013., 334-65.

29 D'AYALA, DINA, 2013., 334-65.

Table 10 Force-based and displacement-based results for the out-of-plane evaluation of the eastern facade rotational mechanism. Damage limit state (q = 1) and life-safety limit state (q = 2) are evaluated.

East facade	Force-based method	Displacement-based method
Damage limit state	0.62	-
Life-safety limit state	1.23	5.19

Table 11 Force-based and displacement-based results for the out-of-plane evaluation of the eastern facade rotational mechanism. Damage limit state (q = 1) and life-safety limit state (q = 2) are evaluated.

East facade	Force-based method	Displacement-based method
Damage limit state	2.15	-
Life-safety limit state	4.29	4.75

Table 12 Force-based and displacement-based results for the out-of-plane evaluation of the eastern facade. Configurations with infinite and finite compressive strength are evaluated.

East facade	Force-based method	Displacement-based method
Infinite comp. strength (q=1)	0.99	-
Finite comp. strength (q=1)	0.94	-
Infinite comp. strength (q=2)	1.99	3.03
Finite comp. strength (q=2)	1.87	2.79

East facade

The first mechanism studied in the east facade is illustrated in Figure 18. According to D'Ayala[33], it corresponds to a facade overturning with diagonal cracks. Two parameters, a and b, have been set and varied to find the minimum value of the compliance factor. This mechanism is observed in the building. The cracks seem indeed to coincide with a facade overturning with diagonal cracks. Here again, the problem can be tackled in two different ways; the rotation of the macroelements with regard to vertical and diagonal hinges, and the overturning of the elements falling out-of-plane. The two configurations are verified below, as it was done with the south facade.

Regarding the rotation of the macroblocks, the results are found in Table 10. When stating a = 3 [m], b = 9 [m] and c = 1.9 [m], the analysis gave the lowest values for the compliance factor. For such geometry, the assessment indicated that the mechanism should have happened, as seen with the compliance factor of the damage limit state lower than 1, which is confirmed by the observed cracks in the house. It is interesting to highlight that the mechanism formed right at the connection with the wooden super beams of the first floor.

The overturning mechanism with the same geometry found previously gives the compliance factors in Table 11. Given the greater factors, this mechanism seems less plausible than the previous one with a rotation of the macroblocks.

The second mechanism of the eastern facade, which is not observed in the building but might be a plausible one because the spandrels are often the weak elements (Fig. 19). Because the connections at the first-floor level are friction ones with only three wooden beams, the plausibility of out-of-plane movements is high. This is why this mechanism might be possible. For this case, two configurations were evaluated. The first one assumes an infinite compressive strength for masonry, whereas the second case shows the influence when assuming a finite compressive strength of the masonry.

The results show that when considering a finite compressive strength, the compliance factors are smaller. This assumption indeed changes the lever arm because the rotation does not occur on a single point at the corner of the element but over a compression length calculated by equilibrium. Moreover, when assessing the mechanism for the damage limit state, the compliance factor indicates that the mechanism should have initiated.

North façade

The mechanism studied in the north facade is illustrated in Figure 20. According to D'Ayala,[34] it corresponds to a facade overturning with vertical cracks. Two configurations are studied; with and without steel ties. This mechanism was observed on the building, due to the vertical cracks seen in the interior of the building right where the extruded part is connected. The vertical cracks in the interior walls seem indeed to indicate a facade overturning. The results

