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PREDGOVOR

Usled ograničenih mogućnosti nametnutih pandemijskom situacijom, Društvo za ispitivanje i istraživanje materijala i konstrukcija Srbije i Udruženje savremene industrije glinenih proizvoda Srbije su oktobra 2022. godine na Divčibarama istovremeno održala svoje Kongrese. Pozitivna iskustva iskazana u vidu velikog interesovanja stručne i naučne javnosti, prispeća značajnog broja radova i velike posećenosti, doprinela su da se donese odluka o ponovnom istovremenom održavanju dva Kongresa.

Kongres DIMK sa Međunarodnim simpozijumom predstavlja odličnu priliku da se inženjeri i istraživači u oblasti građevinskih materijala i konstrukcija iz naše zemlje, ali takođe i iz šireg okruženja, okupe i prikažu rezultate svojih istraživanja u proteklom periodu, kao i da razmene stečena iskustva. Osnovni cilj ovog skupa je da se stručnoj javnosti, kroz prijavljene referate, saopšte rezultati aktuelnih istraživanja u Srbiji i zemljama šireg regiona, kao i da se proceni mogućnost praktične primene novih naučnih dostignuća u oblasti građevinskih materijala i konstrukcija.

S druge strane, izražena je i potreba proizvođača glinenih proizvoda da se na Kongresu okupe, razmene iskustva u pogledu savremene proizvodnje i novih tehnoloških procesa, mogućnosti šire primene proizvoda od gline kao i zaštite životne sredine i energetske efikasnosti. Sve ovo nalaže Udruženju obavezu kontinuiteta održavanja jednog ovako značajnog skupa. Kongresom će biti obuhvaćene teme vezane za geološka istraživanja sirovinske baze, eksploataciju, preradu sirovine, tehnološke procese oblikovanja, sušenja i pečenja i automatizaciju procesa. Posebna oblast kojoj će Kongres posvetiti pažnju je primena građevinskih materijala na bazi gline.

Oba Kongresa i Međunarodni simpozijum su idealna prilika za razmenu iskustava, kao i za analizu ostvarenih rezultata u prethodnom periodu i definisanje budućih pravaca razvoja - u cilju unapređenja našeg građevinarstva na području materijala i konstrukcija. Simpozijum treba da ukaže na orijentaciju glavnih pravaca razvoja u predmetnim oblastima, ali je ovo i prilika da se razmotri stanje naše regulative u ovim oblastima i potreba njenog daljeg usaglašavanja sa evropskim normama. Ciljevi Simpozijuma su da građevinski inženjeri, tehnolozi i stručnjaci drugih struka koji se bave istraživanjima na polju materijala i konstrukcija, kao i proizvodnjom građevinskih materijala i njihovom primenom, kroz izlaganje radova i diskusiju rezimiraju rezultate istraživanja prakse u ovim oblastima. Kao i ranijih godina, Simpozijuma će svakako doprineti napretku struke i nauke, kao i usvajanju kriterijuma i metodologije rada u predmetnim oblastima građevinarstva.

Kongresi i Međunarodni simpozijum obuhvataju veoma širok raspon osnovnih tema, a to su:

- *Istraživanje savremenih građevinskih materijala i njihova primena*
- *Teorijska i eksperimentalna analiza konstrukcija*
- *Projektovanje i građenje objekata sa aspekta održivog razvoja*
- *Aseizmičko projektovanje i građenje*
- *Upravljanje projektima u građevinarstvu*
- *Procena stanja, održavanje i sanacija građevinskih objekata*
- *Standardizacija – tehnička regulativa i sistem kvaliteta*
- *Zaštita životne sredine i energetska efikasnost u savremenom graditeljstvu*
- *Sirovinska baza i novi tehnološki procesi*
- *Aspekti primene proizvoda od gline*

Prateći Zbornik radova obuhvata 57 radova i obima je 525 strana. Publikovani su radovi autora iz Srbije, Hrvatske, Makedonije, Bugarske, Crne Gore, Nemačke, SAD-a. Ovom prilikom upućujem veliku zahvalnost svim autorima na uloženom trudu i posvećenosti razvoju građevinske nauke i struke.

Izražavamo nadu da će predstojeći Kongres DIMK-a, Kongres SIGP-a, kao i međunarodni simpozijum sa prezentovanim radovima, predstavljati značajnu osnovu za stručni rad i konstruktivnu razmenu mišljenja inženjerske zajednice o savremenim izazovima u oblasti graditeljstva.

Realizacija oba Kongresa ne bi bila moguća bez nesebične podrške i angažovanja naših sponzora, koji su svojim učešćem dali nemerljiv doprinos uspešnoj organizaciji i sprovođenju svih aktivnosti. Ovom prilikom izražavamo našu duboku i iskrenu zahvalnost na ukazanom poverenju, saradnji i podršci koju su nam pružili.

Kontinuitet u organizovanju kongresa i simpozijuma je od izuzetne važnosti za stručnjake iz oblasti materijala i konstrukcija, uključujući i domen glinenih proizvoda. Ovakvi naučno-stručni skupovi ne samo da doprinose razmeni znanja i iskustava, već i značajno podstiču unapređenje međuinstitucionalne i regionalne saradnje između Republike Srbije i zemalja okruženja.

Na kraju, ovaj skup bih posvetio nedavno preminulom profesoru dr Dimitriju Zakiću, predsedniku Društva za ispitivanje i istraživanje materijala i konstrukcija Srbije. Njegov izuzetan doprinos razvoju naučne i stručne misli u oblasti građevinskih materijala ostavio je dubok trag u našoj zajednici. Kao istaknuti član DIMK-a, svojim znanjem, autoritetom i neumornim radom, profesor Zakić je značajno doprineo tome da Društvo postane ugledna i priznata institucija, ne samo u Srbiji, već i u međunarodnim okvirima.

Niš, maj 2025.

Zamenik predsednika DIMK Srbije

prof. dr Nenad Ristić, dipl.inž.građ.

UVODNA IZLAGANJA

SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
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Predrag L. Popović[1]

Stručni rad

AVOIDING STRUCTURAL FAILURES DUE TO CREEP IN CONCRETE AND FATIGUE IN STEEL

Summary: *Creep in concrete and fatigue in steel are material properties that are not always fully recoginized by design engineers. Lack of understanding their effects can result in damage and sometimes in collapse of structures. This paper will, through case histories, present instances and projects where creep in concrete caused damage which could have been prevented.*
Fatigue in steel bridges is usually caused by improper details and had resulted in bridge collpases. Crack growth in steel bridges is influenced primarily by detail type, live load stress range, the number of truck passages, and quality of bridge fabrication. Proper inspection and evaluation techniques are critical to successfully identifying fatigue problems and retrofitting them before failure occurs.
Keywords: *creep in concrete, fatigue in steel, failures, damages, collapses.*

SPREČAVANJE OŠTEĆENJA KONSTRUKCIJA ZBOG TEČENJA BETONA I ZAMORA ČELIKA

Rezime: *Tečenje betona i zamor čelika su osobine građevinskih materijala koje nisu uvek dovoljno uzete u obzir tokom projektovanja konstrukcija. Nedovojno razumevanje njihovih efekata može dovesti do oštećenja i čak kolapsa konstrukcija. Ovaj rad, kroz izvedene projekte, prikazuje primere gde je tečenje betona dovelo do oštećenja objekata koja su mogla biti izbegnuta.*
Zamor čelika, zbog pogrešnih detalja, može da dovede do oštećenja i kolapsa mostova. Nastanak prslina u čeliku zavisi od detalja, broja ciklusa opterećenja i kvaliteta obrade čelika. Pažljiva inspekcija i ispitivanje su kritični da se problemi zamora čelika identifikuju i saniraju pre nego što dođe do havarije.
Ključne reči: *tečenje betona, zamor čelika, oštećenja, kolaps, konstruktivni detalji*

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1. INTRODUCTION

Design engineers use allowable and ultimate stresses in concrete and steel to dessign concrete and steel structures. Design codes define these material properties and present design equations to use in design practice. The design methodes have been developed and confirmed over many years in design and construction of structures.

Some other material properties like creep in concrete and fatigue in steel are not always fully recognized by design engineers in their designs and details. Neglecting their influence can result in damage and sometimes in collapse of structures. This paper presents through case histories instances where the damage occurred and could have been prevented.

1.1. Creep in Concrete

It is well known that concrete as a structural material changes its volume over time due to shrinkage, creep and temperature. Shrinkage generally happens when concrete looses moisture during its curing period over time. The volume, especially the length, of concrete varies in proportion of changing its temperature and ambient temperature. The creep is concrete property which is the continuing shortening of the concrete element under constant compressive load. If a compressive load is applied on a concrete element, it will reduce its length in proportion to the applied load. However, if the compressive load stays for a long time, there will be continuous shortening of the concrete element beyond the initial elastic shortening due to the applied compressive load. This shortening will continue over the years with most of it being completed over the first six years.

Typical structures where this occurs are concrete columns in high-rise buildings and in long span post-tensioned concrete slabs and girders. In some high-rise buildings, the length of concrete columns was measured to reduce by over 10 cm over a few years. Similar reduction in lengths were regularly observed in long span post-tensioned concrete structures. If the design and construction details do not take in account this additional shortening, damage or collapse of the structure can occur.

1.2. Fatigue

Fatigue is a fiailure of a material (steel) by cracking resulting from repeated or cyclic stress. Material can fail in fatigue at much lower stress levels than allowable stress we used in design.

Many of today's bridges have exceeded their design lives, are carrying heavier vehicles than for which they were originally designed, and are experiencing traffic counts well in excess of original projections. In addition, at the time many were built, it was not widely known that certain fabrication details and geometry conditions contribute to an increased susceptibility to fatigue damage. These details and conditions have resulted in fatigue cracking in steel bridges including complete fractures of the main girders in some steel bridges.

2. CONCRETE CREEP

Examples of projects where the concrete creep was not properly accounted for in the design are presented here.

A long span post-tensioned parking garage is supported along one side by variable height columns. One of the columns was installed on the top of a wall of the mechanical ventilation shaft and was only about 30 cm high (Figure 1). The long-term movement due to concrete creep could not be accommodated by a stiff short column. The result was structural damage to the column (Figure 2).



Figure 1. Columns supporting parking garage

A similar condition was observed at another parking structure where column height varied due to a sloping ramp. A very short column could not accommodate movements due to concrete creep and it was crashed (Figure 3). In the same structure perimeter columns were built along a ground level wall where the columns were placed in a notch in the wall. As the deck was shrinking due to concrete creep, the column could not move but was wedged in the notch causing severe cracking of the column (Figure 4).



Figure 2. Short column distress





Figure 3. Sloping ramp column



Figure 4. Column wedged in the perimeter wall

In a 20-story apartment building, the lowest floor level was built about 6 m above the ground. A developer decided to use that space to install a singe-story new post-tensioned parking deck in that space. The new deck was built around the existing building columns and was supported on new columns next to the old ones. After several years, the new deck sharnk toward its center with the movements being the largest along the perimeter (Figure 5). This concrete deck movement rresulted in very large lateral forces applied to a mid-height of 6 m span of columns which carried 20-story building. During an inspection of the underside of the lowest floor of the building, the crashing of the new deck against the columns and cracks in the columns were observed (Figure 6). These observations and the follow-up remedial measures prevented the collapse of this 20-story building.

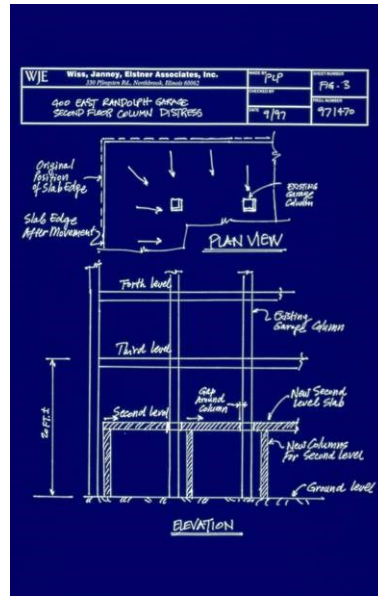


Figure 5. Movement of new deck



Figure 6. Crashing of the deck and cracks in column

3. FATIGUE CRACKING

The causes of fatigue cracking in steel bridges has been widely investigated and documented. Fatigue cracking can be characterized by one of four mechanisms: 1) out-of-plane distortion 2) large initial defects 3) connection restraint and 4) low fatigue

resistance [4]. Several common examples of each class of fatigue cracking are discussed below.

3.1. Out-of-Plane Distortion

Distortion induced cracking is the most prevalent form of fatigue cracking observed. The cracking is the result of out-of-plane distortion of a small gap. The gap is subjected to very high cyclic stresses resulting in cracking at relatively low cycles. Often times this leads to simultaneous cracking at similar details in a bridge.

In plate girder bridges, cracks usually occur in the unstiffened portion of the web between the clipped corner of the vertical connection plate and the toe of the web-to-flange fillet weld. At negative moment regions of continuous girders, the girder top flange plate is laterally constrained by the bridge deck. The floor beam or cross frame is only connected to the girder through a vertical stiffener or connection plate welded to the girder web plate and the girder compression (bottom) flange. Concerns about transverse welding discouraged connecting the stiffener or connection plate to the tension (top) flange. Live loading on the bridge causes end rotation in the floor beam, which in turn induces distortion in the web gap, causing high local bending stresses that result in cracking. The cracking usually initiates at the ends of the vertical connection plate welds, propagating along the weld toe until it turns into the girder web. At the same time, cracking occurs along the horizontal web-to-flange weld on the opposite side of the connection plate. Figure 7 shows a crack extending from a floor beam web gap.



Figure 7. Web gap cracking at top floor beam-to-girder web connection

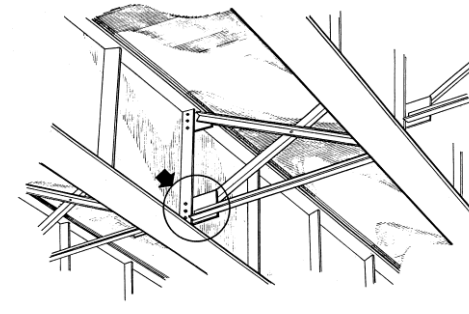


Figure 8. Typical cross frame diaphragm

Out-of-plane distortion can also occur as a result of location of offset cross frames and differential displacements of girders. All types of steel bridges can develop displacement induced fatigue cracking. Figure 8 shows a typical cross frame diaphragm.

The cross frame members are stressed as they try to distribute wheel loads to adjacent girders. The force in the cross frame members is in turn resisted by the welds near the junction of the stiffener and top flange. A cycle of stress occurs with each wheel load, which leads to crack growth. The greatest cross frame force would be expected in the diagonal member connected to the top of the fascia girder, which is consistent with the observed patterns of cracking.

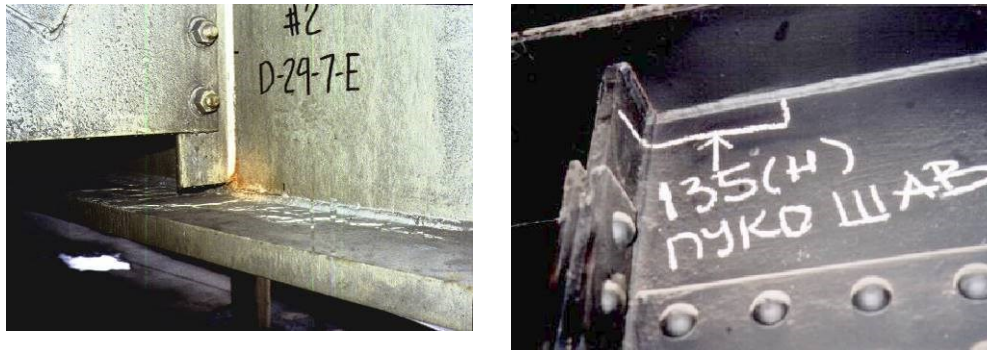


Figure 9. Crack at web-flange junction

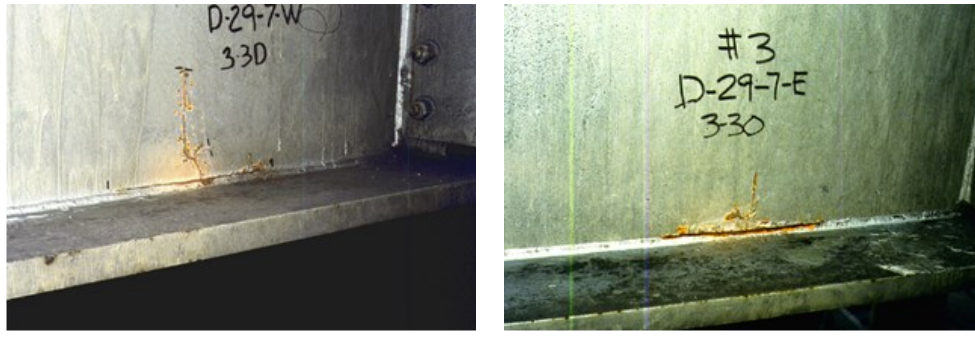


Figure 10. Cracks at offset diaphragm locations

Cracking occurs in the girder webs at the base termination of the vertical weld between a transverse web stiffener plate and the web of a built-up plate girder. It occurs only where an interior diaphragm cross frame is attached to such a stiffener (Figure 9). It most frequently occurs at locations where diaphragms are offset on either side of the web of a beam (Figure 10). In most cases these cracks exist in both the vertical weld and the web of the beam.

Cracks are also caused by differential deflection in adjacent girders. However, in this case, there is no direct connection between the stiffener and the flange. Therefore, a relative deflection of adjacent girders induces an out-of-plane deformation of the girder web at the base of the stiffener. Because the gap between the end of the stiffener and bottom flange is often less than 25 mm., the out-of-plane bending stresses are very large. These large cyclic stresses caused the cracks near the bottom of the stiffener.

3.2. Large Initial Defects

Fatigue cracks generated from initial defects or cracks are typically found in older structures that were constructed before bridge welding quality procedures and associated nondestructive testing measures were widely employed. In other cases, cracking has developed in secondary members or elements not typically subjected to welding quality procedures.

On many plate girder bridges, longitudinal stiffeners are carried across tension regions of the girders for architectural purposes. These stiffeners were periodically spliced using groove welds; however, because they were considered "decorative", the groove welds were not nondestructively tested for defects. Fatigue cracks have initiated from these defects resulting in failure of the girder. Figure 11 shows a cracked longitudinal stiffener groove weld.



Figure 11. Crack in longitudinal stiffener groove weld in tension region of girder



Figure 12. Floor beam cope crack retrofitted using a 25 mm diameter drilled hole

3.3. Connection Restraint

Floor beam, stringer, and diaphragm connections used in bridge framing commonly utilize coped flanges or welded beams where the flanges stop short of the end connections. The flame-cut copes produce residual stresses in rolled sections that approach the material yield point. The change in cross section at the connection by the removal of a flange or portion of flange results in the reduction of the in-plane bending resistance of the members. Figure 12 shows a crack extending from a floor beam cope.

3.4. Low Fatigue Resistance

Over time several details have been found to exhibit poor fatigue performance. Information on these detail types was not known during their design but has become apparent after several failures.

Welded cover plate terminations on girders are one example. Another example includes welded gusset plate connections to girder web plates. The gusset plate connections, whether welded to the vertical connection plate or not, have produced cracking from the combined effects of in-plane and out-of-plane stresses in the web gap. It was found for gusset plates welded to both the web plate and vertical connection plate that the geometric configuration of the detail produces high levels of constraint and stress in the web gaps. The overlapping weld conditions usually found in these details make detection of cracks difficult. Figure 13 is an example of a gusset plate detail with high levels of constraint.



Figure 13. Overlapping welds at lateral gusset plate connection to girder web and vertical connection plate

3.5. Retrofit Installation

Practical retrofit details have been developed to address cracking at fatigue sensitive details caused by the four fatigue cracking mechanisms. Many of the retrofits can be installed by experienced bridge maintenance crews using standard drilling and grinding equipment.

Several retrofit techniques have been used to address out-of-plane bending concerns. Each technique varies with regards to long term performance, constructability, and cost. Out-of-plane cracking in the web gap at top floor beam connections has been successfully retrofitted by installation of large holes adjacent to the web gap as shown in Figure 14. The large holes increase the flexibility of the web gap and thus reduce the bending stresses while capturing any crack tips that extend out of the small gap region. When used in the correct application and installed properly, the large hole retrofit has successfully limited crack extension due to out-of-plane displacement.

Longitudinal stiffeners in tension regions of girders with poor quality groove welds susceptible to crack extension into the girder webs from an initial defect have been successfully retrofitted using several methods. One method involves coring a hole through the girder web plate at the groove weld location to isolate any crack growth in the web plate. Another is shown in Figure 15 and requires coring a hole through the stiffener at the groove weld to remove the weld and grinding the weld terminations to transition the stress flow.

Drilling small diameter holes to arrest the fatigue crack growth in restraint cracking conditions has proven successful as shown in Figure 12. The hole drilling may also include loosening of the connection to remove some of the restraint causing the cracking. Sometimes, fatigue cracks are treated by bridge maintenance personnel by burning off the paint in the vicinity of cracks and drilling large holes at the apparent termination of each crack (Figure 16).

Lateral gusset plate connections to girders webs with overlapping welds at have been found to have low fatigue resistance. The overlapping weld conditions make detection of cracks difficult and produces a condition of high constraint. A retrofit to open the web gap at these details has been installed on numerous steel bridges. An example is shown in Figure 17.

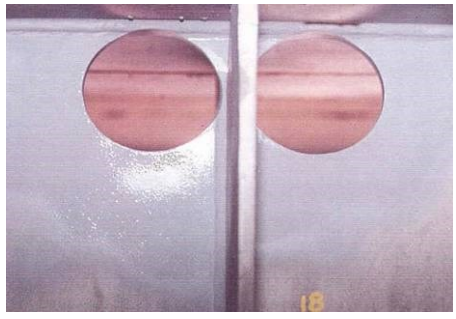


Figure 14. Large hole (100 mm) retrofit installed to address web gap cracking at top floor beam connections in negative moment regions

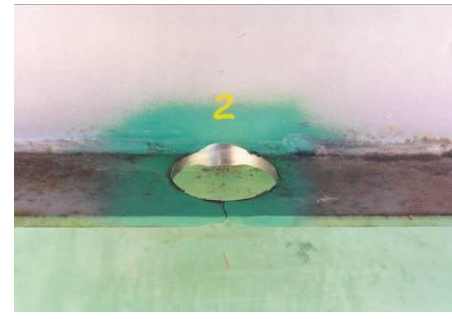


Figure 15. Longitudinal stiffener retrofit to remove defect in groove weld splice



Figure 16. Holes drilled at termination of fatigue cracks to prevent propagation



Figure 17. Lateral gusset plate with overlapping welds removed and gap increased

4. CONCLUSIONS AND ACKNOWLEDGEMENTS

The above-mentioned cases do not provide a comprehensive review of all possible causes of structural failure due to creep in concrete and fatigue in steel. The case histories discussed point out that lack of understanding their effects can result in damage and possible collapse of structure. Potential retrofit methods were also presented.

The case histories of distressed structures described here were einvestigated by engineers from the author's firm and their contribution is gratefully acknowledged.

DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

Milica V. Vasić[1]
Zagorka Radojević[2]

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Originalni naučni rad

ANALIZA KVALITETA SIROVINA ZA PROIZVODNJU KERAMIKE U SRBIJI

Rezime: *Specifična svojstva gline kao sirovine određuju mogućnost primene u različitim vrstama keramičkih proizvoda. U ovom radu dat je prikaz različitih tipova sirovina iz Srbije, na osnovu hemijskog, mineraloškog sastava, kao i ponašanja tokom pečenja. Ispitivane gline se mogu podeliti na one pogodne za proizvodnju opekarskih proizvoda i tzv. fine keramike. Iz svake grupe glina prikazan je po jedan primer i dati su hemijski i mineraloški sastav, kao i ponašanje pri zagrevanju (pečenju). Posebna pažnja posvećena je uticaju nečistoća, kao što su karbonati, oksidi gvožđa i organski materijali, na finalne karakteristike gotovih proizvoda.*
Ključne reči: *Prirodne sirovine, Tradicionalna keramika, Hemijski sastav, Mineraloški sastav.*

ANALYSIS OF THE QUALITY OF RAW MATERIALS FOR THE PRODUCTION OF CERAMICS IN SERBIA

Summary: *The specific properties of clay as a raw material determine the possibility of application in different types of ceramic products. This paper presents a range of raw materials from Serbia, categorized by their chemical and mineralogical composition, as well as their behavior during baking. The examined clays can be divided into those suitable for producing brick products and the so-called fine ceramics. From each group of clays, one example is presented, and the chemical and mineralogical composition, as well as the behavior during heating (baking), are provided. Special attention is paid to the influence of impurities, such as carbonates, iron oxides, and organic materials, on the final characteristics of the finished products.*
Keywords: *Raw clay, Traditional ceramic, Chemical composition, Mineralogical composition.*

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1. UVOD

Kvalitet sirovina predstavlja ključni faktor u proizvodnji keramičkih proizvoda, jer direktno utiče na njihove fizičke, mehaničke i estetske karakteristike. Glina, kao osnovni resurs u tradicionalnoj keramičkoj industriji, pokazuje širok spektar svojstava koja zavise od njenog hemijskog i mineraloškog sastava, kao i prisustva nečistoća. Analiza ovih parametara omogućava definisanje pogodnosti gline za specifične namene, bilo da se radi o proizvodnji opekarskih proizvoda, keramičkih pločica ili sanitarne keramike [1].

Među prirodnim resursima, gline zauzimaju posebno mesto i odavno intrigiraju istraživače i industriju. Njihov heterogeni sastav i tekstura predstavljaju izazove za optimizaciju mogućnosti primene, bilo u poljoprivredi, geotehnici ili industriji. Gline se retko nalaze kao pojedinačni minerali; umesto toga, kaolinit, ilit i montmorilonit često koegzistiraju sa hloritom, mineralima mešovitih slojeva i ne-glinenim mineralima poput peska, feldspata i karbonata. Njihova hemijska i mineraloška svojstva, granulometrija i međusobni odnosi mineralnih faza, definišu njihovu upotrebljivost [2,3].

Posebno su značajne primene gline u proizvodnji keramike, gde se razlike u hemijskom sastavu i ponašanju tokom sinterovanja direktno odražavaju na kvalitet finalnih proizvoda. Proizvodnja tradicionalnih keramičkih materijala, kao što su opeka, keramičke pločice i sanitarna oprema, često zavisi od pažljivo balansiranih sirovinskih smeša. Ključne karakteristike poput plastičnosti, sposobnosti bubrenja i raspodela veliline čestica određuju da li je glina pogodna za određenu namenu [1]. Na primer, visok sadržaj Al_2O_3 i nizak sadržaj Fe_2O_3 često su poželjni za finu keramiku, dok su gline sa većim sadržajem Fe_2O_3 pogodne za grubu keramiku, kao što su crepovi i blokovi od gline [4,5].

Analiza sirovina za keramiku danas zahteva brze i pouzdane metode procene, kao što su rentgenska fluorescencija (XRF) i rentgenska difrakcija (XRD), koje omogućavaju precizno utvrđivanje hemijskog i mineralnog sastava [6]. Brze metode određivanja raspodele veličine čestica, kao i hemijski sastav, predstavljaju prvi korak ka određivanju pogodnosti sirovine za određene namene. Pored toga, korišćenje aditiva i sekundarnih sirovina, kao i razvoj održivih tehnologija proizvodnje ostaju ključni izazovi za ovu granu industrije.

U Srbiji, bogatstvo prirodnih resursa pruža značajan potencijal za razvoj keramičke industrije. Ipak, razlike u sastavu i ponašanju tokom procesa proizvodnje zahtevaju detaljna ispitivanja kako bi se osigurala konzistentnost i kvalitet finalnih proizvoda.

Ovaj rad ima za cilj da prikaže osobine glina iz Srbije i njihovu prikladnost za upotrebu u tradicionalnoj keramici, kao i da predloži metode za održivu i efikasnu eksploataciju. Istraživanje uključuje hemijske i mineraloške analize, kao i ispitivanje ponašanja tokom procesa sinterovanja, kako bi se omogućila pouzdana klasifikacija sirovina na osnovu njihovog potencijala za primenu u različitim keramičkim proizvodima.

2. MATERIJALI I METODE

Uzorci sirovina iz Srbije su označeni prema očekivanoj primeni kao sirovina pogodna za blokove (B), crep (C), keramičke ploćice (P) I vatrostalne elemente (V). Uzorci sirovina su prethodno sušeni na temperaturi od 105°C, a zatim analizirani radi utvrđivanja hemijskog sastava primenom energetsko disperzivne rendgensko

fluorescentne spektroskopske (ED-XRF) analize. Rezultati su upoređivani sa odgovarajućim sertifikovanim referentnim materijalima [6].

Kvalitativna mineraloška analiza obavljena je na praškastim uzorcima u triplikatu korišćenjem difraktometra Philips PW-1050. Ovaj uređaj koristi λCu-Kα zračenje filtrirano pomoću Ni filtera I radi pri brzini skeniranja od 0,05°/s [7].

Instrumentalne metode za termičku analizu uključivale su diferencijalnu termičku I termogravimetrijsku analizu (DTA/TGA) korišćenjem TA Instruments SDT Q600 aparata I dilatometriju (Setaram) u atmosferi kiseonika do temperature od 1000°C. Priraštaj temperature u slučaju DTA/TGA analize je 20°C/min, dok je pri dilatometriji iznosio 10°C/min.

3. REZULTATI I DISKUSIJA

Rezultati koji prikazuju predstavnike određenih glina i njihovog sastava i ponašanja pri zagrevanju (pečenju) dati su u narednim poglavljima.

3.1. Hemijski i mineraloški sastav

Hemijski sastav sirovina (*tabela 1*) Al_2O_3 koji je potreban za proizvodnju pločica i, pogotovo, vatrostalnih materijala [8].

Oznaka uzorka	B	C	P	V
Gubitak žarenjem na 1000°C, %	18,65	6,98	5,95	14,47
SiO_2, %	40,45	60,41	64,08	52,41
Al_2O_3, %	11,40	16,60	22,15	26,54
Fe_2O_3, %	4,41	5,99	1,66	2,62
CaO, %	15,53	2,38	0,24	0,69
MgO, %	6,10	3,15	1,32	0,67
Na_2O, %	0,61	0,81	0,37	0,34
K_2O, %	1,82	2,31	3,37	0,95
SO_3, %	0,08	0,05	0,02	0,03
P_2O_5, %	0,11	0,13	0,05	0,07
MnO, %	0,12	0,10	0,01	0,01
TiO_2, %	0,67	0,73	0,74	0,95
Zbir	99,95	99,64	99,96	99,74
$Al_2O_3 + TiO_2$, %	12,07	17,33	22,89	27,49
$Na_2O + K_2O$, %	2,43	3,12	3,74	1,29

Tabela 1. Hemijski sastav sirovina

Veći udeo Al_2O_3 i TiO_2 je potreban kod vatrostalnih materijala [9], što je evidentno kod ispitivanih uzoraka. Udeo Fe_2O_3 kod opekarskih proizvoda (B i C) je najviši, dok kod vatrostalnih materijala ova vrednost ne sme biti viša od 3,7 % [9]. Prisustvo CaO u nekoj meri može biti i poželjno kod opekarskih proizvoda i keramičkih pločica ako je u, uglavnom u obliku karbonata, on prisutan u fino sprašenom obliku. Kod vatrostalnih materijala udeo CaO ne sme biti viši od 1 % [9]. Ukupan udeo Na_2O i K_2O iznad 3,7 % značajno snižava tačku topljenja materijala i urušava vatrostalnost. Osim ovih oksida, u topitelje spada i SiO_2, ali onaj njegov deo u obliku slobodnog kvarca.

Mineraloškom odredbom ispitivanih sirovina utvrđeno je sledeće: Uzorak B - Najzastupljeniji mineral je kvarc. Detektovan je visok udeo karbonata (kalcita i dolomita). Od slojevitih silikata prisutni su liskun (muskovit), hidroliskun/ilit, kaolinit, hlorit (klinohlorit) i montmorilonit. U manjoj količini su prisutni feldspat (anortit) i hematit. Uzorak C - Najzastupljeniji ilit/muskovit, zatim kvarc, kaolinit, pa feldspat. Od minerala iz grupe slojevitih silikata prisutni su liskuni, kaolinit, ilit, hlorit i manje količine montmorilonita. Identifikovan je i manji sadržaj karbonata (kalcita i dolomita). Uzorak P - Najzastupljeniji mineral je kvarc. Slede slojeviti silikati i to ilit/muskovit i kaolinit. Takođe je utvrđeno i prisustvo feldspata. Uzorak V - Najzastupljeniji mineral je kaolinit, a zatim ilit/muskovit. U značajnoj meri prisutan je i kvarc.

3.2. Ponašanje pri termičkoj obradi

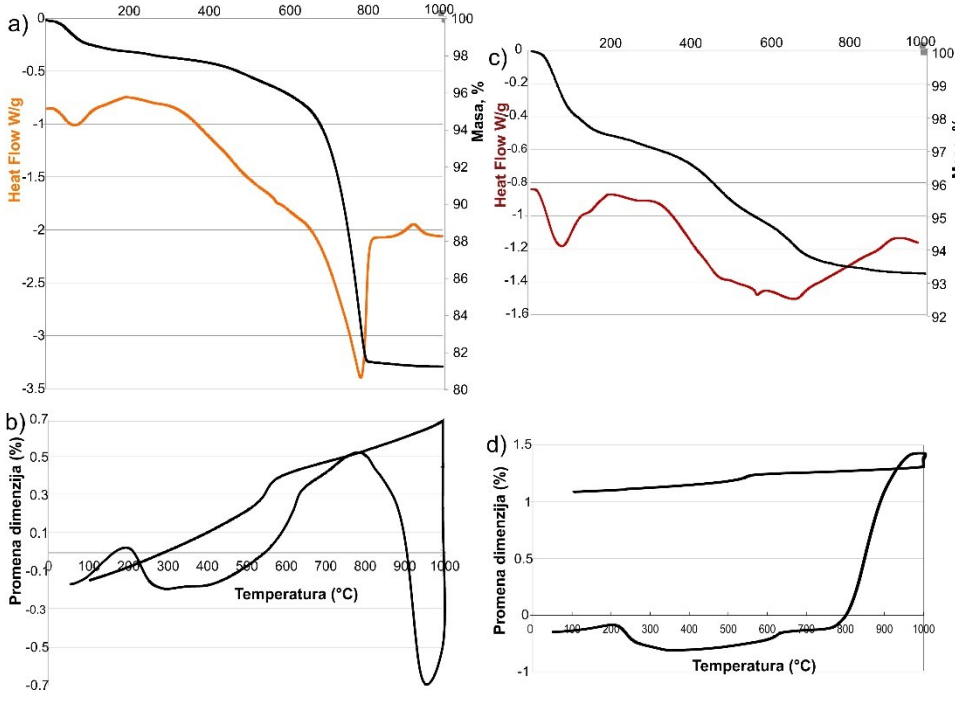
Ponašanje sirovina tokom zagrevanja se razlikuje, što je prikazano na *Dijagramima 1 i 2*. DTA krive uzoraka B i C (*Dijagram 1*) pokazuju najveće u razlike u udelu karbonata, koji se vide u endotermnim pikovima na 783°C i 740°C, respektivno za ove uzorke. Ova razlika se uočava i na TGA krivama u ukupnom gubitku mase (18,67 i 9,57 %, respektivno). U oba uzorka uočeno je prisustvo organske materije, jer se javljaju egzotermni efekti sa pikovima na oko 200°C. Ovaj egzotermni efekat je kod uzorka C prekinut endoternom reakcijom na oko 281°C koja odgovara razgradnji $Fe(OH)_3$ i nastanku Fe_2O_3. Prva dva endotermna pika na obe DTA krive pika prate gubici mase koji odgovaraju odstranjivanju H_2O i OH grupe iz glinenih minerala. Opšte karakteristike pikova ukazuju na prisustvo hidroliskunskih i smektiskih glina, kao i kaolinita. Glavni pik koji ukazuje na razgradnju glinenih minerala se kod uzorka B ne uočava jer je preklopljen endotermnim efektom usled razgradnje karbonata. Kod gline C ovaj pik je slabo uočljiv i razvučen, a javlja se na oko 493°C. Niska temperatura glavnog pika ukazuje na ilit/hidroliskunske gline, hlorite, smektite gde je došlo do izomorfne zamene Al sa Fe, ali i na prisustvo kaolinita neuređene strukture. Mali ali oštar pik koji ukazuje na faznu transformaciju kvarca se uočava na obe krive na oko 578°C. Mali egzotermni pik na 913°C u slučaju gline B vezan je za obrazovanje novih kristalnih faza nastalih razgradnjom glinenih minerala i reakcijom sa CaO iz karbonata (obrazovanje kalcijum silikata). Mali egzotermni pik na 934°C kod gline C može ukazivati na kristalizaciju mulita.

Ponašanje ovih uzoraka tokom zagrevanja pri dilatometrijskom ispitivanju u većem delu (do oko 800°C) se odvija sličnim tokom: prvo se odvija blago širenje do oko 200°C usled gubitka vode, a zatim skupljanje. Intenzivnije skupljanje je uočeno kod gline C, dok je tokom daljeg zagrevanja izraženije širenje uočeno kod uzorka B. Razgradnja getita kod uzorka C u ovom periodu (uobičajeno između 220-290°C) uzrokuje skupljanje. Takođe, intenzitet širenja zavisi od udela kvarca [10]. Dodatno širenje u periodu od 600-

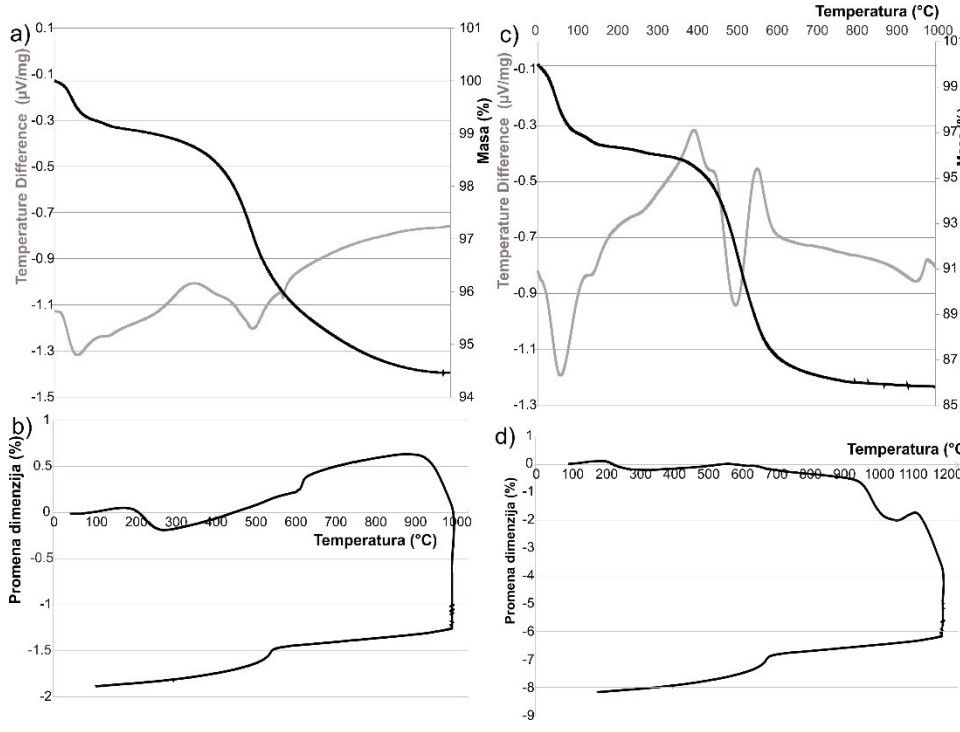
800°C je intenzivnije kod glina koje sadrže karbonate. Početak sinterovanja je registrovan na oko 800°C kod B. Uzorak C se na višim temperaturama intnenzivno širi. Oba uzorka se skupljaju tokom hlađenja uz faznu transformaciju kvarca.

Položaj, oblik i intenzitet pikova (*Dijagram 2*) ukazuju na mešavinu minerala glina i to ilita i kaolinita. Jaki endotermni efekti su detektovani na 61 i 72°C respektivno kod uzoraka P i V. Položaj glavnog pika je kod oba uzorka na oko 500°C, s tim što je on izraćeniji kod vatrostalne gline V. Endotermni pik na DTA krivoj gline V na 441°C ukazuje na prisustvo gvožđe-hidroksida. Ovaj pik je prekloplјen egzotermnom reakcijom koja prati razlaganje organske materije. Slabo izražen endotermni efekat sa pikom na 574°C je uočen kod gline P, dok je ta reakcija kod uzorka V prekloplјena energentski intenzivnijim procesom. Mali egzotermni pik na oko 981°C kod V može ukazivati na kristalizaciju mulita.

Ponašanje ovih uzoraka tokom zagrevanja u većem delu ide sličnim tokom: prvo se odvija blago širenje do oko 200°C usled gubitka vode koje je manje kod kaolinskih glina, a zatim dolazi do skupljanja. Intenzivnije skupljanje je uočeno kod uzorka P, koje je kasnije praćeno i izraženijim širenjem. Razgradnja getita kod V u ovom periodu uzrokuje skupljanje. Takođe, intenzitet širenja zavisi od udela kvarca [10]. Početak sinterovanja je registrovan na oko 867°C (P) i 1116°C (V). Sinterovanje počinje ranije kod glina koje imaju veći sadržaj Na_2O i K_2O.



Dijagram 1. Ponašanje uzoraka pri zagrevanju: Uzorak B: a) DTA/TGA, b) dilatometrijska kriva; Uzorak C: c) DTA/TGA, d) dilatometrijska kriva

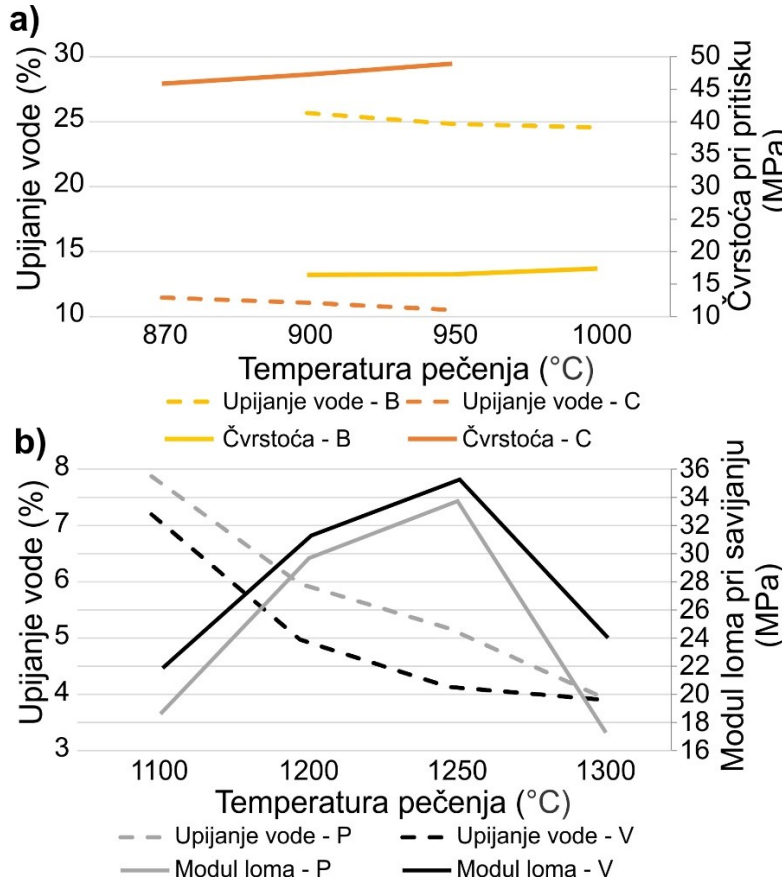


Dijagram 2. Ponašanje uzoraka pri zagrevanju: Uzorak P: a) DTA/TGA, b) dilatometrijska kriva i Uzorak V: c) DTA/TGA, d) dilatometrijska kriva

3.3. Osobine i kvalitet proizvoda nakon pečenja

Karakteristike pečenih proizvoda variraju sa temperaturom pečenja (*Dijagram 3*). Laboratorijski uzorci spravljeni od opekarskih glina su oblikovani korišćenjem ekstruder-prese i pečeni u električnoj peći. Uzorak B pokazuje visoke vrednosti upijanja vode od oko 25 % usled visokog udela karbonata (*Dijagram 3a*). Upijanje vode blago opada sa porastom temperature pečenja od 870-1000°C. Čvrstoća pri pritisku kod laboratorijskih šupljih blokova je znatno viša u slučaju gline C, u odnosu na B, zbog višeg udela glinenih minerala, nižeg udela kvarca i zbog razlike u prisustvu karbonata. Čvrstoća pri pritisku blago raste sa porastom temperature pečenja.

Keramičke i vatrostalne gline oblikovane su postupkom suvog presovanja. Usled razlike u udelu i tipu glinenih minerala, upijanje vode je više kod gline P. Upijanje vode blago opada sa porastom temperature pečenja od 1100-1300°C. Na visokoj temperaturi pečenja, kao što je 1300°C, dolazi skoro do izjednačavanja vrednosti upijanja vode kod ovih uzoraka, što je kombinovan efekat vrste i količine glinenih minerala, i udela topitelja [11]. Kod P se javlja velika razlika u modulu loma pri savijanju u zavisnosti od temperature pečenja. Na visokoj temperaturi, ova vrednost nije pogodna. Svakako je uobičajena industrijska temperatura pečenja keramičkih pločica oko 1200-1250°C, te ova prirodna sirovina zadovoljava zahteve standarda.



Dijagram 3. Osobine pečenih laboratorijskih uzoraka: a) opekarske gline i b) keramičke i vatrostalne gline

4. ZAKLJUČAK

Na osnovu sprovedene analize sirovina iz Srbije za proizvodnju različitih vrsta keramike, može se zaključiti da domaće gline pokazuju značajan potencijal za upotrebu u opekarskoj, keramičkoj i vatrostalnoj industriji, pri čemu hemijski i mineraloški sastav, kao i ponašanje tokom termičke obrade, igraju ključnu ulogu u određivanju njihove pogodnosti. Rezultati pokazuju da sirovine sa većim sadržajem Fe_2O_3 i karbonata više odgovaraju proizvodnji opekarskih proizvoda, dok su one sa višim udelom Al_2O_3 i nižim sadržajem nečistoća pogodnije za finu i vatrostalnu keramiku. Ispitivanje ponašanja pri termičkoj obradi uz pomoć instrumentalnih metoda dodatno je potvrdilo razlike u ponašanju pri zagrevanju, što direktno utiče na osobine gotovih proizvoda kao što su čvrstoća i upijanje vode.

Uzimajući u obzir lokalni resursni potencijal i savremene analitičke metode, moguće je sprovesti selekciju i optimizaciju sirovina na održiv i efikasan način, čime se otvara prostor za dalji razvoj keramičke industrije u Srbiji.

Zahvalnica

Zahvaljujemo se Ministarstvu nauke, tehnološkog razvoja i inovacija Republike Srbije na pomoći i podršci koju nam pruža za sprovođenje istraživačkog rada (Ugovor br. 451-03-136/2025-03/200012).

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
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ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

Ljubomir Jevtović[1], Nemanja Mrđić[2], Emilija Nikolić[3] https://doi.org/29DIMK.019J
Pregledni rad

A SHORT OVERVIEW OF THE PRODUCTION AND USAGE OF CERAMIC BUILDING MATERIALS IN ANCIENT VIMINACIUM

Summary: *Favourable physical characteristics, modularity and standardised, but still various forms and sizes, as well as the aesthetic values, made fired brick one of the most valuable historical building materials. Local geological setting and army organisation made it possible for the Roman Viminacium construction industry to obtain quality raw material in sufficient quantity and easily combine it into a final product. Its CBM production started in the early 2nd century and declined in the end of the 3rd century, although its products were used much longer, for the construction and covering of the walls, arches and vaults, floors and roofs, as well as for water and heating systems. The vast majority of the bricks for funeral masonry constructions, were already used, confirming the rationality in Roman building practice, offering an example of the circular economy we strive for today.*
Keywords: *Viminacium, Roman fired brick, CBM, reuse.*

KRATAK PREGLED PROIZVODNJE I UPOTREBE KERAMIČKIH GRAĐEVINSKIH MATERIJALA U ANTIČKOM VIMINACIJUMU

Резиме: *Повољне физичке карактеристике, модуларност и стандардизовани, али ипак разлнолики облици и величине, као и естетске вредности, учинили су печену опеку једним од највреднијих историјских грађевинских материјала. Локални геолошки контекст и војна организација омогућили су римској грађевинској индустрији Виминацијума да добије квалитетне сировине у довољним количинама и лако их комбинује у финални производ. Производња керамичких грађевинских материјала у Виминацијуму покренута је почетком 2. века, а опала крајем 3. века, иако су се производи користили много дуже, за изградњу и облагање зидова, лукова и сводова, подова и кровова, као и за водоводне и грејне системе. Већина опека у погребним зиданим конструкцијама, већ је била коришћена, потврђујући рационалност римског грађевинарства, нудећи пример циркуларне економије којој данас тежимо.*
Кључне речи: *Виминацијум, римска печена опека, керамички грађевински материјал, поновна употреба.*

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1. INTRODUCTION

Throughout human history, clay-based building materials played an important role in the development of most of the major civilisations. Due to their simple production process, yet being strong, durable, as well as aesthetically pleasing products, ceramic building materials (CBM) were widely used since their first confirmed appearance in the 4th millennium BCE in the valleys of the Nile, Tigris and Euphrates [1,2]. The production and use of a whole range of products culminated during the late Roman Republic and later Roman Empire, when the technology was further developed and the products standardised. Roman production was widespread and lucrative. There were state and privately-owned brick yards, while in the provinces, the most important producer was the army. This paper aims to deliver a short overview of usage of the CBM detected in the ancient Viminacium, taking into account its production and the specificities of its use in the excavated buildings. Viminacium is one of the best explored major Roman agglomerations in present-day Serbia (Figure 1). Virtually all of its segments were at least partially excavated. Moreover, it is the Roman site with one of the largest and best archaeologically researched CBM production facilities in the region [3].

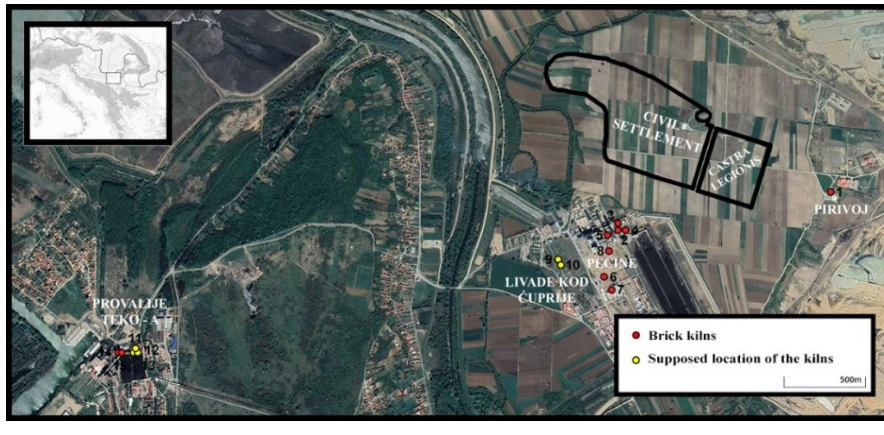


Figure 1. Viminacium and CBM production sites (after [3])

2. VIMINACIUM CBM PRODUCTION SITES

Remains of the Viminacium city and legionary fortress are located in Eastern Serbia, next to the modern town of Kostolac, on the right bank of the river Mlava, near its confluence with the Danube (Figure 1). Viminacium was established as a military outpost in the second half of the 1st century CE, originally as the base of the *Legio III Flavia*, and from the late 1st – early 2nd century as the fortress of the *Legio VII Claudia*. It was in this period that the civilian urban settlement started developing to the west of the fortress. During the reign of Hadrian, in 117, Viminacium was granted the status of municipium, while in 239, during the reign of Gordianus III, it became colonia. The city was destroyed by the Huns in 441 and never again restored its former prosperity [4-7].

Archaeological excavations of Viminacium began in 1882 [8], followed by a brief campaign in 1902–1903 [9]. The excavations resumed only in 1972 [10- 11], being

intensified from 1977 to 1991 due to the construction of the Kostolac B power plant and Drmno coal mine. These primarily protective excavations uncovered extensive necropolises around the city [12-14]. After sporadic work in 1992 and 1997, research resumed continuously from 2001 onward [15], expanding to nearly all parts of the site, including the legionary fortress, city, suburbs, necropolises, production areas, rural estates, and water infrastructure [16].

Throughout these excavations, thousands of pieces of CBM were unearthed, both fragmented and whole, as well as several areas of its production. CBM, including bricks, tiles, pipes and supplementary ceramic building elements, was produced locally in Viminacium, being very important for the construction of the area itself, but also for the other settlements and fortifications along the Danube limes in the province of Moesia Superior, to which it was transported [17-18].

Viminacium excavations uncovered four CBM production zones in the vicinity of the city and fortress, with at least 16 brick kilns grouped together with the kilns for the production of other ceramic products (pottery, lamps, figurines, etc) [3,19] (Figure 1).

