INDIKATORI OŠTEĆENJA OBIČNIH NEARMIRANIH ZIDANIH ZIDOVA IZLOŽENI SEIZMIČKIM DJELOVANJIMA

DAMAGE INDICATORS FOR UNREINFORCED MASONRY BUILDING WALLS SUBJECTED TO SEISMIC ACTIONS

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1 UVOD

Seizmička aktivnost u Bosni i Hercegovini (BiH) uzrokovana je postojanjem dubokih, lateralnih i reverznih rasjeda. Tektonska aktivnost ovog područja povezana je i s činjenicom da drugi po veličini pojas (Alpski pojas) prelazi preko Bosne i Hercegovine, a proteže se od Himalaja, preko Irana Turske i Grčke [1, 2, 3, 4]. Prema evromediteranskoj seizmičkoj mapi rizika, BiH se svrstava u zemlje sa srednjim stepenom seizmičkog rizika, s vršnim ubrzanjem tla (PGA) u rasponu od 0,08 do 0,24 g, dok je jugozapadni dio zemlje okarakterisan visokim rizikom (PGA>0,24 g).

Konstrukcija koja je prikazana u ovom članku (vidjeti sliku 1) karakteristična je za široko područje Zapadnog Balkana, građena pedesetih i šezdesetih godina prošlog stoleća. Više detalja o tipologiji same konstrukcije može se naći na drugim mjestima [1, 2, 3, 4]. Ova vrsta nearmirane (URM) obične zidane konstrukcije izgrađena je od industrijskih opečnih elemenata, ali bez vertikalnih seklača. Osjetljivost ovih konstrukcija povezana je s njihovom velikom visinom, lokacijom nosivih zidova koji se nalaze uglavnom samo u jednom pravcu i činjenicom da nema vertikalnih armiranobetonskih (AB) seklača. Razaranje ove vrste zgrada dobro je dokumentovano nakon zemljotresa u Skoplju 1963. godine (prikazano na slici 2).

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1 INTRODUCTION

Seismic activity in Bosnia and Herzegovina (B&H) is connected to the existence of deep lateral and reverse faults. The fact that the second biggest belt (Alpine Belt), going from the Himalayas over Iran, Turkey and Greece, passes through B&H verifies the tectonic activity of this region [1,2,3, 4]. As per Euro Mediterranean Seismic Hazard Map, B&H falls in the Moderate Seismic Hazard having the peak ground acceleration (PGA) in the range of 0.08 to 0.24g, while a south-west part of the country experiences a High Hazard (PGA>0.24g).

The building presented in this paper (see Figure 1) is characteristic for the wider region of Western Balkans, built in the 1950's and 1960's. More details about the building typology can be found elsewhere [1,2,3, 4]. This type of unreinforced masonry (URM) buildings were constructed with industrial bricks, but without vertical confining elements. The vulnerability of these buildings lies in its high height, structural walls placed mostly in only one direction and the fact that there is no vertical reinforced concrete (RC) confining elements. The devastation of this type of buildings is well documented after the 1963 Skopje earthquake, as illustrated in Figure 2.

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Slika 1. Analizirana zgrada u Sarajevu, Bosna i Hercegovina (fotografisao autor)
Figure 1. Analyzed building in Sarajevo, Bosnia and Herzegovina (authors' photo)

Slika 2. Karakteristična zgrada nakon zemljotresa u Skoplju [14]
Figure 2. Characteristic building damaged after Skopje earthquake [14]

2 DINAMIČKE KARAKTERISTIKE

Dinamičke metode identifikacije često se koriste za određivanje dinamičkih karakteristika (sopstvenih frekvencija i sopstvenih oblika) građevinskih konstrukcija. Ove dinamičke karakteristike važne su za razumijevanje dinamičkog ponašanja konstrukcija izloženih djelovanjem zemljotresa.

Odgovarajući sopstveni oblici i sopstvene frekvencije dobijene pomoću proračuna konačnim elementima (FEM) [1, 2] prikazani su na slikama 3 (a), (b) i (c). Prvi sopstveni oblik jeste preovlađujući način odgovora s najvećim efektivnim doprinosom modalne mase u pravcu x (67,33%). Ovo je „tipično ponašanje” zidne konstrukcije s čvrstim vezama između zidova i ploče (efekat dijafragme).

Vrijednosti frekvencija uspoređene su s podacima datim u [20], pri čemu se ističe to da za više konstrukcije „čak i za konstrukcije koje su na 11 etaža, vrijednosti su blizu 2 Hz iako su konstrukcije napravljene od različitog materijala”. Ovo je veoma važno da bi se model mogao verifikovati s vrijednostima dobijenim na osnovu eksperimenata.

2 DYNAMIC CHARACTERISTICS

Dynamic identification methods are often used for determination of the vibration characteristics (eigen-frequencies and mode shapes) of civil engineering structures. These dynamic characteristics are important for understanding the dynamic behaviour of structures under earthquake actions.

The relevant eigen-modes and eigen-frequencies obtained by the FEM calculations [1, 2] are presented in Figure (a), (b) and (c). The first mode is the prevailing mode of the response with the largest effective modal mass contribution in the x-direction (67.33%). This is a "typical" box behaviour of a masonry structure with stiff connections between the walls and the slab (diaphragm effect).

The value of the frequencies were compared with the data provided by Tomažević [20], indicating that for higher structures, "even up to 11-storeys the values are close to 2Hz even though buildings have been built with different materials". This is very important in order to be able to "verify" the model as there are no experimental data.