33 D'AYALA, DINA, 2013, 334-365

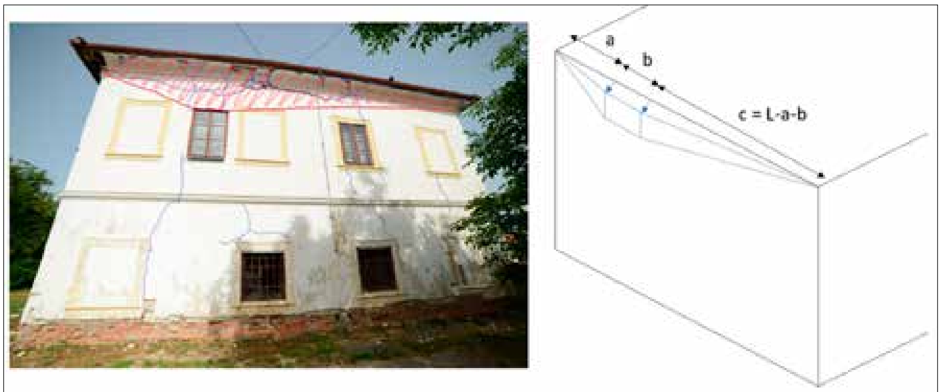
34 D'AYALA, DINA, 2013, 334-365

Tablica 13. Rezultati temeljeni na primjeni sila i na primjeni pomaka za procjenu ponašanja sjevernog pročelja izvan ravnine za slučaj sa i bez čeličnih zatega. Ocjenjuje se granično stanje oštećenja (q = 1) i granično stanje sigurnosti života (q = 2).

Sjeverno pročelje	Metoda temeljena na primjeni sila	Metoda temeljena na pomaku
Bez čelične zatege (q=1)	1,10	-
Sa čeličnom zategom (q=1)	2,98	-
Bez čelične zatege (q=2)	2,19	4,52
Sa čeličnom zategom (q=2)	5,96	12,27

Tablica 14. Rezultati temeljeni na primjeni sila i na primjeni pomaka za procjenu ponašanja zapadnog pročelja izvan ravnine za mehanizam od jednog i dva kata. Ocjenjuje se granično stanje oštećenja (q = 1) i granično stanje sigurnosti života (q = 2).

Zapadno pročelje	θ	Metoda temeljena na primjeni sila	Metoda temeljena na pomaku
Više od dva kata (q=1)	30,1°	1,00	-
Više od jednog kata (q=1)	67,7°	1,00	-
Više od dva kata (q=2)	24,4°	1,00	1,22
Više od jednog kata (q=2)	5,5°	1,00	2,15



18 Out-of-plane mechanism n°1 in the east façade (photo and drawing: Thibaud Maillard, 2023)

Mehanizam izvan ravnine br. 1 na istočnom pročelju (foto i crtež: Thibaud Maillard, 2023.)

Zapadno pročelje

Proučeni mehanizam zapadnog pročelja ilustriran je slikovnim prilogom 21. Prema D'Ayala,[30] odgovara prevrtanju pročelja s dva bočna zida. Proučavane su dvije konfiguracije: prevrtanje cijelog pročelja (višeg od dva kata) i prevrtanje samo jednog kata, što se čini realnijim jer nisu uočene pukotine na spojevima svod-zid na razini prizemlja. Stoga sugerira da se mehanizam izvan ravnine dogodio samo na najvišem katu. Ali dva su mehanizma proučavana kako bi se usporedile njihove vrijednosti i vjerodostojnost s obzirom na indekse sigurnosti.

Variranjem kuta θ, koji definira oblik bočnih zidova, moguće je pronaći minimalni kut potreban da se zadovolji indeks sigurnosti dan metodom primjene sila. Ponavljanjem kuta, za granično stanje sigurnosti ljudskih života, primjerice, θ = 24,4° za slučaj s više od dva kata i θ = 5,5° kada se mehanizam događa samo iznad gornjeg kata. Rezultati su prikazani u tablici 14.

IDEJNI PRIJEDLOZI REKONSTRUKCIJE

Uslijed velikih oštećenja, konstrukcija ne bi mogla izdržati daljnje jake potrese te je njezina stabilnost vrlo upitna. Iz tog razloga zgradu treba ojačati odgovarajućim rješenjima. Idealno rješenje bila bi kombinacija nekoliko metoda. U ovom odjeljku proučavaju se neki mogući zahvati rekonstrukcije kakvi bi mogli biti provedeni u župnom dvoru. Opisuje se njihov koncept te raspravlja o njihovom učinku na konstrukciju. Obrađena su i dodatna istraživanja koje je potrebno provesti nakon uvođenja tih metoda. Uslijed posebnosti građevina povijesne baštine, prikladnost zahvata argumentirana je i nekim kriterijima. Međutim, nije provedena analiza zgrade nakon uvođenja metoda rekonstrukcije. To je tema za daljnji rad.