The production on the site "Pirivoj", where a single kiln was discovered, was conducted between the fourth decade of the 3rd century and the third decade of the 4th century [20-21,3]. The production on the site "Pećine comprised of at least 4 CBM production units (workshops), spread over an area of c. 16,000 m². The most complex one, Unit no. 2 had 3 quadrangular kilns organised in a single structure, 3 circular and 1 small quadrangular kiln, a drying hall(s), a clay pit, a well and remains of several other structures (Figure 2). Units no. 3 and no. 5 (Figure 2) comprised a single kiln each of large capacity, while a large pit was positioned in the vicinity of the Unit no. 5 (105* x 58* m). Unit no. 4 had 2 large quadrangular kilns, placed on the edges of a large borrow pit, as well as a possible workshop located nearby [3, 17, 22, 23, 24, 25, 26]. The production at the "Pećine site" functioned between the middle of the 2nd century and the middle of the 3rd century [3]. The site "Livade kod Ćuprije" is located c. 200–250 m west of the larger production centre at "Pećine" site, and was home to a single unit – Unit no. 6, comprising 2 quadrangular kilns. This production is dated to the 2nd–3rd century, predating a 4th century villa, located in the vicinity [3,27]. The site of "Provalije – TEKO A" is located on the opposite bank of the Mlava River, in the territory of the modern-day "Kostolac A" thermal power plant. At least two production units can be identified, Unit no. 7, comprising 3 kilns, and Unit no. 8 with 2 kilns [3,19,28], while 9 kilns have been identified and destroyed on the site before the archaeological excavations [29]. This production can be dated to the 2nd and the middle of the 3rd century [3].

The analysis of the workshops, dimensions and remains of their facilities indicates that the production on the sites "Pećine" and "Provalije – TEKO A" was characterised by complex spatial and functional organisation and large production. Both sites were operated by the military, namely Legio VII Claudia, and possibly in cooperation with the Legio IV Flavia in the case of the "Provalije – TEKO A". Unfortunately, the production scale at the sites of "Pirivoj" and "Livade kod Ćuprije" is hard to analyse, due to the protective nature of excavations and, thus, their small extent.

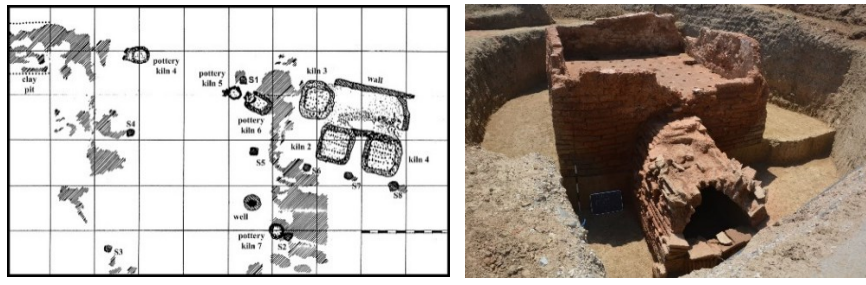


Figure 2. Plan of the Unit no. 2 at the Pećine production site (Documentation of the Institute of archaeology), and the Kiln no. 8 of the Unit. no 5 (after [3].)

3. VIMINACIUM CBM USE

The major usage of CBM was identified on several excavated Viminacium sites in the legionary fortress, city or any suburban area, having function in construction of various buildings – while forming walls, arches or vaults, in floor and wall heating systems, as well as in floor and roof cladding (Figure 3). Its large use, mostly secondary, was detected in the construction of the tombs and graves (Figure 4).

3.1. Legionary fortress

Archaeological research of the Viminacium legionary fortress [30-32] included defensive systems (gates, ramparts, towers and dich) and adjacent buildings, as well as central area of the fortress (*Principium*). Currently, it is possible to identify two main phases of construction. The first phase is dated in the last decades of the 1st century CE and is characterised by the usage of "natural brick" (known locally as "crvenka", metamorphosed local sediment, burnt by the coal fires [33]), as the primary construction material, used to build both the fortifications as well as the walls of buildings inside the fortress. In the second phase, dated to the 2nd century, the ramparts of the fortress were rebuilt with stone (schist and limestone). In the following centuries, there is no clear evidence of any major construction work being done, although several small-scale repair work can be identified between the middle of the 2nd century and third quarter of the 3rd century [34-35]. Based on the current evidence, it seems that northwest part of the fortification was abandoned in late antiquity, most likely in the latter half of the 3rd century, although the principium was used at least until the time of Emperor Constantine [32].

The CBM was not used for construction of Viminacium fortification systems, except for some micro repairs. Additionally, it was almost never used in the construction of the walls of the buildings inside the fortress. Very rarely, it supplemented other materials [35]. It almost completely lacks in the initial phases of the central fortress area (principium), but it is used in all later phases for specific construction elements. In the walls, bricks were used only in the most monumental part of the building (aedes / sacellum) - for supporting walls, arches and vaults. In principium, bricks and tiles were used for usual floor paving, as well as for decorative floors (rectangular bricks and butterfly shaped tiles – Figure 3). Large quantities of CBM were discovered in the rubble of most fortress

buildings, including principium, so we can propose that roof might have been covered with ceramic tiles. The only building inside the fortress, where CBM was originally and widely used, are the legionary baths, built in the first half of the 2nd century and rebuilt during the 3rd century [35]. Whole bricks and tiles of different dimensions were used to construct some of its walls and floors inside and surrounding the building, including the pillars in the hypocaustal space, where the ceramic pipes were additionally used as pillars as well. CBM, especially tegulae, were used for flooring of the canals in both construction phases of the fortress, with occasional use of bricks for their walls, and the tiles used for the flooring of the fountain next to the western fortress gate, attributed to the second phase of the fortress [34-37]. In the later phases of the fortress, from the 3rd century onwards, CBM fragments were used for the repair. This was noticed in the major fortress communications, both *via sagularis* and *via principalis*, but also in some of the buildings. After the abandonment of the fortress, CBM was massively (re)used. Fragmented and whole bricks and tiles were used for the construction of the buildings inside the fortress, on top of its western ditch, and for the construction of the graves both inside and outside the fortress. Two excavated buildings with the ovens, situated along a communication had brick flooring, while fragments were used for the construction of walls, which was also the case in the third building phase dated to the middle of the 3rd century [34-39].

3.2. Urban civilian settlement

The civilian settlement of Viminacium is positioned west of the fortress. The excavations have been focused only on its northwestern part, containing the remains of the large public buildings – the city baths and the amphitheatre, as well as the north and eastern city ramparts.

North and eastern ramparts of the settlement were constructed at the end of the 2nd century. Together with the towers, the ramparts were built of schist or limestone blocks, while brick fragments were rarely used. Bricks, tegulae, imbrices with schist were used for a tower. On the eastern ramparts, bricks were observed only in the eastern gate area. There, fragmented CBM, together with schist and crushed "natural brick" was used for the street substruction and its sidewalk [40].

Amphitheatre is located in the northeastern corner of the settlement. Almost all of its elements were excavated, including the arena and the most of the stands. It was built in the early 2nd century completely out of wood, and was rebuilt in the time of Hadrian (117–138) with the stands made out of wood and the arena wall, entrances and rooms built of stone. At the end of the 2nd century, the eastern and northeastern parts of the amphitheatre were remodelled, as the structure was partially encircled by the city ramparts. Finally, the amphitheatre was abandoned during the 4th century. In general, bricks and tiles were rarely used in its construction. Whole and fragmented bricks were discovered only in a part of the arena wall and its side rooms, being used for repairs (Figure 3), and in the parts of the western and eastern entrances and its side rooms. In the eastern entrance, bricks were used for the construction of two out of eight piers. One of them was partially built using several layers of bricks, while the other was constructed using schist and limestone. Additionally, fragments of bricks, tegulae and imbrices were used for flooring of the entrances, together with "natural brick" and schist, and for the substructure of the outer doorstep. Whole bricks were also used for flooring of the rooms next to the arena wall, connected to some of the later stages of the amphitheatre's construction. Significant usage of CBM was noted in its

northern part, where tegulae were used for flooring of a drainage canal, associated with the second construction phase of the amphitheatre. Finally, bricks with limestone and schist were used in the construction of three small structures (shrines), located underneath the northeastern part of the stands, and linked to various periods of amphitheatre's construction [40-41].

Remains of the city baths (*thermae*) are located in the eastern part of the civilian settlement, near the amphitheatre. The research has identified its three construction periods. First structures at the site were built during the 1st and 2nd centuries. The overground structures preserved up to day are dated to the end of the 2nd and during the 3rd century, and some of them were renewed in the beginning of the 4th century [11,42-44]. The baths comprised of five apses (pools for bathing) situated around a central hypocaustal space and several peripheral rooms. Bricks were used as one of the basic building materials, together with schist. Rectangular bricks were used for the arches, vaults, walls and wall cladding. In combination with square bricks, they were used for piers, arched vaulted openings and canals as well for the pillars of the hypocaustal system The floors inside and around the building were made of rectangular as well as hexagonal, and butterfly shaped bricks (Figure 3). Both whole and fragmented bricks formed part of the wall cores, wall faces and bonding courses, and were an important component of waterproof lime mortar in this building. Finally, fragmented CBM was used for the building repairs, together with schist and pieces of limestone. Specific use of CBM in Viminacium was detected at this building, identifying found ceramic fragments as being parts of vaulting tubes, that is, the modular units which were joined together to make vaults [45], but also the ceramic elements for fixing the stone veneer for the wall.



Figure 3. CBM in Viminacium: the baths: a. hypocaustal space and hexagonal floor tiles (after [44]), b. c. walls built in different techniques (Documentation of the Institute of Archaeology), d. vaulted opening (MoDeCo2000 project); amphitheatre: e. brick repairs in the stone wall; principium: f. butterfly-shaped floor tiles, g. rectangular floor tiles (MoDeCo2000 project); villa "Pirivoj": h. a wall and a vault (MoDeCo2000 project); villa "Nad Klepečkom": i. wall heating (after [44]); j. aqueduct: canal floor (after [48]).

3.3. Suburban villas

Several Roman villas and smaller settlements coming from various periods were excavated in the areas surrounding Viminacium city and the fortress Almost all of them had some elements made using CBM, but two stand out, due to their size and complexity [46-47].

Complex at the "Pirivoj" site comprised at least three buildings, together with a canal, portico, wells, and a necropolis [46,16]. It had six construction phases, thus undergoing intensive remodelling. The most interesting was a building with 13 rooms. Two of them had apses, and four rooms in total had hypocaustal system with bricks pillars. Additionally, discovered tubuli indicated that these four rooms had wall heating. Bricks were used for the construction some walls of the buildings, while hexagonal and butterfly-shaped tiles testify of the floor covering. The drainage canal that ran out of the building was made of bricks, as well as its vaulted construction (Figure 3). The use of bricks was significantly higher in the during the later phases of the complex – for the construction of walls and repair work. The villa was dated to the 3rd century. The villa found at the site "Nad Klepečkom" is the largest one found in Viminacium so far and comprised at least 24 rooms [47]. Its walls were built of stone, the bricks were used for the construction of floors, hypocaustal pillars, while tubuli was used for the wall heating system (Figure 3). Additionally, piles of rubble indicate that the villa had a roof covered with ceramic tiles. The villa was dated to 2nd century.

3.4. Aqueducts

Two aqueducts, that most likely supplied the legionary fortress were discovered to the east of Viminacium, on the site ''Stig'' [48]. The walls of both of these were primarily built out of stone. The floor of one aqueduct was laid exclusively using tegulae (Figure 3), while canals of both of them were covered with bricks. For the network of water infrastructure connecting the built aqueducts with the structures it supplied with water, ceramic pipes were used.

3.5. Necropolises

Viminacium has one of the most thoroughly excavated cemeteries from the Roman period. Only its southern necropolis, located on the sites ''Pećine'' and ''Više Grobalja'' had around 11.500 excavated graves. Although most of the deceased were buried without any grave construction, more than 2.280 of them had at least one brick in the construction [49]. Majority of these represented a simple brick coffin, while large family tombs were masonry structures with arches and vaults built almost exclusively of bricks (Figure 4). The archaeological research has shown that the vast majority of the CBM material present in funeral constructions was already used, originating from other structures, and only rarely newly made bricks were used.



Figure 4. Types of Viminacium graves and tombs made using CBM [after 49]

4. CONCLUSION

Viminacium CBM was produced locally, with the largest producer being the army, running a series of well-organised workshops. Based on the current data, the production started in the early 2nd century, reached its peak in the following centuries and slowly declining until the end of the 3rd century. Having in mind that the output of a single production unit could be as high as several thousands of products, it comes to light that the overall production of the Viminacium workshops was truly staggering.

CBM was used for the construction of the walls in various Viminacium buildings, but mostly for the arches and vaults, for which the suitable stone was not easily available in the area. In the earlier phases of its production (i.e. 2nd and 3rd century), CBM was massively and almost exclusively used in the roof cladding and where water and thermal/fire resistance of the structure were needed. It is noteworthy to mention that the largest building excavated so far and dated to this period, with significant number of elements built out of CBM, were the public baths. In the 3rd and especially in the 4th century, the usage of CBM became widespread for various constructions. Still, it is worth repeating that vast majority of bricks used for the funeral masonry structures was actually reused. Since huge amounts of whole and fragmented CBM material was left behind after the ruination caused by various natural or human influences in the previous period or simply after the remodelling of the buildings, its life was prolonged by using them for making tombs and graves and other buildings. This confirms general rationality and practicality in the use of materials in Roman building practice.

Versatility of use made brick one of the most valuable building materials throughout history. Capable to withstand high temperatures, it was used in heating systems; being capable to face humidity and water it was part of the water and sewer systems. Unlimited

range of shapes made it perfect for creation of various construction and decoration elements. Its modularity made it perfect for fast conduction of construction works. Finally, favourable local geological and natural setting of Viminacium [50], in combination with army organisation, made it possible to obtain quality raw material in sufficient quantity, which can be easily combined into a final, standardised product such as fired brick.

With no intention to judge certain actions of the people from the past, during which the recognition of the importance or heritage values was not developed as it is today, we can emphasize that Viminacium brick longevity and durability is also testified with its reuse in the later historical periods, from the early Byzantine period through the Middle Ages [51, 52], up to today. Viminacium bricks, visible today in in the walls of local Kostolac houses, are still solid and strong as they were when they were first used by the Romans.

The story of Viminacium bricks certainly deserves wide research, contributing to general understanding of CBM production and use in the Roman provinces, but also reinforcing the value of integrated scientific approach in reconstructing the lifecycle of historical building materials, from production to reuse, thus offering us lessons on circular economy we can use today while fighting for sustainability.

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

Kristina Šarić[1]

https://doi.org/29DIMK.030S

GLINE: OD GEOLOŠKOG KONTEKSTA DO KULTURNOG NASLEĐA

Rezime *Gline predstavljaju finozrni prirodni materijal koji u dodiru s vodom postaje plastičan, a prilikom sušenja i pečenja očvršćava. U geološkom smislu, gline su i minerali (filosilikati, glavni predstavnici: kaolinit, monmorijonit, ilit) i stene (klastične pelitske sedimentne stene). Struktura filosilikata se sastoje od TO ili TOT slojeva. Slojeve čine tetraedarski (T) i oktaedarski (O) listovi. Tetraedarski list se sastoji od SiO_4^{4-} tetraedara koji su polimerizovani preko tri zajednička kiseonika. Oktaedarski list je preko četvrtog kiseonika povezan sa tetraedarskim listom. Čine ga oktaedri u čijem centru mogu biti Al, Mg, Fe, okruženi sa šest kiseonika. U zavisnosti od pakovanja listova u TO ili TOT slojeve, filosilikati mogu imati različite karakteristike. Na primer, monmorijonit (TOT) može vršiti razmenu katjona sa okolinom i apsorbovati molekule vode zbog čega bubri, dok kod kaolinita (TO) te karakteristike izostaju. Upotreba gline počinje još u neolitu i u Srbiji su pronađeni brojni jedinstveni artefakti. Mineraloško-petrološkom analizom moguće je utvrditi poreklo gline, temperaturu i uslove pečenja, na osnovu kojih se dodatno interpretira arheološki kontekst (na primer, tehnologija proizvodnje i njena promena kroz vreme). Primeri ovakvih istraživanja kod nas su proučavanja rimskih opeka sa Viminacijuma, ranovizantijske (Caričin Grad) i srednjovekovne keramike (Studenica) i druga.*
Ključne reči: *filosilikati, klastiti, peliti, plastičnost, keramika, mineraloško-petrološka analiza, arheološki kontekst*

CLAYS: FROM GEOLOGICAL CONTEXT TO CULTURAL HERITAGE

Summary *Clays are a fine-grained natural material that becomes plastic in contact with water, and hardens during drying and baking. In geological terms, clays are both minerals (phyllosilicates, main representatives: kaolinite, montmorillonite, illite) and rocks (clastic pelitic sedimentary rocks). The structure of phyllosilicates consists of TO or TOT layers. The layers are made up of tetrahedral (T) and octahedral (O) sheets. The tetrahedral sheet consists of SiO_4^{4-} tetrahedron that are polymerized over three common oxygens. The octahedral sheet is connected to the tetrahedral sheet by a fourth oxygen. It consists of octahedron in the center of which can be Al, Mg, Fe, surrounded by six oxygens. Depending on the packaging of the sheets in TO or TOT layers, phyllosilicates can have different characteristics. For example, montmorillonite (TOT) can exchange cations with the environment and absorb water molecules, causing it to swell, while*

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kaolinite (TO) lacks these characteristics. The use of clay dates back to the Neolithic and a number of unique artifacts have been found in Serbia. By mineralogical-petrological analysis, it is possible to determine the origin of clay, temperature and other firing conditions, on the basis of which the archaeological context is further interpreted (e.g. the technology of production and its change over time). Examples of such research in our country are the study of Roman bricks from Viminacium, early Byzantine (Caričin Grad) and medieval ceramics (Studenica) and others.
***Keywords**: phyllosilicates, clastites, pelites, plasticity, ceramics, mineralogical-petrological analysis, archaeological context*

SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
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Hartmut Pasternak[1], Konstantin Pasternak[2]

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Prethodno saopštenje

WIND POWER PLANTS – POSSIBILITIES, CHALLENGES AND LIMITS

Summary: *Against the background of the EU Climate Law of 2021, the generation of wind energy plays a special role. A brief description of how a wind turbine works is followed by a classification (Onshore and Offshore turbines) with advantages and disadvantages. The components of wind turbines are then discussed, with a particular focus on the foundation, towers and blades and challenges during installation and operation are explained. With artificial intelligence, it seems possible to improve the accuracy of condition assessment of wind power plants, optimize resource allocation and minimize downtime.*
Keywords: *Wind power plants, onshore, offshore, foundation, tower, blade, installation, operation*

VETROELEKTRANE – MOGUĆNOSTI, IZAZOVI I OGRANIČENJA

Rezime: *U svetlu Zakona o klimi Evropske unije iz 2021. godine, proizvodnja energije korišćenjem snage vetra dobija poseban značaj. Nakon kratkog opisa načina rada vetroturbine, sledi klasifikacija (kopnene i morske turbine) sa prednostima i manama svake. Zatim se razmatraju komponente vetroturbina, sa posebnim osvrtom na temelje, stubove i lopatice, a objašnjavaju se i izazovi tokom instalacije i rada. Korišćenjem veštačke inteligencije moguće je unaprediti tačnost procene stanja vetroelektrana, optimizovati raspodelu resursa i smanjiti vreme zastoja.*
Ključne reči: *vetroelektrane, kopnene, morske, temelj, stub, lopatica, instalacija, rad*

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1. INTRODUCTION

The European Union is to become climate neutral by 2050. Climate neutrality means that human activity has no overall impact on the climate. In particular, carbon neutrality means that no CO_2 is emitted or that CO_2 emissions are fully offset. Under the EU Climate Law of 2021, the proportion of CO_2 would have to be reduced to at least 55% by 2030 compared to 1990 levels. The energy production by wind plays an important role in this.

Historically, the utilization of wind power is not new: Persian windmills can be traced back to the 7th century, and in the 1880s there were still 20,000 windmills in Germany. They converted (kinetic) wind energy into mechanical energy (Figure 1).

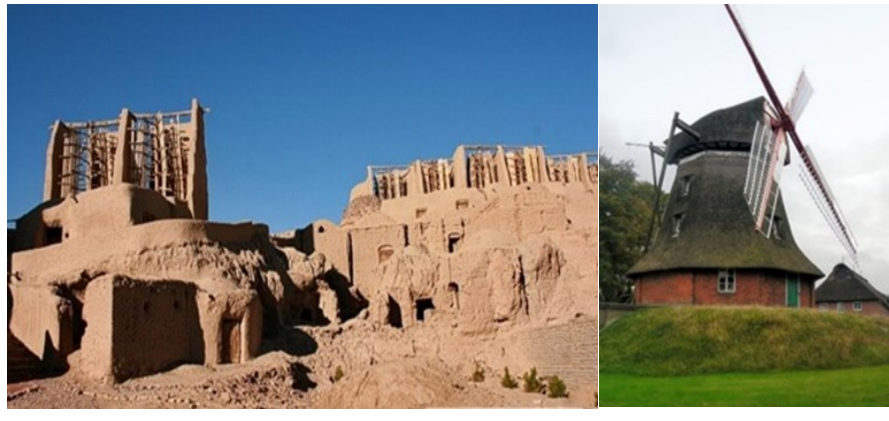


Figure 1. Persian and Dutch windmill

Electricity then led to the utilization of mechanical energy in electrical energy. In the 1950s, wind turbines were first built on land (onshore), followed by offshore wind turbines from 1990 onwards.

The law established by Albert Betz (1885-1968) in 1919 states that a wind turbine can convert a maximum of 59% of the mechanical power that the wind would transport through its projection surface (rotor surface perpendicular to the wind direction) without the braking rotor into useful power. The reason for the limitation is that the energy output is accompanied by a reduction in the flow velocity and a build-up of air, which causes some of the approaching air to avoid the rotor surface.

2. OPERATION MODE OF A WIND TURBINE

Two design principles have become established for wind turbines: Systems with a gearbox (Fig. 2) increase the low speed of the rotor to a speed that is favourable for the generator. In gearless systems, the rotor of the generator sits directly on the rotor shaft. The controller can start up the wind turbine at start-up wind speeds and switch it off again at switch-off wind speeds. There are also aerodynamic, mechanical and electrical losses (Fig. 3).

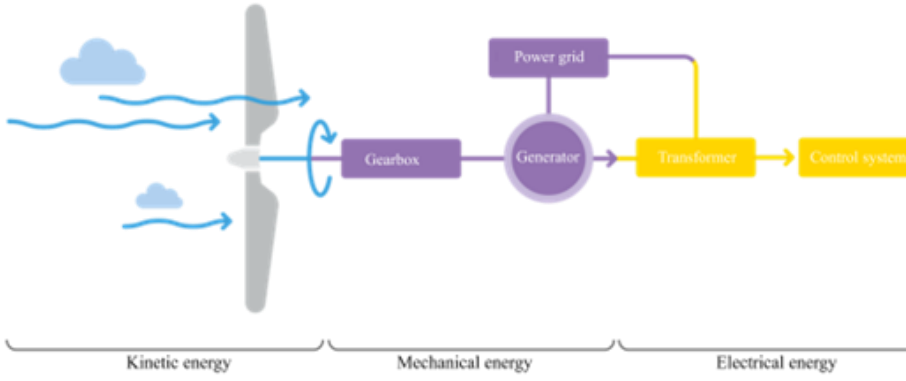


Figure 2. How a wind turbine works

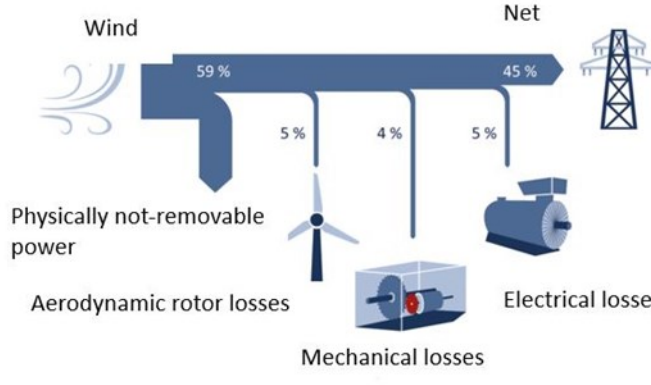


Figure 3. Aerodynamic, mechanical and electrical losses

3. ENERGY CONSUMPTION IN GERMANY

Electricity consumption in Germany (Fig. 4, black curve) in January 2024 fluctuated between 50 GW and 75 GW; electricity from onshore wind turbines is shown in light green, that from offshore in dark green. Photovoltaics are shown in yellow for comparison. As can be seen, wind and solar power supply volatile electricity. Wind energy at sea (offshore) supplies around 3600 full-load hours, or 40% of a year. The utilization of wind energy on land (onshore) is less than 30% (2021: 23%) and that of solar energy is only 10% of maximum output. By 2030, onshore wind turbines with a capacity of 71 GW are to be installed in Germany.

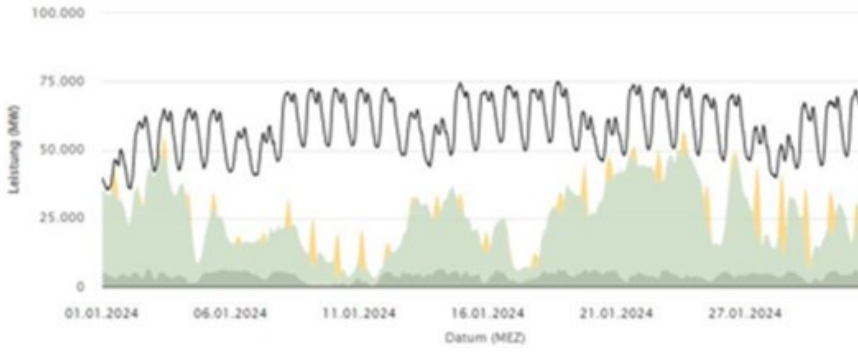


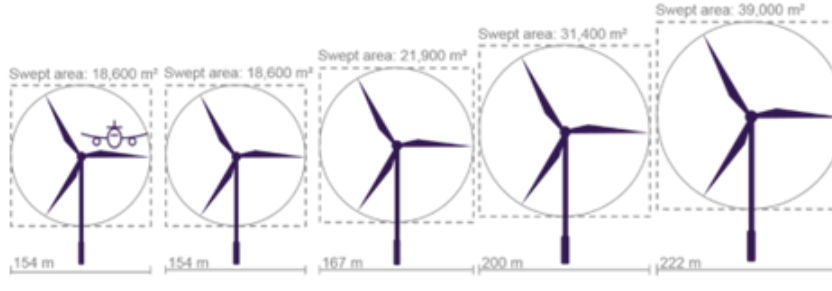
Figure 4. Electricity consumption in January 2024 and the share of wind turbines and photovoltaics

4. WIND POWER PLANTS - CLASSIFICATION AND COMPONENTS

The modern wind power sector can be divided into three categories, depending on where the wind turbines are located and which substructure is used: Onshore turbines (with foundations inland), nearshore turbines (with foundations in the seabed) and floating offshore turbines (on a platform floating above the seabed). Onshore turbines are less complex to install but less powerful, whereas stronger winds with powerful offshore turbines allow more electricity to be generated.

Rotor blades and nacelle

Rotor blades consist of two bonded half-shells made of fibre composites - light and stable glass and carbon fibres - mixed with epoxy and vinyl resins, among other things. Added to this are metals such as iron, copper, aluminium and lead as well as plastic foams, balsa wood and paints with titanium dioxide particles - in other words, a mix of materials that cannot be broken down into its components. However, the first recyclable rotor blades are already on the market. Rotor blades are becoming longer and longer because the power output of a wind turbine depends on the area covered, i.e. it increases in proportion to the square of the diameter (Fig. 5). Today's rotor blades can be up to 115m long (Fig. 6). Current prototypes of the future push the blade length to 135m (SG21-276 DD, 21MW).



	SWT-6.0-154	SWT-7.0-154	SG 8.0-167 DD	SG 11.0-200 DD	SG 14-222 DD
IEC Class	I, S	I, S	I, S	I, S	I, S
Nominal Power	6 MW	7 MW	8 MW	11 MW	14 MW
Blade length	75 m	75 m	81.4 m	97 m	108 m

Figure 5. Wind turbines are getting bigger and bigger



Figure 6. Rotor blade, 115m long



Figure 7. Nacelle (SG 11.0-200 DD)



Figure 8. Prototype with a divisible (SG14 236 DD) generator (E-175 EP5 E2)

Offshore nacelles weigh up to 500 tonnes and are placed on the tower at approx. 100-150m (Fig. 7). To avoid the logistics problems associated with larger onshore generators, in 2025 a prototype has developed a divisible generator for a rated output of 7 MW.

Towers

Today, *tubular steel towers* are the most common and most widely used type of tower. There are several types of steel towers, such as cylindrical and conical. Steel towers are divided into two to five segments, each 20 to 30 m long. Transporting the onshore tower segments of very large wind turbines can be problematic with tubular steel towers, as the road bridges are lower than the diameter of the lowest tower segments. Offshore tower foundations are usually transported by sea. A foundation vessel sails to the wind farm to place to foundation at the respective location Offshore tower sections are delivered by sea to a pre-assembly port. These sections are erected and transported with other main components (nacelle, blades) to the wind farm on the installation vessel (Fig. 9).

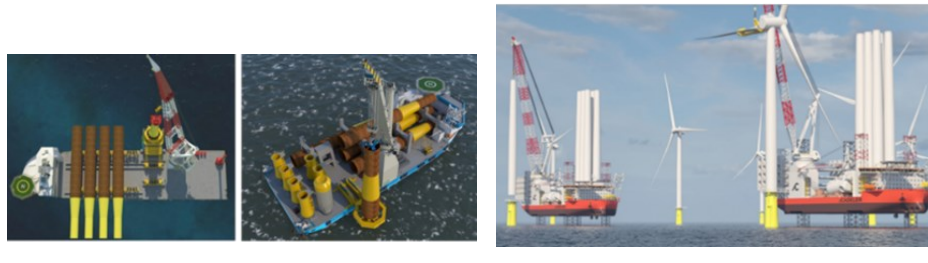


Figure 9. Tubular steel tower – foundation



Figure 10. Lattice tower and head and installation vessel

Lattice towers require less material (about half as much as tubular steel towers) and are therefore lighter and easier to assemble. Nevertheless, they are more expensive in Europe than cylindrical towers, as a lot of labour time is required for production and assembly. Figure 10 shows a 160m high tower with a capacity of 2.5 MW in Lusatia. Wind turbines with a hub height of 300m are being planned.

The lower part of a *hybrid tower* is made of reinforced concrete, the upper part of steel. They are mainly used for tall towers, as they avoid the transport problem of the large diameter of the steel lower sections and therefore make greater hub heights possible. The currently tallest onshore wind turbines with a total height of 246.5m are located in Baden-Württemberg (Fig. 11).



Figure 11. Hybrid towers



Figure 12. Circular foundation

Foundation

The foundation ensures the stability of the wind turbine and transfers all loads from the wind rotor and the turbine's own movement to the ground. Shallow foundations are the most common for onshore wind turbines. Pile foundations are also used on soft ground. Depending on the design of the tower, the basic shape can be octagonal, circular or cross-shaped, for example (Fig. 13). Up to 10,000 tonnes of concrete are sunk into the ground as a foundation for each wind turbine tower. Depending on the location and subsoil, up to 12m deep holes must be drilled. The tower is connected to the foundation by means of the so-called foundation installation part.

Offshore foundations can consist of concrete as well as steel piles or multi-leg structures. Experience with oil platforms has shown that corrosion in the open sea is not a major problem and with electrical corrosion protection such a foundation can be designed for a service life of 50 years. The type of offshore foundation chosen depends on many factors. Water depth, current, wave height and risk of ice are the main ones. With *gravity foundations* (Fig. 13a), the turbines are fixed to the seabed by the weight of the foundation. Caissons are built from steel and concrete in a dry dock on the coast, towed out to the installation site by vessel and filled with gravel and sand after being lowered to the seabed. One advantage of concrete caissons is their high resistance to ice drift. Disadvantages are the high costs at greater depths. So far, gravity foundations have only been utilized in shallow waters with low water depths (< 10 m) and are uneconomical for greater depths. *Monopile* constructions made of steel (Fig. 13b) are the simplest method for offshore foundations. They consist of steel pipes that are driven into the seabed. This is a globally utilized technology, e.g. for 14MW turbines in the North Sea (Scotland). However, heavy pile-driving equipment is required for construction, which can cause very high noise emissions that can disturb and harm marine mammals in particular. Monopile constructions cannot be used on rocky seabeds. *Tripod, quadripod or lattice tower constructions* are required for greater depths (> 20 m) and higher turbine outputs (> 5 MW). This method was derived from the construction of oil drilling platforms. The tower of the wind turbine is connected to a tubular steel frame and distributes the forces over several legs or a lattice tower. These can be anchored to the seabed either with a pile foundation (Fig. 14a) or with a gravity foundation (Fig. 14b). Much smaller cross-sections can be used for the pile foundation than for the monopile. This makes pile driving much easier. For water depths of more than 50 m, foundations that are firmly anchored to the seabed are difficult to realize for wind turbines. The idea of using floating foundations is therefore an obvious one. A floating foundation is anchored to the seabed using ropes (Fig. 15). There is already experience with such foundations in the oil industry. In contrast to this, however, much greater forces act on the floating foundations when used in offshore wind farms.

5. CHALLENGES

The locations selected for wind farms are favoured in windy regions for understandable reasons. The installation of components (e.g. lifting operations using special cranes) is a complicated maneuver and is tied to maximum wind speeds. In particular, the installation of rotor blades offshore on the nacelle must take into account the forces acting on the ‘loose’ blades.

Delays due to so-called weather downtimes are common and cause time delays that are associated with higher costs.

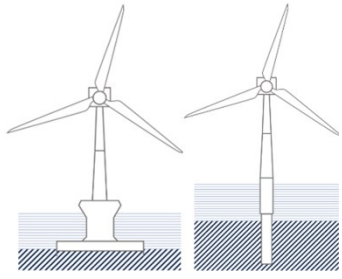


Figure 13. Foundation of wind



Figure 14. a Pile foundation, b Gravity foundation

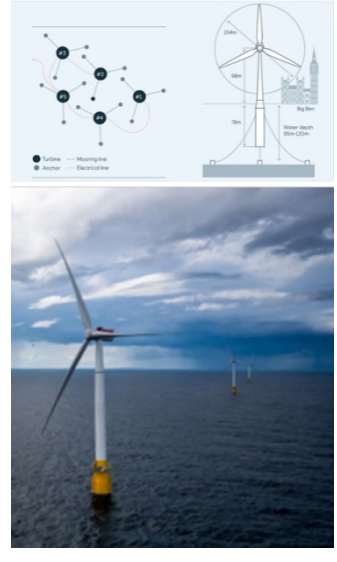


Figure 15. Floating foundation turbines: a Gravity foundation, b Monopile

The short product life cycles in conjunction with the continuous introduction of larger components (tower, rotor, nacelle) within a decade have presented manufacturers with challenges in terms of product quality. Quality deficiencies have an impact on overall equipment effectiveness and ultimately on the wind industry's key indicator of levelized cost of electricity (LCOE). Challenges from a dynamic point of view are e.g. vibration problems of the rotors and the increasing size of the bolts up to M90 (Fig.16).



Figure 16. a Blade, fallen off, b Column buckling

The climatic conditions for offshore turbines must also be taken into account for the operation of the turbine (temperature, humidity, salt water) in order to prevent corrosion, for example.

The ideal solution of reverse logistics incl. disposal on an industrial scale is not in place yet. With the lifetime of 20 years the relevance of this challenge keeps increasing daily. In Germany from 2025 70,000 tonnes of blades per year to be disposed of.

6. CONCLUSION AND OUTLOOK

The current share of wind power in electricity generation, the planned expansion and the rapid development cycles are evidence of the increased relevance of wind power technology. The commercial and technical limits are constantly being pushed. One promising aspect is that the industry can be adapted to a wide range of site conditions onshore and offshore, e.g. using different foundation methods or variable tower heights.

In order to ensure a reliable calculation basis for installation and operation, numerous data points from various disciplines (aerodynamics, material, static and dynamic loads) must be taken into account. Artificial intelligence is increasingly being used to process large amounts of data and complex models and to map digital twins (Fig.17). These enable a comprehensive solution contribution for life cycle management.

The monitoring data collected from wind power plants is fed into a model that simulates the interactions between the various components of the wind turbine under environmental conditions. The results of the simulations are processed with the load transfer from the structure to the components and then to the detail by analyzing the stress ranges and distributions. Following the load analysis, a model is applied to predict the structural deterioration of the structure over time, considering factors such as material roughness, critical crack size, growth rates and failure criteria. This applies, for example, to the assessment of the service life of bolts. It is a major challenge because it is one of the weak points of a wind turbine. Predicting the load-bearing capacity and reliability of bolted joints over the lifetime of a floating structure is difficult, due to both their size and the harsh conditions in which they are used.

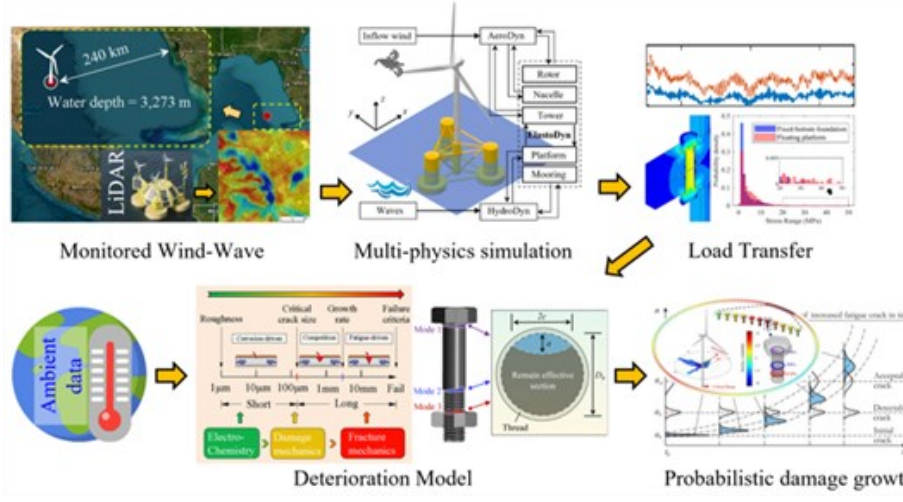


Figure 17. Forward digital twins from condition perception to state assessment [1]

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
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PHYSICAL, MECHANICAL AND ELECTRICAL PERFORMANCE OF METALLIC POWDER INCORPORATED CEMENT MORTARS

Summary: *This study explores the potential use of metallic powders (MP) from the scaffolding industry as a replacement material in cementitious mortars, focusing on circular economy practices. Various mortar mixtures were created by replacing sand with MP at replacement levels of 0.00% (reference), 0.25%, 0.50%, 1.00%, 2.00%, 4.00%, and 8.00% by weight. Following a 28-day water curing period, the samples underwent tests to evaluate their physical, mechanical, and electrical properties. The findings indicate that industrial waste, which is currently non-recyclable, can effectively serve as a substitute in cementitious composites. At low replacement rates of 0.25% and 0.50%, significant improvements were observed in water absorption, porosity, mechanical properties (including compressive and flexural strength), and electrical properties. It is anticipated that mortars incorporating MP will present opportunities for producing composites with enhanced functional and electrical properties.*
Keywords: *Cementitious mortars, waste metallic powders, electrical resistivity*

FIZIČKA, MEHANIČKA I ELEKTRIČNA SVOJSTVA CEMENTNIH MALTERA SA DODATKOM METALNOG PRAHA

Rezime: *Ova studija istražuje mogućnost korišćenja metalnih prahova (MP) iz industrije skela kao zamenskog materijala u cementnim malterima, sa fokusom na principe cirkularne ekonomije. Kreirane su različite smeše maltera zamenom peska MP-om u procentima od 0,00% (referentna serija), 0.25%, 0.50%, 1.00%, 2,00%, 4.00% i 8.00% mase. Nakon 28 dana očvršćavanja u vodi,*

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uzorci su testirani kako bi se procenile njihove fizičke, mehaničke i električne osobine. Rezultati ukazuju na to da industrijski otpad, koji trenutno nije reciklabilan, može efikasno služiti kao zamena u cementnim kompozitima. Pri nižim nivoima zamene od 0.25% i 0.50%, uočena su značajna poboljšanja u apsorpciji vode, poroznosti, mehaničkim osobinama (uključujući čvrstoću na pritisak i savijanje) i električnim karakteristikama. Očekuje se da će malteri koji sadrže MP pružiti mogućnosti za proizvodnju kompozita sa unapređenim funkcionalnim i električnim svojstvima.
Ključne reči: *cemetni malteri, otpadni metalni prahovi, električna otpornost*

1. INTRODUCTION

The environmental impact of cement and concrete production and rising performance demands for construction materials have prompted the incorporation of industrial waste into cement-based composites. Notably, waste metallic powders generated during surface treatment processes, such as sandblasting, have garnered attention due to their unique physical and thermal properties. Recent studies have demonstrated the viability of utilizing metallic waste, such as iron and aluminum powders, steel shavings, and nano-metallic residues, as a replacement for fine aggregates or as functional fillers in concrete and mortar formulations. These materials often originate from the post-processing of metal components, including abrasive cleaning methods like sandblasting, mechanical trimming, or can recycling. They typically consist of spherical metal-rich particles that may be minimally contaminated with trace amounts of siliceous dust or paint residues [6]. In roller-compacted concrete (RCC), adding metallic powders obtained from sandblasting (replacing 0.25% to 2% of the fine aggregate by weight) led to notable enhancements in flexural strength, modulus of elasticity, and impact resistance. However, this improvement was accompanied by a decline in compressive strength and workability. These results indicate that while incorporating metallic waste can significantly enhance ductility and toughness, vital properties for pavements, and high-traffic applications, careful mix design is necessary to effectively manage these mechanical trade-offs [6]. The use of metallic waste for radiation shielding in cement-based systems has been thoroughly researched. Studies by Mahmoud et al. [3] and Van Thuong et al. [9] demonstrated that elevating the proportion of industrial metallic waste in concrete and clay bricks significantly improves their capacity to attenuate gamma rays. This enhancement is primarily attributed to increased density and reduced porosity at higher substitution levels. Such applications are particularly important in the fields of medical, nuclear, and industrial radiation shielding. Nano-scale metallic residues, particularly nano-alumina (sourced from recycled cans) and nano-ferrites, have shown considerable promise in enhancing mechanical durability and radiation attenuation in heavyweight concrete. Research by Tobbala et al. [8] demonstrated that the optimized combination of nano-alumina and nano-ferrite at approximately 2% each by weight significantly improved compressive and tensile strengths. Additionally, this combination increased resistance to chloride and sulfate, which are critical durability concerns for infrastructure subjected to harsh environmental conditions.

In asphalt and bituminous composites, the incorporation of metallic waste—such as steel wool fibers and shavings—has been extensively explored for microwave-induced self-healing. Research conducted by Lou et al. [2] and Norambuena-Contreras et al. [5] demonstrates that adding metallic waste enhances thermal conductivity and promotes more effective microwave absorption. This leads to accelerated crack closure and

improved fatigue resistance. However, the outcomes are influenced by the morphology of the waste materials. Long fibers typically enhance conductivity, whereas crushed shavings may diminish thermal diffusivity and result in inhomogeneous internal structures [4]. Recent research indicates that incorporating small quantities of iron powders or aluminum chips can significantly enhance the fracture energy of hot-mix asphalt. Serin et al. [7] found that the inclusion of up to 1.5% of these materials led to notable improvements in crack resistance indices (CRI), flexibility indices (FI), and the energy dissipated during Semi-Circular Bend (SCB) testing. However, it was observed that the advantages of aluminum chips tended to decrease at higher inclusion ratios. Furthermore, innovative techniques such as electrocoagulation and utilizing metallic waste electrodes for wastewater treatment have emerged. Bani-Melhem et al. [1] validated that scrap iron and aluminum can effectively function as sacrificial anodes, achieving substantial pollutant removal efficiency.

This study aims to examine the potential of waste metallic powders as fillers/aggregates in cement mortars, with an emphasis on their physical, mechanical, and electrical properties.

2. MATERIALS AND METHODS

CEN standard sand conforming to EN 196-1 standard and having a density of 2.56 g/cm3 was used as aggregate in the preparation of mortar mixtures. The granulometry of the standard sand is given in Table 1. Potable tap water was used as mixing and curing water in this study. The pH value of the water is 7.76, the Cl value is 8.7 mg/L, the sulfate value is 10.2 mg/L, and the total hardness value is 11.5 mg/L.

Sieve Size (mm)	2.00	1.60	1.00	0.50	0.16	0.08
Passing (%)	0	7±2	33±2	67±2	87±2	99±1

Table 1. Granulometry of CEN standard sand

This study used waste metallic powders (MPs) analyzed by Sarı et al. [6], sourced from a local industrial scaffolding facility. The majority of the powders are spherical, with diameters from 100 to 500 microns, while non-spherical particles are smaller than 100 microns. EDS analysis shows the material is composed of 90.4% iron, 7.95% carbon, and 1.7% aluminum.

Specimen Code	Mix Proportions (g)			
	Cement	MP	Sand	Water
R (Reference)	450	0	1350	225
MP_0.25		3.37	1346.63	
MP_0.50		6.75	1343.25	
MP_1.00		13.5	1336.5	
MP_2.00		27	1323	
MP_4.00		54	1296	
MP_8.00		108	1242	

Table 2. Mix proportions of the series

Mixing is performed using a mechanical mixer, following the procedures and duration specified by the EN 196-1 standard. The process begins by combining the solids (cement, sand, and MP) in the mixing container. Initially, these solids are dry mixed at a low speed of approximately 140 rpm. Water is then gradually added while the mixer operates at a higher speed of 285 rpm. Finally, the mixture is cast into molds measuring 40x40x160 mm³ and compressed using a jolting apparatus. Following a 28-day water curing period, various tests were conducted on the produced samples to assess their density, water absorption, porosity, ultrasonic pulse velocity, capillary water absorption, flexural strength, compressive strength, and electrical resistivity.

3. RESULTS

3.1. Density, Water Absorption and Porosity

The density values of the samples showed a partial increase with the proportion of MP used. This increase can be attributed to the greater specific gravity of MP compared to the sand it replaces, even when replaced by weight. Furthermore, as expected, the water absorption and porosity values of the mortar samples rose with higher MP content. The addition of metallic powder resulted in a slight increase in density, according to Sarı et al. [6], Mahmoud et al. [3] observed a decrease in density from 2.45 g/cm³ to 2.37 g/cm³ as the content of metallic waste increased from 0% to 30% by weight. In contrast, Van Thuong et al. [9] reported an increase in density from 2.103 g/cm³ at 0% waste to 2.256 g/cm³ at 70% metallic waste.

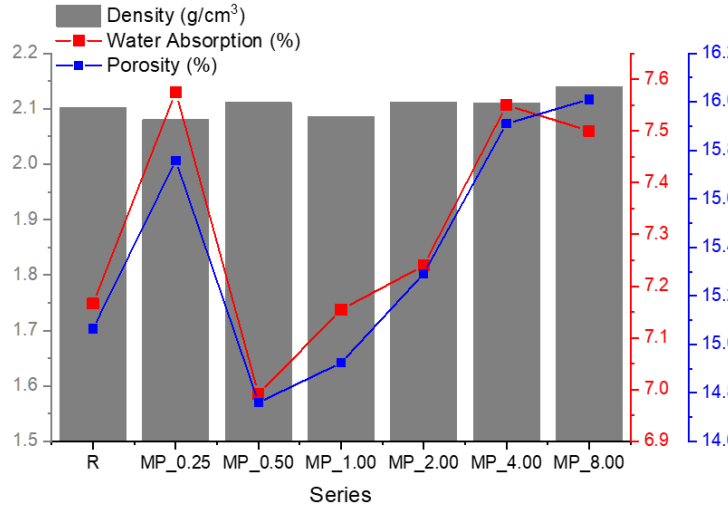


Figure 1. Density, water absorption and porosity values

3.2. Capillary Water Absorption

It is recognized that substituting MP lowers the capillary water absorption coefficient in cement mortars compared to the reference sample, and that metallic powders contribute positively to this outcome. The fine-grained and smooth morphological structure of MP [6] facilitates an advantageous filling effect. This enhancement in the capillary water absorption coefficient is vital for durability, indicating that mortars containing MP substitution can provide significant benefits regarding their service life.

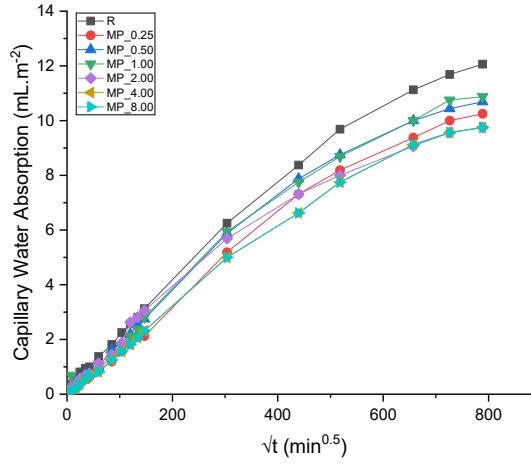


Figure 2. Capillary water absorption values

3.3. Ultrasonic Pulse Velocity

Upon analyzing the Ultrasonic Pulse Velocity (UPV) values, it becomes clear that the non-destructive testing results of the mortar samples, which were produced with metallic powder substitution, closely align with those of the reference concrete. The UPV values for these samples fall within the range of 3.5 to 4.0 km/sec, signifying a classification of "good" quality for the concrete according to the UPV evaluation scale.

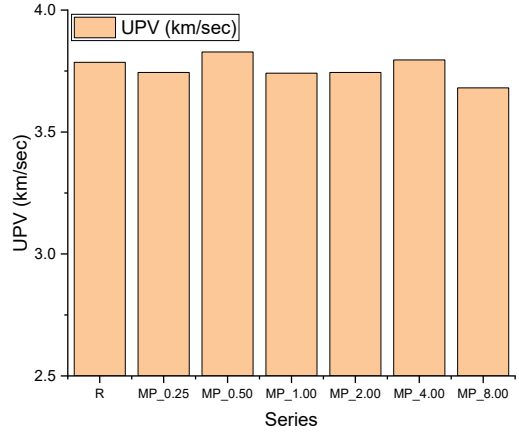


Figure 3. Ultrasonic pulse velocity (UPV) values

3.4. Flexural and Compressive Strength

When the mechanical properties of compressive and flexural strengths were evaluated, the highest flexural strength values were reached in the MP_1.00 and MP_2.00 series. The flexural strength values decreased significantly when the additive ratio exceeded 2.00%. The compressive strength values peaked at 78.67 MPa in the MP_4.00 series, but the flexural strength value was at the lowest level in this series (6.03 MPa). The only series with high compressive and flexural strength values was MP_2.00; in this series, the flexural strength remained at the maximum level of 7.28 MPa, and the compressive strength value remained at a very high level of 77.63 MPa. Therefore, the MP_2.00 series offers optimum performance when considered together in terms of mechanics. This indicates that the MP substitution does not cause a dramatic decrease in the mechanical properties of the samples and can be evaluated within the framework of the circular economy in concrete.

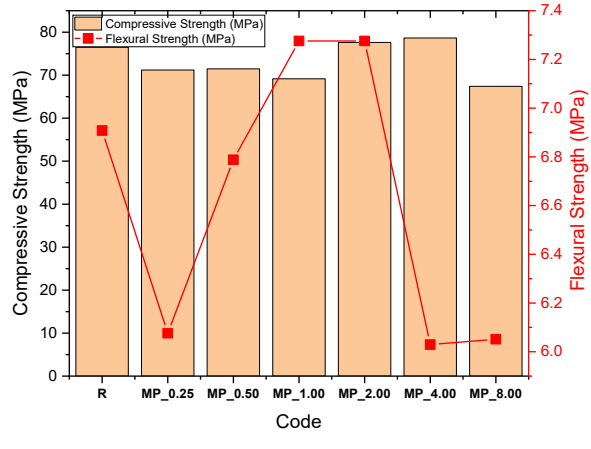


Figure 4. Compressive and flexural strength results

3.5. Electrical Resistivity

Upon reviewing the electrical resistance values obtained from each series on days 7, 14, 21, and 28, we found that resistivity values increased due to the electrical conductivity of the metallic powder at lower doping rates (MP_0.25, MP_0.50, and MP_1.00). This behavior suggests a unique advantage for producing electrically conductive cement mortars, potentially paving the way for developing concrete elements equipped with self-sensing capabilities. On the other hand, analysis of porosity results at higher doping rates of metallic powder revealed a contrasting effect. As the doping rate increased, as MP_2.00 and MP_4.00, the electrical resistance values decreased, which can be attributed to the corresponding rise in porosity. This interplay between doping levels and material properties highlights the complexities in optimizing cement-based composites' performance.

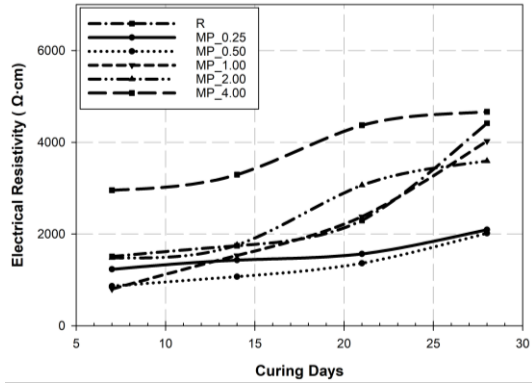


Figure 5. Electrical resistivity results

4. CONCLUSIONS

In this study, metallic powder (MP) was substituted with sand in cement mortars at varying percentages: 0.00% (reference), 0.25%, 0.50%, 1.00%, 2.00%, 4.00%, and 8.00%. After a 28-day curing period, the mortar specimens were subjected to a series of physical, mechanical, and electrical tests. From the findings obtained, the following conclusions can be made.