Slika 3. Sopstveni oblici i sopstvene frekvencije (a) Prvi sopstveni oblik $f_1=2.19$ Hz ($T_1=0.46$ s); (b) Drugi sopstveni oblik $f_2=3.85$ Hz ($T_2=0.26$ s); Treći sopstveni oblik $f_3=4.00$ Hz ($T_3=0.25$ s)

Figure 3. Eigen modes and eigenfrequencies: (a) First mode $f_1=2.19$ Hz ($T_1=0.46$ s); (b) Second mode $f_2=3.85$ Hz ($T_2=0.26$ s); Third mode $f_3=4.00$ Hz ($T_3=0.25$ s)

3 HIPOTEZA DIJAFRAGME

Kada su izloženi djelovanju zemljotresa, oblici loma uzrokovani smicanjem (poprečnom silom) dokazani su tipom objekta koji je analiziran, te se naglašava kritičan aspekt degradacije krutosti, kao indikatora oštećenja, zidanih zidova. Za zidane konstrukcije sa armiranim betonskim međuspratnim tavanicama, horizontalna stropna opterećenja raspoređena su u skladu s krutošću vertikalnih otpornih elemenata. U primjeru postojeće konstrukcije [1, 2] koja je istražena i modelirana, dokazano je to da se međuspratne konstrukcije od polumontažnih „Herbst“ betonskih šupljih elementa (slika 4) ponašaju kao krute tavanice (dijafragme).

Kako bi se verificovala i potvrdila hipoteza o krutom ponašanju tavanica, konstrukcija je izložena djelovanjem horizontalnog ubrzanja u „ $\pm Y$ “ pravcu (vidjeti sliku 5) primjenom *pushover* analize. Odabran je kontrolni čvor u liniji simetrije na nivou krova čvor 44014, kao što je prikazano na slici 5, a također su odabrana i dva dodatna čvora u istoj liniji (čvorovi 44035 i 43935). Numerička analiza sprovedena je upotrebom softvera DIANA 9.4 [8]. Radi jednostavnosti, svi detalji o modeliranju i *pushover* analizi također su dati na drugim mjestima [1, 2].

3 DIAPHRAGM HYPOTHESIS

When subjected to earthquakes, the shear failure mode evidenced by the building typology studied highlights the critical aspect of stiffness degradation, as a damage indicator, of the masonry walls. For the masonry structures with reinforced concrete slabs the horizontal floor loads are distributed according to the stiffness of the vertical resisting elements. In the example of the existing structure [1,2] that has been investigated and modelled, it was proven that slabs made of semi-prefabricated elements "Herbst" concrete hollow elements (Figure 4) behave as a rigid floor (diaphragm).

In order to verify and confirm the hypothesis of rigid floor behaviour, the structure was exposed to a horizontal acceleration in the " $\pm Y$ " direction (see also Figure 5), by means of a pushover analysis. A control node was chosen in the line of symmetry at the roof level, node no. 44014, as shown in Figure 5, and two additional nodes in the same line where also selected (nodes 44035 and 43935). The numerical analysis was performed in DIANA 9.4 software [8]. For the sake of simplicity, all the details about modelling and pushover analysis are provided elsewhere [1,2].

Slika 4. Polumontažni betonski element
Figure 4. Semi-prefabricated concrete elements

Slika 5. Pozicija čvorova; oznaka zidova; pravac horizontalne sile
Figure 5. Location of the nodes; wall labelling; and direction of the horizontal force

Razmatrano je pomjeranje čvorova na vrhu konstrukcije u istoj horizontalnoj liniji (43935, 44035 i 44014, vidjeti sliku 6), kako bi se dokazala pretpostavka o krutim međuspratnim tavanicama. Metoda postupnog naguravanja (pushover) izvršena je u smjeru „ $\pm Y$ “, ovdje će se pokazati samo pravac „ $+ Y$ “ za potvrđivanje hipoteze o krutoj međukatnoj tavanici. Pomjeranja ovih čvorova, kao što je prikazano na slici 6, jesu jednaka, što ukazuje na kruto ponašanje ploča. Pored toga, provjereno je i pomjeranje čvorova 38979 i 39057 i utvrđeno je da postoji jako mala rotacija, manja od 5 stepeni, što se u ovom slučaju može zanemariti. Ovo može biti povezano s „ne u potpunosti“ simetričnom konstrukcijom [1].

The movement of the nodes at the top of the structure in the same horizontal line (43935, 44035 and 44014, see Figure 6) were investigated in order to prove the assumption of the rigid floors. A pushover analysis was conducted in the „ $\pm Y$ “ direction, here only „ $+ Y$ “ direction will be shown for the confirmation of the rigid floor hypothesis. The movements of these nodes, as seen in Figure 6, are the same indicating a rigid behaviour of the slabs. Additionally, the displacement of nodes 38979 and 39057 was checked and it has been seen that there is a very small rotation less than 5 degrees, which in this case can be neglected. This may be connected to the "not completely" symmetric structure [1].