Specifičnost povijesnih građevina jest zahtjev za očuvanjem kulturne vrijednosti građevine, uzimajući u obzir i materijalne i nematerijalne vrijednosti. Kako bi se ispunio ovaj cilj, potrebno je odabrati manje invazivne metode

30 D'AYALA, DINA, 2013., 334-365.

Table 13 Force-based and displacement-based results for the out-of-plane evaluation of the northern facade for the case with and without steel ties. Damage limit state (q = 1) and life-safety limit state (q = 2) are evaluated.

North facade	Force-based method	Displacement-based method
Without steel tie (q=1)	1.10	-
With steel tie (q=1)	2.98	-
Without steel tie (q=2)	2.19	4.52
With steel tie (q=2)	5.96	12.27

Table 14 Force-based and displacement-based results for the out-of-plane evaluation of the western facade for a mechanism over one and two storeys. Damage limit state (q = 1) and life-safety limit state (q = 2) are evaluated.

West facade	θ	Force-based method	Displacement-based method
Over two storeys (q=1)	30.1°	1.00	-
Over one storey (q=1)	67.7°	1.00	-
Over two storeys (q=2)	24.4°	1.00	1.22
Over one storey (q=2)	5.5°	1.00	2.15

in Table 13 show that, as expected, steel ties strongly influence the compliance factor as the presence of steel ties adds an additional restraining force to the static system.

West façade

The mechanism involving the west facade is shown in Figure 21. According to D'Ayala,[35] it corresponds to a facade overturning with two side walls. Two configurations are studied; overturning of the whole facade (over the two storeys) and overturning of only one storey, which seems more realistic because no cracks were observed at the vaults-wall connections at the ground floor level. Therefore, it suggests that the out-of-plane mechanism happened only at the top storey. But the two mechanisms are studied to compare their values and plausibility with regard to the compliance factors.

Varying the angle θ defining the shape of the side walls, it is possible to find the minimum angle needed to satisfy the compliance factor given by the force-base method. Iterating the angle, it results, for life-safety limit state for example, in $\theta = 24.4^{\circ}$ for the case over two storeys and $\theta = 5.5^{\circ}$ when the mechanism happens only over the top storey. The results are shown in Table 14.

CONCEPTUAL RETROFITTING PROPOSALS

Due to a heavy damage, the structure would not be able to withstand further strong earthquakes and its stability is highly questionable. For this reasons, the building should be strengthened with adequate solutions. The ideal solution would be a combination of several methods. This section studies some possible retrofitting interventions that could be introduced in the parish house. Their concept is described and their structural effect is discussed. Additional investigations that should be done after the introduction of these methods are also addressed. Because of the particularity of historical heritage buildings, the suitability of the interventions is also argued with regard to some criteria. However, no analysis of the building after the introduction of retrofitting methods was performed. This is the topic for further work.

The specificity of historical buildings is in the requirement to maintain a cultural value of the building, both considering tangible and intangible values. To fulfill this goal, less invasive methods should be chosen in order to satisfy the "minimum intervention" approach[36] and several more criteria in the following:

- **Aestheticism**

  A method should have the least possible impact on aestheticism and authenticity of the historical building.
- **The invasive/intrusive aspect**

  Non-invasive strengthening measures should be used to ensure the integrity of the heritage structure, meaning it should not cause any additional damage to historical fabric.
- **Effectiveness**