This study investigates the possible evaluation of waste metallic powders (MP) into cementitious composites in line with sustainable production. The findings show that MP increased material density and reduced water absorption, while maintaining ultrasonic pulse velocity within the "good quality" range. An optimal addition of 2% improved both flexural and compressive strength. Moreover, the doping of small quantities of MP enhanced electrical conductivity. This could pose a new approach for developing functional cement materials, such as self-sensing. However, excessive use of MPs resulted in increased porosity and reduced electrical resistivity. These findings show that recycling metallic waste into cement-based materials could be a solution that aligns with circular economy practices. Future research should assess the long-term performance and applications of MP-doped cement composites.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
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XXIX CONGRESS DIMK AND X CONGRESS SIGP
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MICRO-MECHANICAL AND HYDRATION PROPERTIES OF SUSTAINABLE CONCRETE WITH GROUND GLASS POZZOLAN AS A CEMENT REPLACEMENT

Summary: *This study is part of a broader investigation into sustainable concrete, focusing on the use of ground glass pozzolan (GGP) as a substitute for Portland cement (PC) across microstructure, macrostructure, and field applications. To evaluate performance of GGP comprehensively, six concrete mixtures were designed: three control mixes (CM, FA-30, S-40) and three GGP-based mixes (G-20, G-30, G-40), with mortar and cement pastes following the same cement replacement ratios. Hydration properties of cement pastes up to 100 days were analyzed using quantitative and qualitative methods such as SEM/BSE, TGA and XRD. Findings indicated increased CH consumption in GGP systems, particularly in G-40, confirming its pozzolanic reactivity. Micro-mechanical analysis via nanoindentation showed G-40 exhibited the highest elastic modulus and hardness, indicating enhanced material performance. These findings contribute to a deeper understanding of microstructure development and the beneficial role of GGP in cementitious matrices.*
Keywords: *Sustainable Concrete, Recycled Materials, Ground Glass Pozzolan, Micro-Mechanical Properties, Hydration Properties*

MIKROMEHANIČKA I HIDRATACIONA SVOJSTVA ODRŽIVOG BETONA SA MLEVENIM STAKLENIM PUCOLANOM KAO ZAMENOM ZA CEMENT

Rezime: *Ova studija je deo šireg istraživanja o održivom betonu, fokusirajući se na korišćenje mlevenog staklenog pucolana (GGP) kao zamene za Portland cement (PC) na mikrostrukturnom, makrostrukturnom i terenskom nivou. Da bi se sveobuhvatno procenila performansa GGP-a, dizajnirano je šest betonskih mešavina: tri kontrolne mešavine (CM, FA-30, S-40) i tri GGP mešavine (G-20, G-30, G-40), dok su malteri i cementne paste pratili iste procente zamene cementa. Hidratacione osobine cementnih pasti do 100 dana analizirane su kvantitativnim i kvalitativnim metodama, uključujući SEM/BSE, TGA i XRD. Rezultati su pokazali povećanu potrošnju $Ca(OH)_2$ u sistemima sa GGP-om, posebno kod G-40, što potvrđuje njegovu pucolansku reaktivnost. Mikromehanička analiza putem nanoindentacije pokazala je da G-40 ima najveći elastični modul i tvrdoću, što ukazuje na poboljšane karakteristike materijala. Ova studija doprinosi dubljem razumevanju razvoja mikrostrukture i korisne uloge GGP-a u cementnim matricama.*
Kljucne reci: *Odrzivi Beton, Reciklirani Materijali, Mleveni Stakleni Pucolan, Mikromehanicka Svojstva, Hidrataciona Svojstva*

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1. INTRODUCTION

Around 14 billion cubic meters of concrete per year is produced globally (2 tons per person on the planet), requiring significant natural resources for producing cement and aggregates [1]. Today, besides safety and durability, the sustainability requirements present the biggest challenge for the concrete construction industry. Most significant are high energy consumption and CO_2 emission due to $CaCO_3$ calcination and clinker production [2]. The production of cement, as the binding material in concrete, is energy intensive and has raised environmental concerns since one ton of cement produces one ton of CO_2 (~8 % contribution to total global CO_2 emissions). In the USA, about 90 million tons of cement are used annually, with 3 million tons used in New York (CO_2 emissions equivalent to 300 million cars) [3]. To overcome environmental impacts and produce high-performance concretes (HPC), there is a high interest in using recycled materials. Most contemporary concrete applications for buildings and infrastructure use HPC mixtures, which are produced by replacing a percentage of cement with Supplementary Cementitious Materials (SCMs) consisting mainly of fly-ash (FA) class F or C, slag (S), silica fume (SF) and lime stone fines [4],[5]. The added benefit of SCMs besides improved mechanical and durability properties is production of "Sustainable Concrete" by reducing CO_2.

The proper availability and cost of SCMs are paramount for the concrete industry. In recent years, an availability of FA in the USA has decreased significantly due to an EPA regulations limiting hazardous substances [7]-[9]. On the other hand, slag is relatively expensive and generally produced outside the USA [10]. Hence, there is a need for an alternate SCM to overcome the scarcity of FA particularly in the Northeastern region of the USA, and post-consumer glass can be effectively and economically transformed into value-added pozzolanic material for concrete. In the USA a typical Portland cement replacement with SCMs is at 30%. In New York alone, this potentially represents one million tons per year of ground glass pozzolan (one million tons CO_2 reduction) utilizing 6 billion post-consumer beer bottles and creating a $1USD billion market [3],

Non-recyclable waste glass represents an issue for solid waste disposal in many municipalities. The current practice is still to landfill most of the non-recyclable glass. Since the glass is not biodegradable, landfills do not provide an environmentally-friendly solution. Customary, most non-recyclable mixed-color broken glass comes from discarded bottles, containers, and plates. Consequently, there is a strong need to utilize waste glass. The glass recycling industry faces significant challenge and economic hardship processing glass, particularly mixed color glass which has no real market and is generally used as low-cost granulated filler or drainage material. Glass is heavy (about 20% of total solid waste), harsh on processing equipment, and costly to recycle. Glass represents 6% of about 2 billion tons per year of solid waste worldwide. In the USA, about 12 million tons of post-consumer glass is generated annually (~5% of total municipal solid waste), with only about 26.4% being recycled [11].

At the same time, the shortage and cost of SCMs in the USA is of concern to the concrete industry. Mixed-color waste glass from the bottle and jar industry is an inert material that when milled to micro-level particles does not change its chemical composition and provides favorable pozzolanic reactivity. Thus, the lack of market for recycled glass and declining supply of FA, present a unique and transformative opportunity for benefiting both the glass recycling and the concrete production industries through the development and implementation of GGP for concrete.

Over the last few decades, several attempts have been pursued to incorporate waste glass into concrete mixes. Early applications of glass were in the form of granular material. Glass was crushed and used as recycled coarse and fine aggregates in concrete for non-structural applications, as well as for other construction materials such as paving blocks and architectural tiles. Most studies reported that the replacement of natural aggregates with crushed waste glass of up to 20% increased the compressive strength, and 10% was suggested as most effective [12]. However, there were concerns with the potential negative effect of glass on alkali-silica reaction (ASR), due to high silica content in course glass particles. Recent studies that investigated mechanical, physical and chemical properties of concrete with different particle sizes of glass, showed that glass, when milled to micron-level particle size and used as cement replacement, can not only increase compressive strength but also mitigate ASR despite the high alkali content of glass. It was also reported that recycled glass fiber-reinforced polymers (GFRP) when used as a partial coarse aggregate replacement up to 10 %, reduce compressive and tensile strength, but when used as needles, can substantially increase tensile strength. [13]–[15]. There is limited research on applications of GGP in construction projects, and the three most recent relevant studies include: non-structural concrete for sidewalk and driveway applications at Michigan State University Campus in 2011, and for structural concrete elements used in slabs in Australia in 2006 and in slabs and walls in Canada in 2016 [16].

2. RESEARCH SIGNIFICANCE

This study is part of an extensive research on sustainable concrete with GGP as a replacement for Portland cement at all levels simultaneously, including microstructure, macrostructure and field applications. For the purpose of testing GGP performance on all levels, six concrete mixtures were developed, three as a control (CM, FA-30, S-40), and three with GGP (G-20, G-30 and G-40), and mortar and cement pastes followed the same cement replacements. Therefore, this research provides information at all three levels with correlations for all these phases [17]–[20].

A prediction of concrete performance mainly depends on a good understanding of cement paste behavior. Cement paste consists of several phases, most importantly calcium silicate hydrate (C-S-H), calcium hydroxide (CH), anhydrous cement clinker and pores. This microstructure of cement based materials, such as the hydration products and unreacted particles, their chemical composition, amounts and their distribution, make the fundamental link between the early age and long-term properties (mechanical and durability properties) [21]. Calcium hydroxide (CH) or portlandite, the most soluble hydration product formed during the hydration of cement, is considered as a weak link in Portland cement concrete. When the concrete is constantly exposed to running soft water, the portlandite will leach out (dissolve), the porosity will increase and thus it will make the concrete more vulnerable to further leaching and chemical attack. Eventually, this will lead to reduced service life of concrete structures. In GGP concrete, however, CH is consumed by the pozzolanic reaction of glass to produce hydration products, mainly calcium silicate hydrates (C-S-H). The formation of additional C-S-H in GGP concrete contributes to strength development and the improvement of durability properties. The CH content of paste influences the pozzolanic reaction of GGP, the microstructure development and transport properties

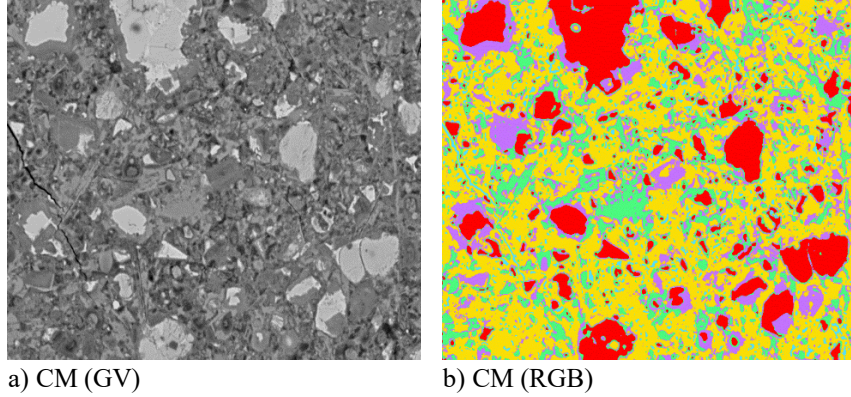
of blended cement systems. The phases of cement paste matrix can be identified and quantified experimentally using different techniques such as Thermogravimetric analysis (TGA), Scanning Electron Microscopy (SEM) and X-ray diffraction (XRD) [22], [23].

The main objective of this study is to determine the multi-phases (hydration products) in the cement matrix, in particular, the CH content of cement pastes blended with GGP in different proportions at ages up to 100 days. It was quantified with SEM and TGA, and verified by qualitative XRD method. Micro- mechanical properties of the phases (average modulus and micro-hardness) were studied by Nano – indentation method [24]. The results obtained in this paper will lead to a better understanding of microstructure development and explain how GGP reacts.

3. EXPERIMENTAL METHODOLOGY

3.1. Scanning Electron Microscopy (SEM/BSE) and Thermogravimetric Analysis (TGA)

Scanning electron microscopy (SEM /BSE / EDX) is one of the most powerful techniques that has been used extensively for studying the chemical and spatial characteristics of cementitious materials. Polished sections were carefully prepared under controlled conditions, and all were observed under SEM. The instrument was operated with accelerating voltage of 15 kV. Cementitious paste samples were only scanned by ESEM in BSE mode in order to acquire images in order to quantify hydration products. The acquired 16-bit BSE images were converted to 8-bit images, and based on the range of gray value (GV) between 0 (black) and 256 (white), they were segmented by using the plugin from ImageJ [25], Trainable Weka Segmentation (TWS) [26] to distinguish between anhydrous phases, portlandite, low and high density CSH, GGP particles, and pores (Figure 1). Since TGA is able to identify X-ray amorphous hydrates, such as C-S-H, it was used complementarily to XRD or SEM/BSE/EDS techniques. Minerals and hydrates can undergo several thermal reactions: dehydration, dehydroxylation, decarbonation, oxidation, decomposition, phase transition or melting, Every mineral or phase is characteristic for a certain temperature region. [23]



a) CM (GV) b) CM (RGB)

Figure 1. SEM/BSE micrograph 500x magnification of pastes at 30 days a) CM (GV), b) segmented phases of CM paste

3.2. X-ray diffraction (XRD)

Samples for XRD were prepared the same way as for TGA with a special care and stored in the desiccator. Right before the testing samples were grounded into fine powder with pestle and mortar, and carefully placed in the aluminum sample holder. The XRD qualitative analysis was used to identify some of the main hydration products (calcium hydroxide and carbonates) on a Philips PW1830/40 powder diffractometer. The machine is operated with an accelerating voltage of 40 kV and an X-ray beam current of 30 mA. The X-ray source is Cu-tube working with characteristic Cu Kα wavelength of 0.15418 nm. Machine is equipped with NI- filter, and analysis were performed in air on cement paste powders with a step size of 0.02° for a 2θ angle range between 5° and 70°. Every step was analyzed for 10 seconds, and the results are presented in Figure 2. Powder samples weighed 3-5 g, and they were tested at 100 days, and results summarized.

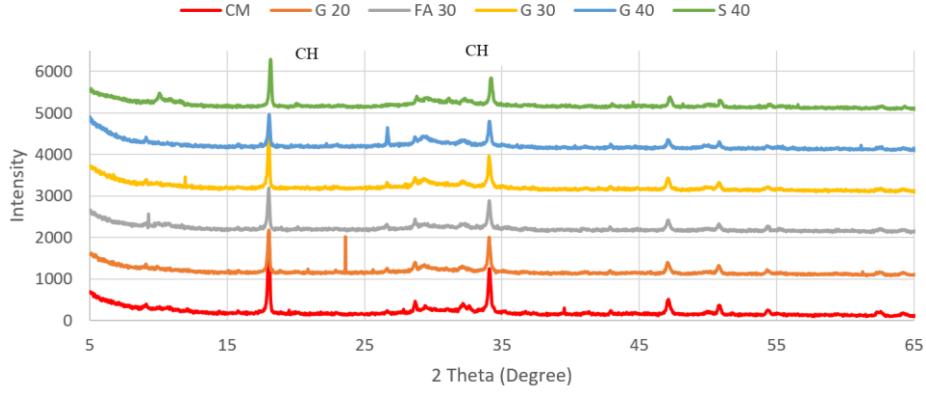


Figure 2. XRD plots for cement pastes at 100 days

3.3. Nanoindentation

At the macroscale, the mechanical and durability properties of concrete are all largely governed by its structural characteristics and material properties at the micro/nanoscale where the deterioration and failure process start. Cement paste and rock materials have traditionally been treated as homogeneous with uniform mechanical properties since mechanical characterization of such materials has been possible only at a macroscale. It is not until recent years that progress and improved availability of advanced instruments and characterization techniques, such as depth-sensing micro/nanoindentation, has made it possible to study nano or micro- mechanical properties of various materials, mostly cementitious systems [27]–[30]. For multiphase materials, a statistical indentation method can be used, by analyzing a large number of indents within a representative sample area [28]. It consists of indenting a sample surface with a diamond indenter, followed by a loading-unloading cycle while continuously recording the load and indentation depth. From the load-displacement slope and calibrated contact area, E_m of the material at the microscale can be determined. Nanoindentation testing was conducted using Agilent Nano Indenter G200 (Keysight, Santa Rosa, CA, USA), equipped with a diamond Berkovich indenter tip.

Nanoindentation was used to examine Es, for cement pastes (binary systems) at 100 days. Sample preparation was performed just prior to testing to avoid carbonation of the tested surface. The tests were set in such a way that the loading was applied when the indenter comes into the contact with the surface of the sample until a specified maximum penetration depth is reached. Afterwards, the load was maintained for 10 s followed by the unloading cycle. A typical example of an indentation curve (load-displacement) is shown in Figure 3. Quartz standard was indented before the test to ensure accuracy and six specimens were tested in total. The indentation depth was 1000 nm. The measurements were obtained from a matrix of 10 x 10 indents, with a distance of 20 μm between indents. This makes 100 indents in total covering an area of 32.4 μm ^{2}. The Continuous Stiffness Method (CSM) developed by Oliver and Pharr [31] was used to calculate the reduced modulus of elasticity (E_r), according to the following equation:

$$E_r = \frac{\sqrt{\pi}}{2}\frac{S}{\sqrt{A_c}}$$

where S is the contact stiffness determined from the load indentation depth slope, and A_c is the contact area under load (mm^2), which is the function of the indentation depth. Since the effect of non-rigid indenter also must be taken into account, the modulus of elasticity of the sample, Es (GPa) is calculated from the following equation:

$$\frac{1}{E_r} = \frac{(1-\nu_s)^2}{E_s} + \frac{(1-\nu_i)^2}{E_i}$$

where ν_s is the Poisson's ratio of the sample, which was assumed to be 0.2 for all measurements. E_i is the modulus of elasticity of the diamond indenter, and ν_i is the Poisson's ratio of the diamond indenter (E_i = 1141 GPa, and ν_i=0.07).
This method enables that E_s is measured as a continuous function of the indentation depth, and not just at the point of initial unloading. Therefore, for each indent, hardness (H) and indentation modulus (E_s) are obtained as a continuous function of surface penetration depth (h).

$$H = P/A_c$$

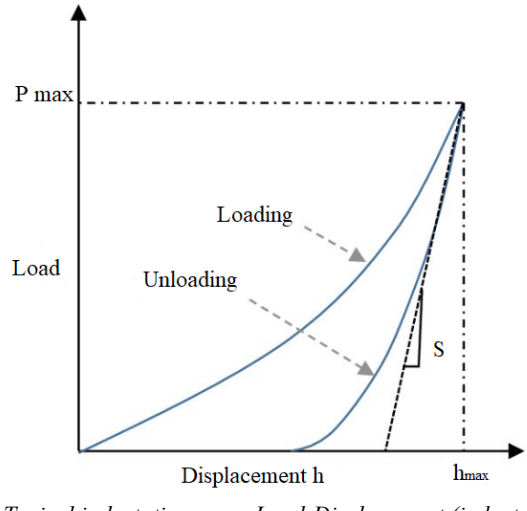


Figure 3. Typical indentation curve Load-Displacement (indentation depth)

4. DISCUSSION OF RESULTS

4.1. Scanning Electron Microscopy (SEM/BSE) and Thermogravimetric Analysis (TGA)

Figure 1a) (CM) shows representative Grey Value (GV) micrographs acquired for studied cement pastes. Different phases in the cement paste matrix show wide range of GV (between 0 and 256), and based on that, the phases were first identified, for instance the unhydrated particles are the brightest, while pores are black (GV = 0). Corresponding segmentations in Red Green Blue (RGB) are shown in (Figure 1b). From the segmented micrographs, quantitative analysis of phases was performed. Phases in CM paste were: Unhydrated particles (red), Inner C-S-H products (purple), outer C-S-H products (yellow), CH (green), and pores (light blue). The proposed quantitative analysis for CM paste works good for all acquired micrographs, however the method for G-20 requires modifications for each micrograph, due to the edge of glass particles that has brighter color (higher GV) than the rest of the particle. It is perhaps an art effect form the SEM, since all micrographs for cement pastes with SCMs have it, (FA, S, and GGP). The GV at the edge of a glass particle falls into two other phases, (inner C-S-H and portlandite). However, after certain modifications (eroding, dilating, subtraction and addition), the proposed method for quantifying of phases works. One of the downsides using BSE for quantitative analysis in 2D, is that is time consuming due to great amount of work which is discussed by Scrivener, however the technique still remains useful tool for quantitative comparison [23]. The method could be improved by coupling with EDS and mapping the phases based on chemical composition. Quantification of phases in FA-30 cement paste cannot be done at all without elemental mapping due to wide variety of GV and chemical composition of the FA particles.

Regarding CH content, G-20 showed 21.4% less portlandite than CM paste. In regards to Inner C-S-H (high density C-S-H) in G-20, increase of 52.9 % was reported, while for outer C-S-H was reduced for 22.9 % in comparison to CM. G-20 showed 22% less unhydrated clinker particles than CM (Figure 4). Although G-20 contains (80% PC and 20% GGP) meaning there is 20% less PC to produce portlandite, it should be bared in mind that the CM replacement is in weight percentage, while quantification of phases is in volume percent. Results confirm that presence of GGP in cementitious matrix modifies the microstructure of composites, and perhaps would be more obvious in higher percentage cement replacements.

The temperature range for calculating CH content by using TGA, is considered to be from 430 ˚C to 550 ˚C and the amount present is shown in Figure 5. As expected CM has the highest amount of CH (`24 %) because it has the highest amount of cement. Pastes with GGP (G-20, G-30, and G-40) show a decreasing trend in CH with increase of cement replacement by GGP (~21 %, 17%, and 14 % respectively). When G-30 is compared with FA-30 (~20 %) and G-40 with S-40 (~16 %), pastes with GGP showed less portlandite than their comparable references.

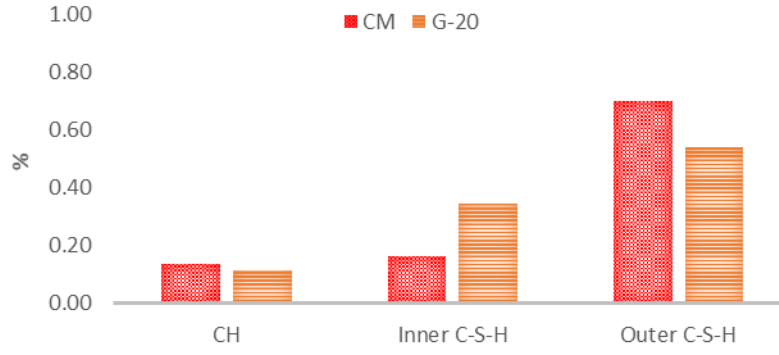


Figure 4. Volumetric fractions of portlandite (CH), high density CSH (inner CSH) and low density CSH (outer CSH) in cement pastes

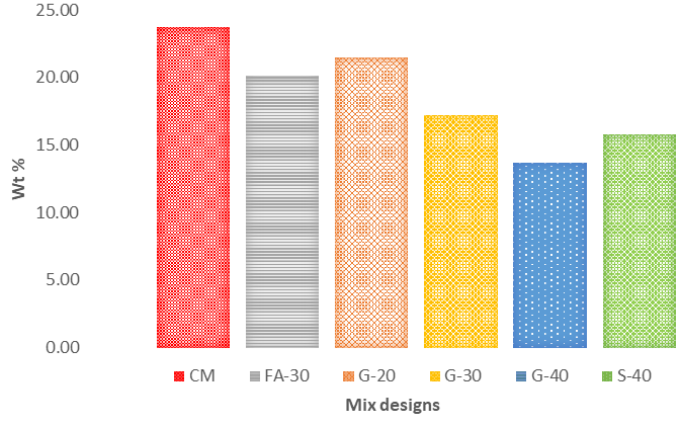


Figure 5. Calcium Hydroxide (Wt %) for all pastes at 100 days

4.2. X-ray Diffraction (XRD) results

Although this method was used in qualitative purposes to identify the crystalline phases in cement pastes up to 100 days, Portlandite (CH) phase is clearly identified at around 18° and 34° 2θ degree. Even though C-S-H phases are amorphous and cannot be captured by XRD technique, they still show a range between 20° and 35°. In Figure 2, all six cement pastes are examined. This technique clearly demonstrates that intensity of the peaks reduces as the cement percent replacement increases. With time the peaks tend to weaken, as hydration and pozzolanic reaction progress. It can be seen from XRD patterns that G-40 has smaller CH peaks than S-40, and G-30 has more reduced CH peaks than FA-30. Pastes with GGP among themselves also show a trend in CH reduction. The XRD results directly reveal that CH is consumed to form an additional C-S-H in pastes with GGP, and that is in an agreement with earlier findings [32]. At

100 days, peaks corresponding to CH can be obviously seen in all pastes, and that indicates that CH remains even after 100 days of pozzolanic reaction with glass powder.

4.3. Nano indentation

The area of indents is visualized by the optical microscope for the reference sample. The mixture has the grey/brownish background which is the gel phase. The brightest uneven shaped particles are the unreacted clinker particles. The intermixing of the phases is expected during the measurements due to inherent heterogeneous nature of the studied materials. The advantage of nanoindentation method is that E_s can be locally tested, which otherwise could not be possible with other techniques. In Figure 6 results are summarized for Average elastic modulus of phases for all pastes at 100 days. In this study, three intervals of E_s were defined and linked to the E_s value of four features in the samples; (1.) pores + low density C-S-H, ($0<E_s < 20$ GPa, (2.) high density C-S-H and some CH crystals, $21 < E_s < 45$ GPa, (3.) unreacted particles $E_s > 46$ GPa. The definition of the three intervals defined in this work is based on the results from the literature [33], [34].

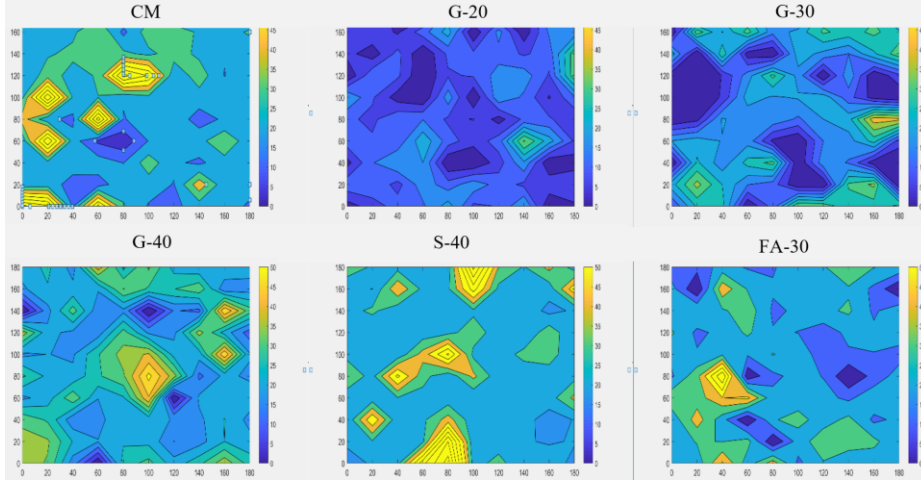


Figure 6. Average modulus of elasticity at 100 days

It was reported in earlier research that elastic modulus of C-S-H, and decalcified C-S-H, ranging between 19.5 and 31.8 GPa, 2.2 and 13.2 GPa, respectively [27]- [32]. In this study, S-40 and G-40 showed denser structure (more high density C-S-H). G-20 and G-30 showed more low density C-S-H and pores. G-30 and G-40 are comparable to their references FA-30 and S-40 (see Figure 6). In Figure 6, E_s is plotted in order to show the location of the phases in the matrix. The highest value of Es is presented in yellow (unhydrated particles), while dark blue represents the pores. In Figure 7, values for hardness were plotted. They are too small in comparison to Es.

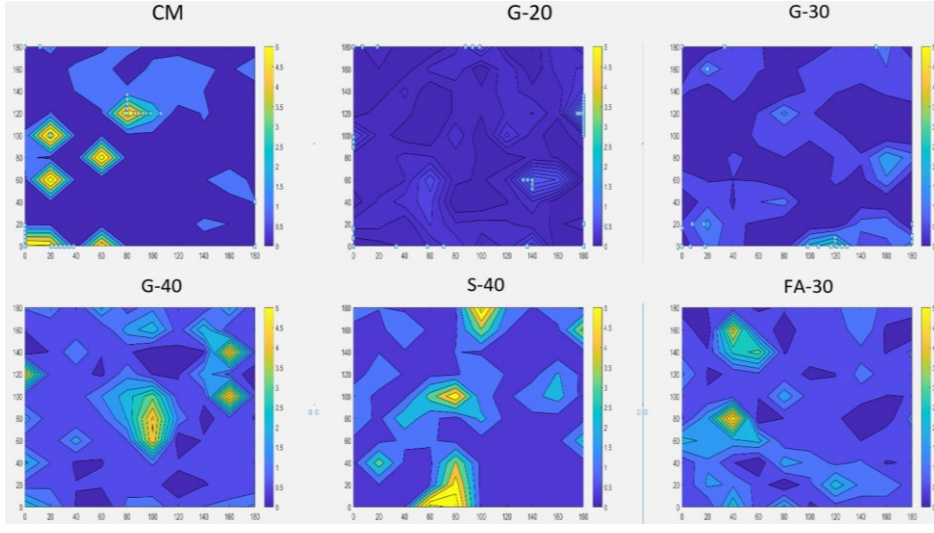


Figure 7. Average hardness at 100 days

5. CONCLUSION

In this study hydration phases were investigated by using SEM/BSE, TGA, and XRD. Quantification and identification of hydration products was performed on cement pastes up to 100 days. Micromechanical properties were also observed with Nano-indentation method.

Calcium hydroxide (CH) was quantified by SEM/BSE and segmentation and TGA. Even though binary systems had less cement for creating CH, when G-30 was compared to FA-30 and G-40 to S-40, results showed that CH was more consumed in the systems when adding more GGP for C-S-H formation. G-40 showed the least portlandite present at all times. That clearly demonstrates the pozzolanic reactivity of GGP with time. Patterns from XRD method justify the CH consumption showed by TGA method. Although the XRD method was qualitative, it clearly shows less CH in pastes with GGP. It can be confirmed that GGP at early age acts as an inert filler, while after 28 days it starts to act as a pozzolanic material by consuming more CH.

Micro mechanical properties are equally important as macro mechanical properties. Micro mechanical properties of cement pastes with GGP are similar to those with FA and S. Among all mixtures with GGP, G-40 showed the highest average modulus and hardness and G-20 the lowest.

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RADOVI

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Sokobanja, May 21 - 23, 2025.

Lena Mihova [1]

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Originalni naučni rad

COMPUTATIONAL MODELS AND ANALYSES OF PILE RETAINING WALLS SUBJECTED TO BASIC AND SEISMIC LOAD COMBINATIONS

Summary: *The paper presents alternative FE models for analysis of retaining walls under static and dynamic (earthquake) actions. Classical 1D beam-models as well as 2D models of the wall-ground interaction are presented. The FE discretization and boundary conditions features are pointed out when performing time history seismic analysis using accelerograms. The main parameters of accelerograms and criteria for selection of appropriate accelerograms are described. Classical and advanced constitutive models are used to describe the mechanical behavior of the soil. Case studies from engineering practice are discussed and comparative analyses of numerical solutions are performed. Computational models of pile walls for strengthening of deep costruction excavations in the city of Sofia are presented.*
Keywords: *pile retaining wall, accelerogram, soil constitutive models, FE.*

RAČUNSKI MODELI I ANALIZE POTPORNIH KONSTRUKCIJA OD ŠIPOVA IZLOŽENIH STATIČKOM I DINAMIČKOM OPTEREĆENJU

Rezime: *U radu su predstavljeni alternativni MKE modeli za analizu potpornih konstrukcija pod statičkim i dinamičkim (zemljotresnim) dejstvima. Prikazani su klasični 1D gredni modeli, kao i 2D modeli interakcije konstrukcije i okolnog tla. Istaknut je značaj MKE diskretizacije i graničnih uslova u sprovođenju seizmičke analize primenom vremenske istorije akcelerograma. Opisani su osnovni parametri akcelerograma i kriterijumi za izbor odgovarajućih akcelerograma. Za opisivanje mehaničkog ponašanja tla primenjeni su klasični i napredni konstitutivni modeli tla. U radu su razmotrene studije slučaja iz inženjerske prakse i sprovedene komparativne analize numeričkih rešenja. Prikazani su računski modeli potpornih konstrukcija od šipova primenjenih za obezbeđenje dubokih iskopa u gradu Sofiji.*
Ključne reči: *potporna konstrukcija od šipova, akcelerogram, konstitutivni modeli tla, MKE.*

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1. INTRODUCTION

The classical theories of active and passive earth pressure according to the theory of limit equilibrium are the most widely applied in the engineering practice for the design of retaining structures. The theories of Rankine and Coulomb define the coefficients of lateral active and passive pressures in the case of non-cohesive soil (φ soil) under static interaction of the retaining wall-ground system. In case of seismic impact, the pseudostatic theory of Mononobe-Okabe [8], [9] is applied, which is an extension of the wedge concept in the Coulomb theory. In the literature, there are numerous solutions based on both the limit equilibrium and limit analysis theories, which develop the classical solutions considering additional factors such as loading on the terrain surface, cohesive soil behind the wall, nonlinear failure mechanism etc. [1], [4], [12], [13], [19]. An overview of pseudostatic solutions for seismic loading of retaining walls is presented in the publication [14]. A major drawback of the pseudostatic solutions is their inability to accurately model the interaction of the wall-ground system, because these solutions use earth pressure diagrams as approach to simulate this interaction. A more precise analysis can be performed using FE models in which both the retaining wall and the soil domain are discretized with FE and appropriate constitutive models for the materials are used.

This paper presents various computational models of pile retaining walls at two construction sites. A comparative analysis between the solutions and an estimation of the applied models are done.

2. PILE WALL FOR SUPPORTING A DEEP CONSTRUCTION EXCAVATION

A commercial building occupying an area of 1760 m^2 is built next to a 16-story residental building and a boulevard with heavy traffic (Figure 1). The 11.5 m deep construction excavation for three basement levels of the building was built in two stages and pile anchor walls were constructed to support the excavation.

Here is considered the pile wall along the side A-A in the plan of the building (Figure 1b). The pile wall is presented in Figure 2.

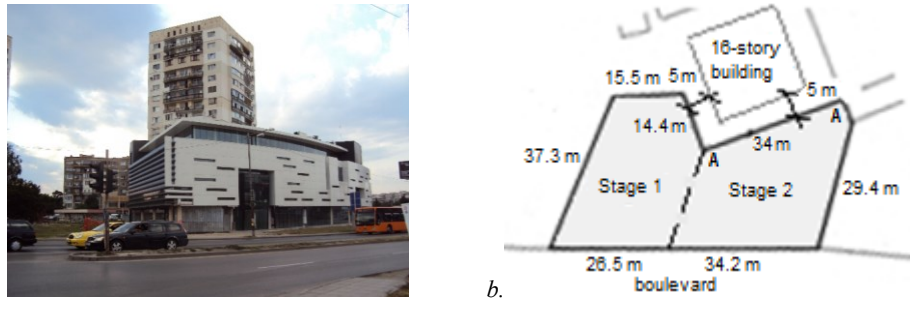


Figure 1. View and plan of the building

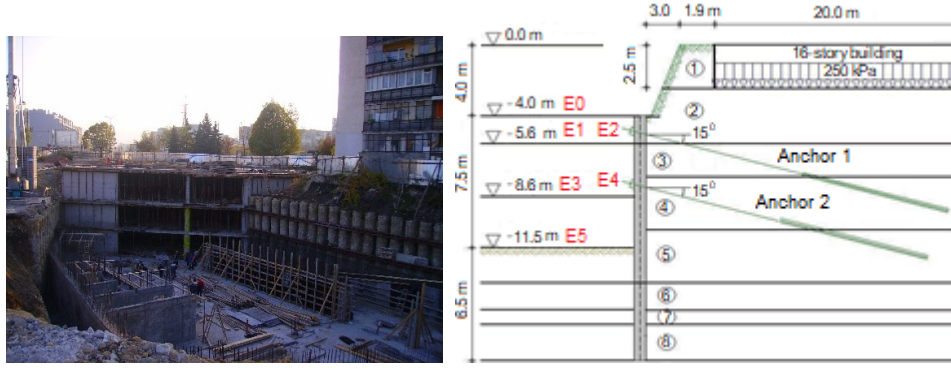


Figure 2. View and cross section of the pile wall along the side A-A in the plan of the building

The pile wall consists of piles 14 m long, 88 cm in diameter. The distance between piles is 1.5 m. The anchoring of the wall is done by installing grouted anchors. The anchors are prestressed with a force of 500 kN. Here a comparative analysis of three computational models of the ground–wall system is performed.

2.1. Computational models of the ground–wall system

A. Classical Model. The impact on the wall is active earth pressure and passive earth pressure which are determined according to the Rankine theory. Fixed horizontal supports are introduced at the anchorage points - at depths of 1.0 m and 4.0 m below the top of the wall. The resultant diagram of active and passive earth pressures to a depth of 4.89 m below the bottom of the excavation are shown in Diagram 3. This depth is obtained from the solution of the equilibrium equations. The bending moments M and shear forces Q in the wall are shown in Diagram 3, too.

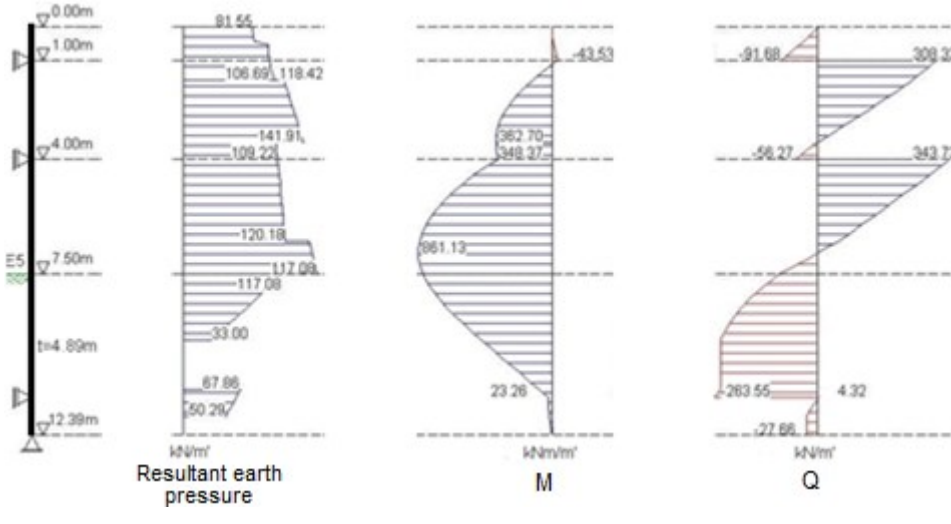


Figure 3. Classical Model: Statical scheme and resultant diagrams

B. Beam-Spring Model (Figure 4**).** The piles are considered as vertical beams elastically supported in the ground along the embedment depth by springs with stiffness, linearly increasing with depth. The anchors were modeled with bars bearing only tensile forces. The prestressed forces in anchors are simulated by introducing axial compressive forces into the bars.

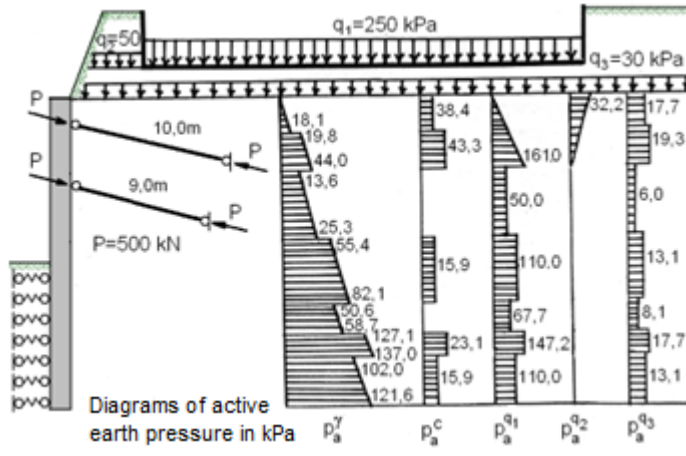


Figure 4. Beam–Spring Model: Statical scheme and resultant diagrams

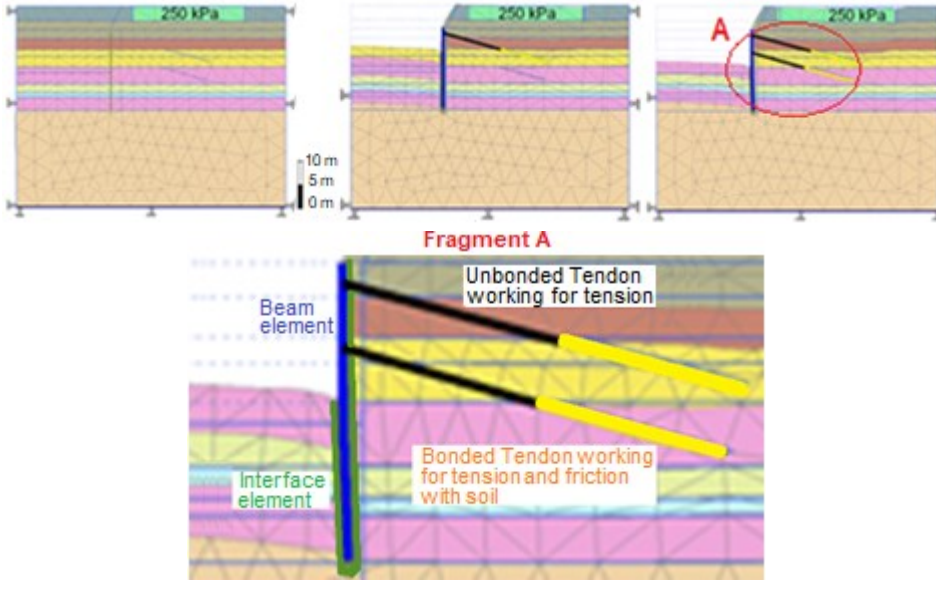


Figure 5. FE Plane Strain Model: In situ phase, E3 phase, E5 phase and Fragment A

C. FE Plane Strain Model (Figure 5**).** In addition to the structural elements (wall and anchors), the model includes a soil domain. The soil is modeled as an elastic-plastic material according to the Mohr-Coulomb constitutive law. The soil–structure interaction

is modeled by introducing an interface defining the friction in the soil–wall contact surface as a part of the soil strength. The anchors are modelled with elements, working for tension and friction with the soil. For simulating the construction phases and service phase of the wall, the solution includes a series of analyses which are marked in Diagram 2 by symbols E_i. The analyses of construction phases are performed using the soil stiffness under unloading behavior due to the excavation works.

2.2. Results and discussion

Model	Phase E_3			Phase E_5			
	Pile		Anchor	Pile		Anchor	
	M_{max} (kNm)	Q_{max} (kN)	A_1 (kN)	M_{max} (kNm)	Q_{max} (kN)	A_1 (kN)	A_2 (kN)
Classical Model	544	462	500	1292	516	500	500
Beam-Spring Model	724	481	564	1136	485	427	623
FE Plane Strain Model	431	378	485	883	434	589	546

Table 1. Results from the analyses

The following conclusions can be drawn from the analyses:

- The internal forces (bending moments and shear forces) in the piles obtained from the FE model are about 20-30% lower compared to the same forces obtained from the other models. The main reason for this difference is the higher precision of the FE analysis;
- More significant differences between solutions are observed in phase E3, that can be explained by the larger wall embedment depth in this phase;
- There is no strict trend in the differences in anchor forces because for anchor behavior influence both the general assumptions of the models and those related to the ability of taking into account the anchor deformability;
- The most conservative results for the internal forces in the piles during the construction of the building basement, i.e. during the service period of the pile wall, are determined from the classical solution. And the most conservative results for the anchors forces in service period of the pile wall are obtained from the FE analysis.

3. FE MODELS FOR SEISMIC ANALYSIS OF BERLIN RETAINING WALL

A Berlin retaining wall, supporting construction excavation in the city of Sofia, (Figure 6) has been analysed by different FE models for seismic impact with a value of design seismic acceleration PHA = 2.3 m/s^2. The wall is braced by strut at a height of 2.5 m above the excavation bottom. The ground is built of clayey and sandy soils. Based on the information from the seismic microzoning of the city of Sofia done in 2001 [10], the fundamental natural period of the soil deposit up to 30 m depth is T_s = 0.75 s (i.e. the fundamental natural frequency is f_s =1.33 Hz). The following computational models and approaches were applied to study the wall seismic response (Figure 7) [7]:

• **Beam–Spring Pseudostatic** model with loading of pseudostatic active earth pressure p_{ae} according to the Mononobe-Okabe theory (Figure 7a);

• **FE Plane–Strain** model of the soil–wall interaction (Figure 7b). This model allows to simulate the construction period of the wall by removing clusters of finite elements from the original model (Figure 8). Seismic impact is included only when the wall construction is completed. Two types of seismic analyses of the wall–ground system have been carried out – Pseudostatic analysis and dynamic (Time history) analysis.



Figure 6. View of Berlin wall supporting the construction excavation

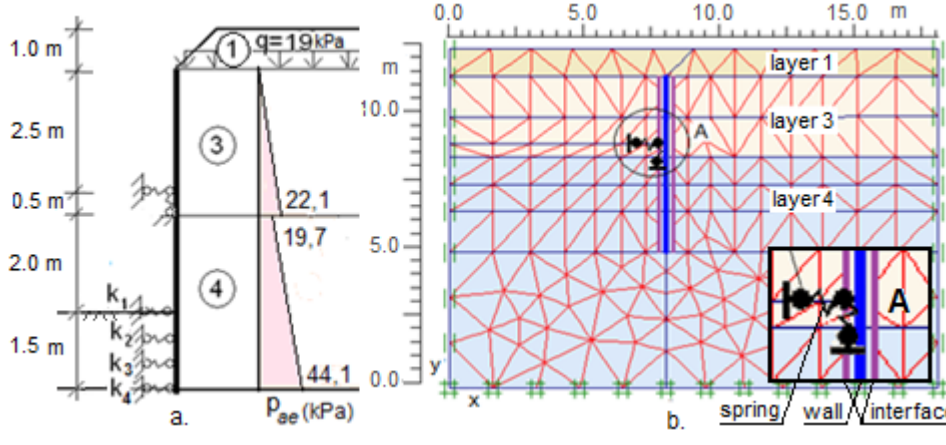


Figure 7. a. Beam–Spring Model; b. FE Plane Strain Model

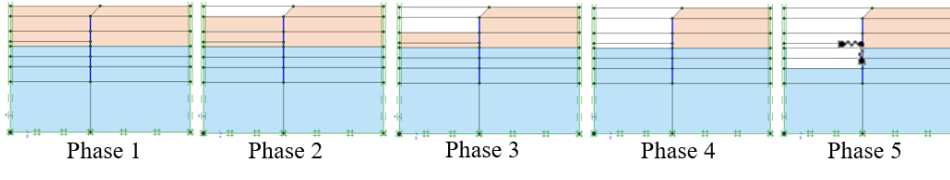


Figure 8. Construction period phases

The pseudostatic seismic loading is defined as horizontal inertial forces which are applied at the finite elements nodes. The inertial force $F_{h,i}$ for each node i is determined using the following formula [4]

$$F_{h,i} = 0.5 \times a \times S \times W_i / g \quad (1)$$

where: $a = 2.3$ m/s^2 $= 0.23g$ is the pseudostatic seismic acceleration for the city of Sofia after the present Bulgarian map of seismic risk; $S = 1.2$ is amplification factor depending on the types of soils in the ground; W_i is the weight of the soil in node i.

The Time history analysis is based on the following assumptions and prerequisites:

- Fulfilling the discretization criteria concerning both the distance between two nodes in the finite element mesh and the interval size in the time domain [6];
- Introducing "dynamic" boundary conditions of the soil domain in the FE model to eliminate seismic waves reflection. *Compliant base boundary* at the bottom of the FE model and *Free field boundary* at the right and left vertical sides of the model are introduced;
- Using Hardening Soil Small (HSS) constitutive model to describe the hysteretic behaviour of the soil and its stiffness reduction under dynamic seismic impact [3];
- Introducing an additional viscous damping according to the Rayleigh theory to correctly account for the hysteretic behaviour of the soil at very small strains [5], [11];
- Simulating the seismic impact by real accelerogram records scaled to the value of *Peak Horizontal Acceleration PHA* = 0.23g.

In general, appropriate accelerograms should be used to perform time history analyses. For this purpose, complex accelerogram selection criteria are applied [16], [18]. These criteria take into account both frequency and intensity characteristics of the seismic signal. Results from seismic analysis of the wall with set of accelerograms are presented in publications [15], [17]. Here, as confirmation for the importance of accurate assessment and selection of accelerograms, the results from two analyses are discussed. The first analysis is carried out using an accelerogram of the earthquake Vrancea (Romania, 1976) and in the second analysis a record of the earthquake Kalamata (Greece, 1986) is used.

Table 2 presents the parameters of the input accelerograms shown in Figure 9, which are applied at the bottom of the FE model and parameters of the accelerograms at the ground surface level obtained from analyses. These parameters are the following:

- Mean Period T_m determined from the accelerogram Fourier Spectrum in the range of frequencies 0.25 Hz $\leq f \leq$ 20.0 Hz by the formula

$$T_m = \frac{\sum_i C_i^2 \left(1/f_i\right)}{\sum_i C_i^2} \text{ at interval for points } i \ \Delta f \leq 0.05\text{Hz} \quad (2)$$

where C_i is the Fourier amplitude at the frequency f_i in point i;

- Predominant Period T_p in the accelerogram Response Spectrum, i.e. the period at which the peak acceleration occurs;
- Pseudo–Spectral Acceleration (PSA) in the accelerogram Response Spectrum at period $T = T_s = 0.75$ s;
- Arias Intensity I_a which is a measure of the seismic impact strength and destructiveness. Arias Intensity is defined by the formula [2]

$$I_a = \frac{\pi}{2g} \int_0^t a^2(\tau) d\tau \quad (3)$$

where t is the accelerogram duration.

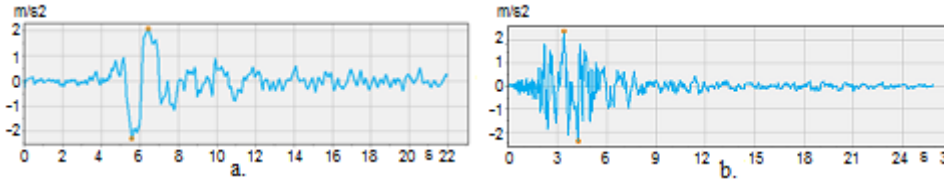


Figure 9. Acceerograms: a. Earthquake Vrancea , 1976; b. Earthquake Kalamata, 1986

Accelerogram of Earthquake	y = 0.0 m (at the bottom)				y = 12.5m (at the top)	
	T_m (s)	T_p (s)	$PSA_{T=Ts}$ (g)	I_a (m/s)	$PSA_{T=Ts}$ (s)	I_a (m/s)
Vrancea (Romania, 1976)	1.75	1.40	0.40	1.6	0.86	4.3
Kalamata (Greece, 1986)	0.69	0.30	0.45	0.7	0.35	1.2

Table 2. Parameters of the used accelerograms

The first criterion for relevance of an accelerogram is the closeness in the values of T_m and T_p to the value T_s = 0.75 s, considering that equality in the values is an indication of a resonance state. As can be seen from Table 2, the Kalamata accelerogram more accurately satisfies this criterion in terms of T_m. But on the other hand, the values of T_p and I_a of the Kalamata accelerogram are much lower compared to the same parameters of the Vrancea accelerogram. More over, as the seismic signal "passes" through the soil deposit in the FE model, the parameters $PSA_{T=Ts}$ and I_a of the Vrancea accelerogram increase about 2-2.5 times, while these parameters of the Kalamata signal change insignificantly. These findings for the data in Table 2 have a significant impact on the results of the time history analyses carried out.

3.1. Results from the models analyses and discussion

Results from the analyses are shown in Tables 3 and 4. It is noticed that the time history analyses determine a stronger reaction of the structure expressed by the bending moments, shear forces and displacements of the wall as well as by the compressive strut force. The most unfavorable response occurs under the seismic loading simulated by the Vrancea accelerogram. As the main reasons for this result, two features of the Vrancea signal are identified. The first feature is the much higher value of the Arias Intensity I_a of the Vrancea accelerogram compared to the Kalamata accelerogram. As can be seen from

Model	Bending moment (kNm)	Shear force (kN)	Strut force (kN)	Horizontal displacement (cm)
Beam-Spring Pseudostatic	45.1	25.0	81.2	1.0
FE Plane-Strain Pseudostatic	48.6	28.4	71.8	2.6
FE Time history – Vrancea, 1976	65.7	38.5	116.2	9.7
FE Time history – Kalamata, 1986	54.4	29.6	96.8	2.4

Table 3. Maximum values of the wall seismic response

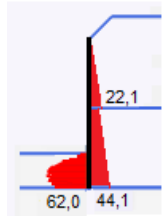
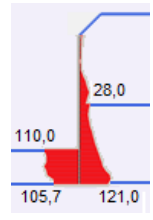
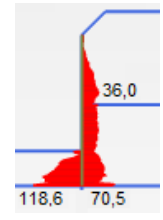
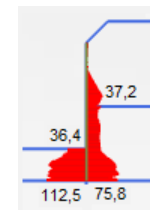
Pseudostatic Beam-Spring	Pseudostatic FE	Time history Vrancea	Time history Kalamata
			

Table 4. Maximum values of the wall active and passive earth pressure

Table 2, the difference in values is up to several times. It is 2 times for the input signals at the bottom of soil domain and 3.5 times for the output signals at the top of the soil domain near the wall. Another feature of the Vrancea accelerogram is the large amplification in Response Spectrum with respect to the Pseudo–Spectral acceleration for period equal to the fundamental period of the soil deposit, i.e. the amplification of $PSA_{T=Ts}$ when the signal "passes" from the bottom of the soil deposit to the top of the soil deposit.

Results in Table 3 show that the Pseudostatic models – Beam–Spring and FE models result in similar response of the structure.

In respect of diagrams of the normal pressures in the soil–wall contact, shown in Table 4, the following is noticeable:

- A gradual increase of active earth pressure is obtained in all analyses. The Mononobe-Okabe formula, used in the Beam-Spring model, determines the lowest value of earth pressure. The value is 2.7 times lower than the value in the Pseudostatic FE model and 1.6 times lower than the value in the FE Time history models.
- The passive earth pressure along the embedment length of the wall has different shapes in computational models, namely parabolic shape in the beam-spring model, uniform shape in the pseudostatic FE model, triangular shape in the time history model using Vrancea accelerogram and trapezoidal shape in the Time history model using Kalamata accelerogram. The Pseudostatic FE model yields the highest value of total passive earth force of 162 kN. The lowest value of total passive earth force of 71 kN is obtained by the Pseudostatic Beam–Spring model.