Slika 6. (a) Kriva kapaciteta za čvorove 44014, 44055 and 43935 u + Y smjeru,
(b) Kriva kapaciteta za čvorove 44014 u + Y smjeru

Figure 6. (a) Pushover Curves for Nodes 44014, 44055 and 43935 in + Y direction,
(b) Pushover Curve for Nodes 44014 in + Y direction

4 DINAMIČKA NEELASTIČNA ANALIZA

Konstrukcija je izložena djelovanju kratkotrajnog zemljotresa (koji se desio u Crnoj Gori, na lokaciji grada Petrovca, 15. aprila 1979. godine, s maksimalnim ubrzanjem tla u vrijednosti od 0,43 g), s ciljem da se što kvalitetnije analizira dinamički odgovor konstrukcije. Ovo je jedan od akcelelograma koji se koristi veoma često prilikom verifikacija odgovora konstrukcija širom bivše Jugoslavije [19, 20]. Potrebno je spomenuti i to da razlike u geofizičkim karakteristikama tla između Petrovca i Sarajeva nisu uzete u obzir u ovom slučaju. Akcelelogram je skaliran s ciljem da ima maksimalno ubrzanje PGA u vrijednosti od 0,1 g, što odgovara Zoni VII MCS-skale, kojoj pripada sarajevsko područje, te gdje je konstrukcija i locirana. Sprovedeno je filtriranje potresa pomoću softvera Seismosignal [18]. Uzeta vrijednost prigušenja (ξ) iznosila je 5%.

4.1 Karakteristike prigušenja

Klasična hipoteza prigušenja, konkretno Rayleigh-ovo prigušenje, slika 7, odabrana je jer konstrukcija ima isti konstruktivni sistem i isti materijal cijelom visinom objekta. Rayleigh-ovo prigušenje sastoji se od dvije komponente – prve koja je proporcionalna masi i druge koja je proporcionalna krutosti, a definirano je putem slijedeće jednačine [6].

$$c = a_0 m + a_1 k$$

gdje su a_0 i a_1 prvi i drugi Rayleigh-jev koeficijent.

4 TIME HISTORY ANALYSIS

The structure was exposed to the 1979 Petrovac short-period earthquake recorded during the earthquake in Petrovac on 15 April 1979, (Montenegro), with peak acceleration of 0.43g, in order to analyze the dynamic response of the structure. This is one of the accelerograms being frequently used for different verifications throughout former Yugoslavia [19,20]. The differences in the soil conditions between Sarajevo and Petrovac in this case were not taken into account in this case. The accelerogram was scaled in order to have a maximum PGA of 0.1g corresponding to the Zone VII (MCS-Scale) for the Sarajevo region, where the building is located, and filtered using the software Seismosignal [18]. The value for the damping ratio (ξ) was taken as 5%.

4.1 Damping characteristics

The classical damping hypothesis, precisely the Rayleigh damping, shown in Figure 7, has been chosen since the building has a similar structural system and structural material over its height. Rayleigh damping composes of mass-proportional and stiffness proportional damping defined by the following equation [6].

$$c = a_0 m + a_1 k$$

where a_0 and a_1 are the first and second Rayleigh coefficients.

Slika 7. Rayleigh-ovo prigušenje [1]
Figure 7. Rayleigh damping [1]

Pretpostavlja se da sopstveni oblici imaju isti iznos viskoznog prigušenja – u vrijednosti od $\xi=5\%$ [5, 6, 9, 13, 19], što je uobičajeno potvrđeno putem eksperimentalnih ispitivanja. Rayleigh-evi koeficijenti mogu se proračunati iz slijedećih jednačina:

$$a_0 = \xi \frac{2\omega_i \omega_j}{\omega_i + \omega_j}, \quad \text{and} \quad a_1 = \xi \frac{2}{\omega_i + \omega_j}$$

It is assumed that the modes have the same damping ratio of $\xi=5\%$ [5, 6, 9, 13,19], usually supported by experimental data, then the Rayleigh's coefficients are calculated by the following equations:

gdje su ω_i, ω_j : ugaone frekvencije za tonove "i" i "j".

Veoma je značajno to što se prilikom izbora tonova i i j odaberu vrijednosti, kako bi se obezbijedila razumna vrijednost prigušenja u svim tonovima koji daju svoje učešće prilikom odgovora konstrukcije. Frekvencije prvog i trećeg tona uzete su u obzir, iako kumulativna efektivna modalna masa iznosi 73%, jer je doprinos ostalih tonova mali. Na taj način, učešće viših (neželjenih) tonova biće izbjegnuto.

Prilikom proračuna, koristila se Newton-Raphson iteraciona metoda, a za sistem intergacije bilo je neophodno da se koristi Hilbert-Hughes-Taylor (HHT) metoda u softveru DIANA 9.4 [8]. HHT metoda (poznata kao α metoda) predstavlja uopšteni oblik Newmark-ove metode. Koristeći navedenu metodu, moguće je uvesti numeričku disipaciju u slučaju viših tonova, a da pri tome ne dođe do degradacije stepena tačnosti [8].

Razlog za primjenu HHT metode leži u činjenici da zidane konstrukcije imaju izrazito nisku čvrstoću na zatezanje, prema tome postoji veoma brz prelaz s linearno-elastičnog ponašanja na trenutno ispućalo stanje, a da pri tome maltene vrijednost krutosti iznosi nula. Ovo sve uzrokuje problem konvergencije, te imajući u vidu to što se matrica krutosti u svakom slijedećem koraku ažurira, to opterećuje sam proračun. Drugo, ova metoda pokazala se kao jako korisna prilikom simulacije dinamičkih djelovanja s velikim brojem stepena slobode kretanja [1, 2, 11].

Odabran je inkrement od $\Delta t = 0,01$ sec zbog vrijednosti najznačajnijih perioda konstrukcije i potrebe da se dobije tačan odgovor u vremenu.

4.2 Konstitutivni zakoni pri modeliranju

Za definisanje nelinearnog ponašanja zidane konstrukcije korišćen je *Total Strain Fixed Crack Model* u softverskom programu Diana [8]. Kod ovog modula, pukotine su fiksirane u pravcu glavnih vektora relativnih deformacija koje se ne mijenjaju prilikom opterećenja konstrukcije.