  The effect the retrofitting method has on the building's behaviour needs to be assessed with nonlinear analysis in further work. Nonlinear analysis allow us to choose less conservative solutions, that fit this set of objectives.
- **Reversibility/removability**

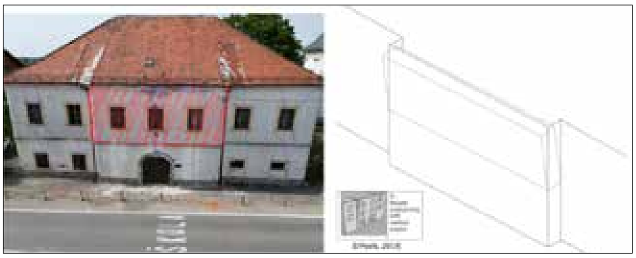
  In the case of a discovery of a new or more suitable retrofitting technique, the present retrofitting intervention should be easily removed to let place for the new and better one.

35 D'AYALA, DINA, 2013, 334-365

36 ICOMOS, 1964

19 Out-of-plane mechanism n°2 in the east façade (photo and drawing: Thibaud Maillard, 2023)

Mehanizam izvan ravnine br. 2 na istočnom pročelju (foto i crtež: Thibaud Maillard, 2023.)

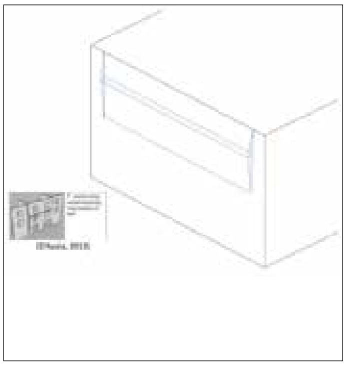


20 Out-of-plane mechanism in the north façade (photo and drawing: Thibaud Maillard, 2023)

Mehanizam izvan ravnine na sjevernom pročelju (foto i crtež: Thibaud Maillard, 2023.)

21 Out-of-plane mechanism in the west façade (photo and drawing: Thibaud Maillard, 2023)

Mehanizam izvan ravnine na zapadnom pročelju (foto i crtež: Thibaud Maillard, 2023.)



22 Western facade plausible out-of-plane mechanism after the introduction of anchors connecting the top slab's beams to the exterior masonry walls (drawing: Thibaud Maillard, 2023)

Vjerojatan mehanizam zapadnog pročelja izvan ravnine nakon uvođenja sidara koja povezuju grede najviše ploče s vanjskim zidanim zidovima (crtež: Thibaud Maillard, 2023.)

kako bi se zadovoljio pristup „minimalne intervencije“[31] te još nekoliko kriterija, navedenih u nastavku:

• **Estetika**

Metoda treba imati najmanji mogući utjecaj na estetiku i autentičnost povijesne građevine.

• **Aspekt invazivnosti/nametljivosti**

Trebalo bi koristiti neinvazivne mjere ojačanja kako bi se osigurala cjelovitost konstrukcije zgrade, što znači da ne bi trebalo uzrokovati dodatna oštećenja povijesnom tkivu.

• **Učinkovitost**

U daljnjem radu je potrebno nelinearnom analizom procijeniti učinak koji metoda rekonstrukcije ima na ponašanje zgrade. Nelinearna analiza omogućava nam odabir manje konzervativnih rješenja koja odgovaraju ovom skupu ciljeva.

• **Reverzibilnost/uklonjivost**

U slučaju otkrića nove ili prikladnije tehnike rekonstrukcije, potrebno je omogućiti lako uklanjanje sadašnjeg zahvata rekonstrukcije kako bi se ostavilo mjesto za novu i bolju tehniku.

• **Kompatibilnost**

Interakcija između novih i izvornih materijala trebala bi biti fizički i mehanički kompatibilna kako bi se izbjeglo svako oštećenje postojećih materijala.