4. CONCLUSION

The present study shows that the numerical modeling of retaining walls seismic response is a complex task. It requires the consideration of a number of factors regarding the soil-structure interaction under the irregular dynamic seismic loading, as well as the apropriate modeling of the seismic loading itself. The formal contradiction that exact but complex computational models lead to difficulties in their implementation can be overcome with the help of modern computer technologies. This paper proves this statement by applying advanced approaches and concepts in modelling the seismic response of retaining walls. The comparative analysis with classical solutions shows that the latter do not always give results in direction of safety in terms of the geotechnical structures design.

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

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Originalni naučni rad

EKSPERIMENTALNO ISPITIVANJE CLT PANELA SA MODIFIKOVANOM ORJENTACIJOM POPREČNIH SLOJEVA

Rezime: *Eksperimentalno ispitivanje je sprovedeno u cilju utvrđivanja naponskog i deformacijskog stanja panela od unakrsno lepljenog lameliranog drveta (CLT-a) izloženih savijanju van ravni kako bi se analizirali efekti različite orijentacije lamela u okviru poprečnih slojeva panela. Ispitani su paneli sa konvencionalnom orijentacijom lamela poprečnih slojeva pod uglom 90° u odnosu na lamele podužnih slojeva i paneli sa modifikovanom orijentacijom lamela poprečnih slojeva pod uglom ±45° u odnosu na lamele podužnih slojeva. Prednosti modifikovanog CLT-a su poboljšane mehaničke performanse i efikasnije korišćenje resursa uzimajući u obzir svojstva materijala. Ponašanje panela pri ekploatacionom i graničnom opterećenju opisano je sa aspekta veze opterećenje-ugib, oblika loma, nosivosti i krutosti.*
Ključne reči: *CLT, eksperimentalno ispitivanje, modifikovana orjentacija lamela, oblici loma.*

EXPERIMENTAL INVESTIGATION OF CLT PANELS WITH MODIFIED ORIENTATION OF TRANSVERSE LAYERS

Summary: *Experimental investigation was conducted to determine the mechanical performance of cross-laminated timber (CLT) panels subjected to out-of-plane bending in order to evaluete the effects of different orientations of laminations within transverse layers of the panels. Panels with conventional orientation of transverse layers at an angle of 90° in relation to longitudinal layers and panels with modified orientation of transverse layers at an angle of ±45° in relation to longitudinal layers were tested. The advantages of modified CLT panels are improved mechanical performance and more efficient use of resources taking into account material properties. Mechanical behavior of the panels under service and limit loads is described from the aspect of load-deflection relations, failure modes, load capacity, stiffness.*
Key words: *CLT, experimental investigation, modified orientation of laminations, failure modes.*

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1. UVOD

Unakrsno lamelirano drvo (*Cross Laminated Timber* - CLT) je pločasti proizvod koji se dobija lepljenjem unakrsno postavljenih slojeva (redova) drvenih lamela (dasaka) tako da poprečni preseci sadrže neparan broj slojeva. Savremena tehnika proizvodnje, u kombinaciji sa dobrim karakteristikama čvrstoće, čini CLT korisnim građevinskim materijalom sa jedinstvenim svojstvima. S obzirom da je osnovna sirovina drvo, CLT predstavlja ekološki prihvatljiv, reciklažan i obnovljiv građevinski material sa dugim eksploatacionim vekom (kada je pravilno zaštićen od vlage). U odnosu na sopstvenu teženu, CLT ima veću nosivost nego drugi građevinski materijali. Odlična energetska efikasnost i kapacitet čuvanja toplotne energije, dobra zvučna izolacija, dobre performanse pri dejstvu požara i dobra otpornost na dejstvo hemijski agresivnog okruženja su dodatne prednosti CLT panela koje su rezultat masivne drvene konstrukcije. Tehnološki proces proizvodnje i kompozicija unakrsno lameliranog drveta pružaju ogromene mogućnosti budući da se paneli mogu obrađivati na bilo koji oblik i veličinu. Sistem unakrsnih slojeva obezbeđuje nosivost CLT panela na opterećenja u ravni i van ravni, te se oni mogu podjednako uspešno koristiti za horizontalne konstruktivne elemente (međuspratne konstrukcije - ploče) i za vertikalne konstruktivne elemente (zidna platna - dijafragme). Puni poprečni preseci osiguravaju da CLT elementi imaju veliku nosivost i krutost, što ih čini pogodnim za stabilizaciju zgrada većih visina. CLT panele karakteriše visok stepen prefabrikacije, sa mogućnostima brzog i jednostavnog povezivanja. Dodatni potencijal unakrsno lameliranog drveta leži u kombinovanju sa drugim materijalima, kao što su beton, čelik i staklo.

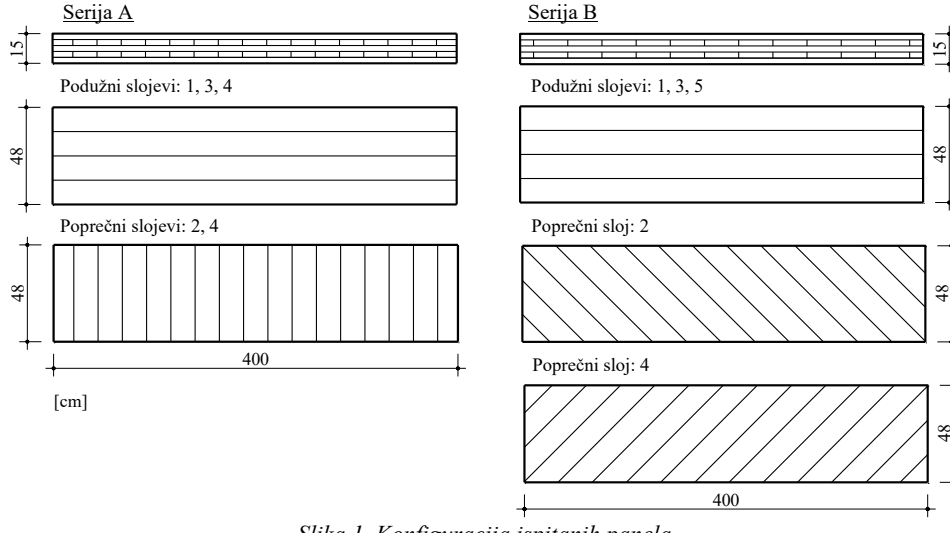
Budući da je CLT materijal sa sve većom primenom u savremenoj građevinskoj praksi, brojna naučna istraživanja su usmerena ka utvrđivanju mehaničkog ponašanja CLT panela za različite vrste naprezanja [1, 12, 15]. Pored analize konvencionalnog proizvoda sa neparnim brojem ortogonalnih slojeva, eksperimentalna i numerička istraživanja su orjentisanja i ka optimizaciji proizvoda kroz promenu orjentacije poprečnih slojeva [2 , 8], varijaciju kompozicije i broja slojeva [6, 7], varijaciju debljina lamela [10, 13] i mogućnost ojačanja panela FRP kompozitima [11]. Kada je reč o modifikaciji orjentacije poprečnih slojeva (postavljanje lamela pod određenim uglom), prednosti modifikovanog CLT-a su poboljšane mehaničke performanse u pravcu nošenja opterećenja (pravac vlakana podužnih slojeva) i efikasnije korišćenje resursa uzimajući u obzir svojstva materijala. Pored toga, prednosti povezane sa postavljanjem lamela poprečnih slojeva pod uglom od recimo 45° umesto 90° u odnosu na lamele podužnih slojeva su i smanjenje varijabilnosti karakteristika unutar panela i povećanje otpornosti na smicanje [2].

U ovom radu je predstavljeno eksperimentalno ispitivanje na savijanje van ravni dve konfiguracije panela od unakrsno lameliranog drveta sa različitom orjentacijom lamela (dasaka) u okviru poprečnih slojeva – konvencionalna sa lamelama poprečnih slojeva pod uglom 90° u odnosu na lamele podužnih slojeva i modifikovana sa orijentacijom lamela poprečnih slojeva pod uglom ±45° u odnosu na lamele podužnih slojeva. Konvencijalni paneli su poslužili kao referentna serija za utvrđivanje efikasnosti predloženog rešenja modifikovanih panela razmatrajući njihovo ponašanje pri ekploatacionom i graničnom opterećenju. Prikazani rezultati predstavljaju deo šireg naučnog istraživanja koje se bavi različitim mogućnostima unapređenja CLT panela sa aspekta graničnih stanja nosivosti i graničnih stanja upotrebljivosti.

2. MATERIJALI I METODE

2.1. Opis eksperimentalnog ispitivanja

Eksperimentalno ispitivanje je sprovedeno u Laboratoriji za ispitivanje konstrukcija Građevinskog fakultetu u Beogradu. Ispitano je ukupno 10 uzoraka panela – 5 uzoraka sa konvencionalnom orijentacijom lamela poprečnih slojeva pod uglom 90° u odnosu na lamele podužnih slojeva (Serija A) i 5 panela sa modifikovanom orijentacijom lamela poprečnih slojeva pod uglom ±45° u odnosu na lamele podužnih slojeva (Serija B). Konfiguracije ispitanih serija su prikazane na slici 1, a izgled uzoraka na slici 2.



Slika 1. Konfiguracija ispitanih panela

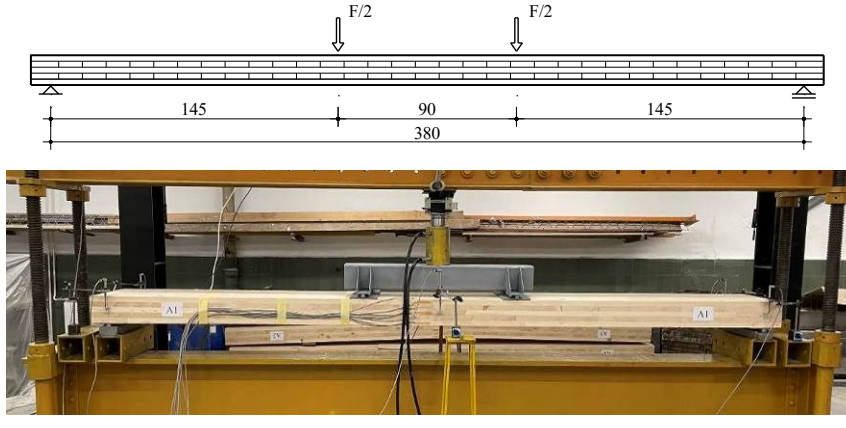
Paneli su proizvedeni u fabrici firme „Kolarević“ iz Pojata i izrađeni su od drveta četinarske građe (smreke) klase čvrstoće C24 prema EN 338 [3] koje je uzeto iz istog šumskog područja u cilju umanjenja uticaja promenljivosti svojstava izazvanih uslovima rasta. Dimenzije ispitanih CLT panela su 48 cm širina × 400 cm dužina × 15 cm debljina. Paneli se sastoje iz pet slojeva, koji su sastavljeni od lamela (dasaka) okvirne širine 12 cm i debljine 3 cm. Podužne lamele su formirane spajanjem dasaka po dužini zupčastom poprečnom lepljenom vezom. Raspored nastavaka u okviru lamela je potpuno proizvoljan. Poprečne lamele nisu sadržale nastavke, već su sve bile iz komada. Poliuretanski lepak je korišćen za zupčaste spojeve i povezivanje podužnih i poprečnih slojeva. Bočne strane susednih lamela u okviru slojeva nisu lepljene. Svi proizvodni parametri bili su u granicama koje zahtevaju CLT standardi. Industrijska CLT proizvodna linija je korišćena za proizvodnju panela, s tim da je prilagođen proces i proizvodna procedura u slučaju panela sa modifikovanim poprečnim slojevima (slika 2).

Svi paneli ispitani su na savijanje kao proste grede raspona 380 cm (približno 25 puta debljina panela) simetrično opterećene sa dve koncentrisane sile na međusobnom rastojanju od 90 cm (6 puta debljina panela), u svemu prema EN 16351 [5]. Ovakvim rasporedom sila, u srednjem delu panela dobijen je konstantan moment savijanja, bez transverzalne sile.



Slika 2. Izgled dostavljenih uzoraka i proces proizvodnje modifikovanih panela

Ispitivanje CLT panela je izvršeno u zatvorenom čeličnom ramu. Opterećenje je aplicirano putem hidraulične prese kapaciteta 350 kN, a transformisano je sa jedne koncentrisane sile na dve sile raspodeljene duž širine panela pomoću čeličnog profila HOP 150x150x5 mm sa zavarenim čeličnim limovima na mestima unosa sile. Slobodno oslanjanje panela omogućeno je primenom valjkastih čeličnih ležajeva koji su korišćeni i na mestima nanošenja sila kako bi se obezbedilo da opterećenje deluje vertikalno. Čelične pločice postavljene su ispod tačaka opterećenja i na osloncima sa ciljem da minimiziraju lokalna oštećenja (gnječenje) drveta. Šematski prikaz dispozicije ispitivanja i fotografija panela u ramu za ispitivanje su dati na slici 3.



Slika 3. Dispozicija ispitivanja panela i izgled panela u ramu za ispitivanje

Nanošenje opterećenja mereno je preko dozne (HBM) nosivosti 2 MN sa tačnošću od 0,05 kN. Ugib panela meren je pomoću induktivnih ugibomera. Ugib u sredini raspona meren je obostrano koristeći dva ugibomera (HBM W100TK, ±100 mm), pozicionirana da omoguće očitavanje u blizini neutralne ose, dok je ugib na osloncima meren obostrano pomoću četiri ugibomera (HBM W20TK, ±20 mm). Svi ugibomeri su tačnosti 0,01 mm. Pored ugiba, u sredini raspona merene su i dilatacije po obimu poprečnog preseka. Podaci o dilatacijama sa mernih traka, o deformacijama sa ugibomera i odgovarajući podaci o opterećenju sa dozne su prikupljeni uz pomoć akvizicionog sistema (HBM MGC). Akvizicija mernih podataka sprovedena je sa frekvencijom od 2 Hz.

Opterećenje je nanošeno kontrolisanom brzinom 12 kN/min, sa ciljem postizanja loma panela za oko 5 minuta. Obe serije CLT panela su ispitane istom procedurom nanošenja opterećenja čime je obezbeđena uporedivost rezultata.

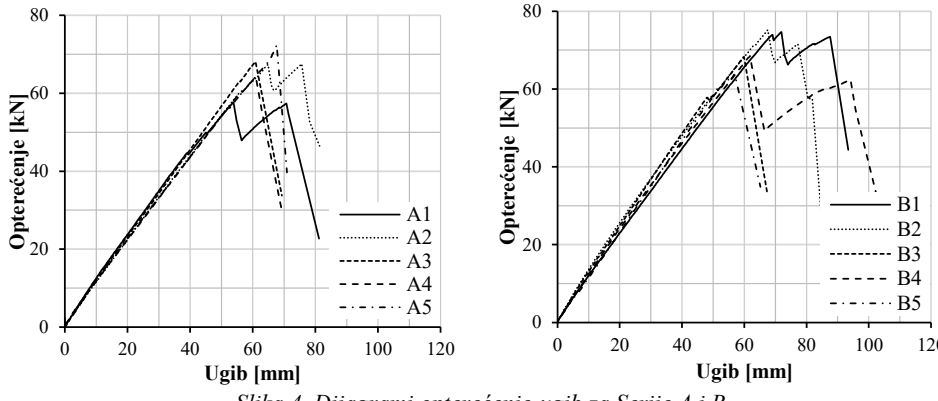
Sve vreme ispitivanja panela, u hali, na mestu neposredno pored rama za ispitivanje, kontrolisana je vlažnost i temperatura vazduha. Vlažnost vazduha izmerena je između 50 i 60%, a temperatura oko 25 °C. Neposredno posle završetka ispitivanja, sadržaj vlage u drvetu meren je pomoću digitalnog vlagomera na različitim lokacijama u okviru panela. Zabeležena vlažnost u svim uzorcima bila je u opsegu 9,8–11,2%.

3. REZULTATI

Ponašanje ispitanih CLT panela do loma je opisano kroz dijagrame opterećenje-ugib, oblike loma, kapacitet nosivosti i deformabilnosti i vrednosti krutosti na savijanje.

3.1. Dijagrami opterećenje-ugib i oblik loma

Dijagrami opterećenje-ugib za sve ispitane CLT panele u okviru Serije A i Serije B dati su na slici 4. Prikazane vrednosti ugiba do loma svakog uzorka predstavljaju srednju vrednost merenja dva induktivna ugibomera postavljena u sredini raspona sa obe strane panela. U okviru eksperimenta, mereni su i ugibi na osloncima u cilju praćenja lokalnih deformacija. S obzirom da su zabeležene izuzetno male vrednosti (približno jednake nuli), ove deformacije su isključene iz dalje analize. Oblici loma svih ispitanih uzoraka su prikazani na slici 5.



Slika 4. Dijagrami opterećenje-ugib za Serije A i B

CLT paneli sa konvencionalnom orijentacijom lamela poprečnih slojeva (Serija A) pokazali su linearno-elastično ponašanje do loma. Kolaps svih uzoraka dogodio se usled loma izazvanog zatezanjem u lamelama spoljašnjeg podužnog sloja. Lom u zategnutoj zoni je praćen izraženim smičućim pukotinama koje se pružaju duž spoja između spoljašnjeg podužnog sloja i susednog poprečnog sloja i/ili kroz poprečni sloj *(rolling shear*). Kao posledica krtog ponašanja drveta pri zatezanju, lom se uglavnom događao iznenada, bez ikakvih naznaka pre dostizanja graničnog opterećenja. Određeni zvučni efekti pucanja vlakana zabeleženi su kod pojedinih uzoraka neposredno pre pojave loma

u lamelama podužnog sloja u zategnutoj zoni. Lom je iniciran na mestima grešaka građe drveta (kvrge) ili na mestima zupčastih spojeva podužnih lamela u oblasti maksimalnog momenta savijanja, između tačaka apliciranja opterećenja. U trenutku kolapsa, kod svih uzoraka je uočeno istovremeno popuštanje više lamela podužnog sloja. Ni u jednom od panela nije bilo tragova plastifikacije u formi zbijanja drvenih vlakana na lamelama podužnog sloja u pritisnutoj zoni.



Slika 5. Oblici loma svih panela Serija A i B

Ponašanje pri savijanju van ravni CLT panela sa modifikovanom orijentacijom lamela poprečnih slojeva (Serija B) je u suštini linearno-elatično do loma, sa mogućnošću pojave određene duktilnosti u fazi neposredno pre konačnog loma. Kao u slučaju konvencionalnih CLT panela, svi uzorci su pokazali donekle sličan mehanizam kolapsa koji je uslovljen čvrstoćom drveta u zategnutoj zoni. Inicijalni lom je izazvan prisustvom defekata i diskontinuiteta drveta u okviru lamela spoljašnjeg podužnog sloja. Pojava inicijalnog loma kod pojedinih uzoraka nije značila trenutni lom, već su paneli nastavili da nose opterećenje još određeno vreme do konačnog loma. Ova pojava se uočava na dijagramu opterećenje-ugib kao nagli pad, a zatim postepeni rast sile. Na taj način, konačan lom ovih uzoraka je bio najavljen. Kapacitet nosivosti, kao i duktilnosti, u fazi nakon inicijalnog loma zavisi od putanje smičućih pukotina. Lamele poprečnih slojeva pod uglom od 45° u odnosu na lamele podužnih slojeva premostile su lokalne defekte u drvetu i na taj način je omogućena preraspodela opterećenja na susedne podužne lamele. Ovaj efekat nije dovoljno izražen zbog koncentracije kvrga u susednim lamelama spoljašnjeg podužnog sloja panela, kao i zbog izostanka lepljenja bočnih strana susednih lamela poprečnih slojeva panela. Tragovi plastifikacije pritisnute zone nisu zabeleženi ni kod jednog od ispitanih uzoraka.

3.2. Kapacitet nosivosti i deformabilnosti

Prikaz eksperimentalnih rezultata u smislu maksimalnog opterećenja i ugiba u sredini pri maksimalnom opterećenju, kao i ugiba u sredini pri konačnom lomu za ispitane serije panela dat je u tabeli 1. Odgovarajuće vrednosti za opterećenja i ugibe su očitane sa eksperimentalno dobijenih dijagrama (slika 4).

Uzorak	Maksimalno opterećenje F_{max} (kN)	Ugib pri max opterećenju w_{odg} (mm)	Ugib pri lomu w_{max} (mm)	Krutost na savijanje EI (kN/mm^2)	Inicijalni oblik loma
A1	57,8	53,8	71,4	$12{,}26 \times 10^8$	zatezanje
A2	67,5	75,6	75,6	$11{,}49 \times 10^8$	zatezanje
A3	68,1	60,9	60,9	$11{,}92 \times 10^8$	zatezanje
A4	64,2	60,8	60,8	$11{,}24 \times 10^8$	zatezanje
A5	72,4	67,7	67,7	$11{,}44 \times 10^8$	zatezanje
Srednja vrednost	66,0	63,8	67,3	$11{,}67 \times 10^8$	-
CV (%)	8,3	12,9	9,7	3,5	-
B1	74,7	71,9	87,6	$11{,}56 \times 10^8$	zatezanje
B2	75,2	67,5	77,2	$12{,}83 \times 10^8$	zatezanje
B3	68,3	59,9	59,9	$12{,}48 \times 10^8$	zatezanje
B4	68,5	61,8	94,2	$12{,}46 \times 10^8$	zatezanje
B5	63,9	56,9	56,9	$12{,}26 \times 10^8$	zatezanje
Srednja vrednost	70,1	63,6	75,2	$12{,}32 \times 10^8$	-
CV (%)	6,8	9,5	21,9	3,8	-
Razlika B/A (%)	6,2	0,7	11,7	5,5	-

CV = koeficijent varijacije

Tabela 1. Rezultati eksperimentalnih ispitivanja

Srednja vrednost graničnog opterećenja za CLT panele sa orijentacijom poprečnih slojeva pod uglom od 90° (Serija A) bila je 66,0 kN, sa koeficijentom varijacije 8,3%. Razlika između najmanje i najveće sile ukazuje na značajnu varijabilnost u čvrstoći drveta. Budući da je drvo prirodni materijal, ovi rezultati su očekivani. Inače, zabeleženi koeficijent varijacije za unakrsno lamelirano drvo je bitno niži nego što je koeficijent varijacije kod lepljenog lameliranog i monolitnog drveta (za čvrstoću na savijanje iznosi 15-25% [9]). Ovo se može objasniti činjenicom da su defekti drveta ravnomernije raspoređeni i da je uticaj svakog pojedinačnog defekta na čvrstoću drveta značajno manji (tzv. efekat lameliranja).

Ispitivanje sprovedeno na CLT panelima sa orijentacijom poprečnih slojeva pod uglom od ±45° (Serija B) pokazalo je srednju vrednost graničnog opterećenja od 70,1 kN,

sa koeficijentom varijacije 6,9%. Kada se izvrši poređenje kapaciteta nosivosti razmatranih konfiguracija panela, kolaps CLT panela sa modifikovanom orijentacijom lamela poprečnih slojeva dogodio se pri većem opterećenju u odnosu na konvencionalne CLT panele. Ako su lamele poprečnih slojeva postavljene pod uglovima manjim od 90°, kao što je 45°, postoji potencijal da se naponi bolje raspodele duž lamela podužnih slojeva, kao i da se umanji varijabilnost njihovih karakteristika. Međutim, zabeleženo povećanje nije opravdalo očekivanja, jer je iznosilo svega 6,2%. Pri tome, zanimljivo je primetiti da su tri uzorka iz Serije B (B3, B4, B5) imala manje granično opterećenje od „najjačeg“ uzorka iz Serije A (A5). Svakako, ovo nije pouzdan pokazatelj efikasnosti primenjene modifikacije jer je lom CLT panela u velikoj meri uslovljen prisustvom defekata i diskontinuiteta drveta.

Generalno, modifikovani CLT paneli (Serija B) pretrpeli su veće deformacije pri konačnom lomu u odnosu na konvencionalne CLT panele (Serija A). Povećanje srednje vrednosti ugiba u sredini panela pri lomu iznosilo je 11,7% (75,2 mm naspram 67,2 mm) Međutim, primetna je velika varijacija vrednosti ugiba pri lomu za uzorke Serije B (21,9%) koja se može objasniti prevremenim lomom i, samim tim, izostankom duktilnog ponašanja uzoraka B3 i B5.

3.3. Krutost na savijenje

Eksperimentalni rezultati krutosti na savijanje za ispitane serije CLT panela dati su u tabeli 1. Reč je o krutosti na savijanje van ravni panela koja je određena na osnovu merenja „globalnog“ ugiba panela. „Globalni“ ugib odražava mehanizam i deformacija savijanja i deformacija smicanja CLT panela. Prema EN 408 [4] krutost na savijanje koja podrazumeva samo mehanizam deformacija savijanja određuje se na osnovu „lokalnog“ ugiba koji predstavlja relativni ugib meren u oblasti između tačaka nanošenja opterećenja (gde je konstantan moment savijanja, bez transverzalne sile). Međutim, vrednosti krutosti izračunate korišćenjem merenja „lokalnih“ deformacija pokazuju veće rasipanje od onih izračunatih iz „globalnih“ deformacija i treba ih razmatrati sa oprezom [4]. Ovo može biti posledica lokalnih varijabilnosti unutar panela koje utiču na „lokalne“ deformacije u većoj meri nego na „globalne“ deformacije. Dati fenomen je u saglasnosti sa istraživanjem [14] koji su pokazali da glavni razlog, za razliku između „globalnih“ i „lokalnih“ vrednosti krutosti, nije smicanje, već varijacija modula elastičnosti unutar uzorka. S obzirom na primenjenu dispoziciju eksperimentalnog ispitivanja u okviru ovog istraživanja (odnos raspona prema debljini panela $l/h = 25{,}3 > 24$ i odnos rastojanja aplicirane sila od bližeg oslonaca prema debljini panela $a/h = 9{,}7 > 8$), uticaj deformacije od transvezalne sile može biti zanemaren i krutost panela se može dovoljno tačno izračunati na osnovu „globalnih“ deformacija.

Krutost na savijanje ispitanih panela je izračunata na osnovu nagiba krivih opterećenje-ugib, za linearno-elastičnu oblast ponašanja između $0{,}1 \cdot F_{max}$ i $0{,}4 \cdot F_{max}$ (F_{max} - maksimalno opterećenje) prema sledećem izrazu:

$$EI_{global} = \frac{3 \cdot a \cdot l^2 - 4 \cdot a^3}{48 \cdot \left(\frac{w_2 - w_1}{F_2 - F_1} \right)} \tag{1}$$

gde je:

EI_{global} „globalna“ krutost na savijanje ;
F_1 sila koja odgovara vrednosti 10% maksimalne sile ($0{,}1 \cdot F_{max}$);
F_2 sila koja odgovara vrednosti 40% maksimalne sile ($0{,}4 \cdot F_{max}$);
w_1 ugib u sredini uzorka meren za vrednost sile F_1;
w_2 ugib u sredini uzorka meren za vrednost sile F_2;
l rastojanje oslonaca uzorka (raspon panela);
a rastojanje između mesta nanošenja opterećenja i bližeg oslonca.

Srednja vrednost krutosti na savijanje za CLT panele sa orijentacijom poprečnih slojeva pod uglom od 90° (Serija A) bila je $11{,}67 \times 10^8$ kN/mm^2, sa koeficijentom varijacije 3,5%. Ispitivanje sprovedeno na CLT panelima sa orijentacijom poprečnih slojeva pod uglom od ±45° (Serija B) pokazalo je srednju vrednost krutosti na savijanje od $12{,}32 \times 10^8$ kN/mm^2, sa koeficijentom varijacije 3,8%. Nizak koeficijet varijacije rezultata zabeležen kod obe serije ukazuje na ujednačen kvalitet drvene građe koja je korišćena za proizvodnju CLT panela. Postavljanje lamela poprečnih slojeva pod uglom od 45° umesto 90° u odnosu na lamele podužnih slojeva dovelo je do povećanja krutosti na savijanje panela od 5,5%. Budući da je kod drveta modul elastičnosti upravno na vlakna značajno manji u odnosu na modul elastičnosti paralelno vlaknima ($E_{90,mean} \approx 1/30 \cdot E_{0,mean}$), može se konstatovati da je zabeleženo povećanje rezultat većeg doprinosa poprečnih slojeva krutosti na savijanje. Međutim, ovaj efekat nije dovoljno izražen zbog položaja poprečnih slojeva unutar poprečnog preseka panela (položaj težišta poprečnih slojeva je relativno blizu u odnosu na neutralnu osu preseka). Osim toga, određeni uticaj ima i to što nije predviđeno lepljenje bočnih strana susednih lamela poprečnih slojeva.

4. ZAKLJUČAK

Na osnovu sprovedenih eksperimentalnih ispitivanja dve serije petoslojnih CLT panela sa različitom orijentacijom lamela (dasaka) u okviru poprečnih slojeva izloženih savijanju van ravni mogu se izvesti sledeći zaključci:

- Ponašanje CLT panela do loma je linearno-elastično. Određeni stepen duktilnosti u fazi neposredno pre konačnog loma moguć je kod panela sa modifikovanom orijentacijom lamela poprečnih slojeva.
- Kolaps CLT panela bez obzira na orijentaciju lamela u okviru poprečnih slojeva događa se usled loma u zategnutoj zoni. Lom je iniciran na mestima grešaka građe drveta (kvrge) ili na mestima zupčastih spojeva podužnih lamela u oblasti maksimalnog momenta savijanja.
- Ne treba zanemariti važnost kontrole kvaliteta, posebno za klasiranje drvene građe, grupisanje defekata drveta u okviru susednih lamela pojedinačnih slojeva panela, izradu zupčastih spojeva i njihovo razmeštanje, sprovođenje procesa lepljenja slojeva panela. Neispunjavanje minimalnih zahteva za proizvodnju može dovesti do prevremenog loma CLT panela.
- Postavljanje lamela poprečnih slojeva pod uglom od 45° umesto 90° u odnosu na lamele podužnih slojeva vodi ka poboljšanju nosivosti i krutosti na savijanje panala. Zabeleženo povećanje graničnog opterećenja od 6,2% i krutosti od 5,5% nije opravdalo očekivanja. Uzimajući u obzir udaljenje od neutralne ose preseka, doprinos poprečnih slojeva nosivosti i krutosti panela bio bi veći kod panela sa većim brojem slojeva (sedam i više).

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
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Novica Staletović[1], Violeta Nikolić[2], Robert Rekecki[3] https://doi.org/29DIMK.082S
Stručni rad

ZNAČAJ PRIMENE STANDARDA ZA QMS, EMS, OHSMS, EnMS I ISHODOVANJE INTEGRISANE DOZVOLE U SEKTORU KERAMIČKIH PROIZVODA

Rezime: *U radu se ukazuje na značaj primene standarda za QMS, EMS, OHSMS i EnMS u ishodovanju integrisane dozvole za sektore na koje se odnosi Zakon o integrisanom sprečavanju i kontroli zagađenja životne sredine. Poslovanje usaglašeno sa međunarodno priznatim standardima ISO 9001:2015, ISO 14001:2015, ISO 45001:2018 i ISO 50001:2018 i zakonskim zahtevima je od ogromnog značaja za održivi razvoj kompanije. Ukoliko se to postavi u kontekst upravljanja promenama na bazi analize rizika i prilika po kvalitet, životnu sredinu, bezbednost na radu i uštedu energije onda se potencira na preventivnost, efikasnost i efektivnost poslovanja što omogućava poslovnim sistemima, da dostignu koncept održivog razvoja.*
Ključne reči: *standardi, integrisana dozvola, QMS, EMS, OHSMS i EnMS, IPPC*

THE IMPORTANCE OF THE APPLICATION OF STANDARDS FOR QMS, EMS, OHSMS, EnMS AND THE OUTCOME OF AN INTEGRATED LICENSE IN CERAMIC PRODUCTS SECTOR

Summary: *The paper points out the importance of applying the standards for QMS, EMS, OHSMS and EnMS in obtaining an integrated permit for sectors covered by the Law on Integrated Prevention and Control of Environmental Pollution. Business compliant with internationally recognized standards ISO 9001:2015, ISO 14001:2015, ISO 45001:2018 and ISO 50001:2019 and legal requirements is of great importance for the sustainable development of the company. If it is placed in the context of change management based on the analysis of risks and opportunities for quality, the environment, safety at work and energy saving, then it emphasizes the efficiency and effectiveness of business, which enables business systems to reach the concept of sustainable development.*
Keywords: *standards, integrated permit, QMS, EMS, OHSMS and EnMS, IPPC*

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1. UVOD

U nacionalnoj i međunarodnoj tržišnoj utakmici primena standarda sistema menadžmenta postaje sve značajniji, a ponegde i presudni faktor poslovnog uspeha.

Zahtevi, odnosno zadaci, koji se danas postavljaju pred privredne subjekte mogu se svrstati u više grupa: ekonomika, kvalitet proizvoda/usluga, zaštita zdravlja i bezbednost na radu, zaštita životne sredine, energetska efikasnost i društvena odgovornost.

Napred navedeni zahtevi mogu se ispuniti samo ako su potpuno usaglašeni sa zakonskom regulativom i zahtevima međunarodno priznatih standarda SRPS ISO 9001:2015, SRPS ISO 14001:2015, SRPS ISO 45001:2018 i SRPS ISO 50001:2018 [6-9]. Da bi se ostvario koncept održivog razvoja i ispunili ovi zahtevi određeni broj postrojenja iz sektora industrije minerala (postrojenja za proizvodnju keramičkih proizvoda pečenjem, a naročito crepa, cigle, vatrostalne opeke, pločica, keramičkog posuđa ili porcelana) [10], u obavezi je da ispuni zakonski zahtev ishodovanja i/ili produženja važenja integrisane dozvole.

Kada je u pitanju ishodovanje i produženje važenja integrisane dozvole, posedovanje sertifikata za QMS, EMS, OHSMS, EnMS nije poseban uslov, ali su ove dve premise, u vrlo tesnoj i funkcionalnoj vezi. Organizacije koje su implementirale napred navedene standarde sistema menadžmenta i ispunile uslove za dobijanje sertifikata QMS, EMS, OHSAS i EnMS, sasvim je sigurno da su u velikoj meri stvorile pretpostavke da ispune i uslove za ishodovanje i/ili produženje važenja integrisane dozvole, kao i obrnuto [3]. Ovakav pristup, svakoj organizaciji, može poboljšati imidž i poziciju: u odnosu na konkurenciju, na unapređenje odnosa sa vladinim i nevladinim sektorom, poboljšati odnos prema društveno odgovornom poslovanja i prema potencijalnim investitorima [4]. Na osnovu toga postupak ishodovanja integrisane dozvole nedvosmisleno usmerava ka uspostavljanju integrisanih sistema menadžmenta QMS, EMS, OHSMS i EnMS.

Integrisani menadžment sistem (Integrated Management System-IMS) predstavlja način za efektivno i efikasno upravljanje organizacijom. Rukovodstvo organizacije ima obavezu da neprekidno odgovara na zahteve tržišta i ostalih zainteresovanih strana (vlasnika, zaposlenih, korisnika proizvoda-usluga, isporučilaca i društva, pa na taj način i da ispuni uslov za pribavljanje integrisane dozvole i obezbedi usaglašenost sa zakonskim zahtevima.

Interes za obezbeđenje usaglašenosti sa zakonskim zahtevima i zahtevima standarda za uspostavljanje integrisanih sistema menadžmenta imaju, ili bi trebalo da imaju, svi: država i njene institucije, vlasnici kapitala, preduzeća, različita udruženja, institucije i građani kao pojedinci [5].

Ishodovanjem i periodičnim produženjem važenja integrisane dozvole i sertifikata za QMS, EMS, OHSAS i EnMS organizacije iz sektora industrije minerala (keramičkih proizvoda) stiču: zavidan kredibilitet, postaju društveno odgovorne organizacije koje posluju na principima tržišne ekonomije i principima održivog razvoja i brinu se za svoje sopstvene interese i interese šire društvene zajednice sve u cilju postizanja održivog razvoja.

2. ANALIZA ZAKONODAVNOG OKVIRA

Na nivou EU ključni instrument u vezi sa integrisanom dozvolom je: IPPC Directive – Council Directive 2008/1/EC od 15. januara 2008. – Official Journal L 24/8 od 29. januara 2008. Ova direktiva je u funkciji unapređenja održive proizvodnje kao ključnog principa održivog razvoja. IPPC Direktiva je transponovana u zakonodavstvo Republike Srbije u decembru 2004. godine, kada je donet Zakon o integrisanom sprečavanju i kontroli zagađenja životne sredine (Službeni glasnik RS br. 135/2004, 25/2015, i 109/2021) [1], a nakon toga usvojeni su sledeći propisi u vezi sa tim i to:

– Uredba o vrstama aktivnosti i postrojenja za koje se izdaje integrisana dozvola (Sl. glasnik RS, br. 84/2005);

– Uredba o kriterijumima za određivanje najboljih dostupnih tehnika, za primenu standarda kvaliteta, kao i za određivanje graničnih vrednosti emisija u integrisanoj dozvoli (Sl. glasnik RS, br. 84/2005);

– Uredba o sadržini programa mera prilagođavanja rada postojećeg postrojenja ili aktivnosti propisanim uslovima (Sl. glasnik RS, br. 84/2005);

– Uredba o utvrđivanju Programa dinamike podnošenja zahteva za izdavanje integrisane dozvole (Sl. Glasnik RS, br.108/2008);

– Pravilnik o sadržini i načinu vođenja registra izdatih integrisanih dozvola (Sl. Glasnik RS, br. 69/2005);

– Pravilnik o sadržini, izgledu i načinu popunjavanja zahteva za izdavanje integrisane dozvole (Sl. Glasnik RS, br. 30/2006, 32/2016, 44/2018, 4/2024);

– Pravilnik o sadržini i izgledu integrisane dozvole (Sl. Glasnik RS, br. 30/2006).

Osnovna načela kojih se treba pridržavati u postupku pribavljanja integrisane dozvole su: načelo predostrožnosti; načelo integrisanosti i koordinacije; načelo održivog razvoja; načelo hijerarhije upravljanja otpadom; načelo "zagađivač plaća" i načelo informisanja i učešća javnosti.

Integrisanom dozvolom odobrava se: rad novog postrojenja i obavljanje njegove aktivnosti (pribavljanje integrisane dozvole pre puštanja u rad), kao i rad i bitne izmene u radu postojećeg postrojenja. U izuzetnim slučajevima dozvola se može odnositi i na prestanak rada postrojenja, a period važenja integrisane dozvole ne može biti duži od 10 godina.

Nadležni organ za izdavanje integrisane dozvole određuje se shodno Zakonu o integrisanom sprečavanju i kontroli zagađenja životne sredine [1], i to može biti: Ministarstvo zaštite životne sredine, Pokrajinski sekretarijat za zaštitu životne sredine i održivi razvoj i Organ jedinice lokalne samouprave.

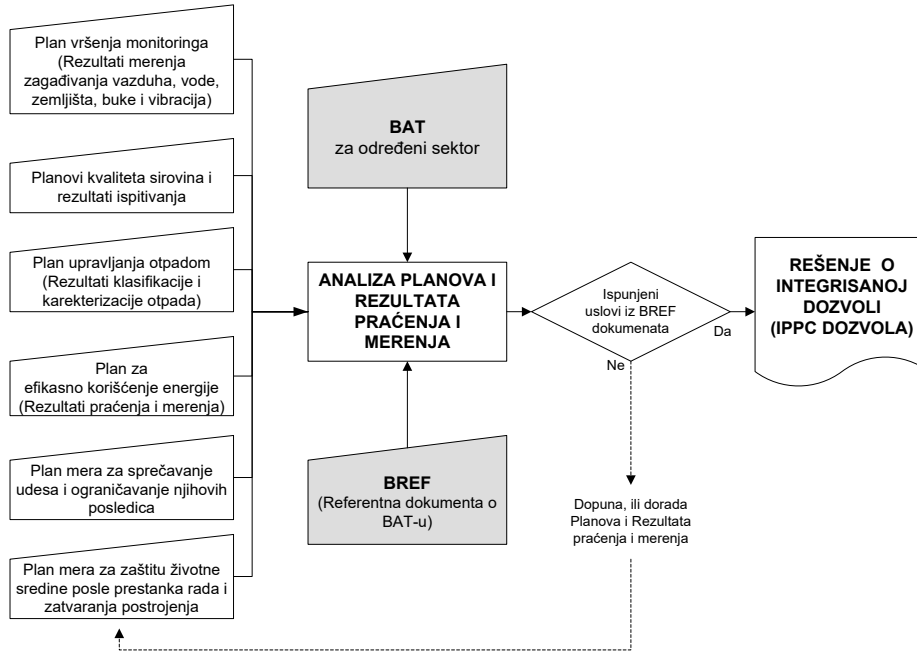
Podnošenje zahteva za izdavanje i/ili produženje važenja integrisane dozvole propisano je Uredbom o utvrđivanju Programa dinamike podnošenja zahteva za izdavanje integrisane dozvole (Sl. glasnik RS, br. 108/2008). Iz sektora industrije minerala zahteve podnose: postrojenja za proizvodnju keramičkih proizvoda pečenjem, a naročito crepa, cigle, vatrostalne opeke, pločica, keramičkog posuđa ili porcelana.....

Obrazac za izdavanje, reviziju ili produženje važenja integrisane dozvole propisano je Pravilnikom o sadržini, izgledu i načinu popunjavanja zahteva za izdavanje integrisane dozvole (Sl. Glasnik RS, br. 30/2006, 32/2016, 44/2018, 4/2024). Poslednjom izmenom ovog pravilnika došlo je i do značajnih promena u obrascu, ali i dalje je sadržan značaj posedovanja sertifikata za QMS, EMS i EnMS.

3. TERMINI, DEFINICIJE I OSNOVNI POJMOVI

Postrojenje jeste stacionarna tehnička jedinica u kojoj se izvodi jedna ili više aktivnosti koje su utvrđene posebnim propisom i za koje se izdaje integrisana dozvola, kao i svaka druga aktivnost kod koje postoji tehnička povezanost sa aktivnostima koje se izvode na tom mestu i koja može proizvesti emisije i zagađenje [2].

Na dijagramu 1. prikazan je šematski prikaz nekih od uslova za pribavljanje integrisane dozvole.



Dijagram 1. Neki od uslova za pribavljanje integrisane dozvole [1]

Integrisana dozvola jeste odluka nadležnog organa doneta u formi rešenja kojom se odobrava puštanje u rad postrojenja ili njegovog dela, odnosno obavljanje aktivnosti čiji sastavni deo čini dokumentacija sa utvrđenim uslovima kojima se garantuje da takvo postrojenje ili aktivnost odgovaraju zahtevima predviđenim zakonskom regulativom [1].

Operater jeste svako fizičko ili pravno lice koje, u skladu sa propisima, upravlja postrojenjem ili ga kontroliše ili je ovlašćen za donošenje ekonomskih odluka u oblasti tehničkog funkcionisanja postrojenja i na čije ime se izdaje integrisana dozvola [1].

Najbolje dostupne tehnike (BAT - Best Available Techniques) predstavljaju - najdelotvornije i najmodernije faze u razvoju aktivnosti i načinu njihovog obavljanja koje omogućavaju pogodniju primenu određenih tehnika za zadovoljavanje graničnih vrednosti emisija, propisanih u cilju sprečavanja ili smanjenja emisija i uticaja na životnu sredinu.

Najbolja – podrazumeva najefikasniji učinak u postizanju visokog opšteg nivoa zaštite životne sredine.

Dostupna – tehnika razvijena do stepena koji omogućava primenu u određenom sektoru industrije pod ekonomski i tehnički prihvatljivim uslovima uključujući i troškove i koristi.

Tehnika – način na koji je postrojenje projektovano, izgrađeno, održavano, na koji funkcioniše i stavlja se van pogona ili zatvara, uključujući i tehnologiju koja se koristi.

BREF dokumenta to su: referentna dokumenta u kojima su definisane smernice kako primeniti najbolje dostupne tehnike. Za sada postoje 33 BREF dokumenta koji pokrivaju sve industrijske grane u kojima se primenjuje IPPC direktiva. BREF dokumenta se definišu na Evropskom nivou.

4. ZNAČAJ INTEGRISANIH MENADŽMENT SISTEMA ZA PRIBAVLJANJE INTEGRISANE DOZVOLE

U Obrascu Zahteva za izdavanje integrisane dozvole u poglavlju II. DETALJNI PODACI O POSTROJENJU, PROCESIMA I PROCEDURAMA eksplicitno su navedeni zahtevi koji se odnose na upravljanje zaštitom životne sredine.

Na prvom mestu su informacije o lokaciji, a zatim na drugom mestu informacije vezane za **upravljanje zaštitom životne sredine** i to:

– da li operater ima **usvojenu politiku zaštite životne sredine**;

– da li operater **primenjuje sistem upravljanja zaštitom životne sredine** (EMS, EMAS) uključujući podatke o standardu koji se primenjuje, odnosno sertifikovanom sistemu (ISO 9001, ISO 14001), datumu sertifikovanja i verifikacije.

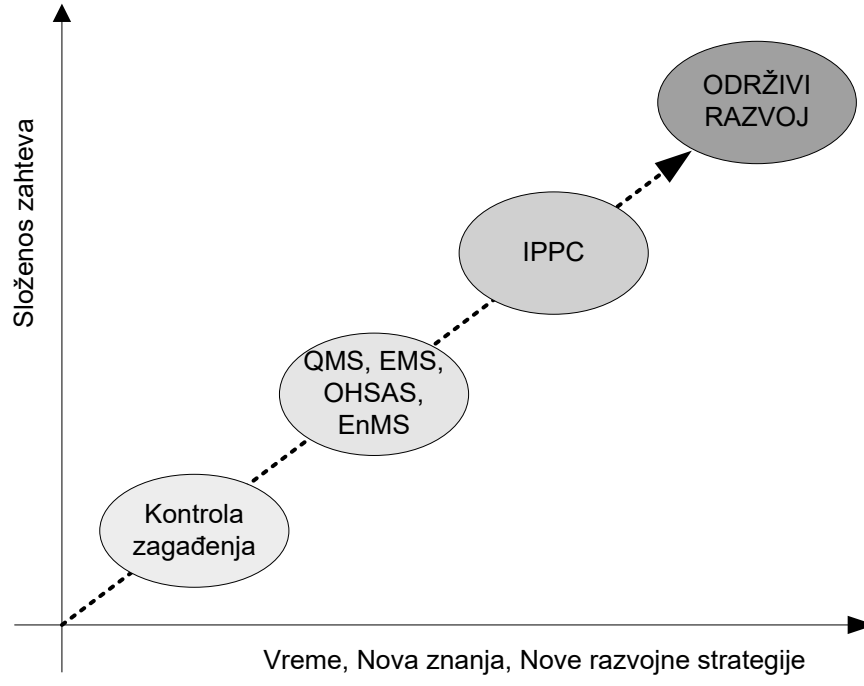
Efikasnost i efektivnost upravljanja procesom pribavljanja integrisane dozvole je teško zamisliva ako se ta razmatranja ne dovode u direktnu vezu sa integrisanim menadžment sistemima QMS, EMS, OHSMS i EnMS.

Svako izdvojeno posmatranje ishodovanja integrisane dozvole bez njenog dovođenja u kontekst sa upravljanjem značajnim aspektima životne sredine, upravljanjem rizicima po životnu sredinu i zdravlje ljudi i upravljanjem aspektima i energetskim faktorima po pravilu svodi se na palijativne-korektivne mere i aktivnosti koje problem ne sagledavaju sistemski.

Sa druge strane, da bi se efektivno upravljalo integrisanim menadžment sistemima potrebno je poznavati faktore koji direktno i/ili indirektno utiču na stepen bezbednosti životne sredine (dijagram 2).

Međunarodno priznati standardi za upravljanje kvalitetom, životnom sredinom i bezbednosti i zdravlja na radu i upravljanje energijom omogućavaju upravljanje aspektima životne sredine na bazi analize rizika i prilika i postizanje održivog razvoja. Primenom sistema upravljanja zaštitom životne sredine efikasno se kontroliše uticaj na zagađenje životne sredine putem Plana monitoringa.

Primenom sistema upravljanja energijom efikasno se ostvaruje Plan mera za efikasno korišćenje energije.



Dijagram 2. Odnos razvoja tehnoloških sistema i zaštite životne sredine

Uz zahtev za izdavanje, reviziju ili produženje važenja integrisane dozvole prilaže se i dodatna dokumentacija i to:

- Projekat za planirano, odnosno izgrađeno postrojenje;
- Izveštaj o poslednjem tehničkom pregledu;
- Plan vršenja monitoringa;
- Rezultate merenja zagađivanja činilaca životne sredine ili drugih parametara u toku trajanja probnog rada;
- Plan upravljanja otpadom;
- Plan mera za efikasno korišćenje energije;
- Plan mera za sprečavanje udesa i ograničavanje njihovih posledica;
- Plan mera za zaštitu životne sredine posle prestanka rada i zatvaranja postrojenja;
- Akt o pravu korišćenja prirodnih resursa;
- Izjavu kojom potvrđuje da su informacije sadržane u zahtevu istinite, tačne, potpune i dostupne javnosti;
- Dokaz o uplaćenoj administrativnoj taksi.

5. ZAKLJUČCI

Uporednom analizom svih zahteva standarda SRPS ISO 9001:2015, SRPS ISO EMS 14001:2015, SRPS ISO 45001:2018 i SRPS ISO 50001:2018 vidi se velika međuzavisnost zahteva koji se odnose na tačku 6. Planiranje QMS, EMS, OHSMS i EnMS i zahteva koji se odnose na ispunjenje tih uslova za pribavljanje INTEGRISANE DOZVOLE.

Standardi za QMS, EMS, OHSMS i EnMS imaju i izričit zahtev za identifikaciju, sprovođenje i vrednovanje usaglašenosti sa zakonskim i drugim zahtevima što je u direktnoj vezi sa ispunjenjem uslova za ishodovanje/izdavanje INTEGRISANE DOZVOLE.

Veliki obim dokumentovanih informacija koje su neophodne za ispunjenje uslova kao dokaz u postupku ishodovanja integrisane dozvole potrebno je kreirati u formi procedura, uputstava i zapisa koji istovremeno ispunjavaju zahteve nadležnog organa, i zahteve navedenih standarda jer se na taj način direktno utiče na PREVENTIVNOST, EFIKASNOST i EFEKTIVNOST poslovanja organizacije u ovom slučaju: postrojenja za proizvodnju keramičkih proizvoda pečenjem, a naročito crepa, cigle, vatrostalne opeke, pločica, keramičkog posuđa ili porcelana. Kao dodatna korist preciznog dokumentovanja procesa je da organizacija postaje konkurentnija u pogledu mogućnosti postizanja viših standarda kvaliteta, zaštite životne sredine, bezbednosti na radu, i efikasnijeg korišćenja energije, i poboljšanja kompetencije zaposlenih.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

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EFFECTS OF ADDITION OF FINELY GROUND CRT GLASS ON THE RESISTANCE OF CONCRETE TO CHLORIDE ION PENETRATION

Summary: *Durability is a critical aspect of reinforced concrete, as it directly affects the service life of structures. The presence of aggressive agents, particularly chloride ions, significantly impacts durability. This study evaluates the resistance to chloride ion penetration in concrete with addition of recycled cathode ray tube glass as a SCMs (supplementary cementitious material). Cement was replaced with cathode ray tube (CRT) glass at two levels: 10% and 20% by mass. In addition to tests conducted on fresh concrete, mechanical properties such as compressive strength and splitting tensile strength were examined. The objective of this research was to assess the effects of addition of finely ground CRT glass on the resistance of concrete to chloride ion penetration, in accordance with the SRPS EN 12390-18 standard. The literature search revealed a lack of papers in which this test on concrete with glass addition was conducted according to the European standard.*
Key words: *CRT glass, durability, chloride ion penetration, mechanical characteristics*.

UTICAJ DODATKA FINO SAMLEVENOG CRT STAKLA NA OTPORNOST BETONA PREMA PRODORU HLORIDA

Rezime: *Trajnost je kritičan aspekt armiranog betona, jer direktno utiče na vek trajanja konstrukcija. Prisustvo agresivnih agenasa, posebno jona hloridna, značajno utiče na trajnost. Ovim istraživanjem procenjuje se otpornost na prodiranje hloridnih jona u beton sa dodatkom recikliranog katodnog stakla kao SCM-a (supplementary cementitious material). Zamena cementa CRT (cathode ray tube) staklom izvršena je u dva nivoa: 10% i 20% po masi. Pored ispitivanja karakteristika svežeg betona, ispitivana su mehanička svojstva kao što su čvrstoća pri pritisku i čvrstoća pri zatezanju cepanjem. Cilj ovog istraživanja bio je da se proceni uticaj dodatka fino samlevenog CRT stakla na otpornost betona na prodiranje hloridnih jona, u skladu sa standardom SRPS EN 12390-18. Pretragom literature utvrđen je nedostatak radova u kojima je ovo ispitivanje na betonima sa dodatkom stakla sprovedeno prema navedenom evropskom standardu.*
Ključne reči: *CRT staklo, trajnost, prodor hlorida, mehaničke karakteristike*

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1. INTRODUCTION

The application of supplementary cementitious materials (SCMs) as partial substitutes for cement presents several advantages, including using industrial by-products and advancing environmentally sustainable production methods. However, it is critical to substantiate their use by demonstrating additional functional benefits, such as pozzolanic reactivity, contributions to strength development, and enhanced long-term durability of the resulting concrete mixtures. In this regard, chloride ion penetration testing is vital for assessing the durability performance of concrete, especially in aggressive environments such as marine settings or regions subjected to de-icing salts, where chloride-induced corrosion poses a significant threat to embedded steel reinforcement.

The most widely used methods for testing chloride penetration include the standardized approach established by the American Society for Testing and Materials (ASTM) C1202 [2] and the method developed in Finland - NT Build 492 [18]. When it comes to domestic regulations in this area, the first standard SRPS EN 12390-18 [4] was published in 2021. The latest standard currently in force in our country carries the designation SRPS EN 12390-18:2024 [5].