Slika 8 pokazuje nelinearno ponašanje zidane konstrukcije, pri čemu je odabrana parabolična veza zakonitosti napon – relativna deformacija za pritisak koji se zasniva na Hill-ovom kriteriju tečenja (*Hill-type yield criterion*), bez lateralnog omeđivanja (*no lateral confinement*) i bez smanjenja pukotina u lateralnom pravcu, s čvrstoćom na pritisak – koja iznosi $f_c = 4,07$ N/mm² i odgovarajućom energijom loma pri pritisku – koja je jednaka vrijednosti od $G_c = 6,51$ N/mm. Ponašanje na smicanje, nakon dostizanja pukotina, definisano je na taj način što se uzeo u obzir faktor zadržavanja (*retention factor*) njegovog linearnog ponašanja, koji smanjuje kapacitet na smicanje prema slijedećoj jednačini:

$$G^{cr} = \beta_r \cdot G \quad (1)$$

gdje je β_r (*retention factor*) koji se kreće u granicama od $0 < \beta_r \leq 1$, a G predstavlja modul smicanja za neispućali materijal.

Zakonitost napon - relativna deformacija za zatezanje određena je prema Rankine-ovom kriteriju tečenja (*Rankine type criterion*), i opisana je putem eksponen-

where ω_i, ω_j are angular frequencies of the i -th and j -th mode.

Correct choice of modes i and j is essential in order to ensure reasonable values for the damping ratios in all the modes contributing to the response. Frequencies of the mode 1 and 3 were taken in the analysis, even though the cumulative effective modal mass was of 73%, as the contribution of the other modes was rather low. In this way the contribution of the undesirable higher modes is avoided.

The Newton-Raphson iteration method was used in this computation, while it was necessary to use an implicit Hilbert-Hughes-Taylor (HHT) method in DIANA 9.4 [8], for time integration. The HHT method (known as α methods) is actually a generalization of the Newmarks method. By this method it is possible to introduce numerical dissipation in higher modes without degrading the order of accuracy [8].

The reasoning behind the adoption of the HHT method lies in the fact that masonry has a very low tensile strength, so there is a rapid transition from the elastic range to the fully cracked stage with development of a large number of distributed cracks, leading to almost no stiffness. This all leads to problems of convergence and, additionally taking into account that the stiffness matrix has to be updated at each step, making the calculation time consuming. Secondly, this method has proven useful in structural dynamic simulations with many degrees of freedom. [1, 2, 11].

The increment value of $\Delta t = 0.01$ sec was selected due to the values of the most important periods of the structure and the need to have an accurate time response.

4.2 Constitutive modelling laws

Physical non-linear behaviour of the masonry walls is defined through the total strain fixed crack model detailed in Diana [8]. In this way the cracks are fixed in the direction of the principal strain vectors being unchanged during the loading of the structure.

For hysteretic behaviour of masonry as indicated in Figure 8 parabolic stress-strain relation for compression, based on Hill-type yield criterion, was chosen with no lateral confinement and no lateral crack reduction, with the compressive strength in the value of, and the corresponding compressive fracture energy amounting to $G_c = 6,51$ N/mm.

The post-cracked shear behaviour was defined by taking into account the retention factor of its linear behaviour, which reduces its shear capacity according to the following equation:

Where β_r is the retention factor $0 < \beta_r \leq 1$, and G is the shear modulus of the uncracked material.

Tension path, based on Rankine-type yield criterion, was described by an exponential tension softening diagram having a tensile strength of $f_t = 0.2$ N/mm², and respectively the tensile fracture energy being equal to

cijalnog dijagrama zatezanje - omekšavanja s čvrstoćom na zatezanje u vrijednosti od $f_t = 0,2 \text{ N/mm}^2$; shodno tome, energija loma pri zatezanju iznosi $G_f = 0,1 \text{ N/mm}$. Retention factor za smicanje, b_r , jednak je 0.01, što je vrijednost koja je zadana u modelu. Ovo znači da će čvrstoća na smicanje biti umanjena za jedan postotak u odnosu na početnu vrijednost čvrstoće na smicanje kada dođe do formiranja pukotina [1, 2].

$G_f = 0,1 \text{ N/mm}$.

The shear retention factor, b_r , was left at the default value of 0.01. This means that the shear strength of the material will be reduced to one percent of the original shear strength when cracks form. [1, 2].

Slika 8. Nelinearne karakteristike materijala [1, 2, 11]
Figure 8. Hysteretic Behaviour of Masonry [1, 2, 11]

4.3 Analiza i diskusija rezultata

Da bi se mogao vidjeti razvoj oštećenja i degradacija krutosti konstrukcije, kao indikatora oštećenja, neophodno je da se analizira razvoj oštećenja (distribucija glavnih zatežućih relativnih deformacija) u različitim koracima. Na osnovu rezultata proračuna, napravljen je dijagram pomjeranja u funkciji vremena za kontrolni čvor 44014, što je prikazano na slici 9. Odabrano je nekoliko tačaka (koraka) tokom djelovanja zemljotresa i izvršena je evaluacija degradacije krutosti. Vremenski koraci koji su odabrani prikazani su na slici 9.

4.3 Analysis and discussion of Results

In order to view the development of the damage and stiffness degradation, as a damage indicator, in the structure it was necessary to investigate the damage pattern (principal tensile strains distribution) in different time steps. The displacement vs. time of the control node 44014 was chosen and plotted as indicated Figure 9. Several time instances during the earthquake action are presented and stiffness degradation evaluated. The time instances that have been selected are indicated in Figure 9.