31 ICOMOS, 1964.

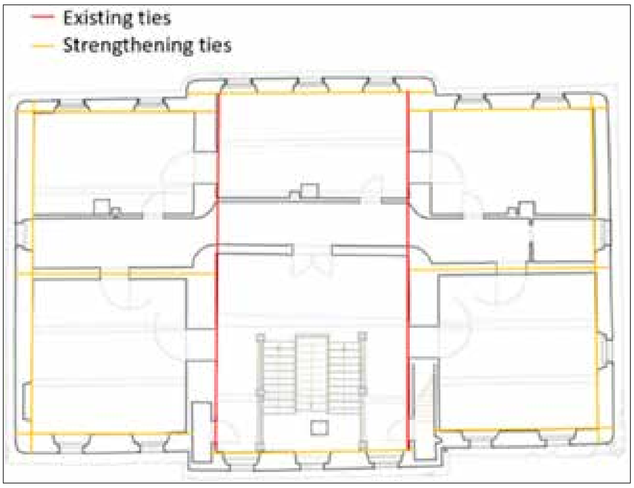
Župna kuća ima nekoliko mehanizama izvan ravnine, kao što je prikazano u prethodnom odjeljku. Glavni razlog kojim se objašnjava takvo ponašanje, bili su neprikladni spojevi između razine prvog kata i vanjskih zidova. Drvene grede jednostavno leže na zidovima, pri čemu se pretpostavlja da je duljina njihova ležaja polovina debljine zida, a pobudama izvan ravnine odupiru se samo silom trenja. Da bi se spriječili takvi mehanizmi, najvažnije je poboljšati spojeve između zidova i najvišeg kata. Stoga, kako bi se smanjila vjerojatnost razvoja mehanizama izvan ravnine, dobro je uvesti sidra za pričvršćivanje drvenih greda na zidove. Takav sustav sastoji se od čelične L-ploče spojene vijcima na drvene grede, a sidrima na zidove zidane konstrukcije. Sidrenjem greda spriječilo bi se klizanje i udaranje greda kada se pojave pobude izvan ravnine te bi se tako ograničila vjerojatnost stvaranja mehanizma loma izvan ravnine. Utjecaj na izračune utvrđen je u činjenici da bi se stabilizirajuća sila koja potječe od trenja između drvenih greda i zidova zidane konstrukcije promijenila u jaču stabilizirajuću silu (moment stabilizacije). Potrebno je kontrolirati provjeru je li sidrenje greda dovoljno. U uzdužnom smjeru postoje samo tri drvene grede, što znači da ova metoda možda neće biti dovoljna. Stoga se mora provjeriti učinkovitost ove tehnike. Unatoč pozitivnim učincima koje bi ta metoda mogla imati na stabilizaciju ovog mehanizma, sidrenje greda moglo bi stvoriti još jedan mehanizam preko najvišeg kata, kao što je prikazano na

• Compatibility

The interaction between the new materials and the original ones should be physically and mechanically compatible to avoid any damage to the existing materials.

The parish house has several out-of-plane mechanisms, as presented in the previous section. The main reason explaining this behaviour were the inappropriate connections between the first-floor level and the exterior walls. The wooden beams are simply lying on the walls, with an embedment length assumed to be half the thickness of the wall, and are resisting out-of-plane solicitations only by friction force. To prevent such mechanisms, the main point is in the improvement of the connections between the walls and the top floor. Therefore, to reduce the likelihood of developing out-of-plane mechanisms, the introduction of anchors fixing the wooden beams to the walls is in a good direction. Such system consists of an L-shaped steel plate connected with screws to the wooden beams and anchors to the masonry walls. By anchoring the beams, sliding and hammering of the beams, would be prevented when out-of-plane solicitations occur and thus limit the probability of the formation of an out-of-plane failure mechanism. The influence on the calculations is found in the fact that the restraining force coming from the friction between the timber beams and masonry walls would change to a stronger restraining force (stabilizing moment). The verification of whether the anchoring of the beams is sufficient needs to be controlled. In the longitudinal direction, only three wooden beams are present, meaning this method might not be sufficient. This is why the verification of the effectiveness of this technique must be evaluated. Despite the positive effects this method might have on stabilizing this mechanism, the anchoring of the beams could create another mechanism over the top storey, as shown in Figure 22. This new mechanism should also be assessed.