A review of the literature revealed that there were no studies in which the testing of chloride penetration on concretes with the addition of waste glass was conducted according to the provisions of the EN standard, which is not surprising, given that European standards were published only a few years ago. The procedure according to NT Build 492 is very similar to the provisions of the SRPS EN 12390-18 standard, however, there are certain significant differences. The most important difference relates to the anode and cathode solutions during testing. In NT Build 492, a 10% NaCl solution (100 g NaCl in 900 g of water) is used for the catholytic solution, while a 0.3 N NaOH solution (12 g NaOH in 1 liter of distilled or deionized water) is used for the anolytic solution. On the other hand, in SRPS EN 12390:2024, the catholytic solution is prepared as 5% NaCl (50 g NaCl in 950 g of 0.3 N NaOH), while the anolytic solution is used as in the case of the Finnish method, 0.3 N NaOH (12 g NaOH in 1 liter of distilled or deionized water).

Jihwan et al. [15] used waste glass sludge (WGS), a by-product of the glass panel manufacturing process, to replace part of the cement in concrete production. The glass sludge passed entirely through a 75 μm sieve and had a specific surface area of 414 m^2/kg according to Blane. The reference concrete had a water-binder ratio of 0.45 and contained 20% fly ash. Replacing 10% of the cement with WGS resulted in a reduction in the chloride migration coefficient D_{nssm} (designation according to NT Build 492), while concrete with 5% glass had almost the same migration coefficient as the reference concrete. The authors attribute this effect of glass on the concrete's resistance to chloride penetration to the fact that WGS had a finer grind than the replaced PC, which resulted in improved concrete compactness and better pore distribution. Lee et al. [17] used two types of waste glass – waste glass powder (WGP) and waste glass sludge (WGS) to replace part of the cement in concrete mixtures. The replacement of cement with glass was made in the amount of 20% by mass. The average particle size of the cement was 24.6 μm, compared to 22.8 μm in the case of WGP, and 14.7 μm in the case of WGS. The results of chloride penetration depth and chloride migration coefficient as a function of replacing part of the cement with two types of waste glass are shown in Figure 1, left. As can be seen from the figure, the WGS20 concrete showed the best performance of all. This is attributed not only to the pore-filling effect caused by the fine particles of WGS but also to the pozzolanic reaction of the glass. The incorporation of WGP resulted in the

highest chloride diffusion coefficient and corresponding greatest penetration depth, indicating poor performance of this addition in terms of chloride ion penetration. WGS20 had the least penetration, which is attributed to the more intense pozzolanic reaction.

Kamali et al. [16] also investigated the durability of concrete with the addition of glass. Two glass powders designated as GP1 and GP2 were used in this study. GP1 is a post-industrial by-product derived from waste glass fiber and GP2 is a post-consumer by-product derived from recycled glass. The average particle size of the glass was 8.4 μm. The impact of the addition of finely ground glass on the value of the chloride ion migration coefficient is shown in Figure 1, right. It can be concluded that concretes with glass powders exhibited lower migration coefficients than the control concrete and concrete with 20% addition of fly ash (FA) at all ages, except for concrete with 10% GP2 at 28 days of curing. This behavior indicated a higher pozzolanic reactivity of GP1 than GP2 leading to an enhanced densification at early age (28 days) and could also be attributed to increased pore solution conductivity in concrete modified with GP2 compared to that with GP1 [16].

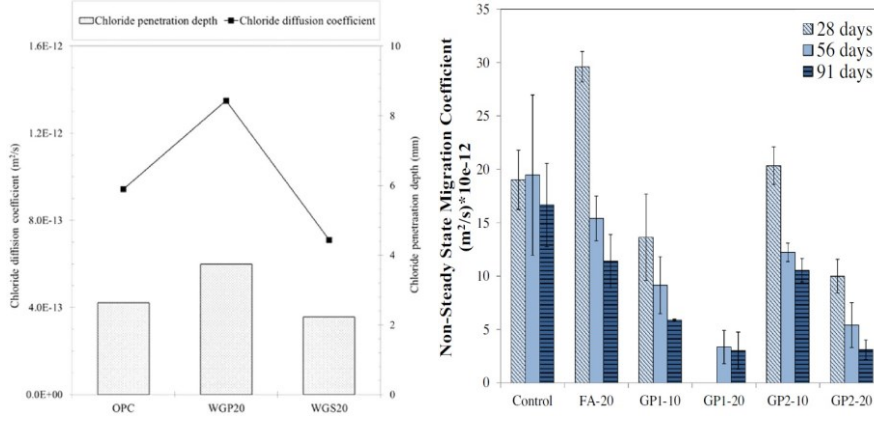


Figure 1. Results of chloride penetration depth and diffusion coefficient (left) [17] and migration coefficient (right) [16]

The results of tests conducted by several authors lead to the conclusion that the addition of finely ground cathode glass to cement composites improves resistance to chloride ion penetration. Furthermore, the extent of chloride ion penetration decreases when the glass is ground to a finer particle size.

2. MATERIALS USED IN EXPERIMENT AND MIX DESIGN

2.1. Materials used in the experiment

For the production of experimental concrete series, pure Portland cement CEM I 52.5R from "Moravacem" Novi Popovac was used, which meets all quality conditions prescribed by the standards EN 196-1 [6], EN 196-3 [7], EN 196-6 [8], and EN 197-1 [9]. The first fraction of river aggregate 0/4 mm from the separation "Trans DN Kop" Čapljinac was used, as well as two fractions of crushed aggregate (4/8 mm and 8/16 mm)

from the company Bisina, Raška. The granulometric composition of the aggregate mixture is shown in Figure 2. CRT glass comes from the recycling center "Jugo - Impex E.E.R." d.o.o. from Niš. Cathode glass was ground using a laboratory ball mill so that it can pass entirely through a sieve with an opening of 0.063 mm. Some physical characteristics of powdered glass and cement are shown in Table 1. As a chemical additive in the production of concrete mixtures, a superplasticizer commercially named MC-PowerFlow Evo 580 was used. It is the latest generation of MC superplasticizers for high rheological requirements of transport concrete with a specific mass of 1.05 g/cm3. Water from the city water supply was used.

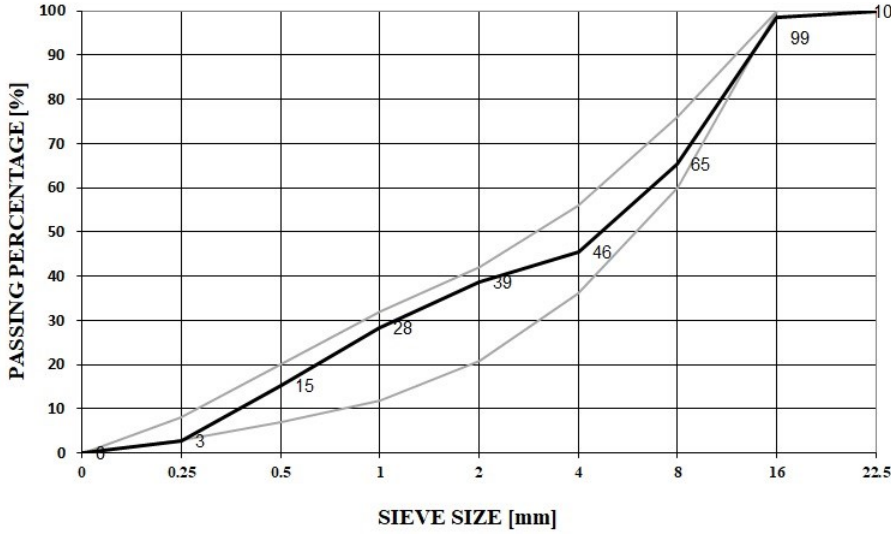


Figure 2. Granulometric composition of aggregate mixture

Material	Specific gravity [g/cm^3]	Specific surface by Blane [cm^2/g]	Fineness on sieve 0.09 mm [%]
CRT	2.84	2450	0.0
CEM I 52.5R	3.15	3940	0.0

Table 1. Some physical characteristics of CRT glass and cement CEM I 52.5R

2.2. Concrete mix design

The reference concrete (R) was made with 360 kg of CEM I 52.5R and 1870 kg of three-fraction aggregate. The water-binding ratio was constant and amounted to 0.486 (effective w/b 0.435). The percentage of cement replacement with CRT crushed glass was 10% and 20%, based on the mass of the cement. The series labels with the glass addition were formed by combining the abbreviation for cathode ray tube glass (CRT) and the percentage of cement replacement with the glass. The complete composition of the concrete mixtures is shown in Table 2. The target slump class for all concrete

mixtures was S4 (minimum slump 160 mm), achieved by applying a chemical additive from the superplasticizer group. During the design of the experimental concrete mixtures, the recommendations of the SRPS U.M1.206 standard, appendix V, for exposure class XD3 were respected, which prescribes a maximum effective water-cement ratio of 0.45, a minimum cement content of 340 kg, and a minimum compressive strength class of C35/45.

Concrete	Aggregate			Cement	CRT	Water	w/b	Additive
	0/4 mm	4/8 mm	8/16 mm	CEM I 52.5 R	<0.063 mm	City Wat. Supply	-	Super-plasticizer
	kg/m^3	kg/m^3	kg/m^3	kg/m^3	kg/m^3	kg/m^3	-	kg/m^3
Reference (R)	840	375	655	360	-	175.0	0.486	2.52
CRT10	840	375	655	324	36	175.0	0.486	2.52
CRT20	840	375	655	288	72	175.0	0.486	2.52

Table 2. Composition of experimental concrete mixtures

3. EXPERIMENTAL RESULTS AND DISCUSSION

Testing of concrete consistency is performed according to the provisions of the standard EN 12350-2 [10], density according to EN 12350-6 [11], and air content in concrete according to the standard EN 12350-7, pressure gauge method [12]. Slump test results are in the range from 160 mm to 170 mm, class S4, which was the goal when designing concrete mixtures. The results of fresh concrete density testing are shown in Figure 3 (left), while the results of air content testing are shown in Figure 3 (right).

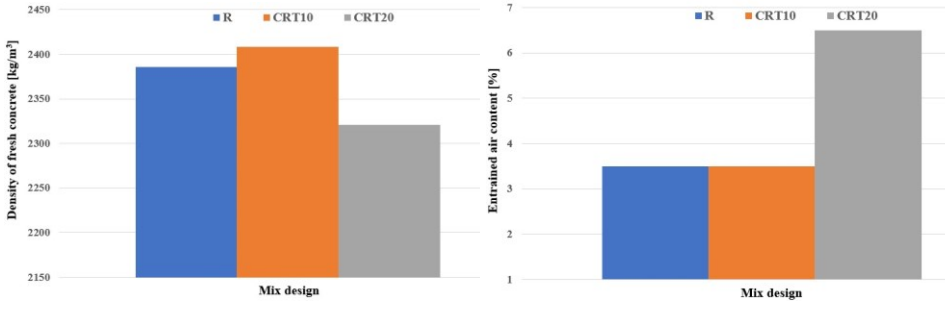


Figure 3. Density (left) and air content (right) of fresh concrete

The density of fresh reference concrete was 2386 kg/m³, while for concrete with 10% cement replacement by glass it was about 1% higher, amounting to 2408 kg/m³. The air content of the reference concrete and CRT10 was 3.5%, which is above the usual value of 2%. Considering that air entraining additives were not used, it can be assumed that the superplasticizer itself leads to increased air entrainment in the mix. Significant

changes in these properties occur in the case of 20% cement replacement by glass. Namely, for the CRT20 concrete series, a very low density was measured - 2321 kg/m³, which is a direct consequence of increased air content in the concrete mix. The average air content in CRT20 concrete was as high as 6.5%, which is significantly higher than usual. The measured significant changes in the properties of fresh concrete with the addition of CRT powder are not in agreement with the authors previous research [3]. In study [3], the maximum level of cement replacement by glass was 35%, and it was found that there was a slight decrease in volume mass within the range of -1% compared to the reference concrete, while the change in air content was at the level of the resolution of the porosimeter device. Additional testing is necessary to provide a more precise answer to why significant air entrainment occurred in the CRT20 concrete mix.

The compressive strength test is conducted on the cube shaped specimens having the edge of 150 mm according to the provisions of the EN 12390 – 3 standard [13]. For the tests, digital, hydraulic press "UTEST UTC - 5740" measuring range 0 - 3000 kN and accuracy class 0,5% is used. Test are conducted at the concrete age of 7 and 28 days. Figure 4 provides the graphic presentation of achieved compressive strengths of concrete mixtures in time.

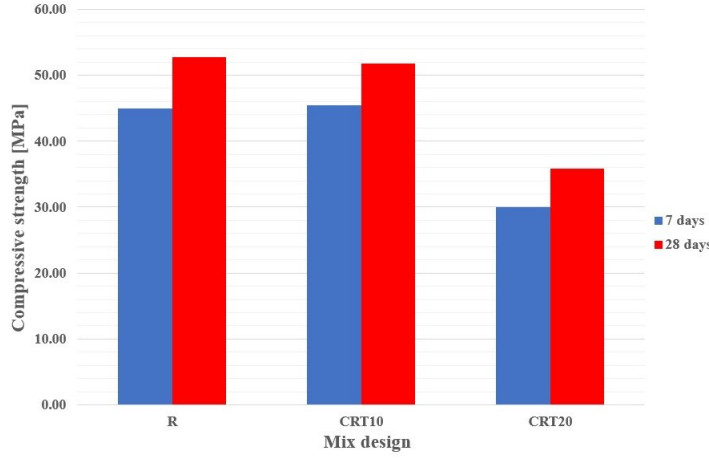


Figure 4. Graphic presentation of achieved compressive strengths of concrete series in time

Replacing 10% of cement with recycled glass did not significantly affect the reduction of concrete compressive strength during testing at 7 and 28 days compared to reference concrete. The CRT20 series, at mentioned ages, had 33% and 38% lower compressive strength compared to the reference series, respectively. Certainly, such a drastic reduction in compressive strength is due to a significant amount of entrained air in this concrete mixture. Based on previous research by the authors, it is expected that the differences in compressive strength will be somewhat less pronounced between CRT20 and the reference series during later testing at 90 days. Namely, the nature of the pozzolanic reaction is such that it takes place later than the cement hydration process, and it is most intensive after 28 days. The obtained results are in correlation with the results of Aliabdo et al. [1] who concluded that the substitution of cement with glass up to level of 10% does not adversely affect the compressive strength at older concrete ages.

The testing of splitting tensile strength is conducted according to the EN 12390-6 standard [14] on the cylindrical specimens with diameter of 150 mm and height 300 mm. Digital hydraulic press "UTEST UTC - 5600" accuracy class 0.5% is used for the test. Effect of substitution of cement with CRT glass powder on the variation of concrete splitting tensile strength at age of 28 days is presented in Figure 5 provides the graphic presentation of achieved tensile splitting strengths of concrete mixtures at the concrete age of 28 days.

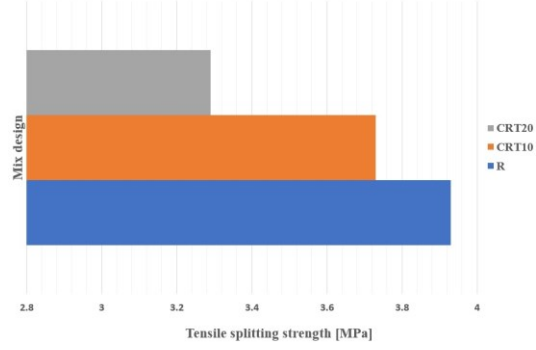


Figure 5. Effect of substitution of cement with CRT glass powder on the variation of concrete splitting tensile strength

Concrete mixtures with partial replacement of cement by CRT glass have lower tensile splitting strength values compared to the reference series at 28 days of testing. The CRT10 and CRT20 series have 5.1% and 16.3% lower tensile splitting strength values compared to the reference series. A corresponding number of samples were also left for later testing at 90 days.

Testing the resistance of concrete to chloride penetration is carried out according to the provisions of the SRPS EN 12390-18 standard [5]. The test was conducted on 3 cylindrical samples with a diameter of 100 mm and a height of 50 mm. These samples were obtained by cutting a larger cylinder with a diameter of 100 mm and a height of 200 mm into which the concrete was initially placed after mixing. The migration cell reservoir is filled with a 5% NaCl solution and connected to the anode (negative charge), while the other cell reservoir is filled with a 0.3 N NaOH solution (12 g NaOH per 1 liter of distilled or deionized water) and connected to the cathode (positive charge). The duration of the test depends on the electrical resistance provided by the sample. Given that the initial measured current strength was in the range of 30 to 60 mA, the voltage during the test was maintained at the initial 30 V, and the test duration was 24 hours. Figure 6 shows the equipment for testing chloride penetration.

After the current test period, the samples were split in half parallel to the direction of chloride penetration. Both halves were sprayed with a silver nitrate ($AgNO_3$) aqueous solution. In reaction with silver nitrate, the part of the sample where chloride penetration occurred changed color to a whitish hue. The depth of chloride penetration was measured in 9 places, excluding the sample edges of 10 mm. Based on the measured individual penetration depths, further calculations are made to compute the migration coefficient according to standard [5]. The average values of chloride ion penetration depth as well as the calculated migration coefficients are shown in Table 3.

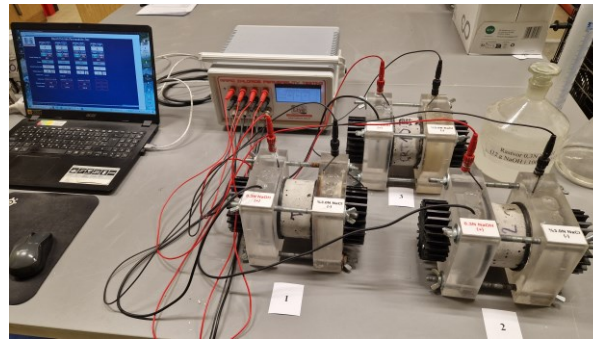


Figure 6. Equipment for determination of the chloride migration coefficient

Mix design	Sample mark	Average depth of chloride ion penetration [mm]	Migration coefficient Mnss [m^2/s]	Average values of the migration coefficient $M_{nss,\ average}$	Resistance class
Reference (R)	E/1	20.5	$9.2 \cdot 10^{-12}$	$9.2 \cdot 10^{-12}$	CL-2
	E/2	18.5	$8.3 \cdot 10^{-12}$		
	E/3	20.0	$9.1 \cdot 10^{-12}$		
CRT10	CRT10/1	20.0	$9.0 \cdot 10^{-12}$	$9.7 \cdot 10^{-12}$	CL-2
	CRT10/2	21.5	$9.6 \cdot 10^{-12}$		
	CRT10/3	23.5	$10.5 \cdot 10^{-12}$		
CRT20	CRT20/1	25.0	$11.4 \cdot 10^{-12}$	$11.4 \cdot 10^{-12}$	CL-1
	CRT20/2	22.0	$10.0 \cdot 10^{-12}$		
	CRT20/3	28.0	$12.9 \cdot 10^{-12}$		

Table 3. Rezultati ispitivanja prodora hlorida prema SRPS EN 12390-18 [5]

The lowest migration coefficient value was measured in the reference series. Based on the test results shown in Table 3, it can be concluded that with the increase in the level of cement replacement with CRT glass, there is an increase in the depth of chloride ion penetration, which is directly related to the increase in the migration coefficient value. Based on the individual and average values of the migration coefficient, concrete can be classified into one of three resistance classes CL-1, CL-2, or CL-3, based on the criteria defined by the standard SRPS U.M1.206, point 5.3.2 [19]. Unfortunately, the obtained results are not consistent with the research results [15-17]. When designing experimental concrete mixtures, the recommendations of the SRPS U.M1.206 standard, Annex V [19], which define the limit values for the effective water-cement ratio, the minimum amount of cement, and the lowest strength class that the concrete should have in order to potentially meet a certain exposure class, were followed. A sufficient number of concrete samples were left to carry out the chloride penetration test at 90 days of age,

with the aim of determining whether the pozzolanic reaction of the glass by that time will lead to better resistance of the concrete to this effect.

Based on the determined resistance classes of the experimental concrete series (Table 3), it can be concluded that the reference concrete (R) and the CRT10 mixture meet the XD2 exposure class, while the CRT20 series meets the XD1 exposure class. According to the SRPS EN 206 standard, Table 1 [20], it can be seen that XD1 exposure classes can be required for concrete surfaces exposed to airborne chlorides in moderately humid environments. Concrete exposed to XD2 conditions is used for swimming pool construction or exposed to industrial waters containing chlorides.

4. CONCLUSIONS

Based on the obtained experimental results, a number of conclusions can be drawn:

- The replacement of cement with ground CRT glass up to a level of 10% insignificantly affects the fresh concrete's volume density, as well as the percentage of entrained air. The CRT20 concrete mixture had a low volume density value of only 2321 kg/m³ and an entrained air content of as much as 6.5%.
- Replacing 10% of the cement with recycled glass slightly reduced the compressive strength of the concrete when tested at 7 and 28 days compared to the reference concrete. The CRT20 series, at the aforementioned ages, had 33% and 38% lower strength compared to the reference series.
- The CRT10 and CRT20 series have 5.1% and 16.3% lower tensile strength by splitting compared to the reference series.
- Based on the determined resistance classes of the experimental concrete series (Table 3), it can be concluded that the reference concrete and the CRT10 mixture satisfy the XD2 exposure class, while the CRT20 series satisfies the XD1 exposure class.
- A sufficient number of samples were left to carry out all tests at the age of 90 days, with the aim of determining whether the development of the pozzolanic reaction of the glass in the period from 28 to 90 days will improve the physical-mechanical properties and durability of the concrete mixtures.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
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COMPRESSIVE STRENGTH OF ALKALI-ACTIVATED MORTARS WITH CONVENTIONAL HYDROXIDE ACTIVATORS AND SUNFLOWER HUSK ASH – COMPARATIVE ANALYSIS

Summary: *Alkali-activated materials (AAMs) are alternative binders with no cement in their composition. They are synthesized from aluminosilicate-rich precursors and alkaline activators – usually hydroxides and silicates. They significantly contribute to the overall CO_2 footprint of AAMs, leading to efforts to replace them with alternative alkali-rich waste materials such as agricultural biomass ashes. The paper presents the comparison of slag-based mortars activated with sunflower husk ash (SHA) – a locally available waste material in the Autonomous Province of Vojvodina, Serbia, and KOH and NaOH. The influence of SHA on the mortars was assessed by comparing the 7- and 28-day compressive strength.*
Keywords: *alkali-activated materials, sunflower husk ash, blast furnace slag, agricultural biomass ash, alternative activator, sustainability*

ČVRSTOĆA PRI PRITISKU ALKALNO-AKTIVIRANIH MALTERA SA KONVENCIONALNIM HIDROKSIDNIM AKTIVATORIMA I PEPELOM SUNCOKRETOVE LJUSKE – KOMPARATIVNA ANALZA

Rezime: *Alkalno aktivirani materijali (AAM) predstavljaju alternativna veziva koja u svom sastavu ne sadrže cement. Sintetišu se iz alumosilikata i alkalnih aktivatora – najčešće hidroksida i silikata. Alkalni aktivatori značajno doprinose ukupnom ugljeničnom otisku AAM-a, zbog čega se intenzivno istražuju mogućnosti njihove zamene alternativnim, otpadnim materijalima bogatim alkalijama, kao što je pepeo poljoprivredne biomase. U radu je prikazano poređejne maltera na bazi zgure aktiviranih pepelom suncokretove ljuske (SHA) – lokalno dostupnim otpadnim materijalom u Autonomnoj Pokrajini Vojvodini u Srbiji – kao i KOH-om i NaOH-om. Uticaj SHA na maltere procenjen je poređenjem rezultata čvrstoće pri pritisku nakon 7 i 28 dana.*
Ključne reči*: alkalno-aktivirani materijali, pepeo suncokretove ljuske, zgura iz visokih peći, pepeo poljoprivredne biomase, alternativni aktivator, održivost*

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1. INTRODUCTION

Alkali-activated materials (AAMs) are binders derived from aluminosilicate-rich precursors and alkaline activators. Along with their competitive performance, they are a potential low-carbon alternative to Portland cement (PC) due to the absence of cement from mix design and the use of industrial-by products (e.g., slag and coal fly ash) as precursors. Alkaline hydroxides, silicates, sulphates and carbonates can be used as activators [1]. The driver of the activation process are alkalis (sodium or potassium) in liquid or solid form, that induce the dissolution of the amorphous aluminosilicates from the precursors, followed by the formation of hydration products [2]. The possibility to use a wide range of materials for producing AAMs results in various properties, depending on the chemical composition of the precursors, type and dosage of the activators. Precursors with low Ca content, such as fly ash or metakaolin have three-dimensional alkali-aluminosilicate hydrate type gel (N-A-S-H) as a reaction product. High-Ca systems are based on the activation of a precursor with high Ca content, such as slag, where the main reaction product is calcium-aluminosilicate hydrate type gel (C-A-S-H) [2,3].

1.1. Conventional chemical activators and alternative alkali sources

A combination of alkali hydroxides and alkali silicates is reported as the most reliable activator for slag-based AAMs. They are usually sodium-based, due to availability and lower price, compared to potassium activators. Hydroxides are commercially available as pellets, while alkaline silicates are available as solutions, also known as waterglass [3,4]. In the AAM systems, alkalis are responsible for early strength development due to the activation and are usually dosed as 2-8% by mass of slag. Silica content contributes to the later increase in strength. Favourable silica-to-alkali ratio in slag systems depends on the slag type [5].

There are two procedures for mixing AAMs. The difference is whether the alkali activator is added in solid form or as a solution. The precursor and solid activator can be dry mixed, followed by addition of water and aggregate. Alternatively, the alkali activator can be prepared as a concentrated water solution, added to dry mixed precursor and aggregates, along with additional water, to obtain satisfactory workability. The first approach, where solid activator is premixed with precursor is named one-part alkali-activation. The second approach, where the activator solution is added to mix is two-part alkali-activation. Although most of the research is focused on the two-part alkali activation due to the better mechanical properties of the obtained concrete, one-part alkali-activation is gaining increased attention [2]. Two-part activation process may impose practical, health and safety concerns, as the dissolution of alkali hydroxides is an exothermic reaction and the produced concentrated alkali solutions are highly caustic [6].

Although utilization of AAMs in construction would significantly reduce the environmental impact compared to PC concrete, the production process of chemical activators also contributes to CO_2 emissions and increases their cost [7]. Hence, there is a strive in the research community to find the alternative alkali and silica sources suitable for the AAM technology.

Contemporary research proposes the use of potassium-rich agricultural biomass ashes as an source of alkalis instead of chemicals [8–12]. Agricultural biomass ash is waste derived from the combustion of agricultural biomass (i.e., crop residues) as energy source

[13]. Most of the published research on this topic is experimental research on slag-based mortars or pastes. The significant number of research papers concern the olive stone biomass ash (OBA) as an alkali source, while the alkali-activation was also proved for almond, hazelnut, nutshell, coffee and mango seed-bark ashes, reaching mortar compressive strength up to 45 MPa after 28 days of curing, depending on the chemical composition of the ashes and mix design [14]. Sunflower husk ash (SHA) and sunflower stalk ash (SSA) was assessed by Zhu et al. [15,16] as an alternative activator for blast furnace slag and black rice husk, to form grout slurries, where the highest compressive strengths were achieved by the activation with SSA.

Besides the reduction AAMs' CO_2 emissions, this approach has the potential for resolving the problems of agricultural biomass landfilling [13]. Furthermore, AAMs with agricultural biomass ashes are produced by one-part alkali-activation process, which is favorable regarding mentioned safety issues associated with two-part alkali activation with conventional chemicals.

Authors of this paper described and discussed the local issue of SHA landfilling in Autonomous Province of Vojvodina, Serbia in previous publications [14,17,18]. It is a potassium rich waste material, generated by eadible oil production companies and Heat and electrical plant (TE-TO) in Sremska Mitrovica, Serbia [14,17]. Our previous work on SHA activated slag mortars showed the effect of SHA content and different curing regimes on compressive strength [14] and SHA influence on the slump flow of the mixes [14,17]. Furthermore, thermogravimetric analysis confirmed formation of the C,K-A-S and C,K-A-S-H gel as primary and hydrotalcite as secondary phase [18].

Valorisation of the locally available SHA as an activator in AAM technology would contribute to the circularity in construction sector through reduction of carbon footprint and prevention of waste landfilling [17]. Therefore, the further research on the properties of SHA activated AAMs is important.

This study aims to compare SHA as an alternative activator and conventional, chemical activators KOH and NaOH for ground granulated blast furnace slag (GGBFS) mortar mixes. The efficiency of the activators was assessed by compressive strength after 7 and 28 days of curing in ambient conditions.

2. MATERIALS AND METHODS

GGBFS provided by Lafarge, Serbia was used as a precursor for alkali-activated mortars. Commercially available KOH and NaOH in the form of pellets were used as conventional activators, with purity 85% and 97%, respectively. SHA collected by oil factory VictoriaOil, Šid, Serbia was used as an alternative, waste-derived activator. Chemical composition of SHA and GGBFS were determined by X-ray fluorescence and are presented in Table 1. Quartz sand was used as aggregate.

Oxide	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	K_2O	MnO	TiO_2
GGBFS	38.19	10.28	0.31	37.04	9.69	0.75	0.87	0.52	0.37
SHA	0.99	0.38	0.27	11.42	7.45	15.68	32.98	0.06	0.01

Table 1. Chemical composition of GGBFS and SHA

2.1. Mix design and mixing procedure

Three groups of mortars were tested in the presented study: two reference mixes activated with KOH and NaOH and the third mix activated with SHA. Each mix had constant slag content, water-to-binder (w/b) ratio 0.45 and aggregate to binder (a/b) ratio of 3. The content of KOH and NaOH were varied to obtain 4, 6 and 8% of alkali content (K_2O and Na_2O, respectively), by mass of GGBFS.

KOH and NaOH activated mortars were mixed as two-part AAMs: they were dissolved in water to form an activating solution. Total water in the reference mixes was calculated as the sum of the water in activating solution and tap water, needed to attain desired workability. Binder content was calculated as the sum of GGBFS and KOH or NaOH pellets. Reference mortars were mixed as follows: GGBFS and aggregates were manually dry mixed after which the water was added, and the mix was homogenized. The alkali solution was added at the end and the mortar was mixed in a mixer for 5 minutes, at high speed.

Mixes activated with SHA were made as one-part AAMs. The binder was equal to the sum of GGBFS and soluble part of SHA. The third group of mortars was activated with SHA dosage 15, 25, and 35 wt% GGBFS. Mixing of solid components was the first step for SHA activated mixes where the GGBFS, aggregates and SHA were also dry mix. Then, they were added to water, manually homogenized and then mixed in a mixer for 5 minutes.

The mix designs of three mortar groups are presented in Table 2 . Reference mixes are denoted according to the alkali type and content, while SHA-activated mix are denoted according to the SHA content.

	GGBFS (g)	**KOH* (g)**	**NaOH* (g)**	**SHA (g)**	**Total water (g)**	**Aggregate (g)**
K_2O/4	403	19.2	-	-	190	1267
K_2O/6		28.8	-	-	194	1296
K_2O/8		38.4	-	-	199	1325
Na_2O/4		-	20.8	-	191	1272
Na_2O/6		-	31.2	-	195	1303
Na_2O/8		-	41.6	-	200	1334
SHA15		-	-	60	194	1292
SHA25		-	-	101	202	1346
SHA35		-	-	141	210	1400
*solid						

Table 2. Mix design of mortars activated with KOH, NaOH and SHA

Reference mortars were mixed as follows: GGBFS and aggregates were manually dry mixed after which the water was added, and the mix was homogenized. The alkali solution was added at the end and the mortar was mixed in a mixer for 5 minutes, at high speed. Mixing of solid components was the first step for SHA activated mixes where the GGBFS, aggregates and SHA were also dry mix. Then, they were added to water, manually homogenized and then mixed in a mixer for 5 minutes.

2.2. Sample casting, curing and testing procedures

Mortar samples were cast in metal prism moulds with dimensions 160x40x40 mm. Cast samples were demoulded after 24 hours and cured wrapped in polymeric films until the day of testing, in ambient conditions. Demoulded samples are shown in Figure 1.

Compressive strength was tested with universal testing machine, after 7 and 28 days of curing in accordance with SRPS EN 196-1 [19].



Figure 1. Example mortar samples activated with: a) SHA; b) KOH; c) NaOH

3. RESULTS AND DISCUSSION

The highest compressive strength after 7 and 28 days of curing was attained for the group of mortars activated with SHA (Figure 4 and Figure 5). Almost all of the mixes within this group outperfomed other mixes at both testing ages, while mix with 15% of SHA had the same strength as mix activated with 6% of NaOH. SHA activated mortars also exhibited the highest growth in the compressive strength from 7 to 28 days of testing (approximatley 20 MPa, in comparisson to 8 MPa for the reference mixes).

Similar results were reported by Zhu et al. in their work on slag and black rice husk ash-based grouts activated with SHA and other similar biomass ashes [15,16]. In these studies, SHA activated grouts had 43% higher 28 day-compressive strength than the mix activated with KOH solution with the concentration 5mol/L. This is explained by high content of soluble potassium salts in the SHA, as well as the filler effect of the ash [15]. Other type of biomass ashes used for producing of the mortar mixes have also been proven to contribute compressive strength development more than conventional chemical

activators [9–11,20], which could be attributed to the combined effect of K_2O and CaO from the SHA on the activation process [10].

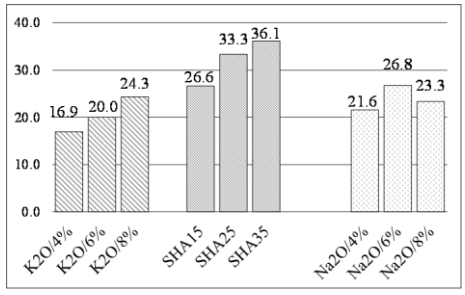


Figure 4. Compressive strength of KOH, SHA and NaOH activated mortars after 7 days of curing

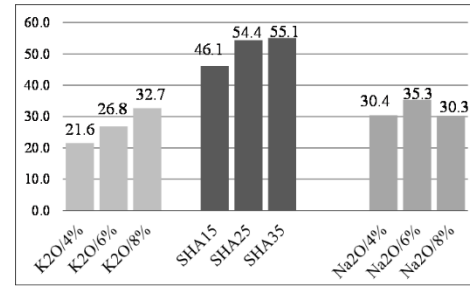


Figure 5. Compressive strength of KOH, SHA and NaOH activated mortars after 28 days of curing

Increase in activator content in mortars activated with KOH and SHA in this study resulted in the increase in compressive strength at both ages, while compressive strength of mortars activated with NaOH decreased for the alkali content above 6%. Mixes SHA25 and SHA35 reached 54.4 and 55.1 MPa, respectively, at 28 days. This could lead to a conclusion that the further increase in SHA content would result in the decrease of compressive strength, which was the reported by Pinheiro et al. for the slag-based mortars activated with OBA [10].

The appearance of samples during and after testing are presented in Figure 6. and Figure 7.

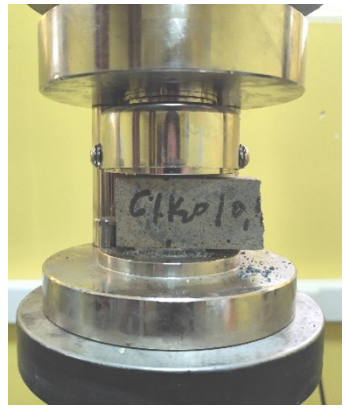


Figure 6. Compressive strength test of mortar samples



Figure 7. Example of a cross section of tested samples

4. CONCLUSIONS

The paper presented comparative analysis of the compressive strength of mortars activated with SHA (15, 25 and 35% by mass of GGBFS) and reference mortars activated with KOH and NaOH (4, 6 and 8% of K_2O and Na_2O, by mass of GGBFS). Compressive strength was tested after 7 and 28 days of curing at ambient temperature.

SHA significantly outperformed conventional chemical activators KOH and NaOH. The highest compressive strentght was attained after 28 days for the mortars activated with SHA25 and SHA35 (approximatley 55 MPa). KOH activated mortars reached highest compressive strength with 8% K_2O (32.7 MPa) and NaOH with 6% Na_2O (35.3 MPa). The highest growth from 7 to 28 day compressive strength was achieved for mortars activated with 25% of SHA.

The use of locally available waste material such as SHA would be beneficial for enchancement of circular economy concepts in the construction sector. While the presented results are promising, the further research is needed to investigate the potential of SHA as an alternative activator in AAM technology.

Acknowledgement

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

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Stručni rad

UPOREDNA ANALIZA ČVRSTOĆE PRI PRITISKU I VODONEPROPUSTLJIVOSTI BETONA − SLUČAJ JEDNE MOSTOVSKE KONSTRUKCIJE

Rezime: *U radu je prikazana uporedna analiza stvarnih i empirijski dobijenih čvrstoća betona pri pritisku. Analiza je obuhvatila rezultate ispitivanja na 1.251 uzorku betona. Dobijeni rezultati su pokazali da empirijski obrazac po Bolomeju daje najmanja odstupanja u odnosu na stvarne vrednosti. Takođe na dve betonske mešavine izvršeno je poređenje klasa vodonepropustljivosti prema povučenim i trenutno važećim standardima.*
Ključne reči: *Beton, čvrstoća pri pritisku, vodonepropustljivost.*

COMPARATIVE ANALYSIS OF CONCRETE COMRESSIVE STRENGTH AND WATER IMPERMEABILITY − CASE STUDY FOR ONE BRIDGE STRUCTURE

Summary: *The paper presents a comparative analysis of actual and empirically obtained compressive strengths of concrete. The analysis included test results from 1,251 concrete samples. The results showed that the empirical model according to Bolomey provides the smallest deviations from the actual values. A comparison of water impermeability classes according to the withdrawn and currently valid standards was also conducted, based on testing of two different concrete mixtures.*
Keywords: *Concrete, compressive strength, water impermeability.*

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1. UVOD

Kako bi se građevinski radovi izvodili kvalitetno, bez nepotrebnih i nepredviđenih troškova neophodno je uložiti veliki trud na izradi potrebne tehničke dokumentacije. Najvažniji deo u izradi predmetne dokumentacije svakako predstavlja statički proračun konstrukcije, koji treba da obezbedi njenu stabilnost i nosivost. U slučaju armiranobetonskih (AB) konstrukcija jedan od ulaznih parametara prilikom proračuna predstavlja odabir klasa izloženosti za svaki konstruktivni element. Standardom SRPS EN 206:2021 [1] i Pravilnikom o tehničkim zahtevima za beton [2] definisane su klase izloženosti AB konstruktivnih elemenata u zavisnosti od tipa konstrukcije, kao i položaja elementa u konstrukciji. U zavisnosti od definisanih klasa izloženosti Pravilnik o tehničkim zahtevima za beton propisuje neke od najbitnijih parametara betona, kao što su: minimalna klasa čvrstoće betona, maksimalni vodocementni faktor, kao i posebna svojstva betona u vidu: vodonepropustljivosti, otpornosti na dejstvo mraza, otpornosti na dejstvo mraza u prisustvu soli za odmrzavanje, otpornosti na habanje i otpornosti na prodor hlorida.

Svi navedeni podaci o karakteristikama betona moraju biti definisani u Projektu za izvođenje (PZI). U skladu sa PZI, izvođač radova je dužan da pre otpočinjanja radova na izgradnji konstrukcije izabere isporučioca betona koji osim tehničkih mogućnosti, poseduje i rezultate prethodnih proba betona kojima dokazuje da predmetni beton zadovoljava sve projektom predviđene karakteristike za beton u svežem i očvrslom stanju.

Kao što je opšte poznato, izrada betonskih mešavina predstavlja jedan iterativan postupak u kom se u početnoj fazi vrši izbor komponentnih materijala, a njihove količine određuju na osnovu nekih prethodnih iskustava i odgovarajućih empirijskih obrazaca [3]. Tako se na primer prilikom definisanja vodocementnog faktora koriste formule: Grafa, Ferea, Bolomeja, Beljajeva, Skramtajeva i drugih, koje uzimaju u obzir projektovanu klasu čvrstoće betona i klasu cementa, a pojedine od njih i kvalitet upotrebljenog agregata.

Jedan od ciljeva ovog rada je bio da se, za različite betonske mešavine koje su isporučene na gradilište, izvrši komparativna analiza stvarnih i računskih čvrstoća pri pritisku na uzorku od 1251 betonske kocke. Stvarne vrednosti čvrstoća pri pritisku dobijene su kao aritmetička sredina rezultata ispitivanja betonskih kocki pri starosti od 28 dana (svi uzorci su ispitani u skladu sa standardom SRPS EN 12390-3:2019 [4]). Računske čvrstoće pri pritisku sračunate su na osnovu efektivnih vodocementnih faktora, za svaku od betonskih mešavina, korišćenjem odgovarajućih empirijskih obrazaca. Predmetna analiza bi trebalo da ukaže koja od navedenih empirijskih formula, za cemente koji se danas koriste u Srbiji, daje najmanja odstupanja u odnosu na stvarne vrednosti čvrstoća pri pritisku.

U radu su, takođe, prikazane i komparativne analize rezultata ispitivanja vodonepropustljivosti, za dve betonske mešavine, prema povučenom standardu SRPS.U.M1.015:1978 [5] i trenutno važećem standardu SRPS EN 12390-8:2019 [6]. Potreba za predmetnom analizom je nastala kao rezultat velikog broja projekata, koji se u poslednjoj deceniji izvode u Srbiji, a kojima se za vodonepropustljiv beton definišu klase vodonepropustljivosti prema povučenom standardu SRPS.U.M1.015:1978. Naime, u postupku kontrole kvaliteta betona, akreditovane laboratorije ispitivanje vodonepropustljivosti sprovode prema trenutno važećem standardu SRPS EN 12390-8:2019 i na osnovu dobijenih rezultata ispitivanja daju ocenu usaglašenosti sa važećim tehničkim zahtevima prema standardu SRPS U.M1.206:2023 [7]. Kako klase vodonepropustljivosti prema prethodno navedena dva standarda nemaju iste oznake ni kriterijume koje treba da zadovolje, često se javljaju nesuglasice na relaciji Izvođač-

Nadzorni organ, a koje se odnose na to da li predmetni beton zadovoljava kriterijume definisane u projektnoj dokumentaciji.

Sprovedena eksperimentalna ispitivanja bi trebalo da ukažu u kom odnosu stoje klase vodonepropustljivosti prema prethodno navedenim standardima i da u tom smislu donekle olakšaju donošenje odluka u spornim situacijama, kako je to prethodno pomenuto.

2. ODABRANE BETONSKE MEŠAVINE

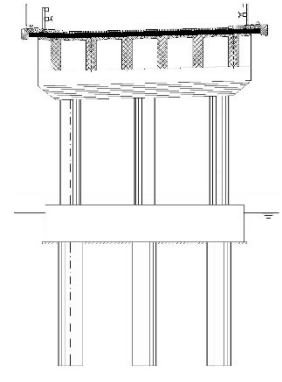
2.1. Betonske mešavine korišćene u komparativnoj analizi čvrstoća pri pritisku

Svi rezultati ispitivanja koji će biti prikazani u delu koji se odnosi na komparativnu analizu stvarnih i empirijskih čvrstoća pri pritisku odnose se na betone koji su isporučeni prilikom izrade mostovskih konstrukcija na brzoj saobraćajnici u Srbiji.

Za spravljanje betona korišćeni su sledeći komponentni materijali:

- agregat: prirodni rečni separisan na standardne frakcije: 0/4 mm, 4/8 mm, 8/16 mm i 16/31.5 mm,
- cement: CEM II/A-M(S-L) 42.5 R proizvođača „TITAN“ Srbija i CEM I 42.5R proizvođača „ANTEA CEMENT SH.A“ Albanija,
  CEM I je korišćen za prethodno-napregnute glavne nosače, dok je CEM II korišćen za sve ostale elemente konstrukcije,
- voda: bunarska,
- hemijski dodaci: superplastifikator SIKA Viscocrete-4077, aerant SIKA Control AER 100 RS i retarder SIKA Plastiment-20 R.

Na slici 1 dat je poprečni presek mostovske konstrukcije.



Slika 1. Karakterističan poprečni presek mostovske konstrukcije

U tabeli 1 prikazana su projektovana svojstva betona za pojedine pozicije na mostovskoj konstrukciji, kao i neki od osnovnih podataka vezanih za sastav betonskih mešavina.

Pozicija	Projektovana svojstva	Vodocementni faktor	Količina cementa	Uvučeni vazduh
		(-)	(kg/m^3)	(%)
Šipovi	C25/30 sa 400 kg/m^3 cementa	0.388	400	1-3
Naglavne grede	C25/30	0.500	350	1-3
Srednji stubovi	C25/30,V-2, M-1, MS-1	0.428	370	4.8-6.5
Obalni stubovi	C25/30,V-2, M-1, MS-1	0.428	370	4.8-6.5
Krilni zidovi	C25/30, V-2, M-1, MS-1	0.428	370	4.8-6.5
Kolovozna ploča, poprečni nosači, ležišne grede	C35/45, M-2, V-2, MS-2	0.375	440	4.8-6.5
Montažni prednapregnuti nosači	C35/45 i C40/50, M-2,V-2, MS-2	0.359 i 0.325	440 i 460	4.8-6.5
Prelazne ploče	C25/30	0.500	350	1-3
Ivični venci i pešačke staze	C35/45, M-2, V-2, MS-2	0.378	440	4.8-6.5
Temelji i obloga kegle	C25/30	0.548	300	1-3
Podložni beton	C16/20	0.548	270	1-3

Tabela 1. Projektovane betonske mešavine za ispitivanje čvrstoće pri pritisku

2.2. Betonske mešavine korišćene u komparativnoj analizi vodonepropustljivosti

Ispitivanja vodonepropustljivosti sprovedena su na dve različite betonske mešavine projektovanih klasa čvrstoća C25/30 i C35/45, koje ujedno predstavljaju i najčešće korišćene vrste betona u Srbiji [8]. Osnovni podaci o navedenim mešavinama dati su u tabeli 2.

Projektovana svojstva	Vodocementni faktor	Količina cementa	Uvučeni vazduh
	(-)	(kg/m^3)	(%)
C25/30, VDP	0.629	310	1-3
C35/45, VDP	0.513	380	1-3

Tabela 2. Projektovane betonske mešavine za ispitivanje vodonepropustljivosti

3. REZULTATI ISPITIVANJA

3.1. Čvrstoća pri pritisku

Uzorkovanje i negovanje betonskih uzoraka vršeno je na fabrici betona u skladu sa standardima SRPS EN 12350-1 [9] i SRPS EN 12390-2 [10] paralelno sa izgradnjom mostovske konstrukcije u periodu februar 2021 − jul 2023. godine. Ispitivanje čvrstoće pri pritisku vršeno je u skladu sa standardom SRPS EN 12390-3 [4] na betonskim kockama ivice 15 cm, starosti 28 dana. Svaki od 1251 rezultata ispitivanja predstavlja prosečnu vrednost dobijenu na dve betonske kocke.

S obzirom na to da je za svaku mešavinu bila poznata vrednost efektivnog vodocementnog faktora (ω), kao i vrste i količine upotrebljenih materijala, sračunate su očekivane čvrstoće pri pritisku koristeći empirijske obrasce date u tabeli 3.

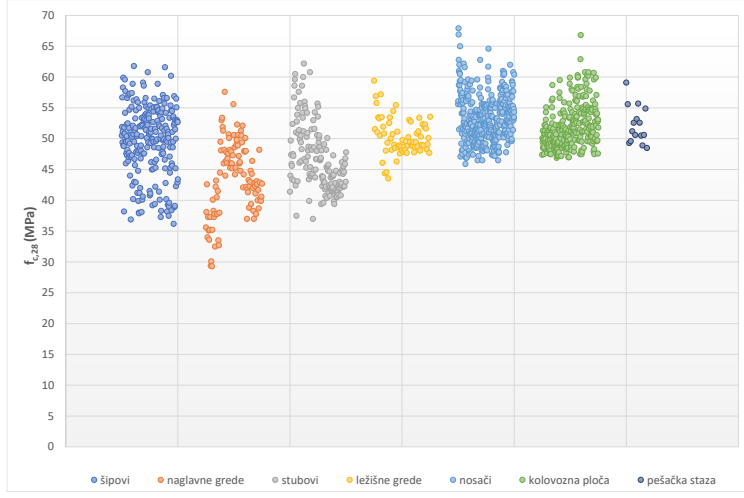
Naziv	Obrazac	Usvojeni parametri
Fere	$f_{c,28} = \frac{k}{\left(1 + \omega * \frac{\gamma_{sc}}{\gamma_{sv}}\right)}$	$k = 320$ $\gamma_{sc} = 3100\ kg/m^3$ $\gamma_{sc} = 1000\ kg/m^3$
Beljajev	$f_{c,28} = \frac{f_{pc}}{k * \sqrt{\omega^3}}$	$k = 4$ $f_{pc} = 42{,}5$
Bolomej	$f_{c,28} = \mathrm{A} * f_{pc} * \frac{1 - 0.5\omega}{\omega}$	$A = 0{,}6$ $f_{pc} = 42{,}5$
Skramtajev	$f_{c,28} = A_2 * f_{pc}(\frac{1}{\omega} + 0{,}5)$ за ω≤0,4 $f_{c,28} = A_1 * f_{pc}(\frac{1}{\omega} - 0{,}5)$ за ω≥0,4	$A_1 = 0{,}6;\ A_2 = 0{,}4$ $f_{pc} = 42{,}5$
Graf	$f_{c,28} = \frac{f_{pc}}{A * \omega^2}$	$A = 6{,}0\ (za\ CEMI)$ $A = 6{,}5\ (za\ CEMII)$ $f_{pc} = 42{,}5$

Tabela 3. Korišćeni empirijski obrasci za proračun računske čvrstoće pri pritisku betona

Na slici 2 su grafički prikazane sve sračunate čvrstoće pri pritisku betonskih uzoraka, dobijenih na osnovu izmerenih sila loma.

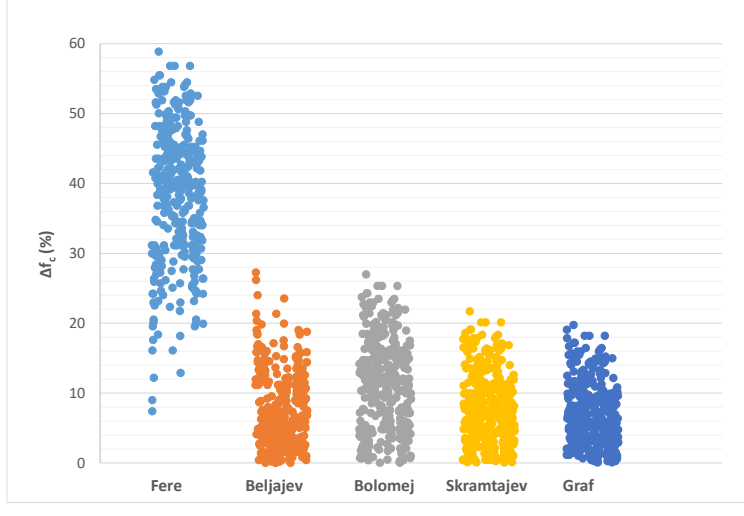
Ocena stepena tačnosti korišćenih empirijskih obrazaca sa konkretnim recepturama betona prikazana je sračunavanjem odstupanja čvrstoće pri pritisku u odnosu na stvarno izmerene vrednosti prema sledećoj formuli:

$$\Delta f_c = \frac{f_{c,stv} - f_{c,emp}}{f_{c,stv}} \cdot 100(\%).$$

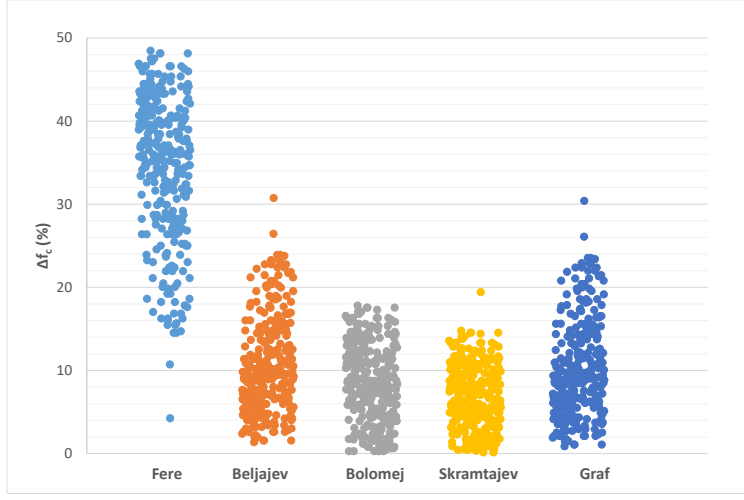


Slika 2. Dobijene čvrstoće pri pritisku betonskih kocki

Na slikama 3 i 4 grafički su prikazana predmetna odstupanja za reprezentativne primere betonskih mešavina spravljanih sa CEM I i CEM II cementima.



Slika 3. Odstupanja čvrstoća pri pritisku betonskih mešavina za poziciju prethodno napregnutih nosača (CEM I)



Slika 4. Odstupanja čvrstoća pri pritisku betonskih mešavina za poziciju kolovozne ploče (CEM II)

Za glavne nosače korišćen je beton projektovanih svojstava C35/45, V-2, M-2, MS-2 sa 440 kg/m^3 cementa CEM I i vodocementnim faktorom ω=0,359. Najveći stepen tačnosti pokazao je obrazac Grafa sa prosečnom vrednošću odstupanja od 7,1%, dok su prema ostalim obrascima prosečne vrednosti odstupanja iznosile: 7,3% Beljajev, 7,9% Skramtajev, 11,3% Bolomej i 38,3% Fere .