Slika 9. Vremenski tok pomjeranja kontrolnog čvora 44014
Figure 9. Time history of displacement of the control node 44014

U slijedećim paragrafima, razvoj oštećenja i In the following paragraphs, the damage develop-

degradacija krutosti za zid Z-Y6 (vidjeti također sliku 5) biti će prodiskutirani. Prvi „vrh” je evidentiran u vremenu $t_1 = 3,29$ s, a vid oštećenja prikazan je na slici 10. Formiranje prve naprsline smješteno je na mjestu veze podruma koji je izgrađen od armiranog betona i prizemlja koje je izgrađeno od zidanih elemenata. Ista slika oštećenje može se vidjeti u vremenu $t_2 = 3,41$ s samo na suprotnoj strani zida (slika 10 a) i b)).

Slika 10. Glavne relativne dilatacije zatezanja za nosivi zid Z-Y6 (a) $[(t)]_5 = 23,85$ s i (b) $[(t)]_2 = 3,41$ s
Figure 10. Principal tensile strains in load bearing wall W-Y6 (a) $[(t)]_5 = 23,85$ s i (b) $[(t)]_2 = 3,41$ s

Drugi par „vrhova” jeste u vremenu ($t_3 = 7,65$ s) i ($t_3 = 7,79$ s) u dva suprotna smjera. Razvoj naprslina prikazan je na slici 11 (a) i (b) respektivno. Konačno, slika oštećenja na konstrukciji nakon prestanka djelovanja zemljotresa, dakle, u vrijeme $t_s = 23,87$ s prikazana je na slici 12. Koncentracija oštećenja smještena je na mjestu otvora, kao i na mjestu prizemlja i prvog sprata. Formiranje tipične „X” naprsline – koja je karakteristična za ponašanje zidane konstrukcije na djelovanje poprečne sile – prikazana je na slici 12.

ment and stiffness degradation of the wall W-Y6 (see also Figure 5) will be discussed. The first peak is evident at $t_1 = 3.29$ s and the damage pattern is shown in Figure 10. The formation of the first cracks is located at the contact between the basement, made of reinforced concrete, and ground floor, made of masonry. The same pattern is kept at the time $t_2 = 3.41$ s just in the opposite side of the wall (Figure 10a) and b)).

The second sets of peaks are and ($t_3 = 7.79$ s) in the two opposite direction. The crack development is shown in Figure 11(a) and (b) respectably. Finally, the damage pattern on the structure after the earthquake, meaning at the time $t_s = 23.87$ s is shown in Figure 12. The concentration of damage is located at the openings as well as in the ground level and the first floor. The creation of the typical "X" cracks for shear behaviour is evident in Figure 12.

Figure 11. Glavne relativne dilatacije zatezanja za nosivi zid Z-Y6 (a) $[(t)]_3 = 7,65$ s i (b) $[(t)]_2 = 7,79$ s
Figure 11. Principal tensile strains in load bearing wall W-Y6 (a) $[(t)]_3 = 7,65$ s i (b) $[(t)]_2 = 7,79$ s

Figure 12. Glavne relativne dilatacije zatezanja za nosivi zid - Z-Y6 $[(t)]_5 = 23,85$ s
 Figure 12. Principal tensile strains in load bearing walls W-Y6 $[(t)]_5 = 23.85$ s

5 DEGRADACIJA KRUTOSTI I DISIPACIJA ENERGIJE

Određivanje ili bolje rečeno, procjenjivanje tačne krutosti veoma je važno, na primjer, za projektovanje zidanih konstrukcija na osnovu metode proračuna, koja se zasniva na deformacijama [12, 17]. Histerezna kriva – petlja može se koristiti za procjenu deformacionog kapaciteta zidova. Obično se modul elastičnosti (Young-ov modul elastičnosti) i modul smicanja određuju iz važećih standarda, a ne putem eksperimentalnih ispitivanja. Uobičajeno je da se karakteristična čvrstoća na pritisak – određena putem eksperimenata – koristi kao osnova za izračunavanje drugih mehaničkih karakteristika zidova. Ova vrijednost ne predstavlja stvarno naponsko stanje u zidanim zidovima koji su podvrgnuti cikličnom poprečnom opterećenju [12].

5.1 Degradacija krutosti

Krutost zida na poprečnu silu procenjuje se iz krive histereze prikazane na slici 13. Razmjera (visine naspram dužine) zida iznosi 1,50. Maksimalna otpornost definisana je vrijednostima: H_{max} dobijen iz proračuna iznosi 474,88 kN i $d_{Hmax} = 8,53$ mm. Maksimalno pomeranje pri ultimnoj nosivosti iznosi $d_{max} = 11,38$ mm, a odgovarajuća vrijednost H_u jeste 430,73 kN.

5 STIFFNESS DEGRADATION AND ENERGY DISSIPATION

Determining, or rather estimating the correct stiffness is of the utmost importance, for example for the deformation-based design of masonry structures [12, 17]. It is the hysteresis curve that can be used for the estimation of the deformation capacity of masonry. Usually the Young's and shear modulus is determined from the applicable codes and not through experimental tests. It is commonly the characteristic compressive strength that is determined from experiments and then used as the basis for calculation of other mechanical characteristics of masonry. This value does not represent the actual stress state in the masonry wall subjected to cyclic shear loading [12].