In case this method does not bring enough resistance, another complementary alternative should be considered. Possible solution is in combining the anchors with steel ties in the walls, as shown in Figure 23. This additional restraining force provided by the ties, in particular by the interior walls which are connected at the middle of the facade, would have a beneficial effect on the stability of the new mechanism presented in Figure 22 above. The introduction of steel ties, such as in the plan view presented in Figure 23, would at the same time reduce the deformations of the spandrels, bringing additional resistance and ductility to the building. Because most of the in-plane damages are located in spandrels, it might be useful to add horizontal tension



23 Steel ties introduced at the first floor level to strengthen the building (in orange) and existing ties (in red). The ties are placed in the interior side of the building for perimeter walls and in the middle of the thickness for the two interior walls (floor plan and drawing: Thibaud Maillard, 2023)

Čelični vodoravni serklaži uvedeni na razini prvog kata za ojačanje zgrade (narančasti) i postojeći vodoravni serklaži (crveni). Serklaži se izvode na unutarnjoj strani zgrade za obodne zidove te u središnjem dijelu za dva unutarnja zida (tlocrt i oznake: Thibaud Maillard, 2023.)

ties in every wall. The concept is to provide compressive stress in the masonry walls, in this case horizontally, impacting both the peak and residual behaviour of spandrels.[37] The ties would bring a confining effect to the wall and its strength and ductility would improve which will reduce the likelihood of developing cracks in the spandrels. The displacement capacity of the structure would be increased which will ensure sufficient capacity in case of a future earthquake stronger than the one reported in the codes by the response spectra. Nevertheless, the ties should be treated adequately to prevent any possibility of corrosion.

But before all, repairing existing damages is the first step in a strengthening procedure. To do so, mortar repointing would be the first step. This method consists in replacing partially the mortar in the joints with a new one. It should be applied in particular when the cracks are localized in the mortar. The new material should be carefully chosen to ensure compatibility with the original ones. However, this technique is only used when the damages are localized in the mortar. Therefore, where large cracks happened and in highly damaged zones, it should be investigated whether bricks have also failed or not in the damaged portion. In that case, partial reconstruction by removing the broken bricks and replacing them with new bricks of a similar material might be necessary.

37 BEYER, KATRIN, 2012, 533-547

slikovnom prilogu 22. Taj novi mehanizam također treba procijeniti.

U slučaju da ova metoda ne osigurava dovoljnu otpornost, treba razmotriti drugu komplementarnu alternativu. Moguće rješenje je u kombinaciji sidara sa čeličnim zategama u zidovima, kao što je prikazano na slikovnom prilogu 23. Ova dodatna stabilizirajuća sila koju osiguravaju zatege, posebice unutarnji zidovi koji su povezani na sredini pročelja, imala bi povoljan učinak na stabilnost novog mehanizma prikazanog u gornjem slikovnom prilogu 22. Uvođenje čeličnih zatega, kao na tlocrtu prikazanom na slikovnom prilogu 23, istovremeno bi smanjilo deformacije nadvoja, čime bi zgrada dobila dodatnu otpornost i duktilnost. Budući da do većine oštećenja u ravnini dolazi u nadvojima, moglo bi biti korisno dodati vodoravne vlačne zatege u svaki zid. Idejno rješenje jest osigurati tlačno naprezanje u zidovima zidane konstrukcije, u ovom slučaju vodoravno, što bi utjecalo i na vršno i na rezidualno ponašanje nadvoja.[32] Zatege bi za zid imale stabilizirajući učinak, a njegova čvrstoća i duktilnost bi se poboljšali, što bi smanjilo vjerojatnost razvoja pukotina u nadvojima. Kapacitet pomaka konstrukcije bi se povećao, čime bi se kroz spektre odgovora osigurao dovoljan kapacitet u slučaju budućeg potresa jačeg od onog navedenog u propisima. Unatoč tome, zatege treba prikladno obraditi kako bi se spriječila svaka mogućnost korozije.