Za ležišne grede, kolovozne ploče, poprečne nosače i pešačke staze sa ivičnim vencima korišćen je beton projektovanih svojstava C35/45, V-2, M-2, MS-2, sa 440 kg/m^3 cementa CEM II, i vodocementnim faktorom ω=0,375. Najveći stepen tačnosti u ovom slučaju pokazao je obrazac Skramtajeva sa prosečnom vrednošću odstupanja od 6,7%, a odmah zatim i obrazac Bolomeja sa 8,3% i Grafa sa 10,2%, dok su prema obrascima Beljajeva i Ferea prosečna odstupanja iznosila 10,7% i 34,4%, respektivno.

3.2. Vodonepropustljivost

Kao što je prethodno navedeno, jedan od ciljeva ispitivanja prikazanih u ovom radu bio je da se da poređenje klasa vodonepropustljivosti definisanih standardima: SRPS.U.M1.015:1978 i SRPS.U.M1.206:2023 (koji daje ocenu klase vodonepropustljivosti betona ispitanog u skladu sa standardom SRPS EN 12390-8:2019).

Kako bi poređenje navedenih standarda, a samim tim i klasa vodonepropustljivosti definisanih ovim standardima bilo što jasnije, u tabeli 4 su uporedo prikazani svi relevantni podaci koji se odnose na ispitivanje betona prema gore navedenim standardima.

Standard	SRPS.U.M1.015:1978	SRPS EN 12390-8:2019
Godina objavljivanja	1978.	2010. (2019. novo izdanje)
Godina povlačenja	1998.	Trenutno važeći
Klasa vodonepropustljivosti	V-2, V-4, V-6, V-8, V-12	V-1, V-2, V-3
Potreban broj uzoraka	6 uzoraka	3 uzorka
Postupak ispitivanja	Početni pritisak 1 bar, a zatim se do dostizanja zahtevanog pritiska na svakih 8 h, pritisak poveća za 1 bar. Na primer, za klasu V-6: (8 h...1 bar, 8 h...2 bara, 8 h...3 bara, 8 h...4 bara, 8 h...5 bara, 8 h...6 bara)	72 h, 5 bara
Ocena postignute klase vodonepropustljivosti	Ne sme se zapaziti pojava kapi na gornjoj površini na pet od šest ispitanih uzoraka, s tim da je prvu nižu klasu zadovoljilo svih šest uzoraka	Najveći dozvoljeni prodor vode u svakom uzorku je: 5 cm za klasu V-1, 3 cm za klasu V-2, 2 cm za klasu V-3 (definisano standardom SRPS.U.M1.206:2023)

Tabela 4. Poređenje standarda za ispitivanje vodonepropustljivosti

Ispitivanje prodora vode pod pritiskom izvršeno je za dve betonske mešavine (C25/30 VDP i C35/45 VDP) na sledeća tri načina:

- prema SRPS.U.M1.015:1978 do maksimalnog pritiska od 6 bara,
- prema SRPS.U.M1.015:1978 do maksimalnog pritiska od 8 bara,
- prema SRPS EN 12390-8:2019.

Radi poređenja efekta metode, svi tretirani uzorci su pocepani po izvodnici u cilju određivanja dubine prodora vode. Dobijeni rezultati ispitivanja prikazani su u tabeli 5.

Kao što se vidi iz rezultata ispitivanja prikazanih u tabeli 5, betoni klase čvrstoće C35/45 imali su manje prodore vode u odnosu na betone klase C25/30, za istu metodu ispitivanja. Ovakvi rezultati su i logični s obzirom na to da beton više klase čvrstoće ima nižu vrednost vodocementnog faktora. Takođe, kada se porede vrednosti prodora vode pod pritiskom prema standardu SRPS U.M1.015:1978 za obe vrste betona, očekivani rezultati se dobijaju kada se uzme u obzir vrednost maksimalnog pritiska vode. Naime, uzorci koji su izlagani maksimalnom pritisku vode od 8 bara imali su 3,1 mm i 2,2 mm veće prosečne vrednosti prodora vode, kod betona klasa čvrstoće C25/30 i C35/45, respektivno.

Beton C25/30 VDP			Beton C35/45 VDP			Metoda ispitivanja
Prodor vode	Prosek	Max	Prodor vode	Prosek	Max	
h (mm)	h_{sr} (mm)	h_{max} (mm)	h (mm)	h_{sr} (mm)	h_{max} (mm)	
26	25,7	28	23	22,3	24	SRPS U.M1.015 (V-6)
22			24			
25			20			
26			24			
27			20			
28			23			
32	28,8	32	24	24,5	27	SRPS U.M1.015 (V-8)
31			23			
32			26			
25			25			
28			27			
25			22			
36	34,0	36	27	24,3	27	SRPS EN 12390-8
30			24			
36			22			

Tabela 5. Rezultati ispitivanja vodonepropustljivosti

Betonski uzorci koji su ispitivani u skladu sa standardom SRPS EN 12390-8:2019, prema standardu SRPS U.M1.206:2023 zavodoljavaju klasu V-2 (C35/45) i V-1 (C25/30). S obzirom da ni na jednoj seriji betonskih uzoraka koji su ispitivani u skladu sa standardom SRPS U.M1.015:1978 nije došlo do pojave kapi vode na gornjoj površini betona, može se konstatovati da su ispunjeni uslovi za klase vodonepropustljivosti V-6 i V-8, u zavisnosti od maksimalnog pritiska kom su uzorci izlagani.

Poredeći prosečne i maksimalne dubine prodora, za obe ispitivane betonske mešavine, može se zaključiti da su više vrednosti prodora vode dobijene u slučaju kada su uzorci izlagani vodi pod pritiskom u skladu sa standardom SRPS EN 12390-8:2019. Drugim rečima, ukoliko betonski uzorci zadovolje klasu vodonepropustljivosti V-2 prema standardu SRPS U.M1.206:2023, može se smatrati da će se najverovatnije zadovoljiti i klase V-6 i V-8 prema standardu SRPS U.M1.015:1978.

Ovde se napominje da se navedeni zaključak donosi na osnovu rezultata ispitivanja na ukupno dve betonske mešavine i da je za opštu primenu ovakvog zaključka potrebno sprovesti dodatna eksperimentalna ispitivanja na većem broju betonskih mešavina. Svakako u nedostatku drugih sličnih ispitivanja, ovakvi rezultati mogu biti dragocena smernica Izvođačima i Nadzornim organima u slučaju da se u projektnoj dokumentaciji definišu klase vodonepropustljivosti po povučenom standardu.

4. ZAKLJUČAK

U radu je prikazana komparativna analiza stvarnih čvrstoća pri pritisku dobijenih na kontrolnim betonskim kockama prilikom izgradnje mostovske konstrukcije na brzoj saobraćajnici u Srbiji i empirijski sračunatih vrednosti prema obrascima Ferea, Beljajeva, Bolomeja, Skramtajeva i Grafa. Ispitivanja su obuhvatila ukupno 1251 rezultat ispitivanja dobijen na deset različitih betonskih mešavina.

U slučaju betonskih mešavina spravljenih sa cementom tipa CEM I, najveći stepen tačnosti pokazao je obrazac Grafa, koji je za poziciju prethodno napregnutih nosača imao najveći stepen tačnosti od 7,1%, dok su prema ostalim obrascima prosečne vrednosti odstupanja iznosile: 7,3% Beljajev, 7,9% Skramtajev, 11,3% Bolomej i Fere 38,3%.

U slučaju betonskih mešavina spravljenih sa cementom tipa CEM II, najveći stepen tačnosti pokazao je obrazac Skramtajeva, koji je za poziciju kolovozne ploče imao prosečnu vrednost odstupanja od 6,7%, a odmah zatim i obrazac Bolomeja sa 8,3% i Grafa sa 10,2%, dok su prema obrascima Beljajeva i Ferea prosečna odstupanja iznosila 10,7% i 34,4%, respektivno.

U radu su takođe prikazani i rezultati ispitivanja vodonepropustljivosti betona prema povučenom standardu SRPS U.M1.015:1978 i prema trenutno važećem standardu SRPS EN 12390-8:2019. Dobijeni rezultati, za slučaj dva betona različitih klasa čvrstoće (C25/30 i C35/45) pokazali su veće prodore vode pod pritiskom u slučaju ispitivanja u skladu sa standardom SRPS EN 12390-8:2019. Takođe, na ispitivanim betonskim mešavinama betoni koji su zadovoljili klasu vodonepropustljivosti V-2 prema SRPS U.M1.206:2023, zadovoljavaju i klasu V-8 prema standardu SRPS U.M1.015:1978.

Zahvalnost

U radu je prikazano istraživanje koje je pomoglo Ministarstvo prosvete, nauke i tehnološkog razvoja Republike Srbije i Građevinski fakultet Univerziteta u Beogradu kroz ugovor o saradnji (broj 200092).

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CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

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THE INFLUENCE OF CRUMB RUBBER ON THE FREEZE-THAW RESISTANCE OF CONCRETE

Summary: *Waste tires are a persistent challenge in waste management. To reduce environmental impact and utilize this material more effectively, its use in concrete mixtures has been studied. This research aims to incorporate waste materials into concrete production while ensuring acceptable performance. The study presents results where fine aggregate was partially replaced with crumb rubber (2.5% and 5% by volume). All mixtures were designed with a w/c= 0.55 and compared with a control and a mixture with a chemical air-entraining admixture. Fresh concrete properties were monitored after 10, 30, and 60 minutes. After 60 minutes, rubber-modified mixtures showed increased air content and decreased density. The hardened concrete with rubber had lower compressive strength but better resistance after 200 freeze-thaw cycles compared to the control.*
Keywords: *crumb rubber, air content, compressive strength, frost resistance.*

UTICAJ DROBLJENE GUME NA OTPORNOST BETONA PRI SMRZAVANJU I ODMRZAVANJU

Rezime *Otpadne gume predstavljaju trajan izazov u upravljanju otpadom. Radi smanjenja ekološkog uticaja i efikasnijeg iskorišćenja ovog materijala, istražena je njegova primena u betonskim mešavinama. Cilj ovog istraživanja je uključivanje otpadnih materijala u proizvodnju betona uz obezbeđenje zadovoljavajućih performansi. U radu su prikazani rezultati u kojem je sitni agregat delimično zamenjen drobljenom gumom (2,5% i 5% po zapremini). Sve mešavine su projektovane sa w/c=0,55 i upoređene sa kontrolnom i mešavinom sa hemijskim dodatkom aeranta. Osobine svežeg betona su praćene nakon 10, 30 i 60 minuta. Nakon 60 minuta, kod mešavina sa gumom zabeležen je veći sadržaj vazduha i manja zapreminska masa. Očvrsli beton sa dodatkom gume imao je manju čvrstoću pri pritisku, ali bolju otpornost nakon 200 ciklusa smrzavanja i odmrzavanja u poređenju sa kontrolnom mešavinom.*
Ključne reči: *drobljena guma, sadržaj vazduha, čvrstoća pri pritisku, otpornost na dejstvo mraza.*

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1. INTRODUCTION

Globally, the accumulation of waste tires (WRT) continues to grow, with approximately 1.5 billion tires discarded annually, posing significant environmental and logistical challenges [1]. In the European Union, 2.135 million tons of WRT were produced in 2020, averaging 4.815 kg per capita [2]. In Serbia, data from the Environmental Protection Agency (SEPA) show that around 360,000 tons of tires were introduced to the market between 2011 and 2021, while only about 50,000 tons were treated in compliance with regulations, and a negligible quantity ended up in landfills.

Due to their long degradation period—often between 50 and 80 years—waste tires are classified among special waste streams that require regulated collection, transportation, and treatment [3]. Among various disposal strategies, material reuse in construction, particularly in asphalt and concrete, is considered the most sustainable and environmentally friendly solution [4–6]. The use of crumb rubber in concrete, forming rubberized concrete (RC), has shown beneficial effects on durability, including improved freeze-thaw resistance [7–9], reduced chloride permeability, and better acid resistance, making it suitable for aggressive environments such as marine infrastructure [10].

Additionally, crumb rubber contributes to the thermal insulation of concrete and reduces CO_2 emissions, thereby enhancing the sustainability of construction materials [9]. Despite these benefits, knowledge gaps remain regarding the water impermeability and absorption characteristics of rubberized concrete. This study explores the potential of replacing fine aggregate with crumb rubber at 2.5% and 5% volumetric ratios. Four mixtures were designed and assessed for fresh-state properties (slump, bulk density, and air content) at intervals of 10, 30, and 60 minutes. After 28 days, hardened concrete was evaluated for water absorption and impermeability. The objective is to develop rubberized concrete that could serve as an alternative to chemical air-entraining admixtures, contributing to sustainable waste management and concrete durability in accordance with Net-Zero environmental goals.

This study investigates the potential application of crumb rubber in amounts of 2.5% and 5% as a volumetric replacement for fine aggregate in concrete mixtures. All mixes were prepared with a water-cement ratio of 0.55, and the testing program included the evaluation of both fresh and hardened concrete properties. In addition to standard compressive strength testing, special emphasis was placed on assessing the concrete's resistance to 200 cycles of alternating freezing and thawing. The results were compared with those obtained from a reference concrete mixture without crumb rubber, as well as with a mixture containing a chemical air-entraining admixture, aimed at enhancing freeze-thaw resistance.

2. MATERIALS AND CONCRETE MIXTURE

2.1. Component Materials

In this study, concrete mixtures were prepared using crumb rubber (CRC) as a partial volumetric replacement for fine aggregate in proportions of 2.5% and 5%. All concrete mixes were designed with a constant water-cement ratio of 0.55. The materials used included ordinary Portland cement, natural crushed stone aggregates (fractions 0/4 mm, 4/8 mm, and 8/16 mm), crumb rubber particles, a superplasticizer, and potable water.

The cement used was Ordinary Portland Cement (OPC) of strength class 42.5 R, produced by Lafarge BFC. It had a specific gravity of 3110 kg/m³ and a specific surface area of 4320 cm²/g. The crumb rubber was supplied by ECO-RECYCLING d.o.o., obtained through mechanical grinding of end-of-life tires. It had a specific gravity of 1170 kg/m³, with sharp-edged, smooth-surfaced particles and a grain size of 0/1 mm. The volume mass without voids ranged between 1050 and 1150 kg/m³, while the bulk density ranged from 260 to 460 kg/m³.

As a natural aggregate, crushed limestone was used in all three fractions (0/4 mm, 4/8 mm, and 8/16 mm), with a specific gravity of 2650 kg/m³ and a fineness modulus of 3.06. A high-range water-reducing admixture based on polycarboxylates (Sika Viscocrete 4077x) was included to improve workability. Tap water was used as the mixing water.

The particle size distribution of both natural aggregates (fine and coarse) and crumb rubber (CR) is illustrated in Figure 1.

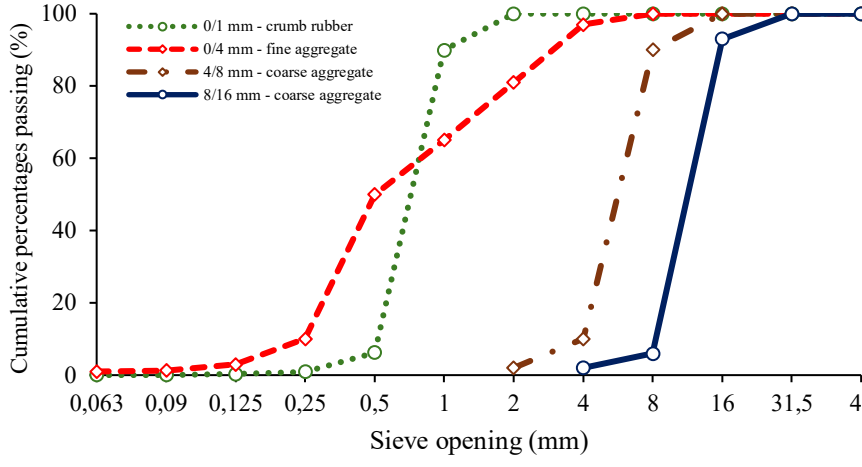


Figure 1. Particle size composition of crumb rubbers (CR) and natural aggregate

A chemical composition analysis of the cement and crumb rubber powder was performed using the Spectro Xepos system, identifying the main oxides and trace elements present in the materials (see Figure 2).

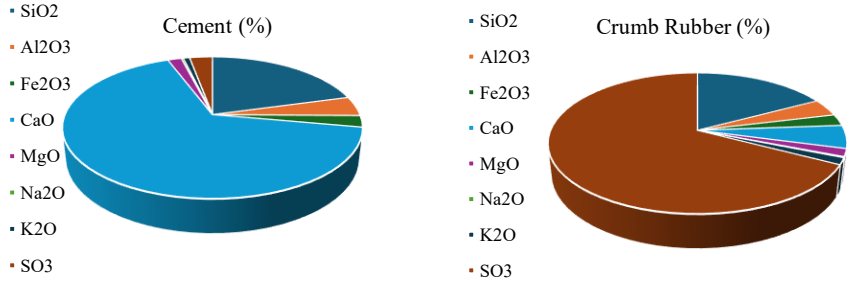


Figure 2. Chemical composition of cement and crumb rubber

2.2. Concrete Mixture

This study included four types of concrete mixtures: a reference concrete made with ordinary Portland cement (OPC), a concrete mixture containing a chemical air-entraining admixture (AER), and two mixtures with crumb rubber (CRC) used as a partial volumetric replacement of fine aggregate at levels of 2.5% and 5%. All mixtures were produced using a constant water-cement ratio of 0.55.

The reference concrete consisted of 45% sand (0/4 mm), 15% gravel (4/8 mm), and 40% gravel (8/16 mm).

The air-entrained concrete had the same aggregate composition as the reference mixture, with the addition of a chemical admixture designed to improve resistance to freeze-thaw cycles.

In the CRC mixtures, crumb rubber (0/1 mm) was introduced in amounts of 0.5% and 1.0%, replacing an equivalent volume of sand. As a result, the sand content was adjusted to 44.5% in the 2.5% CRC mix and 44.0% in the 5% CRC mix. The proportions of gravel remained unchanged—15% for the 4/8 mm fraction and 40% for the 8/16 mm fraction.

All mixtures were mixed for 90 seconds. The exact proportions of the constituent materials are provided in Figure 3.

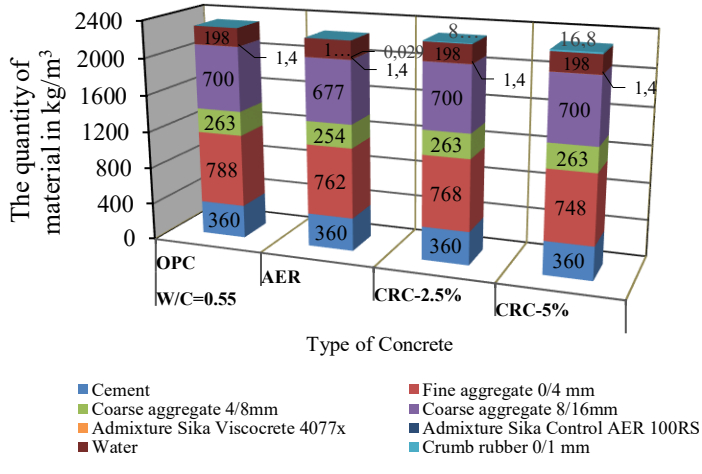


Figure 3. Mix design proportions of constituent materials for OPC, AER, CRC–2.5%, and CRC–5% concretes

3. RESULTS AND DISCUSSION

3.1. Properties of Fresh Concrete

The concrete mixtures were proportioned to achieve consistency class S4 (160–240 mm) according to the slump test, with measurements taken 10 minutes after water addition. Each mixture included 360 kg/m³ of cement, and the dosage of chemical admixture was adjusted to reach slump values within the range of 160 to 210 mm.

Fresh concrete was tested for slump (SRPS EN 12350-2), air content (SRPS EN 12350-8), and bulk density (SRPS EN 12350-6) at three intervals: 10, 30, and 60 minutes after mixing with water.

The results of these tests are shown in Figures 4, and 5, illustrating the development of slump, air content, and bulk density over time.

Slump test results (Figure 4) indicate the following:

At 10 minutes, all concrete mixtures meet the requirements of consistency class S4 (160–210 mm).

After 30 minutes, a reduction in slump is observed. Mixtures containing crumb rubber as a partial replacement for fine aggregate (CRC) fall into consistency class S3 (130–150 mm), while the reference concrete (OPC) and the concrete mixture containing a chemical air-entraining admixture (AER) remain within consistency class S4 (160–210 mm).

After 60 minutes from water addition, the mixtures with 2.5% and 5% volumetric replacement of fine aggregate with crumb rubber (CRC–2.5% and CRC–5%) exhibited consistency within class S2 according to the slump test method. The mixture containing a chemical air-entraining admixture (AER) maintained consistency class S3, while the reference concrete without additives (OPC) remained in consistency class S4.

Overall, a declining trend in the consistency of fresh concrete over time was observed across all mixtures based on slump test measurements.

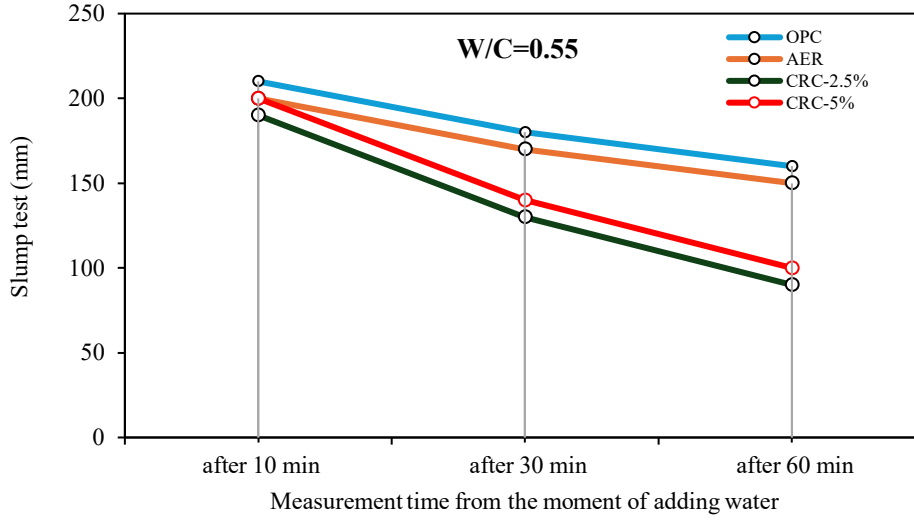


Figure 4. Obtained values of consistency of fresh concrete by slump test for Type of Concrete OPC, AER, CRC-2.5% and CRC-5%, w/c=0.55

As shown in Figure 5, the bulk density of fresh concrete varied over time depending on the composition of the mixture. The reference concrete (OPC) and the mixture containing a chemical air-entraining admixture (AER) exhibited a slight increase in bulk density within the time interval from 10 to 60 minutes after water addition. In contrast, the mixtures with 2.5% and 5% volumetric replacement of fine aggregate with crumb rubber (CRC–2.5% and CRC–5%) showed a gradual decrease in bulk density over the same period. The most pronounced reduction was observed in the CRC–5% mixture, where the

bulk density after 60 minutes was 3.8% lower compared to OPC, while the decrease in CRC–2.5% and AER mixtures amounted to 3.0%. Regarding air content, the CRC mixtures exhibited a significant increase over time, reaching 4.8% after 60 minutes, which is 2.8% higher than the final value measured in OPC concrete. The AER mixture reached an air content of 2.3%, representing a moderate increase of 0.3 percentage points. On the other hand, the OPC concrete showed a gradual reduction in air content from 2.3% to 2.0%, indicating a lower capacity to retain entrained air compared to the other mixtures. The obtained results highlight the significant influence of both chemical admixtures and recycled materials on the stability of fresh concrete properties.

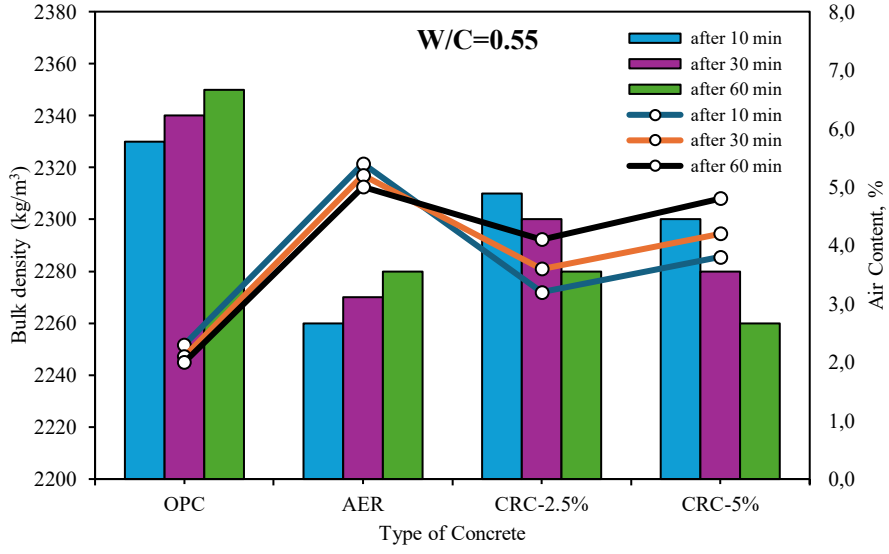


Figure 5. Obtained values of bulk density and air content for Type of Concrete OPC, AER, CRC-2.5% and CRC-5%, w/c=0.55

3.2. Properties of Hardened Concrete

After evaluating the properties of fresh concrete, cubic specimens with an edge length of d = 150 mm were prepared and subjected to water curing at a temperature of +20 °C until the time of testing. The physical and mechanical properties of the concrete, including bulk density and compressive strength, were evaluated at 3, 7, and 28 days of age, while frost resistance was assessed on a dedicated series of specimens subjected to cyclic freeze–thaw exposure. To assess long-term durability, all types of hardened concrete mixes were exposed to alternating freeze–thaw cycles in accordance with the national standard SRPS U.M1.206:2023. After 200 cycles, the specimens were examined to determine their resistance to frost-induced deterioration.

3.2.1. Bulk Density

Bulk density is a key parameter that enables the establishment of a functional correlation between the mechanical properties of hardened concrete primarily its

compressive strength. The determination of bulk density was performed on cube-shaped specimens with edge dimensions of d = 150 mm. The value was calculated as the ratio of the specimen's mass to its volume, with volume determined using hydrostatic weighing. The obtained bulk density values provide insight into the porosity of the concrete matrix and serve as an indicator of the expected mechanical performance of the hardened material.

The evaluation of bulk density over time was conducted at concrete ages of 3, 7, and 28 days, with the corresponding results presented in Figure 6.

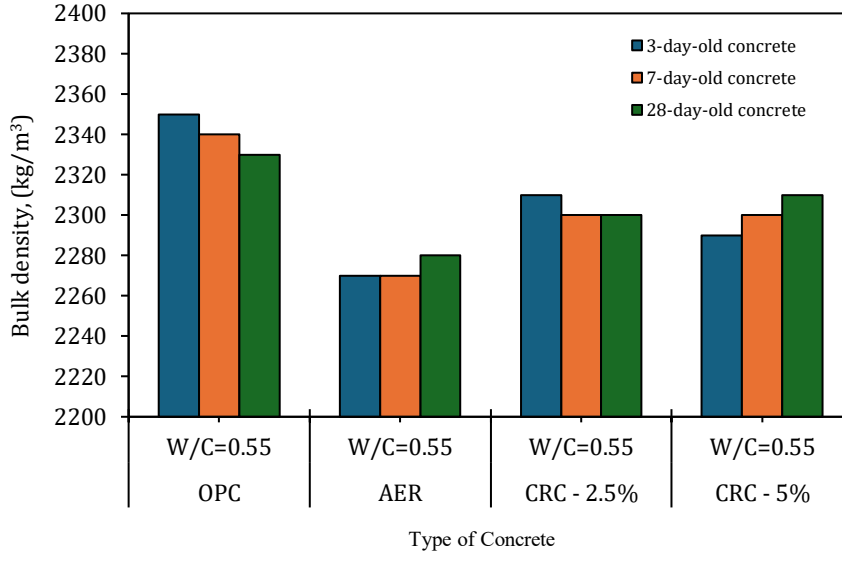


Figure 6. Obtained values of bulk density for Type of Concrete OPC, AER, CRC-2.5% and CRC-5%, w/c=0.55

Compared to the reference OPC concrete, all alternative mixes (AER, CRC–2.5%, and CRC–5%) exhibited lower bulk density values at all tested ages. The AER concrete showed the most significant decrease, with approximately 3.0% lower density than OPC, indicating a strong effect of the chemical air-entraining admixture. The CRC–2.5% mix showed a slightly smaller reduction (~1.5%), while CRC–5% had a further decrease (~3.0% lower than OPC) due to the higher replacement of fine aggregate with crumb rubber. Over time, the bulk density of the AER mix remained the lowest, while CRC mixtures showed a slight increase. At 28 days, CRC–5% exceeded CRC–2.5%, suggesting potential microstructural densification and improved integration of crumb rubber particles within the cement matrix.

3.2.2. Compressive Strength

Compressive strength testing was conducted on cube specimens with edge dimensions of d = 150 mm (Figure 20), following water curing at 3, 7, and 28 days of age. The tests were performed using a digital hydraulic testing machine with a maximum capacity of 3000 kN. The load was applied continuously at a rate of 0.6 MPa/s until failure. All specimens exhibited a uniform and typical fracture pattern upon reaching the maximum load.

After the test was completed, the results were obtained and are presented for all types of concrete, as shown in Table 1.

Type of Concrete	Water–cement ratio	Compressive strength, (N/mm^2)		
		Age of 3 days	Age of 7 days	Age of 28 days
OPC	W/C = 0.55	32.0	43.4	56.5
AER	W/C = 0.55	28.8	38.4	48.8
CRC - 2.5%	W/C = 0.55	30.2	39.5	49.6
CRC - 5%	W/C = 0.55	26.1	36.5	44.4

Table 1. Obtained values of compressive strength for Type of Concrete OPC, AER, CRC-2.5% and CRC-5%, w/c=0.55

Compared to the reference OPC concrete, all alternative mixes (AER, CRC–2.5%, and CRC–5%) exhibited reduced compressive strength values at all tested ages. In the case of AER concrete, compressive strength was lower by 10.0%, 11.5%, and 13.6% at 3, 7, and 28 days, respectively, indicating a consistent negative effect of aeration on strength development. The CRC–2.5% mixture showed a smaller reduction relative to OPC—5.6% (3 days), 9.0% (7 days), and 12.2% (28 days)—suggesting that a moderate addition of crumb rubber has a less pronounced impact on strength. The greatest reduction was observed in the CRC–5% mixture, with compressive strength decreases of 18.4%, 15.9%, and 21.4%, respectively, confirming the significant adverse effect of higher crumb rubber content on the mechanical performance of concrete.

3.2.3. Freeze–Thaw Resistance of Concrete

The freeze–thaw resistance of concrete was determined as the ratio of the compressive strength of specimens subjected to freeze–thaw cycles to that of reference specimens of equivalent age not exposed to such cycles, in accordance with SRPS U.M1.206:2023, Annex D. Cube-shaped specimens with edge dimensions of 150 mm were air-cured for 24 hours and then water-cured until the time of exposure. Testing was conducted when the specimens were 74–75 days old, with a total of 200 freeze–thaw cycles performed in a digital chamber. Each cycle lasted 8 hours (4 hours of freezing in air at –20 °C, followed by 4 hours of thawing in water at +20 °C), with three cycles carried out per day. The reference specimens were continuously stored in water until the day of compressive strength testing.

The obtained compressive strength results for the reference specimens and after exposure to 200 freeze–thaw cycles are presented in Table 2.

Type of Concrete	Water–cement ratio	Compressive strength, (N/mm^2)	
		reference specimens	after 200 freeze–thaw cycles
OPC	W/C = 0.55	58.0	42.1
AER	W/C = 0.55	52.3	45.7
CRC - 2.5%	W/C = 0.55	52.9	40.7
CRC - 5%	W/C = 0.55	47.6	42.1

Table 2. Compressive strength of OPC, AER, CRC–2.5%, and CRC–5% concretes (w/c = 0.55) for reference specimens and after 200 freeze–thaw cycles.

The obtained compressive strength ratios between the reference specimens and those subjected to 200 freeze–thaw cycles are presented in Figure 7.

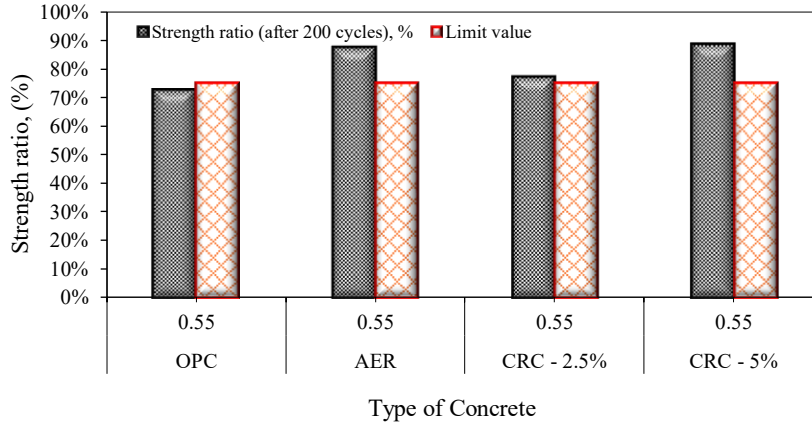


Figure 7. Compressive strength ratio of reference specimens and specimens exposed to 200 freeze–thaw cycles for OPC, AER, CRC–2.5%, and CRC–5% concretes

According to the testing results, concrete is considered resistant to freeze–thaw action if the strength ratio after 200 cycles is greater than or equal to 75%. The reference OPC concrete exhibited a strength ratio of 73%, falling below the specified threshold, indicating its limited durability under freeze–thaw conditions. In contrast, all modified concretes — AER, CRC–2.5%, and CRC–5% — achieved strength ratios above the required limit, thereby meeting the criteria for freeze–thaw resistance. Notably, the CRC–5% mix demonstrated the highest strength retention (95%), suggesting that the incorporation of crumb rubber at higher replacement levels can significantly enhance the freeze–thaw durability of concrete, likely due to the elastic properties of crumb rubber and its ability to mitigate internal stresses within the cementitious matrix.

4. CONCLUSION

This study evaluated the effects of incorporating crumb rubber as a partial volumetric replacement for fine aggregate on the freeze–thaw resistance and mechanical properties of concrete. All mixtures were produced with a constant water–cement ratio of 0.55 and compared against a reference concrete and a mixture with a chemical air-entraining admixture.

The results demonstrated that the inclusion of crumb rubber in concrete leads to increased air content and reduced bulk density over time in the fresh state. Although the addition of crumb rubber caused a decrease in compressive strength at all tested ages, particularly at the 5% replacement level, its impact on freeze–thaw durability was significantly positive. After 200 freeze–thaw cycles, all modified concretes (AER, CRC–2.5%, and CRC–5%) exhibited compressive strength retention above the 75% threshold defined by SRPS U.M1.206:2023, Annex D, indicating satisfactory freeze–thaw resistance. The CRC–5% mix showed the best performance, with a strength ratio of 95%, outperforming both the control and the chemically air-entrained concrete.

These findings confirm the potential of crumb rubber as an effective material for enhancing freeze–thaw durability, offering a sustainable alternative to chemical admixtures and contributing to waste tire management.

Future research should focus on mixtures with higher crumb rubber content (above 5%) and evaluate their resistance under a greater number of freeze–thaw cycles (300 or more), to confirm long-term durability under real service conditions.

Acknowledgments

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

Slobodan Živaljević[1], Matija Zejak[2],
Radomir Zejak[3], Goran Gorašević[4]

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SOME MECHANICAL PROPERTIES OF SHOTCRETE AT HIGH TEMPERATURES

Summary: *Despite the importance of tunnel fire protection, relatively few studies are related to the behavior of shotcrete at high temperatures. The paper presents the results of uniaxial compressive strength tests conducted on shotcrete samples after exposure to temperatures ranging from 100 to 800 degrees Celsius and cooling (residual tests). A total of 59 cylindrical samples of shotcrete were cut with a rotary machine from panels with dimensions of 60x60x12cm. The panels were obtained from the construction site of the Vežeščnik tunnel on the Bar-Boljare highway, Montenegro, where the shotcrete was sprayed onto molds using the wet process technology. The results indicate a drop in uniaxial strength of up to 70%, a loss of mass of up to 13% and a drop in the speed of ultrasonic waves through the sample up to 50% in the tested temperature range.*
Keywords: *tunnels, shotcrete, temperature, compressive strength, mass loss*

NEKA MEHANIČKA SVOJSTVA MLAZNOG BETONA PRI VISOKIM TEMPERATURAMA

Rezime: *Uprkos važnosti problematike protivpožarne zaštite tunela, relativno mali broj istraživanja je vezan za ponašanje mlaznog betona pri visokim temperaturama. U radu su predstavljeni rezultati testova jednoaksijalne pritisne čvrstoće sprovedeni na uzorcima mlaznog betona nakon njihovog izlaganja temperaturi u opsegu od 100 do 800 stepeni celzijusa i hlađenja (rezidualni testovi). Ukupno 59 cilindričnih uzoraka mlaznog betona je izrezano rotacionom mašinom iz panela dimenzija 60x60x12cm. Paneli su pribavljeni sa gradilišta tunela Vežeščnik na auto putu Bar-Boljare, Crna Gora, gdje je mlazni beton bio ugrađen u pripremljene kalupe tehnologijom mokrog postupka. Rezultati ukazuju na pad jednoaksijalne čvrstoće do 70%, gubitak mase do 13% i pad brzine ultrazvučnih talasa kroz uzorak do 50% u ispitanom opsegu temperatura.*
Ključne reči*: tuneli, mlazni beton, temperatura, pritisna čvrstoća, gubitak mase*

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1. INTRODUCTION AND PROBLEM STATEMENT

It is well known that the effects of fires pose a particularly pronounced problem in tunnels compared to other types of structures. The enclosed space, heat accumulation, rapid fire spread and difficulty in extinguishing fires often lead to temperatures that very quickly exceed 1000 degrees Celsius [3, 4, 6, 8]. Naturally, such conditions have a detrimental effect on the tunnel structure, which consists of the primary lining, waterproofing layer and secondary lining [10]. In addition to the primary lining, recent practice has seen the secondary lining also constructed using sprayed concrete – shotcrete. This is done even in situations requiring rapid construction, cost-effective execution and improved bonding between the primary and secondary linings [1].

While low water impermeability of shotcrete was a concern in the past, the issue is now resolved through the application of various waterproofing layers. However, another critical challenge arises in the form of fire resistance. Specifically, the high temperatures generated during tunnel fires induce spalling of shotcrete due to the vaporization of entrapped water within concrete. This process generates internal pressure, resulting in surface cracking and subsequent delamination. The loss of the protective layer, combined with damage and overheating of the reinforcement, inevitably weakens the mechanical properties of the shotcrete and compromises the integrity of the primary lining. Depending on the severity of the fire exposure, such conditions may jeopardize both the safety and serviceability of the entire tunnel structure.

Despite its current relevance and potential implications, limited data is available in the literature on behaviour of shotcrete under high-temperature exposure. To date, empirical models for uniaxial compressive strength of reinforced concrete have predominantly been applied [7] under the assumption that shotcrete and conventional reinforced concrete-both being stone aggregate and cement-based materials-exhibit a similar thermal response [1]. This approach, however, warrants scrutiny, as the mix design and placement methods for these two concrete types differ significantly.

This paper presents the results of uniaxial compressive strength, mass loss and ultrasonic pulse velocity tests conducted on shotcrete specimens after they were heated to temperatures ranging from 100 to 800 degrees Celsius, allowed to cool naturally/slowly and then tested (residual tests). The obtained results may serve to compare the behaviour of shotcrete with that of conventional reinforced concrete, evaluate its alignment with standardized constitutive models in the new MEST EN regulations and inform engineering practice.

2. LABORATORY TESTING OF THE MECHANICAL PROPERTIES OF SHOTCRETE

2.1. Specimen preparation

The shotcrete tested in this study was utilized in the construction of the primary and secondary linings of the Vežešnik tunnel, a structure located on the Bar-Boljare highway section. The wet-mix shotcrete technology was employed, with concrete sprayed into three prefabricated wooden molds measuring 600 x 600 x 120 mm. During application, the molds were positioned adjacent to one another and tilted at a 45˚ angle relative to the horizontal plane. The concrete used was supplied by the “Gornje Mrke”

concrete plant, operated by “BEMAX” LLC, Podgorica, Montenegro. The mix design is provided in table 1.

Components		Mass (kg)	Volume (m^3)
Cement		460	0.148
Water		200	0.200
Chemical additive	Sika 1020X	2.8	0.003
Entrained air		/	0.030
Fraction	0/4	1300	0.495
	4/8	325	0.124
Total		2288	1.000

Table 1. Concrete mix of the tested panels

Limestone aggregate was used. The water-cement ratio was 0.43, with a concrete strength class of C30/37, water impermeability class W8 and frost resistance class F100.

The resulting panels were cured on-site under relatively high humidity conditions within the tunnel environment for a minimum of 28 days, after which they were transported to the laboratory. Upon arrival, the panels were stored in a designated area and further cured at room temperature under constant humidity conditions

At 35 days of panel age, cylindrical specimens with dimensions of diameter/height d/h = 54/100 mm were extracted from the panels via coring (Figure 1). The specimens were categorized according to the panels from which they were cut, yielding 19 samples from the first panel, 26 from the second and 24 from the third, totaling 69 specimens. The sampling is illustrated in figure 2.



Figure 1. Panel 1 from which specimens were extracted

All core specimens were subsequently machined to achieve plane-parallel surfaces.

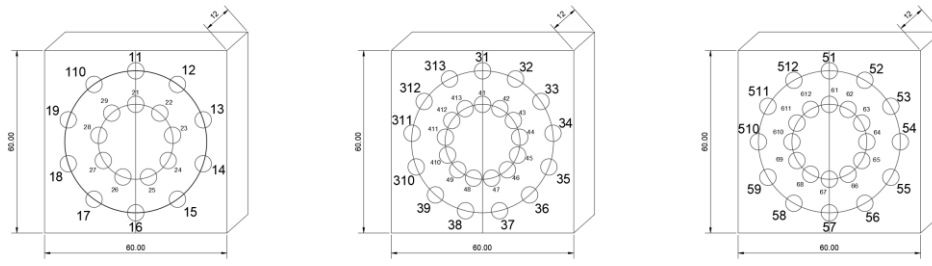


Figure 2. Specimen sampling scheme

2.2. Laboratory equipment and testing procedure

For specimen heating, a muffle furnace (shown in figure 3) was used. The furnace, manufactured by "ELEKTRON – Banja Koviljača" has a maximum operating temperature of 1100 degrees Celsius.

Figure 4 illustrates the experimental setup for determining the uniaxial compressive strength of the specimen. The press, manufactured in Russia, has a load capacity of 250 tons (2500 kN).

Ultrasonic pulse velocity testing through the specimens was conducted using an Ultrasonic TICO device manufactured by "PROSEQ SA – Switzerland".

The first series, comprising specimens from panel 1, was tested at an age of 42 days, while the second series, consisting of specimens from panels 2 and 3, was tested at an age of 49 days.

The testing procedure for the group of specimens not exposed to elevated temperatures (T = 20˚C) was conducted as follows: First, the specimen mass was measured, followed by the ultrasonic pulse velocity measurement along the longitudinal axis of the cylindrical specimen. Finally, the uniaxial compressive strength was determined using the hydraulic press.

The testing procedure for the group of specimens pre-exposed to elevated temperatures (T ≥ 100˚C) differed as follows: First, the specimen mass and ultrasonic pulse velocity were measured. Subsequently, the specimens were heated over a 24 hour period, repeated measurements of mass and ultrasonic pulse velocity were conducted, followed immediately by the determination of uniaxial compressive strength using the hydraulic press.

Given that intense fire exposure is typically estimated to last between 1.5 to 2 hours, which is shown in curves (RAB, HYD, RWS) [6, 8, 10] used to simulate fire effects in tunnel design, material fire resistance standards evaluate performance under short-term elevated temperature conditions. Accordingly, all tested shotcrete specimens exposed to elevated temperatures were subjected to the designated temperature for a duration of 2 hours, excluding the time required to attain the target temperature.

The specimens were cooled under laboratory-controlled conditions at a room temperature of approximately 20˚C and testing was performed after a 24-hour resting period. It should be noted that the equipment or capability to test specimens in a heated („hot tests") state was unavailable.

The following chapter presents the individual raw results obtained from testing, without specific statistical processing, grouping or „smoothing“ of the data, given the stochastic nature of the tested shotcrete's properties.

The results were subsequently visualized using Excel spreadsheets and presented in graphical form, where discernible trends and patterns could be identified for further discussion. This approach mitigated excessive data presentation, thereby avoiding overburdening the reader with extraneous details and minimizing the need for additional space.



Figure 3. Muffle furnace



Figure 4. Uniaxial compression strength test

3. RESULTS AND DISCUSSION

3.1. Table of tested specimens with results

The experimental results are compiled in Table 2.

Specimen label	T (˚C)	m_1 (g)	m_2 (g)	Δm (g)	Δm (%)	v_1 (m/s)	v_2 (m/s)	v_1/v_2	f_c (MPa)
11	0	538.3	538.3	0	0.00	-	-	-	47.59
12	0	526.6	526.6	0	0.00	-	-	-	41.15
13	0	527.6	527.6	0	0.00	-	-	-	16.37
14	0	518.4	518.4	0	0.00	-	-	-	29.04
15	0	525.11	525.11	0	0.00	-	-	-	21.40
16	0	520.1	520.1	0	0.00	-	-	-	34.93
17	0	533.8	533.8	0	0.00	-	-	-	27.07
18	150	529	507.9	21.1	3.99	-	-	-	15.28
19	150	529.7	509.3	20.4	3.85	-	-	-	17.68
110	150	526.8	505.5	21.3	4.04	-	-	-	28.38

21	300	521.4	491.5	29.9	5.73	-	-	-	9.61
22	300	530.6	498.4	32.2	6.07	-	-	-	16.59
23	300	538.6	508.4	30.2	5.61	-	-	-	21.40
24	500	532.6	495.6	37	6.95	-	-	-	17.25
25	500	528.1	495.5	32.6	6.17	-	-	-	13.75
26	500	533.4	489.1	44.3	8.31	-	-	-	22.92
27	800	533.2	465.8	67.4	12.64	-	-	-	10.92
28	800	526.1	458.5	67.6	12.85	-	-	-	7.20
29	800	531.6	462.2	69.4	13.05	-	-	-	8.95
33	300	493.5	474.3	19.2	3.89	4010	3380	0.84	30.65
35	200	491.2	471	20.2	4.11	4140	3670	0.89	35.15
36	0	497.4	-	-	-	3960	3960	1.00	37.11
37	700	503	459.4	43.6	8.67	4070	2290	0.56	14.15
38	500	498.7	472.5	26.2	5.25	4040	2460	0.61	18.00
311	700	497.8	446.7	51.1	10.27	3980	2230	0.56	12.43
312	0	501.2	-	-	-	4040	4040	1.00	34.06
41	500	497	467.7	29.3	5.90	4090	2240	0.55	15.65
44	0	495.5	484.80	10.7	2.16	4070	4070	1.00	38.58
46	300	499.7	477.6	22.1	4.42	4140	3280	0.79	24.43
47	500	503.2	474.7	28.5	5.66	4110	2350	0.57	20.15
49	200	500.6	471.8	28.8	5.75	4040	3510	0.87	29.58
411	200	495	476.3	18.7	3.78	4060	3520	0.87	23.58
413	0	494.2	-	-	-	4110	4110	1.00	36.44
51	0	500.1	-	-	-	3880	3880	1.00	26.20
52	500	486.8	463.9	22.9	4.70	3760	2220	0.59	11.15
53	700	488.3	460.6	27.7	5.67	3780	2130	0.56	13.50
54	0	496.9	-	-	-	3880	3880	1.00	32.75
55	300	496.7	481.2	15.5	3.12	3930	3430	0.87	26.79
59	700	487.7	455	32.7	6.70	3720	2290	0.62	10.72
511	300	506.7	490.3	16.4	3.24	3870	3180	0.82	30.65
512	200	493.3	481.3	12	2.43	3810	3540	0.93	27.86
62	700	492.5	443	49.5	10.05	3870	2010	0.52	10.07
64	200	499.5	484.8	14.7	2.94	3960	3590	0.91	32.15
65	0	501.1	-	-	-	3960	3960	1.00	39.73
66	500	501.5	475.6	25.9	5.16	3940	2220	0.56	19.72
610	300	502.6	485.1	17.5	3.48	3870	3140	0.81	17.58
611	0	503.4	-	-	-	3910	3910	1.00	36.68

Table 2. Results of tested specimen

– *T - temperature to which the specimen was heated*
– m_1 *- specimen mass prior to heating*
– m_2 *- specimen mass after heating*
– Δm *- loss of mass after heating*

– v_1 - *ultrasonic pulse velocity through the specimen prior to heating*
– v_2 - *ultrasonic pulse velocity through the specimen after heating*
– f_c - *uniaxial compressive strength of the specimen*

3.2. Uniaxial compressive strength

The results of the uniaxial compressive strength test are presented in figure 5. It should be noted that all specimens were sourced from the same batch, cured under identical conditions and tested using the same methodology, with the exception of the testing timeline: specimens from panel 1 were tested 7 days earlier. Despite these standardized conditions, the diagram reveals expected clustering of results, albeit with somewhat greater dispersion in the experimental data.

It can be observed that specimens from panel 1 exhibit slightly lower compressive strength compared to the other specimens, which may be attributed to their earlier age, as previously noted.

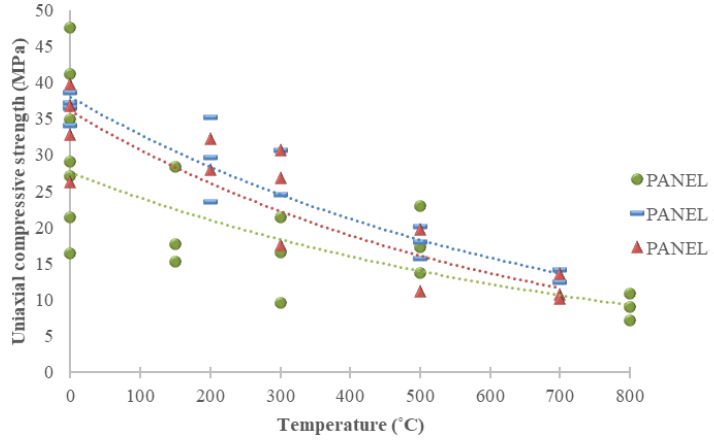


Figure 5. Diagram of compressive strength vs. Temperature for tested specimen

Figure 6 illustrates the relationship between the compressive strength of shotcrete at elevated temperatures and its compressive strength under normal temperature conditions (normalized diagram), which exhibits a hyperbolic distribution/trend. For comparative purposes, the diagram also includes values for reinforced concrete derived from so called “hot” tests, as recommended by Eurocode 1992-1-2 [7]. In these tests, uniaxial compressive strength is determined immediately after heating the specimen, without allowing it to cool.

As shown, the strength ratios of shotcrete specimens are lower than the prescribed values for reinforced concrete. The primary reason is that “hot” tests consistently yield higher uniaxial compressive strength values compared to those obtained from “cold” (residual) tests, which were conducted in this study. A parabolic approximation is also

discernible but was not confirmed in our experiments (limestone-based aggregate was used).

Since the diagram includes recommended curves from Eurocode 1992-1-2 [7] for two aggregate types, the experimental results should be compared to the limestone-based aggregate curve. Similar results are observed at baseline (room temperature) and maximum temperatures of 800˚C; however, significant deviations occur within the intermediate range, including differing trends in curve approximation.

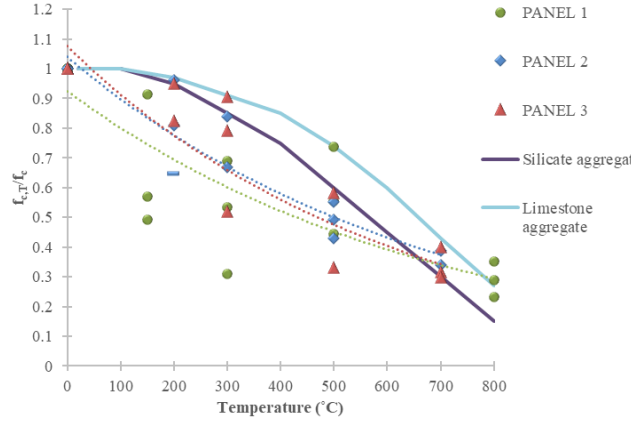


Figure 6. Ratio of compressive strength at elevated temperatures to reference compressive strength (20˚C)

3.3. Mass loss at elevated temperatures

Figure 7 presents the mean mass loss values of tested shotcrete specimens exposed to elevated temperatures, expressed as percentages. As clearly shown in the figure, specimens from panel 1 exhibit the highest mass loss, reaching up to 13%. This can be attributed to their higher initial mass at the time of testing compared to specimens from panel 2 and 3. Specifically, the average initial mass of panel 1 specimens was 530 g, approximately 30 g greater than the average initial mass of panels 2 and 3 (500 g). This discrepancy may stem from improved compaction during the installation of panel 1.