5.1 Stiffness degradation

The shear wall stiffness is assessed from the hysteresis curve shown in Figure 13. The aspect ratio (height over length) of the wall is 1.50. Maximal resistance is defined with values: H_{max} obtained from the calculations is 474.88kN and $d_{Hmax}=8.53$ mm. Maximal displacement at the ultimate resistance amounts to $d_{max} = 11.38$ mm and the corresponding H_u amounts to 430.73kN.

Slika 13. Histerezisna kriva – veza između horizontalnog pomjeranja na krovu (kontrolni čvor) 44014 i seizmičke

Figure 13. Hysteresis curve of the relationship between the horizontal displacement at the top floor (Control Node 44014) and the seismic force in the $\pm Y$ direction

Kao što se može videti, degradacija krutosti kao indikator oštećenja, javlja se prilikom režima smanjivanja smicanja zbog obrnutih cikličnih opterećenja. Horizontalna opterećenja raspoređena su u skladu s krutostima zidova i to se vidi prenosom broja pukotina iz jednog zida na drugi zid. Kako se pukotine pojavljuju na jednom zidu, prenos opterećenja se mijenja jer se dobija drugačija distribucija u odnosu na preostalu krutost zida [1, 20].

Da bi se procenila degradacija zida, potrebno je izračunati sekantnu krutost ($K_{s,i}$) za svaki ciklus. Ova vrijednost izračunata je prema sledećoj formuli:

$$K_{s,i} = \frac{H_{max,i}}{d_{max,i}} \quad (2)$$

gdje je:

$K_{s,i}$ – sekantna krutost za i ciklus

$H_{max,i}$ – horizontalna sila pri maksimalnom pomaku za i ciklus

$d_{max,i}$ – maksimalni pomak za i ciklus

Degradacija krutosti jasno je vidljiva na slici 14.. Kako se pomjeranje povećava, smanjuje se sekantna krutost zida, što dovodi do jasnog zaključka o oštećenju zida i njenom smanjenom kapacitetu. Rezultati [19, 21] ukazuju na to da je oblik degradacije krutosti – kao funkcije bočnog pomicanja u nedimenzionalnom obliku – prilično sličnog oblika, bez obzira na tip zidova (obični, omeđeni ili armirani).

As it can be seen the stiffness degradation as a damage indicator occurs at shear failure modes due to reverse cyclic loads. The horizontal loads are distributed according to the stiffness of the walls and this is seen by the transmission of the number of cracks from one wall to the other. As the cracks occur on one wall, the transfer of the load changes as different distribution is obtained in respect to the residual wall stiffness [1,20].

In order to evaluate the degradation of the wall it was necessary to calculate the secant stiffness ($K_{s,i}$) for each cycle. This value was calculated according to the formula:

where:

$K_{s,i}$ – secant stiffness as i cycle

$H_{max,i}$ – horizontal load at maximum displacement at i cycle

$d_{max,i}$ – maximum displacement at i cycle

Degradation of the stiffness is clearly seen in Figure 14. As the displacement increases the secant stiffness of the wall decreases leading to the clear conclusion regarding the damage of the wall and its reduced capacity. The results [19, 21] indicated that the shape of the stiffness degradation as a function of lateral displacement in the non-dimensional form is of a rather similar form regardless of the masonry type (plain, confined or reinforced masonry).

Slika 14. Degradacija krutosti
Figure 14. Stiffness degradation

Tomažević i Klemenc [19, 21] predložili su stepenu funkciju koja se može koristiti za degradaciju krutosti zidanih zidova podvrgnutih cikličnim opterećenjima u ravni. Ova funkcija glasi:

$$\frac{K_{s,i}}{K_s} = \alpha_1 \left(\frac{d_{max,i}}{d_{Hmax}} \right)^\beta \quad (3)$$

gdje je :

K_s – sekantna krutost pri elastičnom stanju

d_{Hmax} – pomak pri maksimalnoj horizontalnoj sili

α_1, β – parametri degradacije krutosti

U odsustvu eksperimentalnih podataka, predloženo je da se koristi vrijednost $\alpha_1 = 0,3$, i $\beta = -0,85$ za slučaj normalnih napona pritiska, koji ne prelaze 20% čvrstoće na pritisak zidanog zida i cikličnih bočnih opterećenja [6, 22]. U ovom slučaju, parametri α_1, β dobijeni su regresionom analizom izračunate krive i dobijene vrijednosti jesu $\alpha_1 = 0,348$, i $\beta = -0,869$. Dobijeni koeficijent određivanja ili koeficijent višestruke determinacije za višestruku regresiju jeste jednak – $R^2=0,91$. Predložena stepena funkcija na dobar način prati trend degradacije krutosti.

where:

K_s – secant stiffness at elastic limit

d_{Hmax} – displacement at maximum horizontal load

α_1, β – stiffness degradation parameters

In the absence of experimental data it was suggested to use the value of $\alpha_1 = 0,3$, and $\beta = -0,85$ for the case of normal compression stresses, not exceeding 20% of masonry's compressive strength, and cyclic lateral loads [6, 22]. In this case the parameters α_1, β were obtained by the regression analysis of the calculated curve resulting in $\alpha_1 = 0,348$, and $\beta = -0,869$. The obtained coefficient of determination, or the coefficient of multiple determination for multiple regression is equal to $R^2=0,91$ and the proposed power function follows the trend of stiffness degradation in a good manner.