No, prije svega, sanacija postojećih oštećenja je prvi korak u postupku ojačanja. Da bi se to postiglo, prvi bi korak bio ponovno fugiranje žbukom. Ta se metoda sastoji se u djelomičnoj zamjeni morta u fugama novim. Naročito ga treba primijeniti ako su pukotine u žbuci lokalizirane. Potreban je poman odabir novog materijala kako bi se osigurala njegova kompatibilnost s izvornim materijalom. Međutim, ova tehnika se koristi samo ako su oštećenja lokalizirana u žbuci. Stoga, tamo gdje su se pojavile velike pukotine i u jako oštećenim zonama, treba u oštećenim dijelovima ispitati stanje opeka. U tom slučaju može biti potrebna djelomična rekonstrukcija uklanjanjem razbijenih opeka i njihovom zamjenom novom opekom od sličnog materijala.

ZAKLJUČCI

Zidane konstrukcije mogu biti vrlo osjetljive na potrese. Ključna je potpuna i točna procjena njihova ponašanja pod seizmičkim djelovanjem. Cilj ovoga rada jest predložiti temeljitu metodologiju za procjenu zgrada zidane konstrukcije kroz studiju slučaja župne kuće u Selima, u Hrvatskoj. Opći cilj ove studije bio je pomoći u budućoj seizmičkoj procjeni i rekonstrukciji sličnih zidanih zgrada te dati prvi model župne kuće.

U ovom radu korištene su dvije različite metode za procjenu župne kuće: numerička analiza korištenjem softvera Tremuri te ručni izračuni kojima se proučava i ponašanje u ravnini i izvan ravnine. Da bi se to postiglo, izrađen je model zgrade s nearmiranom zidanom konstrukcijom upotrebom metode ekvivalentnih okvira (Equivalent Frame Method ili EFM) i obrađen upotrebom softvera Tremuri. Usporedba ponašanja dvaju modela u ravnini pokazuje da su ručni izračuni, prema očekivanjima, konzervativniji pri procjeni župne kuće. Slično ponašanje uočeno je pri usporedbi pojedinačnih zidova dvaju modela. Rezultati za cijelu zgradu, nakon modifikacije nekih parametara preuzetih izravno iz Tremurija, razlikuju se za otprilike 13 % do 10 % za pobude u osima x odnosno y. Što se tiče kapaciteta pomaka, konstrukcija je sposobna podnijeti od 28% do 115% za pozitivnu x, odnosno y os. Procjena izvan ravnine provedena je kinematičkom analizom na temelju uočenih ili vjerojatnih mehanizama. Čini se da su četiri mehanizma nestabilna u pogledu njihovih indeksa sigurnosti za granično stanje oštećenja, ali svi su stabilni u pogledu graničnog stanja sigurnosti ljudskih života.

Konstrukcija župne kuće je preslaba da izdrži buduće potrese. Potrebni su ozbiljni zahvati na stabilizaciji zidova i posebice prvog kata. Ipak, kada su u pitanju povijesne građevine s baštinskom vrijednošću kao što je župna kuća, dodatni izazovi se pojavljuju u pogledu mjera rekonstrukcije. Kulturna baština se mora očuvati, što pak dovodi do pitanja kako ojačati staru zgradu bez utjecaja na njezinu povijesnu vrijednost. Nekoliko je mogućnosti izvedivo, ali treba pronaći pravu ravnotežu između izvedbe i nametljivosti. Zadatak je složen, ali su u radu dane neke smjernice i raspravljeni su njihovi učinci na konstrukciju. Čini se da bi kombinacija ojačanja spojeva prvog kata sidrima i uvođenje čeličnih zatega za izbjegavanje lomova izvan ravnine mogla biti najizvedivija metoda. Uz to je potrebno pomno ispitati postojeće pukotine i oštećenja radi popravka oštećene žbuke odgovarajućim novim materijalima ili rekonstrukcije dijelova nadvoja u slučaju prevelikog oštećenja tih zona.