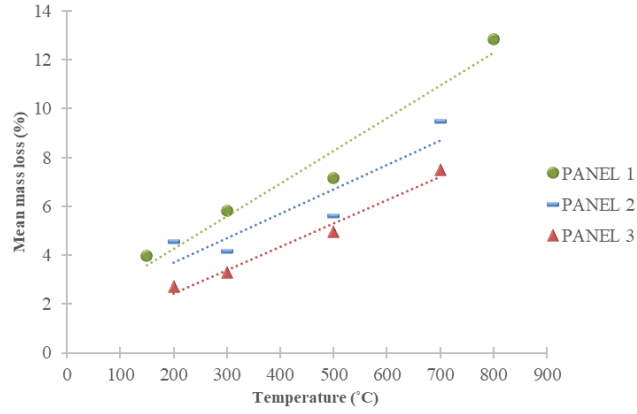


Figure 7. Diagram of mean mass loss values of specimens at elevated temperatures

3.4. Ultrasonic pulse velocity at elevated temperatures

Figure 8 illustrates the relationship between the ultrasonic pulse velocity through specimens at elevated temperatures and that under normal temperature conditions. This property was analyzed exclusively for specimens derived from panels 2 and 3. The diagram reveals a marked reduction in ultrasonic pulse velocity, reaching 44% for panel 2 specimens subjected to heating at 700˚C. The results demonstrate a clear trend, explaining the significantly increased porosity of specimens treated at elevated temperatures.

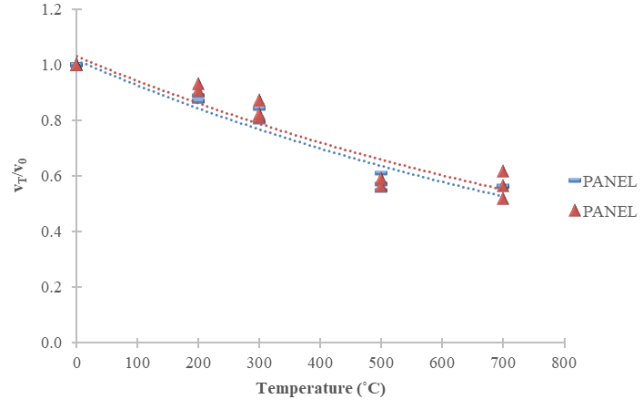


Figure 8. Ratio of ultrasonic pulse velocity at elevated temperatures to reference velocity (20˚C)

4. CONCLUSION

This study presents selected results from an experimental research of the influence of elevated temperatures on the mechanical properties of shotcrete, aiming to provide a deeper understanding of this phenomenon. The results indicate a reduction in the uniaxial compressive strength of shotcrete by up to 70%, a mass loss of up to 13% and a decline in ultrasonic pulse velocity through the specimens by up to 50% within the tested temperature range.

The initial hypothesis - that shotcrete exhibits distinct behaviour from conventional reinforced concrete under elevated temperatures due to short-term fire exposure-appears to be partially validated, at least in terms of the observed trends in compressive strength loss, mass reduction and increased porosity. However, the baseline and threshold values at 800˚C show reasonable agreement with the constitutive functions provided in Eurocode 1992-1-2.

The research conducted in this study can serve to supplement the limited existing database in this field while simultaneously providing practical value to engineers addressing these challenges in real-world scenarios related to the design and remediation of shotcrete structures.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
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EXPERIMENTAL INVESTIGATION OF THE INFLUENCE OF CHEMICAL ADMIXTURES ON THE BASIC PROPERTIES OF 3D PRINTED CONCRETE

Summary: *This study analyzes the impact of chemical admixtures on the basic properties of 3D printed concrete. The investigation was carried out using two commercial chemical admixtures, the superplasticizer SikaViscoCrete®-5500 Iron and the hydration controller SikaTard®-930, both manufactured by Sika. The experimental program included the testing of concrete properties in fresh and hardened states, using four different mixtures (named A, B, C, and D), which were prepared by varying the dosages of the used admixtures. The open time and extrudability were determined in the fresh state for all mixtures, and the extrudability test was used as an indicator of the printability of the concrete mixture. For selected mixtures (A and D), the bulk density and compressive strength was tested at various ages on mold cast and 3D printed samples. Based on the obtained results, an analysis of the impact of chemical admixtures was performed, highlighting the main conclusions regarding the effects of admixtures on the basic properties of 3D printed concrete.*
Keywords: *3D printed concrete, chemical admixtures, experimental investigation, density, open time, extrudability, compressive strength.*

EKSPERIMENTALNO ISPITIVANJE UTICAJA HEMIJSKIH DODATAKA NA OSNOVNA SVOJSTVA 3D ŠTAMPANOG BETONA

Rezime: *U ovom radu analiziran je uticaj hemijskih dodataka na osnovna svojstva 3D štampanih betona. Ispitivanje je sprovedeno korišćenjem dva komercijalna hemijska dodatka proizviđača Sika: superplastifikatora SikaViscoCrete®-5500 Iron i kontrolora hidratacije SikaTard®-930. Eksperimentalni program obuhvatio je karakterizaciju betona u svežem i očvrslom stanju. Analizirane su četiri mešavine (označene kao A, B, C i D), koje su dobijene različitim doziranjem navedenih dodataka. U svežem stanju, za sve mešavine određivani su vreme obrade i ekstrudabilnost, svojstvo kojem se ocenjuje kvalitet štampe betonske mešavine. Za odabrane mešavine (A i D) ispitivana je zapreminska masa i čvrstoća pri pritisku u različitim starostima, na livenim i 3D štampanim uzorcima. Na osnovu dobijenih rezultata izvršena je analiza uticaja hemijskih dodataka, pri čemu su istaknuti ključni zaključci u vezi sa njihovim efektima na osnovna svojstva 3D štampanih betona.*
Ključne reči: *3D štampani beton, hemijski dodaci, eksperimentalno ispitivanje, zapreminska masa, vreme obrade, ekstrudabilnost, čvrstoća pri pritisku.*

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1. INTRODUCTION

1.1. 3D concrete printing technology

3D concrete printing technology represents a modern way to produce concrete structures. It is based on the technology of 3D printing of plastic, where the printing material is a fresh concrete mixture. The intensive development and wider application of this technology began in the early 21st century, particularly with the development of the Contour Crafting technique [1,2]. Over time, driven by the growth of research institutions and increasing interest in automation in construction, numerous printing techniques have been developed and tested, including Fused Deposition Modeling (FDM), Inkjet Printing, and Shotcrete 3D Printing (SC3DP) [3–6]. The main part of this technology is the 3D printer with a mechanical system that provides movement of the printer according to an earlier prepared printing path. The fresh concrete mixture is extruded or sprayed in form of layers, making structural elements [1]. The main advantages of 3D concrete printing technology include increased construction speed without formwork, material savings, reduced labor, enhanced site safety, and the potential for sustainable development [2], [7–9]. There are several challenges and limitations, such as high initial investment, the need for skilled labor, insufficient knowledge about material performance, and lack of design and construction standards [10–12]. Despite these limitations, significant progress has been made in applying this technology in the construction industry. A variety of structures have been built using 3D printed concrete, including one-story and two-story buildings, pedestrian bridges, and individual elements such as columns, walls, and non-structural elements like sculptures, planters, and outdoor furniture [3].

1.2. The effect of admixtures on the basic properties of 3D printed concrete: State of the art

The literature review was prepared using the following methodology. Comprehensive review articles [13–15] focusing on the effects of chemical admixtures on the basic properties of 3D printed concrete were first identified and analyzed. These reviews served to find original research papers, which were then used to compile *Table 1*, and to present global application and impact of chemical admixtures. The *Table 1* summarizes the type of admixture, the applied dosage, and its influence on fresh and hardened properties of printed concrete. The influence of superplasticizers and hydration controllers on open time, extrudability, and compressive strength is only considered in this study.

Reference	Type of admixture	Name of admixture	Dosage [%]	Impact
Souza et al. [16]	Superplasticizer Retarder	Polycarboxylic ether (PCE) $C_{12}H_{22}O_{11}$, CSR	0.05-0.30 0.10	Higher content of PCE decrease buildability
Le et al. [17]	Superplasticizer Retarder	Polycarboxylate Methylene phosphonic and citric acids and formaldehyde	1.00 0.50	Increase of open time for 100 min
Chen et al. [18]	Water reducing agent Retarder	Polycarboxylate Tartaric acid (TA)	0.30 0.00-0.30	Higher content of TA increase open time and have no effect on compressive strength
Rahul et al. [19]	Superplasticizer	Polycarboxylic ether (PCE)	0.00-0.18	Higher content of PCE of 0.10% and more allowed good extrusion without any defects and have no effect on compressive strength

Table 1. Summary of experimental studies on the influence of chemical admixtures

The use of chemical admixtures with various dosages is essential in 3D concrete printing technology. The dosages of admixtures are often determined by the trial-and-error method. The optimal dosage of superplasticizers is around 0.30%, with values ranging from 0.05% to 1.00%. For hydration controllers or retarders, the optimal dosage is also 0.30%, with reported values between 0.10% and 0.50%.

1.3. Objective

This study analyzes the impact of two commercial chemical admixtures, superplasticizer Sika ViscoCrete®-5500 Iron and hydration controller SikaTard®-930 on the basic properties of 3D printed concrete. The open time and extrudability were determined in the fresh state for all mixtures. For selected mixtures (A and D), the bulk density and compressive strength was tested at various ages on mold cast and 3D printed samples.

2. EXPERIMENTAL PROCEDURE

2.1. Materials and mixtures proportion

For the preparation of the fresh concrete mixture, a ready-to-use product with the commercial name Sikacrete®-751 3D [20], manufactured by Sika, was used. Sikacrete®-751 3D is a one-component micro-concrete specifically designed for use in 3D concrete printing technology. The primary components of the premix are Portland cement, quartz flour, quartz sand, and specific chemical admixtures, with a maximum particle size of 1 mm. To obtain the fresh concrete mixture, the premix should be mixed only with the water, with the optional addition of other chemical admixtures.

In this study, two chemical admixtures were used: the superplasticizer Sika ViscoCrete®-5500 Iron and the hydration controller SikaTard®-930, both manufactured by Sika. ViscoCrete®-5500 Iron is a polycarboxylic polyether polymer designed for water reduction, shrinkage control, and the achievement of early strength. SikaTard®-930 is a liquid hydration controller specifically developed for the stabilization of concrete mixes for long periods without negatively influencing the quality.

The reasons for using the superplasticizer ViscoCrete®-5500 Iron in this was motivated by its ability to provide high early strength and reduce shrinkage in the printed elements. The use of the hydration controller SikaTard®-930 provides extended printing time (i.e. open time) for the concrete mixture and the high quality of the printed structure. *Table 2* presents the quantities of component materials – premix, water, and chemical admixtures for the four concrete mixtures considered in this study, which differ only in the amount of used admixtures.

Mixture	Premix [kg]	Water [kg]	ViscoCrete®-5500 Iron [kg]	SikaTard®-930 [kg]
A-ref	15.00	2.10	0.036 (0.24%)	0.012 (0.08%)
B	15.00	2.10	0.036 (0.24%)	0.041 (0.27%)
C	15.00	2.10	0.020 (0.13%)	0.012 (0.08%)
D	15.00	2.10	0.018 (0.12%)	0.041 (0.27%)

Table 2. Mixture proportions for considered concrete mixtures

Mixture A was designed according to the recommendation for the minimum amount of chemical admixtures. Mixtures B and C were designed by varying the dosage used in mixture A. In Mixture B, the amount of hydration controller admixture was increased more than three times, while in mixture C, the amount of superplasticizer was reduced by half. Through testing of properties in the fresh state—extrudability and open time for mixtures A,B and C, the optimal dosage of chemical admixtures was determined, resulting in mixture D.

2.2. Mixing and printing process, sample preparation and curing

The procedure for preparing the fresh concrete mixture and printing the samples is described in detail in the [21]. The preparation of the concrete mixture was carried out in several steps. First, 15 kilograms of the premix were placed into the mixer and mixed for 30 seconds. After that, water was gradually added over a period of 60 seconds, with a water-to-binder ratio of 0.14. Mixing then continued for an additional 240 seconds, during which the mixture transitioned from a dry to a plastic consistency, which is essential for the 3D printing process. The total mixing time was 330 seconds, or 5.5 minutes. The first university 3D printer for concrete in the Republic of Serbia was used for sample production. A circular nozzle with a diameter of 12.5 mm was used, with a printing speed of 1200 mm/min and a pump pressure of 4 bar.

The mold cast samples were obtained by pouring the fresh concrete mixture into prepared molds with dimensions of 100×100×100 mm, followed by vibration for 5 seconds. The 3D printed samples were manufactured by cutting plates formed through 3D concrete printing. The approximate dimensions of the plates were 500×500×100 mm *(Figure 1)*, while the target dimensions of the samples were identical to the mold-cast samples *(Figure 2)*. After printing, the plates were covered with wet blankets, and they were cut at the age of one day and placed into the water tank. The curing of both series was in the water tank.



Figure 1. 3D printed plates



Figure 2. Measumerent of 3D printed cube samples

2.3. Testing procedure

2.3.1. Density

The bulk density was tested in fresh and hardened state according to the SRPS EN 12390-7:2019 [22]. The test were conducted using mold-cast and 3D printed samples. Micro ruler was used to determine the average values of all dimensions (length, width, and height) for each 3D printed sample, which were then used to calculate the sample volume and bulk density.

2.3.2. Open time

The open time represents the period during which the mixture is workable, i.e. printable. To assess the open time, the results of the extrudability test were used. The criterion used followed the methodology described in [18], according to which open time is defined as the time interval during which the width of the extruded layer does not change by more than 10% from its initial value. It considered the width of the layer at the midpoint, specifically 200 mm from the start of the print because the ends of the layer were not representative for analysis, because the certain layer length is required to stabilize the pump pressure.

2.3.3. Extrudability

Extrudability is one of the main properties of 3D concrete printing technology, as it indicates the ability of a mixture to be successfully extruded through a nozzle in the desired form. Currently, there is no standardized method for testing this property. According to a review of the available literature, extrudability is most commonly tested by measuring dimensions of a single layer and a visual inspection of the shape of the layer. In this study, the methodology described in the [18] was applied to evaluate the extrudability of the mixture. After mixing, the fresh concrete mixture was put into the pump, and single layers with a length of 400 mm were printed with constant printing speed and pump pressure at various time intervals.

2.3.4. Compressive strength

The compressive strength was determined on mold-cast and 3D printed samples. For the testing of the printed samples, three samples were subjected to compressive forces applied perpendicular to the direction of the layers, while another three samples were tested with forces applied parallel to the layers (lateral direction). The testing was conducted in accordance with SRPS EN 12390-3:2014 [23] on an Amsler press with a capacity of 600 kN.

3. RESULTS AND DISCUSSION

3.1. Effects on physical-mechanical properties

The bulk density test was performed for reference mixture (mixture A) and for mixture with optimal dosing of chemical admixtures determined through the extrudability test and open time test (mixture D). The results in the fresh and hardened states as shown in *Table 3*. For printed samples from mixture A, the bulk density in the fresh state is equivalent to that of the mold-cast samples. The bulk density is slightly below 2200 kg/m³ for both mixtures and types of samples, which is typical values for mortar mixtures that don't contain coarse aggregate, like the used premix in this study. In this case, the differences in value of density between the mixtures were a few percent, suggesting that variation in admixture dosage had minimal influence on bulk density in the fresh and hardened states, as expected. Nevertheless, mixture A demonstrated excellent matching

between the mold-cast and printed samples in the hardened state. This indicates high print quality, with good compactness and minimal presence of voids and defects.

Mixture	Type of sample	Density – fresh state [kg/m^3] - Average	Density – hardened state [kg/m^3] - Average
A-ref	Mold-cast	2176 (σ=31)	2162 (σ=26)
A-ref	3D printed	2176 (σ=31)	2156 (σ=47)
D	Mold-cast	2149 (σ=16)	2143 (σ=18)

Table 3. Bulk density results of mold-cast and 3D printed samples for mixture A and D

Compressive strength test results for mixtures A and D are presented in *Table 4*. The results show that mixture A exhibited higher compressive strength for all ages of concrete. The most difference was observed at the age of 1 day, where mixture D demonstrated significantly lower strength, primarily due to the internal voids in the samples. These defects indicate insufficient compactness, resulting from a reduced dosage of superplasticizer, which produces a stiffer fresh mix and difficult production of samples. Compressive strength values for printed specimens in both testing directions are very close to the mold-cast samples, confirming the high print quality and showing that the layered structure did not affect strength. The higher value of compressive strength was obtained for the lateral direction, particularly at the ages of 1 and 7 days. Based on the obtained results, mixture A achieved strength class C40/50 for both types of sample, as well as class C35/45 for mixture D.

Mixture	Type of sample	Direction of force	Strength – 1 day [MPa] - Average	Strength – 7 day [MPa] - Average	Strength – 28 day [MPa] - Average
A-ref	Mold-cast	-	37.80 (σ=0.23)	56.73 (σ=1.80)	63.92 (σ=1.37)
A-ref	3D printed	Perpendicular	27.99 (σ=1.30)	42.57 (σ=1.55)	61.16 (σ=1.40)
A-ref	3D printed	Lateral	34.37 (σ=1.09)	55.21 (σ=0.99)	61.38 (σ=0.55)
D	Mold-cast	-	8.06 (σ=3.56)	44.26 (σ=4.03)	54.07 (σ=1.31)

Table 4. Compressive strength results of mold-cast and 3D printed samples for mixture A and D

3.2. Effects on printability

Figure 3 presents the results of the extrudability test for all considered mixtures. The geometry of the printed layers is shown at different time intervals, with a target layer width of 70 mm. It can be observed that mixtures with a higher dosage of superplasticizer (A and B) require more time to reach the target layer width. This behavior refers to the more time needed for the mix to develop sufficient stiffness, which resulte with desired width of printed layer. Furthermore, mixtures containing a higher amount of hydration controller (B and D) demonstrated a longer open time during which the layer width is close to the target value, indicating a more extended period suitable for quality printing of the mixture.

The calculation of the open time property is presented in *Table 5*. For all mixtures, the layer width at the midpoint (200 mm from the start of the print) was presented. The start of the open time interval was defined as the moment when the layer for the first time printed with a consistent width, without defects, and with a width that does not differ more than 10% compared to the target value. The end of the interval was marked by the point at which the layer width decreased by more than 10% compared to the initial width. The results clearly show that mixtures containing higher amounts of hydration controller (B and D) achieved a significantly longer open time—more than twice as long as that of mixtures with lower dosages (A and C).

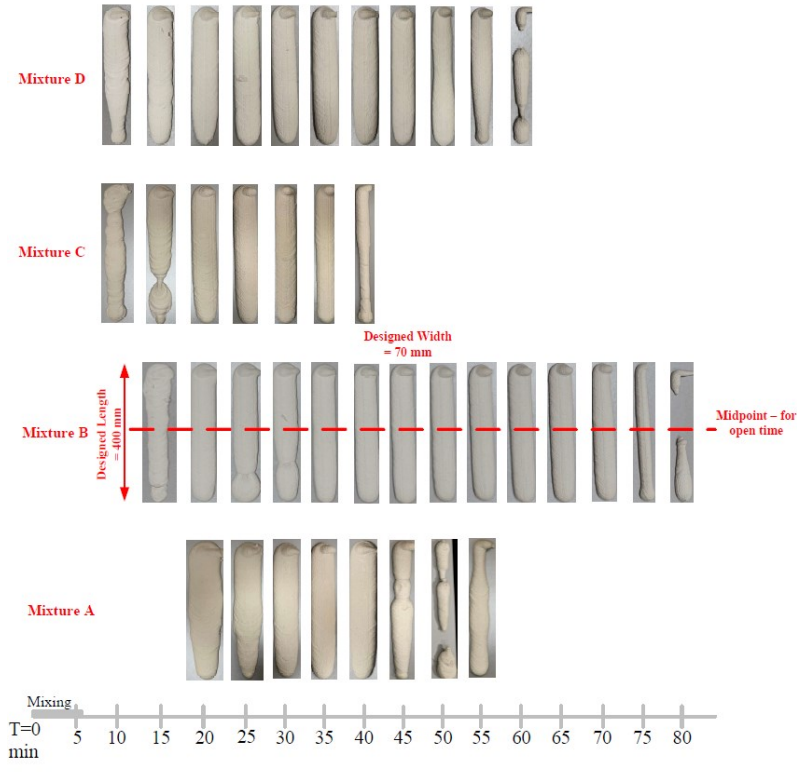


Figure 3. Extrudability test results for all considered mixtures

Mixture	Start [min]	Width of the layer on the start [min]	End [min]	Width of the layer on the end [min]	Open time [min]
A	30.00	77.04	40.00	69.24	10.00
B	35.00	70.65	70.00	62.81	35.00
C	15.00	69.52	30.00	64.88	15.00
D	20.00	75.40	45.00	66.64	25.00

Table 5. Detailed calculation of open time for each considered mixture

4. CONCLUSION

Based on the review of relevant literature and the results of the conducted experimental investigations, the following conclusions can be drawn:

- The technology of 3D concrete printing imlies the use of various chemical admixtures. These admixtures have a notable influence on print quality, as well as the physical and mechanical properties of the printed elements and structures. However, their application of admixture also leads to higher costs of this technology and requires a series of trial mixes to determine optimal dosage.
- The bulk density test indicated that variations in chemical admixture dosage had no significant effect on this property. Nevertheless, the compressive strength test obtained that a reduced amount of superplasticizer led to a stiffer fresh mix, which negatively affected workability and resulted in lower compressive strength. The high cement content in the premix contributed to achieving strength class C40/50 for mixture A and C35/45 for mixture D.
- Chemical admixtures have a significant influence on the open time and extrudability of printed concrete. An increased dosage of superplasticizer delays the point at which the mix reaches the optimal consistency for printing, while a higher amount of hydration controller extends the open time during which the mix can be extruded with consistent layer width and adequate quality. In the case analyzed, the optimum dosage of superplastisizer and hydration controler were 0.12% and 0.27%, respectively.

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

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BETON MODIFIKOVAN GRAFEN-OKSIDOM

Rezime: *S obzirom na to da je beton trenutno najrasprostranjeniji građevinski materijal, njegova čvrstoća i izdržljivost su njegove bitne karakteristike. Nedavna dostignuća u vidu primene vlakna nanodimenyije, kao što su ugljenične nanotube i grafen oksid, poboljšala su performanse betona. Grafen oksid (GO) je dvodimenzionalni planarni list komprimovanih atoma ugljenika u konfiguraciji saća debljine 0,335 nm. GO je obećavajući nanomaterijal zbog svojih jedinstvenih svojstava, naime, dobio je međunarodnu pažnju zbog svojih izvanrednih mehaničkih, optičkih i termičkih svojstava. Pored regulacije sekundarnog mehanizma hidratacije, dodavanjem GO betonu poboljšavaju se njegove mehaničke performanse, izdržljivost, samozarastanje i druge multifunkcionalne karakteristike. Ova studija predstavlja ispitivanje svojstava betona sa dodatkom GO. Pripremljena su četiri različita projekta betonske mešavine. GO je dodat u količinama od 0,01, 0,02, 0,03 i 0,06 %. Hidratacija je analizirana diferencijalnom termičkom analizom. Praćene su mehaničke čvrstoće u trajanju od 28 dana i otpornost na mraz sa solima za odleđivanje tokom 56 ciklusa.*

Ključne reči: *nano-additives; building materials; cement; microstructure; hydration; advanced properties.*

GRAPHENE OXIDE-MODIFIED CONCRETE

Summary: *Given that concrete is currently the most widely used building material, its strength and durability are essential qualities. Recent advances in new nanoscale fibers like as carbon nanotubes and graphene oxide have improved the performance of concrete. Graphene oxide (GO) is a two-dimensional planar sheet of compressed carbon atoms in a honeycomb configuration with thickness 0.335 nm. GO is a promising nanomaterial due to its unique properties, namely, it has received international attention for its outstanding mechanical, optical, and thermal properties. In addition to regulating the secondary hydration mechanism, adding GO to concrete improves its mechanical performance, durability, self-healing, and other multifunctional characteristics. This study presents an investigation of the properties of concrete with GO addition. Four different concrete mix designs were prepared. GO was added in quantities of 0.01, 0.02, 0.03, and 0.06 %. Hydration routes were analyzed by differential thermal analysis. Mechanical strengths were tracked in duration od 28 days and frost resistance with deicing salts during 56 cycles.*

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Keywords: *nano-additives; building materials; cement; microstructure; hydration; advanced properties.*

1. UVOD

Brzi razvoj savremene nauke i tehnologije, kao i kontinuirana težnja za višim kvalitetom života, podstiču razvoj materijala na bazi cementa ka novim kompozitima sa integrisanom strukturom, funkcionalnošću i inteligencijom. Materijali na bazi cementa predstavljaju najrasprostranjenije građevinske materijale u svetu zahvaljujući odličnim mehaničkim svojstvima i izdržljivosti. Međutim, s jedne strane, materijali na bazi cementa imaju slaba zatezna, savojna i deformaciona svojstva usled svoje krutosti. S druge strane, njihova funkcionalna svojstva često ne zadovoljavaju zahteve savremenog inženjerstva kada je reč o multifunkcionalnoj primeni građevinskih materijala.

Grafen, kao obećavajući dvodimenzionalni nanomaterijal, poseduje izuzetna mehanička i funkcionalna svojstva, uključujući visoku električnu i toplotnu provodljivost. Na primer, u pogledu mehaničkih svojstava, grafen dostiže zateznu čvrstoću veću od 50 GPa i modul elastičnosti od približno 1 TPa [1], pa njegovo dodavanje cementnim materijalima može značajno povećati žilavost i smanjiti njihovu krtost. U pogledu funkcionalnih karakteristika, pokretljivost elektrona i toplotna provodljivost grafena dostižu vrednosti od $2,5 \times 10^5$ $cm^2/(V\ s)$ [2] i 5300 W/(m K) [3]. Zbog toga, dodavanje grafena materijalima na bazi cementa može zadovoljiti specifične inženjerske zahteve, poput otapanja snega i leda u hladnim predelima [4], pregleda velikih objekata, kao i prevencije i ublažavanja katastrofa [5,6].

Zbog izraženih dvodimenzionalno-morfoloških i nukleacionih efekata proisteklih iz nanometarske veličine grafena [7], dolazi do formiranja mreže, unapređenja strukture i procesa hidratacije cementa, što rezultira zgusnutom unutrašnjom strukturom i poboljšanjem ukupnih svojstava cementnih kompozita [8,9].

Prethodna istraživanja uglavnom su bila usmerena na uticaj grafena na mehanička i funkcionalna svojstva cementnih materijala. Wei i Rhee i saradnici koristili su grafen za pripremu cementnih kompozita sa visokom toplotnom provodljivošću i ustanovili značajan porast toplotne provodljivosti [10,11]. Chu i saradnici su pokazali da je dodavanjem grafena došlo do neznatnog smanjenja mehaničkih svojstava, dok je toplotna difuzivnost povećana za 28% [12,13]. Neki istraživači su mešali nanometarske grafenske čestice u cementne kompozite i utvrdili da je prag perkolacije iznosio 15% u odnosu na masu cementa [14], a najbolja mehanička svojstva postignuta su pri sadržaju grafena od 0,80% mase [15]. Takođe je zabeleženo poboljšanje električne provodljivosti i smanjenje apsorpcije vode uvođenjem nano-grafena [16,17].

Dodavanje grafen oksida (GO), koji sadrži funkcionalne grupe sa kiseonikom, u cementne materijale takođe doprinosi poboljšanju njihovih svojstava. Studije su pokazale da GO značajno povećava nepropusnost i mehanička svojstva cementnih pasti i maltera [18–22]. Takođe je uočeno da GO ima izražen nukleacioni i "šablonski" efekat u cementnoj matrici, čime se podstiče hidratacija cementa i formiranje većih količina C-S-H gela, što odlaže širenje mikropukotina [23,24]. Ipak, oksidacija GO dovodi do uništavanja konjugovane kristalne strukture grafena, zbog čega GO gubi većinu svojih električnih i toplotnih karakteristika. Kao rezultat, njegova funkcionalna svojstva u cementnim materijalima nisu uvek jasno izražena.

Zbog toga su u primeni sve češće i grafenski nanolistovi (GNP), koji mehanički i funkcionalno unapređuju cementne materijale, dok njihovi derivati dodatno poboljšavaju strukturu i funkcionalna svojstva kompozita. Dodavanje ovih derivata dovelo je do povećanja zatezne čvrstoće od 100%, 144% i 132% u 3., 7. i 28. danu očvršćavanja, redom [25], dok su efikasnost elektromagnetne zaštite i apsorpcije povećane 1,6 i 7 puta [26].

Kompatibilnost i disperzija grafena predstavljaju ključne faktore za poboljšanje svojstava cementnih materijala, zbog čega je od suštinskog značaja pripremiti stabilnu vodenu disperziju. Najčešće metode za dispergovanje grafena uključuju jaku kiselinsku oksidaciju, tretman surfaktantima, ultrazvučni tretman i indukciju električnog polja [27–29]. Na primer, Lotia i saradnici su predložili metodu disperzije grafena tretmanom surfaktantom (SDS) sa koncentracijom od 0,3 mg/ml, uz ultrazvuk niske snage tokom 400 h i korišćenje natrijum-hlorata kao disperzanta, čime je postignuta ravnomerna raspodela grafena [27,30]. Opšte je prihvaćeno da je kombinacija surfaktanta i ultrazvuka efikasna metoda za pripremu disperzije grafena u cementnim materijalima [31]. Alternativno, Horszczaruk i Kim i saradnici su dodavanjem disperzanta i silicijum-dioksida u matricu dobili nanomaterijal sa ujednačenom disperzijom [32,33].

Iako je postignuta disperzija male količine grafena u cementnoj matrici [34], moguće je da koncentracija ne bude dovoljna za zadovoljenje zahteva kompozita na bazi cementa (betona), pa se ne postižu očekivani rezultati. Zbog toga se, radi prevazilaženja navedenih ograničenja, koristi grafen oksid (GO) kao zamena za grafen. GO se lako disperguje i ima dobru kompatibilnost zahvaljujući funkcionalnim grupama sa kiseonikom na svojoj površini.

Ova studija predstavlja ispitivanje svojstava betona sa dodatkom GO. Pripremljene su četiri različite betonske mešavine. GO je dodat u količinama od 0,01%, 0,02%, 0,03% i 0,06%. Hidratacija je analizirana diferencijalno-termičkom metodom, a praćena su mehanička svojstva tokom 28 dana i otpornost na dejstvo mraza i soli za odmrzavanje u trajanju od 56 ciklusa.

2. EKSPERIMENTALNI RAD

Za spravljanje betonskih mešavina korišćeni su sledeći mateijali:

- prirodni frakcionisani rečni agregat, frakcije 0/4, 4/8 i 8/16 mm, proizvođač "Hidro baza", Ostružnica,
- cement CEM I 42.5 R, proizvođača "Lafarge", Beočin,
- hemijski dodatak: superplastifikator, Sika Viscocrete 4077, "Sika Srbija", Beograd,
- grafen oksid
- voda iz vodovoda.

Spravljeno je pet betonskih mešavine i to: referentni beton (E) i četiri betona sa dodatkom GO u količinama od 0.01, 0.02, 0.03 i 0.06 % u odnosu na masu cementa (G1, G2, G3 i G6), Tabela 1. Sve mešavine su spravljene sa istom količinom komponentnih materijala osim grafen oksida. Udeo frakcija je bio sledeći: 0/4-45%, 4/8-15% i 8/16-40%.

Oznaka	Cement	Agregat	VSC 4077	Voda	w/c

E	360	1842	2.88	162	0.45

Tabela 1. Sastav referentne betonske mešavina (kg/m^3)

2.1. Svež beton

Izvršeno je ispitivanje sledećih svojstava betona u svežem stanju: konzistencije metodom sleganja, sadržaja uvučenog vazduha i zapreminske mase.

Rezultati ispitivanja su prikazani u Tabeli 2.

Mešavina	Konzistencija sleganjem (mm)	Zapreminska masa (kg/m^3)	Sadržaj vazduha (%)
E	110	2340	2.5
G1	50	2350	2.3
G2	60	2340	2.9
G3	50	2320	3.1
G6	65	2350	2.3

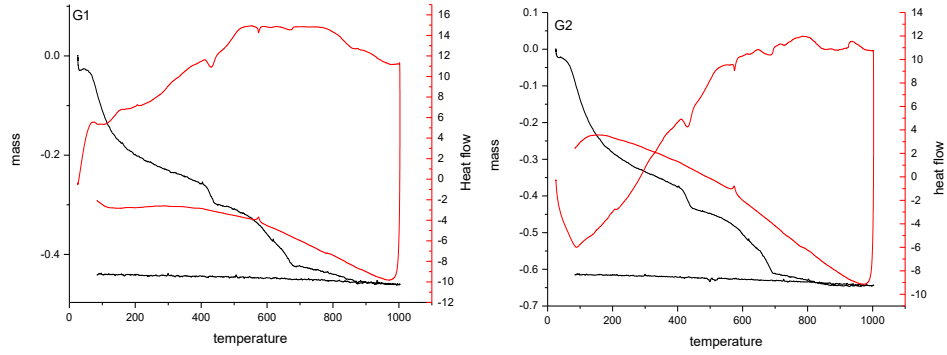
Tabela 2. Svojstava svežeg betona

2.2. Očvrsli beton

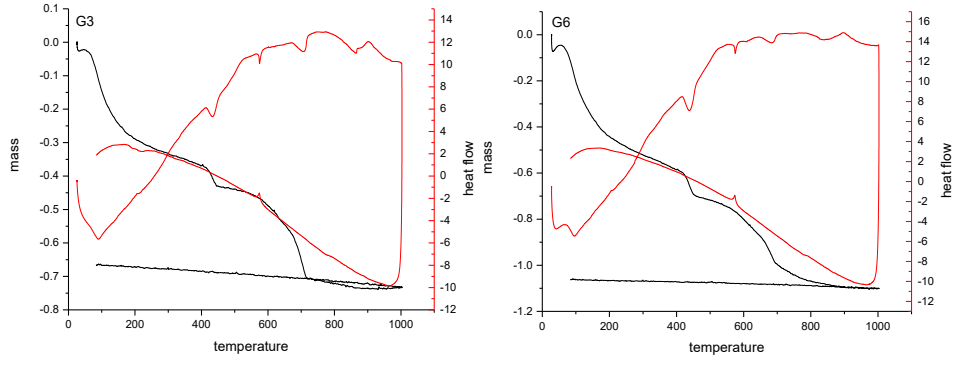
Smeše praha su podvrgnute diferencijalnoj termičkoj analizi (SETSIS TG/DTA/DSC, SETARAM Instrumentation), Slike 1,2.

Uzorci (30 g) su stavljeni u posudu od alumine i grejani od 20 °C do 1000 °C. Brzina zagrevanja je bila 10 °C/min.

Uzorak G1 je pokazao egzotermni efekat između 20 i 400°C. Uzorci G2-G6 umesto egzotermnog pokazuju endotermni efekat u pomenutom rasponu temperatura. Gubitak mase je približno isti za sva četiri razmatrana uzorka.



Slika 1. Diferencijalna termička analiza SETSIS TG/DTA/DSC, SETARAM Instrumentation, betona G1 i G2



Slika 2. Diferencijalna termička analiza SETSIS TG/DTA/DSC, SETARAM Instrumentation, betona G3 i G6

2.2.1. Čvrstoća pri pritisku

Urađeni su uzorci za ispitivanje čvrstoće betona pri pritisku. Beton je ugrađivan vibroiglom u kalupe oblika kocke, dimenzija ivica d=100 mm koji su očvršćavali u vodi na temperaturi od +20°C do trenutka ispitivanja prema standardu SRPS EN 12390-2 [9].

Ispitivanje čvrstoće betona pri pritisku u starosti od 1, 2, 7 i 28 dana sprovedeno je prema SRPS EN 12390-3 [10]. Rezultati su prikazani Tabeli 3.

Oznaka mešavine	Čvrstoća pri pritisku (N/mm^2) / Starost (dana)			
	1	2	7	28
E	29.0	37.2	44.1	55.2
G1	26.2	32.8	41.1	55.0
G2	27.1	35.1	42.4	49.1
G3	28.8	34.3	41.8	45.0
G6	27.0	34.2	41.6	48.3

Tabela 3. Čvrstoća pri pritisku betona u različitoj starosti

2.2.2. Otpornost betona na dejstvo mraza (u prisustvu soli za odmrzavanje)

Betonske konstrukcije izložene efektima zamrzavanja i odmrzavanja treba da budu trajne, da imaju pogodnu otpornost na ove uticaje, a u slučaju saobraćajnih konstrukcija – zamrzavanje i odmrzavanje u prisustvu soli za odmrzavanje. Ovo se primenjuje na betonske mešavine, betonske proizvode, prefabrikovani beton, betonske elemente ili beton spravljen na gradilištu.

Ukoliko beton nema pogodnu otpornost na zamrzavanje i odmrzavanje, tada će se ovakav uticaj odraziti na dva tipa oštećenja – ljuštenje (atmosfersko dejstvo na površinu) i unutrašnje oštećenje strukture.

Ispitivanje otpornosti površine za zamrzavanje i odmrzavnje betona se zasićava i sa vodom i sa NaCl rastvorom. Može se koristiti za poređenje novih konstitutivnih materijala ili novih Mešavina betona u odnosu na komponetne meterijale ili mešavine za koje se zna da daju pogodne performanse u lokalnom okruženju ili za prihvatanje ispitinih rezultata prema nekim apsolutnim numeričkim vrednostima zasnovanih na lokalnom iskustvu.

Za sprovođenje ovog ispitivanja iz izrađenih kocki ivice d=150 mm isečeni su pločasti uzorci, koji su izloženi zamrzavanju i odmrzavanju u prisustvu soli 3% rastvora NaCl. Otpornost na zamrzavanje i odrmzavanje se procenjuje merenjem oljuštenog materijala sa pločastih uzoraka posle 56 ciklusa. Rezultati su prikazani u Tabeli 4.

Oznaka mešavine		Srednje vrednosti količine oljuspanog materijala (kg/m^2)
28 ciklusa	E	0.55
	G1	0.00
	G2	0.00
	G3	0.00
	G6	0.00
56 ciklusa	E	1.02
	G1	0.05
	G2	0.06
	G3	0.05
	G6	0.00

Tabela 4. Srednje vrednosti količine oljuspalog materijala u kg/m^2

3. ZAKLJUČAK

U poređenju sa referentnom mešavinom, smanjenje čvrstoće pri pritisku kod mešavine G2 u starosti od 28 dana iznosi 11.0%, kod mešavine G3 je 18.5%, a kod mešavine G6 je 12.5%.

Na osnovu dobijenih rezultata ispitivanja može se zaključiti da referentna mešavina nije zadovoljila u pogledu otpornosti betona na dejstvo mraza u prisustvu soli za odmrzavanje. Svi uzorci sa dodatkom grafen oksida su zadovolji za klasu izloženosti betona XF4, a najbolje rezultate je imao beton sa 0.06% dodatka grafen oksida jer na njegovoj površini nije uočeno ljuspanje betona.

Može se zaključiti da je dodavanjem 0.01% grafen oksida u odnosu na masu cementa uz neznatno smanjenje čvrstoće pri pritisku na 28 dana dobijen beton koji zadovoljava MS-S2 u pogledu klase otpornosti na dejstvo mraza i soli za odmrzavanje. Dodavanjem 0.06% grafen oksida dobijen je beton kod koga nije uočeno ljuspanje posle 56 ciklusa, ali je čvrstoća pri pritisku na 28 dana smanjena 12.5%.

Zahvalnost

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
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WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

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SELF-HEALING CONCRETE: MATERIALS AND MECHANISMS

Summary: *Cracks in concrete structures represent a persistent issue that compromises their reliability and durability. Traditional repair methods are expensive and often impractical, especially for large-scale infrastructure projects. Self-healing concrete offers an innovative solution through autogenous and autonomous healing approaches. According to research, autogenous mechanisms are effective for cracks up to 150 µm, while autonomous methods can seal cracks up to 500 µm, improving mechanical properties, water tightness, and resistance to external influences. This paper analyzes materials, and mechanisms of self-healing concrete, highlighting its potential for sustainable and resilient infrastructure in the future.*
Keywords: *Cracks; Autogenous; Autonomous; Mineral Additives; Fibers; Nanofillers; Curing agents; Electrodeposition; SMA; Capsules; Vascular networks; Microbial technologies.*

SAMOZACELJUJUĆI BETON: MATERIJALI I MEHANIZMI

Rezime: *Prsline u betonskim konstrukcijama predstavljaju stalni problem koji ugrožava njihovu pouzdanost i trajnost. Tradicionalne metode sanacije su skupe i često nepraktične, posebno kod velikih infrastrukturnih pojekata. Samozaceljujući beton nudi inovativno rešenje zahvaljujući autogenim i autonomnim pristupima zaceljivanja. Prema istraživanjima, autogeni mehanizmi efikasni su za prsline do 150 µm, dok autonomne metode mogu zatvoriti prsline/pukotine širine do 500 µm, poboljšavajući mehanička svojstva, vodonepropusnost i otpornost na spoljašnje uticaje. Ovaj rad analizira materijale i mehanizme samozaceljujućeg betona, ističući njegov potencijal za održivu i otpornu infrastrukturu u budućnosti.*
Ključne reči: *Prsline; Autogeno; Autonomno; Mineralni dodaci; Vlakna; Nanofileri; Sredstava za unutrašnju negu betona; Elektrodepozicija; SMA; Kapsule; Vaskularne mreže; Mikrobiološke tehnologije.*

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1. INTRODUCTION

Concrete's adaptability and affordability make it the most popular engineering material in the world. This composite material consists of multiple components, phases, and proportions, but the process for its production is relatively simple. With mixed proportions that can be changed to satisfy needs, it is typically composed of cement, water, aggregates, and chemical and mineral additives. As urbanization continues to expand, particularly in developing countries, and the global population grows, concrete will remain the most produced and utilized construction material. Finding a sustainable alternative to concrete remains a major challenge [9]. Despite its widespread use, concrete has inherent weaknesses such as brittleness, low tensile strength, limited deformation capacity, and proclivity for multiple cracks. Crack formation is particularly detrimental because it compromises the structural integrity and load-bearing capacity of concrete elements. Over time, this can reduce the service life of concrete structures, increase maintenance costs, and have serious technical, economic, and environmental implications. Penetration of water, gases, and fluids containing aggressive chemicals into cracks accelerates material degradation. As modern infrastructure projects become increasingly large and complex, and are built in extremely aggressive environments, preserving durability of concrete becomes even more challenging [29].

Traditional repair methods are often cost- and labor-intensive, especially for large infrastructures. Damage can be hard to access due to crack location and operational demands, like in highways and tunnels. These challenges have resulted in the development of self-healing concrete, a material that possesses the inherent ability to repair structural damage autogenously or with little assistance from an outside stimulus. Using such innovative solutions, it is possible to significantly extend the lifespan and safety of structures while also preserving infrastructure [3]. The aim of the research presented in this paper is to define and classify self-healing concrete and review key approaches for achieving it.

2. SELF-HEALING CONCRETE: DEFINITION, CLASSIFICATION AND DEVELOPMENT

2.1. Definition and classification

To create a common framework for understanding and applying self-healing concepts, definitions for key terms have been provided. Self-healing is any process initiated by the material itself that leads to the recovery and improvement of material performance that had previously been reduced due to some form of damage [3]. Self-healing mechanisms can be classified as autogenous and autonomous:

- Autogenous self-healing mechanism refers to recovery processes that rely on components naturally present within the material, even if the material was not specifically designed for self-healing (own generic materials). Autogenous self-healing methods include the application of supplements, the most common of which are mineral additives, fibers, nanofillers and curing agents [3].
- Autonomous self-healing mechanism refers to recovery processes that involve engineered components not originally found in the material, specifically introduced to enable active healing (engineered additions). Autonomous approaches most often include

shape memory alloys, electrodeposition, capsules, vascular networks, and microbial technologies [3].

2.2. Historical development

The phenomenon of self-healing in cement-based materials has evolved significantly from early observations in the 19th century to advanced engineered solutions in the 21st century. Table 1 provides a summarized overview of key milestones and contributions that have shaped the development of self-healing concrete over time.

Year/Period	Event/Discovery	Researcher(s)/Institution
1836	First observation of self-healing in water-retaining structures	French Academy of Science [3]
Late 19th – Early 20th century	Reports of leakage reduction and natural crack sealing in hydraulic structures	Various engineers/researchers [3]
1930s–1980s	Experimental studies on moisture-driven crack healing and strength recovery	Loving [18], Wagner [28], Lauer and Slate [14], Hannant and Keer [10]
1970s	Observation of healing in hydraulic concrete	Ivanov and Polyakov [11]
1980s	Healing at the steel fiber–cement interface under water curing	Gray [7]
1980s–1990s	Studies on factors influencing cracks' natural ability to heal themselves: material properties, healing kinetics, crack width and moisture availability	Clear [1], Ripphausen [23], Meichsner and Wuenschig [19], Edvardsen [6]
1990s	Use of silica fume for self-healing in concrete subjected to freeze-thaw cycles	Jacobsen et al. [12]
Late 1990s	Concept of microencapsulation for self-healing in concrete	Dry and Corsaw [4] and Dry [5]
Early 2000s	Development of Engineered Cementitious Composites (ECC) with self-healing capabilities	Li V.C. [17]
Early 2000s	Introduction of bacteria-based self-healing concrete ("bioconcrete")	Jonkers [13]
2000s–2010s	Advanced studies on capsule systems, bacterial healing, and vascular networks	Schlangen, De Belie and Van Tittelboom [3]
2005–2012	Formalization of autogenic vs. autonomic healing concepts; systematic research coordination	RILEM Technical Committee 221-SHC [3]
2010s	Use of superabsorbent polymers (SAPs) and microfibres to enhance the self-healing capacity of cementitious materials	Snoeck et al. [26]
Today	Rapid expansion of engineered self-healing technologies; growing focus on sustainable infrastructure	Global research community

Table 1. Historical development of self-healing concrete approaches

3. AUTOGENOUS SELF-HEALING MECHANISM

Numerous processes contribute to concrete's autogenous healing of cracks, which refers to the material's innate capacity to plug micro and macrocracks without the need for outside assistance. These mechanisms are categorized into three main groups: physical, chemical, and mechanical causes, shown in Figure 1.

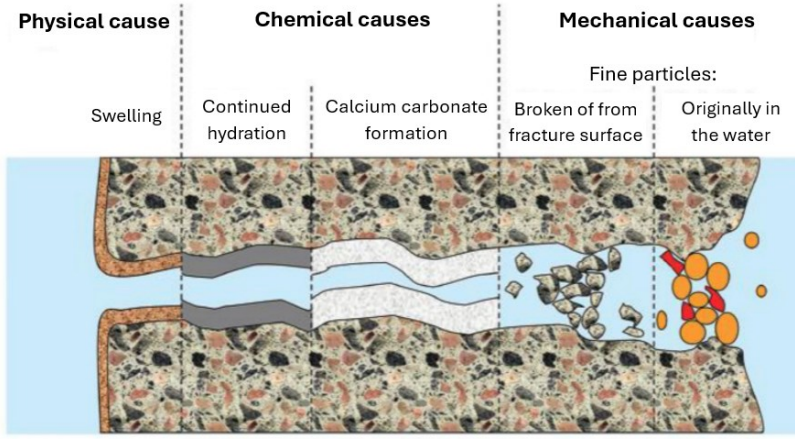


Figure 1. The mechanisms of autogenous healing concrete [3]

Physical causes occur as expansion of hydrated concrete in the sides of cracks. **Chemical** causes involve continued hydration and the formation of calcium carbonate ($CaCO_3$). Continued hydration happens when unhydrated cement particles react with water that penetrates the crack, producing additional hydration products that fill the voids. In the presence of moisture and atmospheric carbon dioxide, calcium hydroxide, one of the cement hydration products, reacts to form calcium carbonate, which effectively seals cracks. **Mechanical** causes refer to the migration and deposition of fine particles within the crack. These particles may originate from the mixing of water or breaking off from the crack surfaces. Their accumulation inside the crack contributes to its filling and potential closure [3]. These mechanisms support autogenous healing, especially in moist environments where water activates the process. Supplements, such as mineral additives, fibers, nanofillers and curing agents, described below, are used to promote the efficiency of the healing process.

3.1. Mineral additives

Mineral admixtures are among the most widely explored materials to enhance autogenous self-healing in cementitious composites. Commonly used additives include fly ash, blast furnace slag, carbonated steel slag, expansive materials, geo-materials, crystals and chemical additives. These materials promote healing through continued hydration and precipitation of healing products such as calcium silicate hydrate (C–S–H), calcium carbonate ($CaCO_3$), and ettringite. For example, blast furnace slag, with its high calcium oxide (CaO) content, provides a favorable environment for calcite formation, enhancing crack sealing efficiency. Expansive agents contribute by physically sealing

cracks through volume expansion, while crystalline materials generate healing products over time via slow chemical reactions (Figure 2). Healing is most effective for cracks below 150 μm, with optimum results observed under water or high humidity curing conditions. Nevertheless, effectiveness depends on multiple factors including admixture type and dosage, water-to-binder ratio, curing regime, and crack characteristics [29].

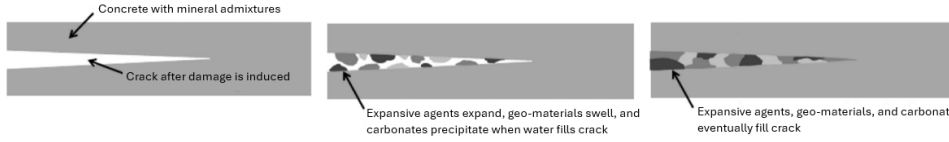


Figure 2. Schematic illustration of self-healing approach by incorporating mineral admixtures [16]

3.2. Fibers

Incorporation of fibers into concrete serves a dual role: limiting crack width and facilitating healing. Commonly used fibers include polyvinyl alcohol (PVA), polyethylene (PE), polypropylene (PP), steel and carbon fibers, and also natural fibers, glass fiber and other synthetic fibers. Fibers help control crack width, thereby increasing the efficiency of the self-healing process since less healing material is required to seal the cracks. Then, fibers contribute to crack bridging by serving as anchors for crystallization products, further promoting the self-healing effect. By bridging cracks and reducing their width, fibers increase the likelihood that cracks will close autonomously. In strain-hardening cementitious composites (e.g., ECC), microcracks induced during loading are typically below 100 μm, making them ideal for autogenous healing. Additionally, fibers can act as nucleation sites for the deposition of healing products, enhancing their local accumulation. Studies have shown that specimens containing hybrid fibers (e.g., steel + PE) demonstrate improved mechanical recovery and water tightness, even under cyclic loading. However, steel fibers require careful handling in moist environments due to corrosion risks. Overall, fiber-reinforced systems demonstrate high potential for both passive and active self-healing enhancement [29].

3.3. Nanofillers

Due to their unique structure, nanoparticles can significantly enhance cementitious composites and support self-healing. It has been confirmed that nanomaterials can be used to create self-healing composites, including epoxidized natural rubber, self-healing epoxy nanocomposites, and nanocomposite elastomers. Three primary mechanisms have been proposed to explain how nanofillers enhance the self-healing behavior of concrete and it is shown in Figure 3.

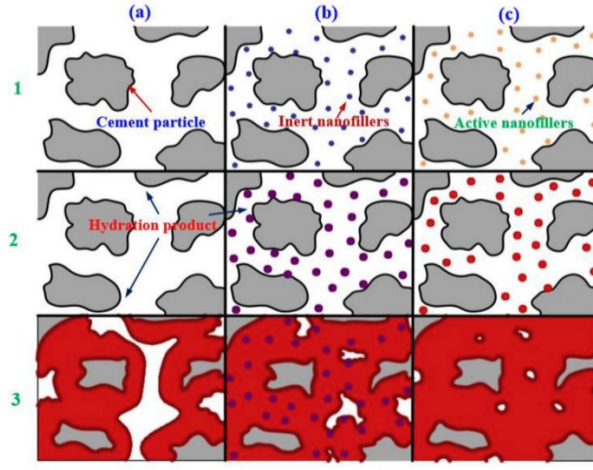


Figure 3. Hydration of plain cement compared to composites with inert nanofillers [8]

First, nanofillers act as nucleation sites within the pore solution, facilitating the precipitation of hydration products and promoting continued hydration of unreacted cement particles. Subfigure 3a illustrates the hydration process in plain cement paste, where hydration products are concentrated around cement grains. In Subfigure 3b, the presence of inert nanofillers provides additional surfaces for hydration, leading to a more dispersed distribution of products. Subfigure 3c goes a step further, showing that active nanofillers - due to their surface reactivity - significantly enhance hydration by acting as highly effective nucleation sites, resulting in a denser and more interconnected matrix. Second, the inclusion of nanofillers strengthens the internal structure of the cement matrix by refining the pore structure, encouraging the formation of finer microcracks, and distributing stress more evenly. This structural refinement helps in arresting crack growth and creates favorable conditions for self-healing to occur. The comparative density of the matrix, as shown in the third row of Figure 3, is visibly higher in nanofiller-reinforced systems (Subfigure 3b and Subfigure 3c) than in plain cement (Subfigure 3a), highlighting the improved compactness and durability brought by the fillers. Third, active nanofillers - such as nano-silica (SiO_2) or nano-titania (TiO_2) coated with silica - participate in pozzolanic reactions by reacting with calcium hydroxide $Ca(OH)_2$, a by-product of cement hydration. This chemical contribution complements the physical effects described above, making active nanofillers especially effective in enhancing self-healing performance. However, dispersion and dosage control are critical, as excessive amounts may impair workability or lead to agglomeration [29].

3.4. Curing agents

Curing agents promote autogenous healing by acting as internal water reservoirs. When there is enough water, they absorb and store it, and as the humidity gradient occurs, they gradually release the retained water to unhydrated cement to enable continuing hydration. Common agents include lightweight aggregates (LWA) such as ceramsite and pumice, superabsorbent polymers (SAP), and shrinkage-reducing

admixtures (SRA). LWA acts as internal reservoirs that release water slowly, supporting ongoing hydration of unreacted cement particles. SAP not only provides internal curing but also physically blocks cracks through swelling when exposed to moisture (Figure 4). While SAP enhance water tightness and delay crack propagation, their shrinkage during dry cycles can create voids that compromise mechanical integrity. The effectiveness of curing agents depends on their water retention capacity, particle size, and interaction with the surrounding matrix. Optimized use of internal curing systems has been shown to reduce autogenous shrinkage, improve crack closure, and enhance durability properties in both early and long-term stages [29].