5.2 Disipacija energije

Drugi, prilično važan parametar u analizi cikličnih odgovora konstrukcija jeste koeficijent disipacije energije zidova. Ovakav kapacitet procjenjuje se putem koeficijenta ekvivalentnog viskoznog prigušenja (ζ_{eq}),

5.2 Energy dissipation

Another rather important parameter in the cyclic response analysis of structures is the energy dissipation coefficient of the walls. This capacity is evaluated through the coefficient of equivalent viscous damping

koji se dobija kada se disipirana energija u vibracionom ciklusu stvarne konstrukcije odredi da bude jednaka ekvivalentnom viskoznom sistemu [6]. Indikatori oštećenja koji uzimaju u obzir disipaciju energije koji se odražavaju u čvrstoći i krutosti mogu se uspješno koristiti za kontrolu parametara prilikom proračuna [16].

Da bi se približno odredilo ekvivalentno viskozno prigušenje povezano s histerezičkim ponašanjem, može se koristiti koncept disipirane (E_D) i uskladištene (E_{so}) energije. Disipirana energija u zidu data je površinom E_D zatvorene petlje histereze u jednom ciklusu opterećenja.

Energija disipirana viskoznim prigušenjem u jednom ciklusu harmonijskih vibracija, kako je definisano u [9], data je jednačinom (4)

$$E_D = 2\pi\zeta \frac{\omega}{\omega_n} k u_o^2 \quad (4)$$

gdje je:

ω – frekvencija pobude

ω_n – sopstvena frekvencija

ζ – koeficijent prigušenja

k – krutost

u_o – amplituda pomjeranja u *steady state*

Potencijalna energija se definiše jednačinom (5)

$$E_{so} = \frac{k u_o^2}{2} \quad (5)$$

Izjednačavanje energije disipirane viskoznim prigušenjem u jednom ciklusu harmoničnih vibracija definisanih u (4), sa energijom koja se disipira u stvarnoj konstrukciji, dovodi do:

$$E_D = 4\pi\zeta_{eq} \frac{\omega}{\omega_n} E_{so} \quad \text{ili / or} \quad \zeta_{eq} = \frac{1}{4\pi} \frac{\omega}{\omega_n} \frac{E_D}{E_{so}} \quad (6)$$

Određivanje E_D treba izvršiti kada je $\omega = \omega_n$, pošto je odgovor sistema najosjetljiviji na prigušenje. Ovo, konačno, vodi do sledećeg:

$$\zeta_{eq} = \frac{1}{4\pi} \frac{E_D}{E_{so}} \quad (7)$$

Grafički prikaz odgovarajućih energija dat je na slici 15.

Izračunate su vrijednosti pohranjene potencijalne energije i disipirane energije, kao što je objašnjeno u prethodnom tekstu za ovaj konkretan slučaj (prikazano na slici 16). Sličan trend disipacije energije dobijen je u [7], što je uporedivo s obzirom na to što su karakteristike ulaznih podataka – u vezi s karakteristikama materijala zidane konstrukcije – prilično slične [1, 2, 7]. Odnos između disipirane i ulazne energije kreće se u opsegu od 14% do 49%.

(ζ_{eq}), which is obtained when the energy dissipated in a vibration cycle of the actual structure is set to be equal to the equivalent viscous system [6]. Damage indicators that consider the energy dissipation reflected in the strength and stiffness deterioration can be used successfully to control the design parameters [16].

In order to make an approximation of the equivalent viscous damping connected to the hysteretic behaviour, the concept of dissipated (E_D) and stored (E_{so}) energy can be utilized. The dissipated energy in the wall is given by the area E_D enclosed by the hysteresis loop in one cycle loading.

The energy dissipated by viscous damping in one cycle of harmonic vibrations, as defined in [9], is given by equation (4)

where:

ω - exciting frequency

ω_n - natural frequency

ζ - damping ratio

k - stiffness

u_o - steady state displacement amplitude

The stored/strain energy is defined by equation (5)

Equating energy dissipated by viscous damping in one cycle of harmonic vibrations defined in (4) with the energy dissipated in the actual structure leads to:

Determination of the E_D should be done when $\omega = \omega_n$ as the response of the system is most sensitive to damping. This finally leads to

Graphical representation of the respective energies is illustrated in Figure 15.

The calculated values of the strain/stored potential energy and dissipated energy, as explained above for this particular case, is presented in the Figure 16. A similar trend of the energy dissipation was obtained in [7], which is comparable as the input material characteristics of the masonry structure are quite similar [1, 2, 7]. The ratio between dissipated and input energy was in the range from 14 to 49%.

Slika 15. Proračun disipirane i potencijalne energije
Figure 15. Calculation of dissipated and stored energy

Slika 16. Disipirana i pohranjena potencijalna energija
Figure 16. Dissipated and stored potential energy

Na osnovu dobijenih vrijednosti disipirane i pohranjene potencijalne energije i korišćenjem jednačine (7), prosječna vrijednost koeficijenta ekvivalentnog viskoznog prigušenja (ζ_{eq}), s obzirom na to što je početno viskozno prigušenje (ζ_0) jednako nuli, što ukazuje na to da zavisi samo od histeričnog ponašanja, iznosi 11,48%.

Kada se govori o URM, ove veličine u funkciji su oblika loma, iz eksperimentalne i analitičke studije [10]. U istraživanju, Magenes i Calvi predložili su ultimni međuspratni pomak od 0,5% i 1,0%, a ekvivalentno prigušenje jednako je 10% i 15%, usljed otkazivanja na smicanje i savijanje respektivno. Priestley i ostali [15], u istraživanju koje je sprovedeno nekih deset godina nakon [10], smanjuje ove granice međuspratnog pomjeranja za 20%, tj. 0,4% za smicanje i 0,8% za lom

On the basis of obtained values of dissipated and stored potential energy and utilizing the equation (7), the average value of the coefficient of equivalent viscous damping (ζ_{eq}), taking into account that the initial viscous damping (ζ_0), is equal to zero, indicating that it is dependent only on the hysteretic behaviour, amounts to 11.48%.