Što se tiče mogućnosti poboljšanja ovoga članka, glavna točka koja u njemu nedostaje za potpunu seizmičku procjenu zgrade jest procjena ponašanja konstrukcije nakon primjene metoda rekonstrukcije. Razmatranje prikladnosti i učinkovitosti zahvata na nekoj oštećenoj građevini je bitna točka koju treba razmotriti u budućem radu.

Daljnja istraživanja s drugim modelima mogla bi biti smjer za poboljšanje razumijevanja ponašanja konstrukcije. Konkretno, simulacije u softveru, gdje je moguća interakcija načina loma i u ravnini i izvan ravnine, mogle bi biti zanimljiva tema za daljnja istraživanja. Ta bi posebnost mogla biti vrlo zanimljiva upravo u slučaju župne kuće jer su u konstrukciji uočeni mnogi mehanizmi izvan ravnine sloma uslijed nedavnih potresa.

32 BEYER, KATRIN, 2012., 533-547.

CONCLUSIONS

Masonry buildings can be highly vulnerable to earthquakes. A complete and accurate assessment of their behaviour under seismic actions is crucial. This work aims at proposing a thorough methodology for assessing masonry buildings through the case study of the parish house of Sela, in Croatia. The overall goal of this study was to help with future seismic assessment and retrofitting of similar masonry buildings and to provide a first model of the parish house.

This paper used two different methods to evaluate the parish house; a numerical analysis using the software Tremuri and hand calculations studying both in-plane and out-of-plane behaviour. To do so, a model of an unreinforced masonry building using the Equivalent Frame Method was created and run using Tremuri software. A comparison of the in-plane behaviour of the two models shows that the hand calculations are, as expected, more conservative when assessing the parish house. The similar behaviour was observed when comparing the two models wall by wall. The results for the whole building, after modification of some parameters taken directly from Tremuri, differ from about 13% to 10%, respectively for positive x and y axis solicitations. Regarding displacement capacity, the structure is capable of supporting from 28% to 115%, for positive x and y axis respectively. The out-of-plane assessment was done with kinematic analysis based on observed or plausible mechanisms. Four mechanisms seem unstable regarding their compliance factors for the damage limit state but all of them are stable with regard to the life-safety limit state.

The parish house structure is too weak to withstand future earthquakes. Heavy interventions is needed to stabilize the walls and the first floor in particular. Nevertheless, when it comes to historical buildings with heritage value such as the parish house, additional challenges comes up regarding retrofitting measures. The cultural heritage must be conserved and it leads to questions on how to strengthen an old building without impacting its historical value. Several possibilities are feasible but the right equilibrium between performance and intrusiveness should be found. The task is complex but some directions are given in the paper and their structural effects are discussed. It appears that a combination of a reinforcement of the first-floor connections with anchors and the introduction of steel ties to avoid out-of-plane failures might be the most feasible methods. Additionally, the existing cracks and damages should be carefully investigated to repair the damaged mortar with appropriate new materials or reconstruct portions of the spandrels when the zones are too heavily damaged.

Concerning the possibilities of improving this work, the main point missing in this paper for having a complete seismic assessment of the building is the evaluation of the structural behaviour after the introduction of retrofitting methods. Considering the suitability and effectiveness of an intervention on a damaged building is an essential point that should be considered in the future work.

Further investigations with other models might be a direction to improve the understanding of the behaviour of the structure. In particular, simulations in software where the interaction of both in-plane and out-of-plane failure modes are possible could be an interesting point for further studies. This particularity might be of high interest in the case of the parish house as many out-of-plane failure mechanisms have been observed in the structure due to the recent earthquakes.

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