Regarding URM, these quantities are indicated in function of the failure mode, from an experimental and analytical study [10]. In their research, Magenes and Calvi suggested an ultimate drift of 0.5% and 1.0%, and an equivalent damping equal to 10% and 15%, for shear and flexural failure respectively. Priestley et al. [15], in a research carried out some 10 years later than [10], reduced these drift limits by 20%, i.e. 0.4% for shear and 0.8% for flexural failure. These are also the limits

usljed savijanja. Ovo predstavljaju i granice koje se nalaze u italijanskim tehničkim propisima. Dakle, sada se pretpostavlja da ekvivalentno viskozno prigušenje iznosi 15% za ponašanje na smicanje i 10% za ponašanje na savijanje. Dobra usaglašenost dobijena je vrijednostima koje su predložene u [10, 15].

6 ZAKLJUČAK

U predstavljenom radu, prikazano je seizmičko ponašanje tipične zidane konstrukcije na području Zapadnog Balkana, pod djelovanjem cikličnog opterećenja. Čini se da je njeno seizmičko ponašanje kontrolisano osnovnim oblicima ponašanja.

S povećavanjem pomaka, koncentracija oštećenja smještena je kod otvora, kao i na mjestu prizemlja i prvog sprata. Zidovi koji su paralelni s djelovanjem opterećenja prikazuju dijagonalne naprsline uzrokovane djelovanjem horizontalnih sila, kao i dijagonalni vid pukotina u obliku slova „X”, usljed djelovanja cikličnog opterećenja, što vodi smanjenju kapaciteta.

Na osnovu proračuna, potvrđeno je to da i za ovaj objekat oblik degradacije krutosti prati trend stepene funkcije. Dobijena vrijednost koeficijenta ekvivalentnog viskoznog dampinga ζ_{eq} jednaka je 11,5%, što je u skladu s prijedlozima drugih istraživača. Istraživanja potvrđuju to da uticaji čvrstoće pri cikličnom opterećenju u ravni i degradacija krutosti jesu krucijalne prilikom određivanja moguće lateralne dinamičke nestabilnosti. Ovdje pokazatelji oštećenosti uzimaju u obzir disipaciju energije što se ogleda u pogoršanju čvrstoće i krutosti te predstavlja jasan pokazatelj ponašanja obične (URM) zidane konstrukcije.

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imposed by Italian Technical Code. So, now the equivalent viscous damping is assumed to be 15% for shear and 10% for flexural failure behaviour. A good consistency is obtained with the values proposed in [10, 15].

6 CONCLUSION

In the presented paper, the seismic behaviour of a masonry building typical from Western Balkans under cyclic loading is presented and exploited. Its seismic behaviour seems to be controlled by the fundamental global modes.

With the increase of displacement, the concentration of damage is located at the openings as well as in the ground level and the first floor. The walls parallel to the load direction exhibit diagonal cracking caused by horizontal forces, as well as the diagonal "X" type cracking due to cyclic loading, leading to its reduced capacity.

From the calculations, it was confirmed that also for this building the stiffness degradation shape follows a power function trend. The obtained value of the coefficient of equivalent viscous damping ζ_{eq} amounts to 11.5%, which agrees with the proposals given by from other researchers. Investigations confirm that the effects of in-plane cyclic strength and stiffness degradation are crucial in determining the possibility of lateral dynamic instability. Here damage indicators take into account energy dissipation which is reflected in the strength and stiffness deterioration giving a clear indication of the behavior URM building.

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REZIME

INDIKATORI OŠTEĆENJA OBIČNIH NEARMIRANIH ZIDANIH ZIDOVA IZLOŽENI SEIZMIČKIM DJELOVANJIMA

Naida ADEMOVIĆ
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Disipacija energije i degradacija krutosti su važni seizmički indikatori i degradacije ponašanja unutar numeričkog modeliranja konstrukcija, a u ovom slučaju zidane konstrukcije. Oni su veoma značajni za procjenu seizmičkog učinka zidanih zidova. U ovom članku analiziran je primjer postojeće zidane stambene zgrade koja se nalazi u Sarajevu, u Bosni i Hercegovini. Uočena je očigledna degradacija krutosti zidanih zidova prilikom odgovora zgrade na horizontalna djelovanja.

Ključne riječi: zidane konstrukcije, degradacija krutosti, nelinearno modeliranje, raspodjela oštećenja, disipacija energije, indikatori oštećenja, seizmička djelovanja

SUMMARY

DAMAGE INDICATORS FOR UNREINFORCED MASONRY BUILDING WALLS SUBJECTED TO SEISMIC ACTIONS

Naida ADEMOVIC
Daniel V. OLIVEIRA

Energy dissipation and stiffness degradation are important seismic performance indicators within the numerical modelling of structures, and in this case particularly of masonry. They are of great importance for evaluation of the seismic performance of masonry walls. An example of an existing masonry residential building located in Sarajevo, Bosnia and Herzegovina was investigated in this paper. It was observed that during the response of the building to horizontal actions the stiffness degradation of masonry walls was evident.

Key words: masonry structures, stiffness degradation, nonlinear modelling, damage pattern, energy dissipation, damage indicators, seismic actions