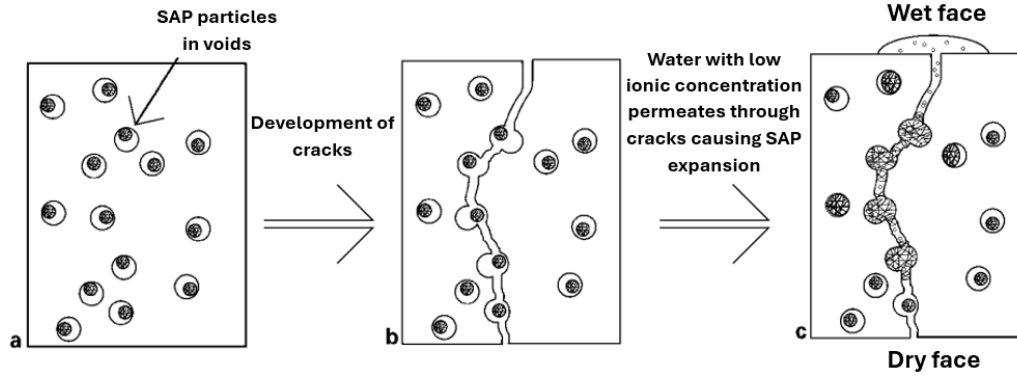


Figure 4. Schematic showing potential mechanism of self-sealing cracks using SAP [15]

4. AUTONOMOUS SELF-HEALING MECHANISM

4.1. Shape Memory Alloys (SMA)

Shape Memory Alloys, particularly NiTi-based SMAs, are metallic materials capable of returning to their pre-deformed shape upon thermal activation. In concrete, SMA elements can be embedded as reinforcement or stitched across crack-prone zones. Upon heating - typically through electrical stimulation - SMA components undergo a phase transformation from martensite to austenite, generating recovery stress that pulls cracked sections together. This process effectively closes cracks and restores structural continuity. Studies have shown that SMA can significantly reduce mid-span deflection in beams and recover both flexural strength and stiffness. SMA-based self-healing systems also demonstrate repeated healing potential, making them suitable for applications involving cyclic loading. While SMA offers rapid and repeatable healing, its cost and temperature sensitivity remain challenges for broader implementation [29].

4.2. Electrodeposition

Electrodeposition is a promising electrochemical technique for autonomous crack repair in reinforced concrete, particularly suitable for marine and wet environments. The process involves applying a low-intensity direct or pulsed current between embedded

reinforcement (cathode) and an external anode in the presence of an electrolyte solution. This leads to the precipitation of insoluble compounds such as $Mg(OH)_2$, $CaCO_3$, ZnO, and CuO within cracks and on concrete surfaces, forming a sealing barrier [24]. This mechanism is shown in Figure 5. The efficiency of the method depends on several factors, including the type and concentration of the electrolyte, current density, duration, and the porosity of the cement matrix. Pulsed current has shown superior healing results compared to constant current, producing denser and more uniform deposition layers. Electrodepositions can achieve complete crack filling for widths up to 1 mm, significantly enhancing the mechanical integrity and durability properties such as water tightness, carbonation resistance, and chloride ion impermeability [21].

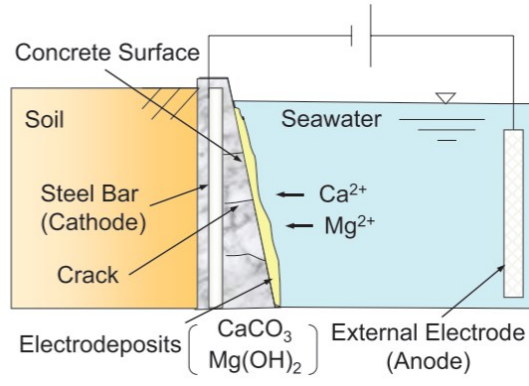


Figure 5. Application of electrodeposition in marine structure [21]

4.3. Capsules

Capsule-based self-healing concrete involves the integration of micro- or macro-capsules filled with healing agents into the cementitious matrix. When a crack propagates through the matrix, it ruptures the capsules, releasing the encapsulated material into the crack. The healing agents - typically epoxy resins, sodium silicate solutions, or polymer precursors - then undergo chemical or physical processes such as polymerization or precipitation to seal the crack. This process is confirmed by an Environmental scanning electron microscopy (ESEM) at Figure 6, which shows a ruptured microcapsule after the healing mechanism is activated and microcapsule images with different magnification [22].

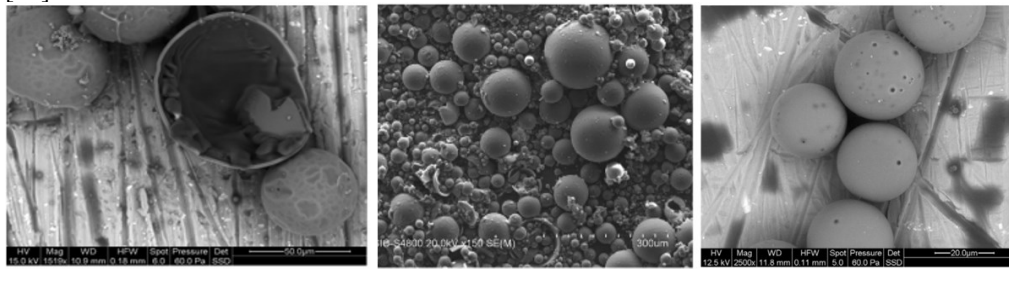


Figure 6. ESEM images of the epoxy containing microcapsules [22]

Various capsule types have been developed, including polymeric, ceramic, glass, and clay capsules, with sizes ranging from a few micrometers to several millimeters. Key parameters affecting performance include shell material, capsule size and distribution, and compatibility with the cement matrix. While this method allows for precise targeting of crack healing, challenges include ensuring capsule survivability during mixing and achieving uniform dispersion. Hybrid systems using both capsules and fibers or porous aggregates have shown improved performance in sealing wider cracks and recovering durability [29].

4.4. Vascular Networks

Vascular self-healing systems mimic biological circulatory systems and are designed to transport healing agents through embedded channels or tubes within cement matrix. These networks can be single-use (one-time release upon crack formation) or multiple-use (refillable systems), and are typically constructed using glass, polymeric, or metallic capillaries. Upon cracking, the healing agent flows from the reservoir through the vascular network to the damaged area, filling and sealing the crack (Figure 7). Healing agents such as epoxy, cyanoacrylate, and sodium silicate are commonly used. Vascular networks offer significant advantages over capsule systems, such as reusability and extended healing over large areas. However, the complexity of design and construction, risk of blockage, and potential reduction in mechanical strength due to the presence of hollow channels present key challenges [29].

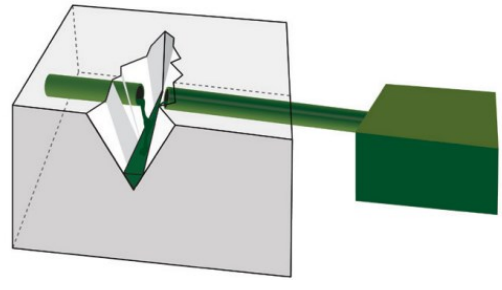


Figure 7. Schematic illustration one-channel vascular based self-healing approache [27]

4.5. Microbial Technologies

Microbial self-healing utilizes specific types of bacteria, primarily bacilli, that induce precipitation of calcium carbonate ($CaCO_3$) through metabolic processes, such as ureolysis or nitrate reduction. These sporogenic and alkaliresistant bacteria are embedded into the cement matrix, often encapsulated in lightweight aggregates, hydrogels, or microcapsules to ensure survival and delayed activation. When cracks occur and moisture infiltrates, bacterial spores germinate and begin metabolic activity which includes biomineralization, effectively sealing cracks. Urea, calcium lactate or similar nutrients are provided to support bacterial metabolism. Bacteria structure and biomineral production are illustrated in Figure 8. Bacteria negatively charged cell walls attract calcium ions from the environment, while metabolic degradation of urea provides dissolved inorganic

carbon ions in microenvironment of cells. As a result, calcium carbonate ($CaCO_3$) begins to precipitate on the bacterial cell wall. Over time, the bacterial cell becomes fully immobilized by the precipitated material. Figure 8 shows the imprints of bacterial cells that participated in the carbonate precipitation process [2].

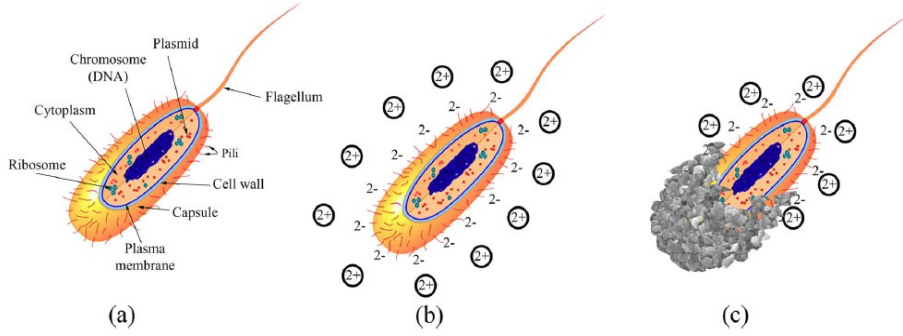


Figure 8. a) Bacteria structure. b) Negatively charged cell wall and the presence of positive charged ions. c) Biomineral production by means of binding ions to cell wall [25]

Microbial healing can close cracks up to 800 µm, depending on species, cell concentration, nutrient availability, and environmental conditions. This bio-based approach offers a sustainable and environmentally friendly alternative to traditional methods. Nevertheless, challenges such as limited healing rates under dry conditions, long activation times, and long-term viability of bacteria in the alkaline concrete environment need further exploration [29]. In this regard, the studies obtained by Šovljanski et al. [30] and Milović et al. [20] showed that the indigenous individual and co-culture *Bacillus* bacteria could survive and remain active under varying environmental conditions, including wet-dry cycles and extreme pH levels in cement matrix.

5. CONCLUSIONS

Self-healing concrete concept offers an innovative approach to improve the durability and longevity of concrete structures by allowing them to cure themselves from damage. Autogenous self-healing, driven by ongoing hydration and carbonation, can be considerably improved by using mineral additives, fibers, nanofillers, and curing agents, which help microcrack closure and promote healing product formation. Autonomous self-healing mechanisms such as electrodeposition, shape memory alloys, encapsulated healing agents, vascular networks, and microbial technologies expand the healing potential beyond the limitations of natural reactions. These technologies allow for repetitive or externally activated healing and can handle larger cracks. Research indicates that autonomous procedures can seal cracks up to 500 µm, greatly enhancing mechanical characteristics, water tightness, and resistance to external effects. Autogenous processes are efficient for cracks up to 150 µm.

In general, autonomous healing offers a more reliable solution for structural maintenance, though autogenous repair is simpler and durable. Future smart and resilient concrete infrastructure may benefit best from a combination of both strategies, according to application requirements and environmental circumstances, while continuing to

address the challenges posed by new approaches, methods and materials, as a continuation of the most recent innovative researches.

Acknowledgements

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

Milica V. Vasić[1], Zagorka Radojević[2]

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CIRKULARNA EKONOMIJA U SLUŽBI ODRŽIVOG RAZVOJA GRAĐEVINSKIH PROIZVODA NA BAZI GLINA

Rezime: *Pred proizvodnjom tradicionalnih proizvoda na bazi gline postavljaju se brojni izazovi na globalnom nivou. I pored toga, oni predstavljaju jedan od osnovnih materijala sa primenom u građevinarstvu. U radu se predstavljaju moguća poboljšanja u proizvodnji: optimalno korišćenje prirodnih mineralnih resursa, korišćenje otpadnih materijala, smanjenje potrošnje energije i korišćenje zelene energije, smanjenje emisije opasne po zdravlje i životnu sredinu, smanjenje emisije CO2, analiza uticaja proizvoda na životnu sredinu, rekultivacija ležišta i dr. Ključni parametri kod kojih treba obratiti pažnju su vezani za značajan doprinos opekarske industrije globalnoj emisiji CO2, uglavnom zbog energetski intenzivnih procesa proizvodnje i korišćenja neobnovljivih izvora energije. Ključni pravci razvoja obuhvataju prelazak na „zelenu“ energiju i korišćenje veštačke inteligencije za praćenje i optimizaciju proizvodnje. Ovaj rad daje sveobuhvatan pregled trenutnog stanja opekarske industrije i nudi preporuke za buduća istraživanja i inovacije za promovisanje održivosti životne sredine.*
Ključne reči: *Opekarska industrija, Potrošnja energije u procesu proizvodnje, CO2 otisak.*

CIRCULAR ECONOMY IN THE SERVICE OF SUSTAINABLE DEVELOPMENT OF CLAY-BASED CONSTRUCTION PRODUCTS

Summary: *The production of traditional clay-based products faces numerous challenges at the global level. In addition, they are one of the basic materials used in construction. The paper presents possible improvements in production: optimal use of natural mineral resources, use of waste materials, reduction of energy consumption and use of green energy, reduction of emissions hazardous to health and the environment, reduction of CO2 emissions, analysis of the impact of products on the environment, reclamation of deposits, etc. The key parameters to pay attention to are related to the brick industry's significant contribution to global CO2 emissions, mainly due to energy-intensive production processes and the usage of non-renewable energy sources. Key areas of development include the transition to "green" energy and the use of artificial intelligence to monitor and optimize production. This paper provides a comprehensive overview of the current state of the brick industry and offers recommendations for future research and innovation to promote environmental sustainability.*
Keywords: *Ceramic industry, Energy consumption in the production process, CO_2 footprint.*

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1. UVOD

Industrija građevinskih materijala na bazi gline suočava se sa sve većim izazovima na globalnom nivou, usled povećanih zahteva za održivošću i smanjenjem negativnih uticaja na životnu sredinu. Ipak, glina ostaje jedan od ključnih materijala u građevinarstvu, zahvaljujući svojim prirodnim svojstvima, širokoj dostupnosti i dugoj tradiciji upotrebe. Građevinski proizvodi na bazi glina predstavljaju nezaobilazne materijale zbog njihovih prirodnih karakteristika, dostupnosti sirovina i mogućnosti recikliranja [1].

Tradicionalna keramika i dalje predstavlja jedan od glavnih gradivnih materijala u građevinskoj industriji. Uprkos tehnološkom napretku, ova industrija značajno zavisi od primarnih, neobnovljivih mineralnih sirovina i fosilnih goriva. Pored toga, tradicionalni procesi proizvodnje generišu velike količine otpada, a pečenje keramičkih proizvoda podrazumeva visoku potrošnju energije i značajnu emisiju CO_2 [2].

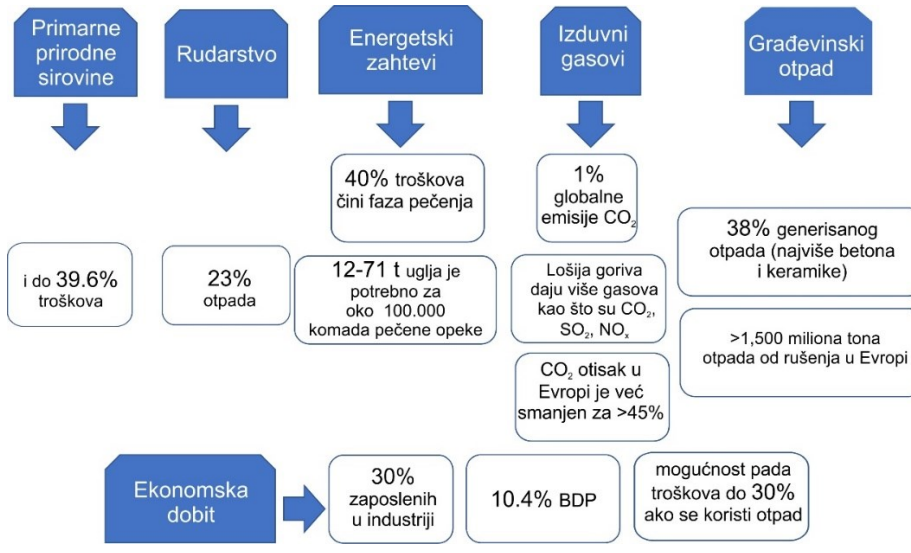
Industrija keramike na globalnom nivou, koja obuhvata proizvode poput opeke, crepa, keramičkih pločica i sanitarne keramike, beleži značajan rast. U poslednjih nekoliko godina porasla je i potražnja za ekološki prihvatljivim građevinskim materijalima, što otvara nove tržišne prilike za "zelene" keramičke proizvode [3]. Ovi proizvodi se odlikuju upotrebom obnovljivih izvora i/ili korišćenjem sekundarnih sirovina, optimizovanim procesima proizvodnje s naglaskom na sniženu potrošnju energije, smanjenje emisije CO_2, te primenom principa cirkularne ekonomije [4,5].

Primena principa cirkularne ekonomije u ovoj oblasti otvara brojne mogućnosti za unapređenje proizvodnih procesa. Jedan od važnih ciljeva je i produženje životnog ciklusa proizvoda kroz obnovu i inovaciju, čime se smanjuje iscrpljivanje prirodnih resursa. Proizvodi od glina mogu se reciklirati i ponovo koristiti u različitim fazama procesa pri čemu se vraćaju kao sirovina u proizvodnju (direktno unutrašnje ponovno korišćenje). Ostatak se može reciklirati eksterno (npr. u izgradnji puteva) ili, u krajnjoj nuždi, odlagati na deponijama [6]. Reciklažom otpadnih materijala iz drugih industrija takođe je moguće smanjiti potrošnju sirovina, ali i energije [7,8]. Zatim, smanjenje potrošnje energije korišćenjem energetski efikasnijih tehnologija u procesima proizvodnje doprinosi uštedi i smanjenju negativnih uticaja po životnu sredinu [9]. Korišćenje lokalnih resursa i energetska efikasnost proizvoda od glina smanjuje emisije štetnih gasova. Primena zelene energije, odnosno obnovljivih izvora energije, u proizvodnim procesima može značajno smanjiti emisiju štetnih gasova.

Ovaj rad analizira dosadašnje pristupe integraciji sekundarnih sirovina i otpada u proizvodnju keramičkih materijala, istovremeno ukazujući na izazove i predstojeće perspektive održivosti kroz cirkularnu ekonomiju.

2. IZAZOVI I POTENCIJAL

Tradicionalna industrija keramike spada u jedan od najvećih potrošača prirodnih resursa nemetaličnih mineralnih sirovina. U proizvodnji keramike, prvi suštinski element je dostupnost sirovina, koje mogu da čine i do 40 % ukupnih troškova proizvodnje, posebno u proizvodnji keramičkih pločica [10]. Širom sveta prisutna je sve veća zabrinutost oko bezbednog i pristupačnog snabdevanja sirovinama. S druge strane, u okviru Evropske Unije, rudarstvo je odgovorno za stvaranje oko 23 % otpada [11]. Podaci pronađeni u literaturi su prikazani na *Dijagramu 1*. Oni ukazuju na neke od problema sa kojima je neopodno suočiti se radi postizanja bolje zaštite životne sredine, ali i kompetititvnosti na tržištu.

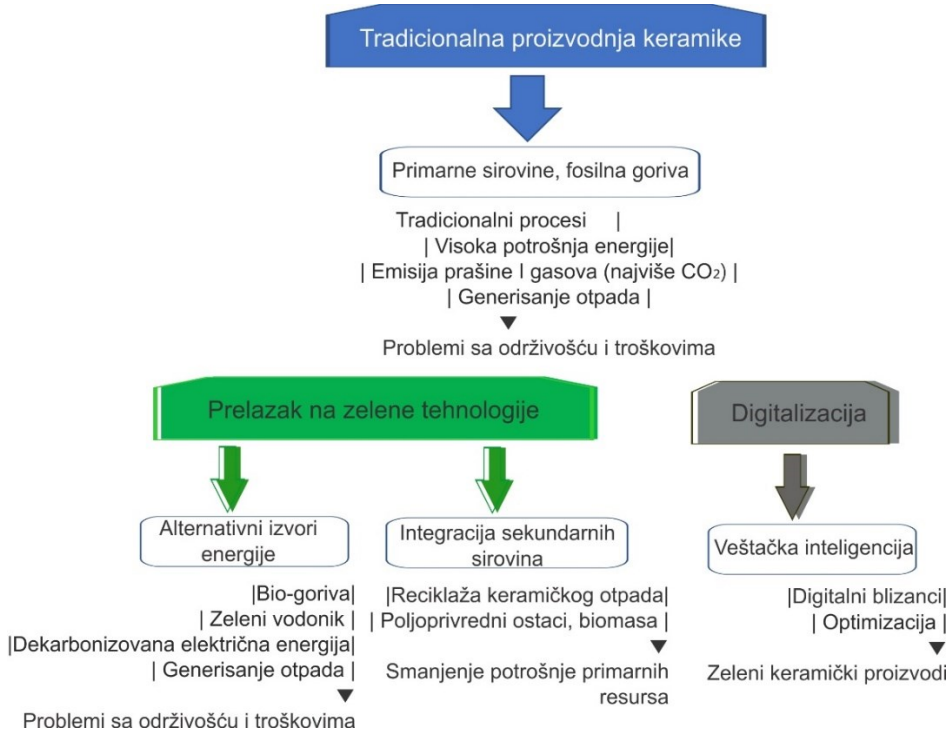


Dijagram 1. Tradicionalna keramička industrija u brojevima.

Uobičajen lanac proizvodnje keramike, gde se koriste primarne sirovine i fosilna goriva, rezultira visokim energetskim zahtevima, velikim emisijama CO_2 i generisanjem otpada (*Dijagram 2*). Prelazak ka održivijim rešenjima razdvaja glavni put u dve smernice. Alternativni izvori energije podrazumevaju upotrebu bio-goriva, zelenog vodonika i dekarbonizovane električne energije, čime se omogućava smanjenje emisija CO_2 i optimizacija procesa. Dekarbonizovana električna energija odnosi se na električnu energiju koja je proizvedena uz minimalnu ili nikakvu emisiju CO_2 i drugih gasova sa efektom staklene bašte. To znači da je proizvodnja ove energije zasnovana na obnovljivim izvorima, kao što su solarna energija, vetroenergija, hidroenergija, ili nuklearna energija, koji ne emituju CO_2 tokom procesa proizvodnje. Takođe, može uključivati tehnologije za smanjenje emisija, kao što su hvatanje i skladištenje CO_2, koje omogućavaju smanjenje emisija iz fosilnih goriva. Dekarbonizacija energetske proizvodnje ključna je za borbu protiv klimatskih promena, jer smanjuje uticaj na globalno zagrevanje i pomaže u ostvarivanju ciljeva smanjenja emisija CO_2, koji su postavljeni u okviru međunarodnih klimatskih sporazuma poput Pariskog sporazuma [12].

Integracija sekundarnih sirovina koja obuhvata reciklažu otpada (keramički otpad, poljoprivredni ostaci, biomasa) omogućava smanjenje potrošnje primarnih resursa i unapređenje ekološkog profila proizvoda. Primena tehnologija poput "digitalnih blizanaca" omogućava optimalno upravljanje procesima, što dodatno doprinosi energetskoj efikasnosti i smanjenju emisija. Rezultat prelaska je proizvodnja zelenih keramičkih proizvoda – visokokvalitetnih materijala sa znatno nižim ugljeničnim otiskom. "Digitalni blizanci" predstavljaju matematičke modele fizičkih objekata, proizvodnih linija ili celokupnih proizvodnih sistema, koji omogućavaju simulaciju, praćenje i optimizaciju procesa proizvodnje u realnom vremenu. Ovi "soft sensing" modeli se koriste za poboljšanje efikasnosti, smanjenje troškova, poboljšanje kvaliteta proizvoda i smanjenje negativnog uticaja na životnu sredinu [13]. U keramičkoj industriji, digitalni blizanci mogu npr. da simuliraju rad peći praćenjem temperature, protoka energije, efikasnosti sagorevanja i emisije gasova, omogućavajući optimizaciju procesa pečenja i smanjenje

potrošnje energije. Takođe, mogu se koristiti za predviđanje kada će biti potrebna održavanja ili kada će nastati kvarovi, što poboljšava pouzdanost sistema [14]. Neke kompanije u Italiji i drugim evropskim zemljama implementiraju digitalne modele koji simuliraju celokupan rad keramičkih peći. Ovi alati omogućavaju realno-vremensko podešavanje procesa pečenja, optimizaciju temperature i vremena, te na taj način značajno smanjuju potrošnju energije. Efikasnije upravljanje procesom doprinosi smanjenju emisija i povećanju ukupne energetske efikasnosti.



Dijagram 2. Mogući pravci poboljšanja keramičke industrije

Izvori energije koji se koriste u keramičkoj industriji uglavnom zavise od dostupnosti, cene i primenjenog proizvodnog sistema. Tradicionalni izvori goriva su ugalj, prirodni gas, električna energija, pa čak i petrol koks [3,15]. Savremeni sistemi obuhvataju biogoriva, biogas ili solarne izvore energije [3]. Ugalj je gorivo sa relativno visokom toplotnom moći od oko 10 do 25 MJ/kg. Problem je što peći na ugalj često rade neefikasno, s obzirom da se veliki deo energije gubi kroz izduvne gasove. Takođe, korišćenje uglja ima značajne negativne uticaje na životnu sredinu, uključujući emisije gasova i sitnih čestica. Prirodni gas ima veću toplotnu moć, od oko 42 do 55 MJ/kg [16]. Prirodni gas je efikasan izbor za peći u savremenoj keramičkoj proizvodnji, jer daje značajnu količinu toplote uz relativno niske emisije. Električne peći su takođe široko korišćene u keramičkoj industriji, posebno za manji obim proizvodnje. Međutim, njihov uticaj na životnu sredinu u velikoj meri zavisi od izvora električne energije, što je ranije objašnjeno.

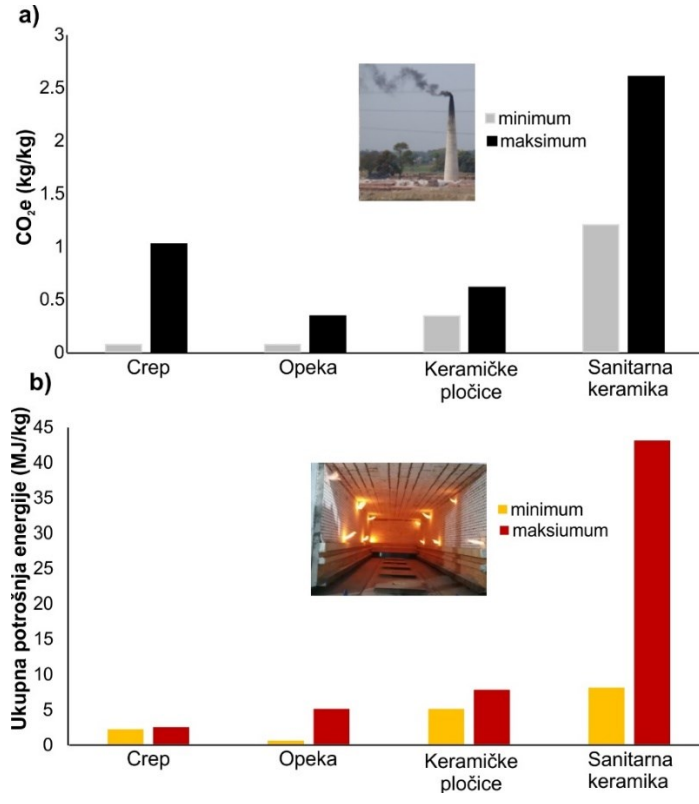
Biomasa se sve više koristi zbog svojih gotovo nultih emisija CO_2 i zagađivača, neiscrpnih zaliha i visoke reaktivnosti. Ugljen-dioksid koji se oslobađa tokom sagorevanja

nadoknađuje se ugljen-dioksidom koji su biljke apsorbovale tokom rasta. Zbog relativno niske energetske gustine koja je uporediva sa ugljem, biomasa se može koristiti i kao gorivo u procesu ko-gasifikacije. Pored toga, njena efikasnost zavisi od kvaliteta i vlažnosti biomase, kao i od sposobnosti peći da održava ravnomernu raspodelu toplote, što potom utiče na ukupni uticaj na životnu sredinu [17].

Energetska efikasnost peći u keramičkoj industriji zavisi od njihovog dizajna, goriva koje se koristi i tehnologije sagorevanja, a sve to utiče i na emisiju čestica i drugih zagađivača u vazduhu. Jedno od tradicionalno korišćenih rešenja za smanjenje potrošnje energije je povrat toplote iz peći i njena primena u procesu sušenja. U nekim evropskim keramičkim pogonima integrišu se lokalni obnovljivi izvori energije (npr. solarni paneli ili vetroturbine) za napajanje pomoćnih sistema poput sušenja ili za napajanje dodatne opreme u proizvodnji. Ovo smanjuje indirektne emisije povezane sa korišćenjem električne energije koja dolazi iz termoelektrana zasnovanih na fosilnim gorivima.

Tokom pečenja tradicionalne keramike, emituju se različiti dimni gasovi, koji uglavnom uključuju CO_2, CO, SO_2, NO_x i PM čestice [18]. Ovi gasovi nastaju sagorevanjem goriva i hemijskim reakcijama koje se dešavaju unutar peći. Ukupna količina gasova obično se preračunava u ekvivalent CO_2 koji predstavlja ukupnu količinu emisija gasova sa efektom staklene bašte koje se generišu tokom proizvodnog procesa. Ovaj ekvivalent omogućava sagledavanje ukupnog uticaja na životnu sredinu, jer osim CO_2, proizvodnja keramike može emitovati i druge gasove, kao što su metan (CH_4), azotovi oksidi (NO_x), i drugi, koji takođe doprinose globalnom zagrevanju. Tako je moguće da jedna fabrika troši manje energije za proizvodnju, ali da ostavlja veći CO_2 otisak. Ovo je u velikoj meri povezano sa vrstama goriva koja se koriste (uglavnom su problem neobnovljivi izvori energije) i gubicima energije u proizvodnom sistemu, naročito u peći. Najveći efekat ekvivalentnog CO_2 otiska primećuje se u fazama proizvodnje, ali i transporta od kapije do lokacije ugradnje. Najmanji CO_2 otisci su registrovani npr. kod proizvodnje opeke sagorevanjem biogasa [19]. Kada značajna potrošnja energije korespondira sa niskim otiskom gasova, to ukazuje na značajnu integraciju čistih izvora energije. Neke opekarske industrije, naprimer u Danskoj, primenjuju značajne količine obnovljivih izvora energije, što je rezultiralo značajno nižim otiskom na životnu sredinu. Ovo ukazuje na hitnu potrebu za reformama u tradicionalnoj keramičkoj industriji i u drugim zemljama, sa fokusom na povećanje upotrebe obnovljivih izvora energije. CO_2 "capture" sistemi u keramičkim pećima su još u ranoj fazi industrijskog prilagođavanja, oni predstavljaju značajan korak ka direktnom smanjenju emisija nastalih tokom procesa pečenja. Time se otvara mogućnost da se emisije u budućnosti dodatno minimiziraju, čime se doprinosi ostvarivanju ambicioznych klimatskih ciljeva.

Prema dostupnim podacima iz literature (*Dijagram 3*), CO_2 otisak tokom proizvodnje keramičkih proizvoda varira u širokom opsegu [19-23], a najviši je kod sanitarne keramike i crepa. Paraleleno, ukupna potrošnja energije u proizvodnji po kilogramu proizvoda je najviša kod sanitarne opreme i crepova od gline [20-24]. Sanitarni proizvodi pokazuju najveći ekološki uticaj zbog svojih složenih oblika, više faza pečenja i većih energetski zahteva. Varijacije ova dva parametra zavise od mnogih faktora, kao što su osobine polaznih sirovina, temperatura pečenja, konstrukcija peći, efikasnost peći i vrsta goriva. Intenzivnija energetska potrošnja glaziranih keramičkih pločica je zbog dodatnog procesiranja. Kod sanitarne keramike, veći CO_2 otisak i intenzivnija potrošnja energije se javljaju kod složenijih oblika proizvoda.



Dijagram 3. Ukupni CO_2 otisak i potrošnja energije kod različitih vrsta proizvoda tradicionalne keramike

Faza pečenja u proizvodnji keramike predstavlja najveći izvor potrošnje energije i emisije CO_2. Iako većina evropskih fabrika koristi prirodni gas, u mnogim regionima, posebno u zemljama u razvoju, u proizvodnji se i dalje oslanja na ugalj. Ključni aspekti za budućnost industrije uključuju prelazak na održivije tehnologije i izvore energije, što predstavlja izazov s aspekta troškova adaptacije, ali direktno utiče i na globalnu konkurentnost keramičke industrije. U nekim evropskim keramičkim pogonima testiraju se i već se primenjuju mešavine bio-goriva umesto fosilnih goriva kao što su ugalj ili petrol koks. Ovo omogućava smanjenje emisije CO_2, a istovremeno smanjuje korozivno delovanje gasova u pećima. Primeri iz pilot projekata [25,26] pokazuju da zamenom dela tradicionalnih goriva energetski intenzivna faza pečenja može postati manje štetna po životnu sredinu. Iako se još nalaze u fazi istraživanja i pilot projekata, neke kompanije eksperimentišu sa korišćenjem zelenog vodonika u fazama sušenja i pečenja, dok se dekarbonizovana električna energija koristi kao dodatak ili zamena za tradicionalne izvore toplote. Ovi pristupi imaju potencijal da u srednjoročnom periodu kompletno transformišu energetsku strukturu proizvodnih pogona.

Sumarno, tradicionalna proizvodnja keramike zahteva značajne energetske resurse i često rezultira emisijom štetnih gasova, uključujući CO_2. Pored toga, intenzivna eksploatacija prirodnih mineralnih resursa može ostaviti dugoročne posledice na ekosistem. Implementacija cirkularne ekonomije u građevinskoj industriji zahteva

inovativne tehnologije, edukaciju i saradnju različitih aktera, od proizvođača do krajnjih korisnika. Iako postoje izazovi, poput regulative i početnih investicija, potencijal za dugoročni profit je značajan.

3. UTICAJ KERAMIČKIH PROIZVODA NA ŽIVOTNU SREDINU

Analiza uticaja građevinskih proizvoda na bazi gline na životnu sredinu postaje sve važnija tema. Deklaracije o zaštiti životne sredine proizvoda (Environmental Product Declarations - EPDs) za keramičke proizvode ključne su za postizanje održivosti u građevinskoj industriji. Procena životnog ciklusa (Life Cycle Assessment - LCA) ovih proizvoda obuhvata sve faze njihovog postojanja, pružajući podatke o njihovom uticaju na životnu sredinu [27]. Ključni aspekti ovakvih dokumenata uključuju praćenje CO_2 otiska i njegovo smanjenje. Smanjenje emisija koje negativno utiču na zdravlje lokalnih zajednica ključno je za postizanje održivosti. Energetski intenzivni procesi na prvom mestu zahtevaju prelazak na obnovljive izvore energije. Takođe važan aspekt je vođenje računa o životnoj sredini na taj način što se vrši rekultivacija ležišta nakon eksploatacije sirovina, radi revitalizacije područja kako bi se očuvali biodiverzitet i ekosistemi.

4. PRIMENA SEKUNDARNIH SIROVINA

Fokus istraživanja na laboratorijskom nivou je na ispitivanju uticaja različitih vrsta otpada na svojstva keramičkih proizvoda. Do sada su proučavani kako organski (npr. ostaci poljoprivredne proizvodnje poput pirinčanih ljuspica, suncokretovih ljuspica, piljevine, itd), tako i neorganski otpadni materijali (poput različitih vrsta pepela nastalih sagorevanjem uglja, keramičkog otpada) u formulacijama koje se koriste za proizvodnju opeke i crepa. Rezultati pokazuju da se dobijaju proizvodi smanjene zapreminske mase i niža toplotne provodljivosti. Pozitivan efekat je to što se dodavanjem određenih vrsta otpada mogu poboljšati izolacijska svojstva proizvoda. Povećanje poroznosti usled upotrebe otpada može smanjiti čvrstoću, što zahteva pažljivo balansiranje količina dodataka. Primena otpada smanjuje potrebu za primarnim mineralnim sirovinama i potencijalno može dovesti do uštede energije u procesu pečenja, ukoliko se procesi optimizuju u uslovima koji što verodostojnije oslikavaju industrijsku proizvodnju. Ovi nalazi ukazuju na značajan potencijal, ali i na potrebu za daljim istraživanjima koja bi laboratorijske uslove što više približila realnim industrijskim procesima [3].

Prevod istraživačkih nalaza sa laboratorijskog nivoa na industrijsku proizvodnju predstavlja ključni izazov. Iako su neke pilot studije pokazale uspešne primene – na primer, korišćenje 10–20% keramičkog otpada u proizvodnji opeke ili kombinacija keramike sa pirinčanim ljuspicama u proizvodnji crepa – industrija još uvek postupa vrlo oprezno. Ovo se uglavnom odnosi na zabrinutost zbog mogućih problema kao što su oštećenja peći, prekoračenje emisijskih normi ili neadekvatna mehanička svojstva konačnih proizvoda. U Španiji, gde opekarska industrija beleži rast, neki proizvođači su počeli da upotrebljavaju do 20% keramičkog otpada (npr. keramička glina odbačena tokom proizvodnje) u formulacijama za proizvodnju crepa i opeke. Sličan princip primenjen je i u proizvodnji crepa, gde upotreba mešavine pirinčanih ljuspica i keramičkog otpada (u rasponu od 10–20%) ne samo da smanjuje potrošnju primarnih sirovina već i unapređuje toplotna izolaciona svojstva proizvoda [3].

Konkretni primeri iz laboratorijskih i pilot studija pokazuju da se otpadne materije poput pepela različitog porekla, ostataka iz poljoprivredne proizvodnje i čak ambalažni stakleni otpad (kao zamena za pesak) uspešno integrišu u kompozicije za proizvodnju keramičkih materijala, čime se postiže smanjenje ukupnog ugljeničnog otiska [3]. Implementacija principa cirkularne ekonomije kroz ponovnu upotrebu otpada ne samo da smanjuje pritisak na primarne resurse, već i doprinosi smanjenju ekološkog otiska industrijskog sektora. Ipak, za široku primenu potrebno je razviti standarde za kvalitet sekundarnih sirovina, kao i poboljšati komunikaciju između akademske zajednice i industrije.

Sve navedeno je značajno povezano sa ciljevima održivog razvoja i odgovornom potrošnjom i proizvodnjom [28].

5. REKULTIVACIJA

Rekultivacija gliništa je proces obnove i unapređenja područja koja su bila podložna eksploataciji a ostala su degradirana zbog iskopavanja glina. Nakon što je iskopavanje završeno, prvi korak je obnova površinskog sloja zemljišta. To može uključivati dodavanje plodnog sloja zemlje, hranjivih materija, biljnih ostataka i drugih materijala koji pomažu u obnavljanju plodnosti tla. Takođe se mogu koristiti i određene vrste tla koje su potrebne za buduće planirane aktivnosti. U zavisnosti od tipa zemljišta i specifičnih ekoloških uslova, često se sadi autohtono bilje koje je prilagođeno lokalnom ekosistemu. Ovo pomaže u stabilizaciji tla, sprečava eroziju i poboljšava biodiverzitet. Rekultivacijski projekti često uključuju mere za kontrolu erozije, kao što su sadnja pogodnih vrsta vegetacije ili postavljanje zaštitnih struktura. Osim toga, u mnogim slučajevima, rekultivacija uključuje i upravljanje površinskim i podzemnim vodama kako bi se sprečilo zagađenje i osiguralo pravilno zadržavanje vode u obnovljenom području. Nakon završetla rekultivacije, važno je kontinuirano pratiti ekosistem kako bi se procenila uspešnost programa. Ovo može uključivati analize kvaliteta tla, vode, biodiverziteta i uslova rasta biljaka. Uključivanje lokalne zajednice u planiranju i radu na rekultivaciji može biti od velike važnosti za dugoročnu održivost projekata i podizanje svesti o važnosti očuvanja životne sredine.

Rekultivacija glinista može biti izazovna, ali pravilno planiranje i implementacija mogu rezultirati pozitivnim ishodima za okolinu i zajednicu. Usmeravanjem na obnovu ekosistema, ovaj proces može poboljšati kvalitet životne sredine i pomoći u oporavku lokalnih zajednica.

6. ZAKLJUČAK I PREPORUKE

Globalni trend urbanizacije rezultira intenzivnijom gradnjom i potrebom za inovativnim građevinskim materijalima. Keramička industrija, kao ključni segment građevinske industrije, ima direktnu priliku da kroz implementaciju ekoloških rešenja zadovolji zahteve tržišta. Građevinski sektor, naročito arhitekte i investitori u održive projekte, sve više traže materijale koji ispunjavaju stroge ekološke standarde. Ovo stvara potražnju za proizvodima nižeg ugljeničnog otiska. "Zeleni" keramički proizvodi, sa smanjenom potrošnjom fosilnih goriva i optimizovanim procesima, postaju atraktivniji kako za proizvođače, tako i za krajnje korisnike. Proizvođači imaju mogućnost da se pozicioniraju na tržištu kao lideri u održivosti nudeći proizvode s ekološkom oznakom ili sertifikacijom.

Postoje značajne mogućnosti za saradnju između istraživačkih institucija i proizvođača. Ova saradnja može doprineti razvoju novih, održivijih tehnologija, kao i boljem prenošenju inovacija iz laboratorijskih uslova do industrijske proizvodnje.

Kako bi se unapredila održivost u opekarskoj industriji, neophodno je:

- Inovirati tehnologije proizvodnje,
- Podsticati istraživanja koja se fokusiraju na korišćenje otpadnih materijala,
- Razvijati politike koje promovišu korišćenje obnovljive energije, i
- Povećati saradnju između naučne zajednice, industrije i donosilaca odluka kako bi se postigli zajednički ciljevi održivosti.

Izazovi obuhvataju visoke početne troškove i potrebu za edukacijom tržišta.

Ključne smernice za dalja istraživanja i industrijsku praksu obuhvataju uspešan prelaz na "zelene" tehnologije koji ne samo da će unaprediti ekološki profil proizvoda, već i otvoriti nove mogućnosti za diferencijaciju na tržištu, povećanje konkurentnosti i dugoročnu ekonomsku održivost.

Zahvalnica

Zahvaljujemo se Ministarstvu nauke, tehnološkog razvoja i inovacija Republike Srbije na pomoći i podršci koju nam pruža za sprovođenje istraživačkog rada (Ugovor br. 451-03-136/2025-03/200012).

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

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Prethodno saopštenje

PRIMENA POVRŠINSKI AKTIVNE SUPSTANCE ARKOPALA N 100 U CIGLASTVU

Rezime: *U radu je ispitivana mogućnost primene ARKOPALA N 100 u ciglarskoj industriji. Pomenuta supstanca je dodata u vodu sa kojom se vlažila prethodno osušena i prerađena sirovina. Formirana su četri kompozita. Koncetracija ARKOPALA N 100 je varirala od 0 do 0,07%. Proces sušenja praćen je na uzorcima pločica. Snimljen je set izotermskih eksprimenata za svaki kompozit. Konstrujisane su zavisnosti efektivnog koeficijenta difuzije u funkciji od sadržaja vlage (Deff-MR). Oblikovani uzorci kockica su sušeni na 25^{0}C pri relativnoj vlažnosti od 75% dok apsolutni sadžaj vlage u njima nije spušten do 15, 10 i 5 %. Formirane su zavisnosti promene prtisne čvrstoće oblikovanih uzoraka u funkciji sadržaja vlage. Dobijene krive omogućile su nam da objektivno pratimo generisanje pukotina tokom sušenja i utvrdimo optimalu koncetraciju ARKOPALA N 100 koja omogućava da se iste izbegnu.*

APPLICATION OF THE ARKOPAL N 100 SURFACTANT IN BRICK& TILE INDUSTRY

Abstract: *The application of ARKOPAL N 100 in brick&tile industry was examined. Surfactant was added to the water and four composites were formed. It's concentration was varied from 0 to 0.07 %. Drying process was monitored on tile samples. A set of isothermal experiments was recorded for each composite. Curves effective diffusivity vs moisture content were constructed. Cube samples were dried (25^{0}C and 70% RH) until it's absolute moisture content was reduced to 15, 10 and 5%. Compressive strength vs moisture content curves were also constructed. These curves allowed us to monitor the generation of cracks during drying and to determine the optimal concentration of ARKOPAL N 100 that allows drying without crack formation.*

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1. UVOD

Neadekvatan sastav siroviskog komopozita (na primer nedostatak vezivne „glinene“ komponente, nedovoljna plastičnost itd...) i velika razlika između brzine isparavanja sa površine uzorka i brzine dotoka vlage ka površini uzorka tokom brzog sušenja su glavni razlozi pojave unutrašnjih naprezanja u sušenom proizvodu. U zavisnosti od veličine naprezanja proizvod se može izvitoperiti, iskriviti (plastične-viskoplastične deformacije) ili pući. Intenzivno sušenje često voditi ka pojavi pukotina. Ovaj fenomen je direktno povezan sa naprezanjima koja nastaju kada raspodela vlage ili temperature u uzorku postane nelinearna [1,2]. Iako su u literaturi opisane različite tehnike modelovanja i specijalizovani softveri za optimizaciju parametara vazduha za sušenje u industrijskim sušarama za crep, efikasnost njihove primene je ograničena karakteristikama merne i regulacione opreme koja je instalirana u pogonu. Ovo je posebno uočljivo ako se projektnim zadatkom zahtevala uspostava režimima brzog sušenja [3-5]. Održavanje teoretski propisanog opsega vlažnosti od 95-98% u industrijskim sušarama tokom početne faze brzog sušenja, koja ima za cilj da se proizvod zagreje na više temperature bez pojave pukotina, je veoma izazovno i često tehnički neostvarivo. U praksi stabilna kontrola vlažnosti vazduha se može ostavariti u rasponu od 85 do 88% u početnoj fazi sušenja [6].

Ovaj rad ima za cilj pronalaženje rešenja za sprečavanje nastanka pukotina tokom intenzivne faze zagrevanja u režimu brzog sušenja. Nekoliko mera može se primeniti. Prva je prilagođavanje "prirode" sirovine (npr. sadržaj glinenih minerala, raspodela veličine čestica, plastičnost) dodavanjem glinenih ili peskovitih frakcija. Druga mera podrazumeva nadogradnju ili instalaciju mernih i regulacionih uređaja unutar sušara, u skladu sa teorijskim zahtevima [4,6]. Treća mera odnosi se na poboljšanje transporta vlage iz unutrašnjosti proizvoda tokom sušenja i postizanje ujednačenije raspodele vlage, što se može postići upotrebom površinski aktivnih supstanci (PAS).

Kovalski i njegov istraživački tim proučavali su uticaj nekoliko PASa (Rokacet R26, Rokanol IT6, SDS i FC 4430) na smanjenje naprezanja izazvanih sušenjem u proizvodima od kaolinske gline. Metoda akustične emisije korišćena je za praćenje razvoja pukotina u realnom vremenu, a najbolji rezultati postignuti su kada je glina zasićena vodom koja sadrži PAS u koncentracijama ispod ili blizu kritične micelarne koncentracije (CMC) [7,8]. Gulzena je koristila Triton TX-100 (nejonski PAS) u proizvodnji glinenih opeka sa dominantnom ilitnom glinom. Razvoj pukotina beležen je digitalnom kamerom i analiziran pomoću softvera Motic Images Plus 2.0. Najbolji rezultati postignuti su pri koncentraciji PAS blizu CMCa [9].

Upoređivajući različite nejonske PAS Bisvald je otkrio da povećanje koncentracije PASa postepeno smanjuje Hamekerovu konstantu i rad adhezije, dostižući plato iznad CMCa. Ustanovljeno je da Igepal CO-630 ima nižu CMC vrednost (0,08 mmol/L) i bolje svojstvo kvašenja pri niskim koncentracijama od Triton TX-100 (0,15 mmol/L). Zbog manje potrošnje i potencijalno veće biorazgradivosti Igepal CO-630 je ocenjen kao efikasnije sredstvo [10]. U literaturi nisu pronađena istraživanja u kojima je proučavan uticaj koncentracije nejonskih surfaktanata nešto iznad CMC na smanjenje naprezanja izazvanih sušenjem [11].

U ovom radu pratila se efikasnost različitih koncentracija Akropalna N 100 na smanjenje naprezanja izazvanih sušenjem. Na bazi eksperimentalnih podataka konstrujisane su krive promene efektivne difuzivnosti u funkciji sadžaja vlage (Deff–MR) i pritisne čvrstoće materijala u funkciji sadržaja vlage (MS–MR). Krive Deff–MR

korišćene su za identifikaciju mehanizama transporta tečne vode i pare tokom sušenja. Karakteristične tačke na tim krivama, zatim su transponovane na MS–MR krive. Analizom podataka utvrđena je optimalna koncentracija ARKOPALa N 100.

2. MATERIJALI I METODE

Sirovina je osušena na 60^0C a potom usitnjena na laboratorijskom kolnom mlinu. Potom je izvršeno vlaženje sa vodenim rastvorom Arkopala N 100 (Ataman Chemical, CAS No.:9016-45-9), nakon čega je glinena masa propuštana kroz par laboratorijskih valjaka, sa zazorom od oko 3 mm. U radu je korišćena pomenuta gradacija Arkopala jer je u tehničkom listu proizvođača navedeno da je ona rastvorna u vodi na sobnoj temperaturi za razliku od gradacija N 40 ili N 60. Pripremljena masa je odležala 48 h radi homogeniziranja vlage, a potom je ponovo propuštena kroz par laboratorijskih valjaka, podešenih na zazor od oko 1 mm. PAS koncentracija je data u tabeli 1. Oblikovana su probna tela crepova (120 x 50 x 60 mm) i kocki (33 x 33 mm) na laboratorijskoj presi tipa „Hendle“, pod konstantnim pritiskom od 94 kPa. Uzorci su odmah upakovani u plastične kese kako bi se sprečile fluktuacije sadržaja vlage tokom skladištenja.

Kompozit	A	B	C	D
PAS koncentracija (%)	0	0,07	0,007	0,0007

Tabela 1. PAS koncentracija

Uzorci obika crepa su sušeni izotermski. Snimljena su 4 izotermska režima za svaki kompozit.Temperatura vazduha za sušenje u prvom eksperimentu bila je podešena na 25 °C, a zatim je povećavana za 5 °C u svakom narednom eksperimentu. Brzina od 3 m/s i relativna vlažnost vazduha od 90% su održavane konstantnim tokom procesa sušenja. Ovi parametri su regulisani unutar sušare sa tačnošću od ±0,2 °C, ±0,2% i ±0,1 m/s. Masa crepova i odgovarajuće linearno skupljanje kontinuirano su praćeni i beleženi tokom procesa sušenja. Tačnost ovih merenja bila je 0,01 g i 0,2 mm. Dobijeni podaci korišćeni su za konstrukciju Deff-MR krivih.

Po dve kocke iz istog kompozita sušene su izotermski na 25 °C, 70% relativne vlažnosti i brzini vazduha od 1 m/s. Ovi eksperimenti su omogućili proračun vremena sušenja (u minutima) potrebnog za dostizanje bilo kog željenog sadržaja vlage između početnog (formiranog) i ravnotežnog sadržaja vlage. Potom je definisano dvanaest različitih nivoa vlage između početne i ravnotežne vrednosti. Po dve kocke iz istog kompozita je sušeno do svakog prethodno definisanog nivoa vlage. Nakon toga uzorci su pakovani u kese i skladišteni su u eksikatoru 24h. Eksikator je držan u klimatizovanoj prostoriji na 25 °C. Uzorci su potom vađeni kako bi im se izmerile dužina, širina i visina. Nakon toga uzorci su postavljani u presu. Brzina prirasta sile iznosila je 2 N/s. Uzorci su opterećivani do pucnja. Dobijeni podaci korišćeni su za konstrukciju MS-MR krivih. Opisana procedura ponovljena je za svaki kompozit.

3. REZULTATI I DISKUSIJA

Kritična micelarna koncentracija (CMC) je koncentracija PASa u rastvoru pri kojoj počinje formiranje micela. Ispod ove koncentracije, molekuli PASa uglavnom postoje kao pojedinačni monomeri. Kada se dostigne CMC, svaki dodatni PAS molekul formira micele umesto da ostane slobodan u rastvoru. Ovo je izuzetno važan parametar jer označava tačku u kojoj prestaje dalje smanjenje površinskog napona, a svojstva poput solubilizacije, emulgovanja i deterdžentnosti se značajno menjaju. Faktori kao što su temperatura, pritisak, jonska jačina i sama struktura PASa mogu uticati na vrednost CMC-a. Iako je CMC za Igepal CO-630 dobro dokumentovan (49,36 mg/L ili 0,005%), specifične vrednosti CMC-a za Arkopal N-100 nisu navedene u tehničkom listu proizvođača. Međutim, s obzirom na njihove strukturne sličnosti, može se pretpostaviti da bi CMC za Arkopal N-100 mogao biti u sličnom opsegu. Rezultati karakterizacije sirovine sumirani su u tabeli 1 i 2.

Sastav	[wt.%]
Kvarc	25
Feldspati (ortoklas, i plagioglas)	15
Iliti + muskovit	20
Kaolin	6
Montmorionit	6
Hlorit	4
Kalcit	7
Dolomit	6
Ostalo	11
Veličina čestica (μm)	[wt.%]
Glina (< 2)	29
Prašina (2 – 20)	51
Pesak (2 – 200)	20

Tabela 1. Fazni i granulometrijski sastav kompozita A

Na osnovu granulometrijske analize, utvrđeno je da uzorak pripada prašinasto glinovitim sirovinama. U sirovini su identifikovani ilit, muskovit, montmorilonit, hlorit, kaolinit, velike količine kalcita i dolomita, kvarc i feldspat.

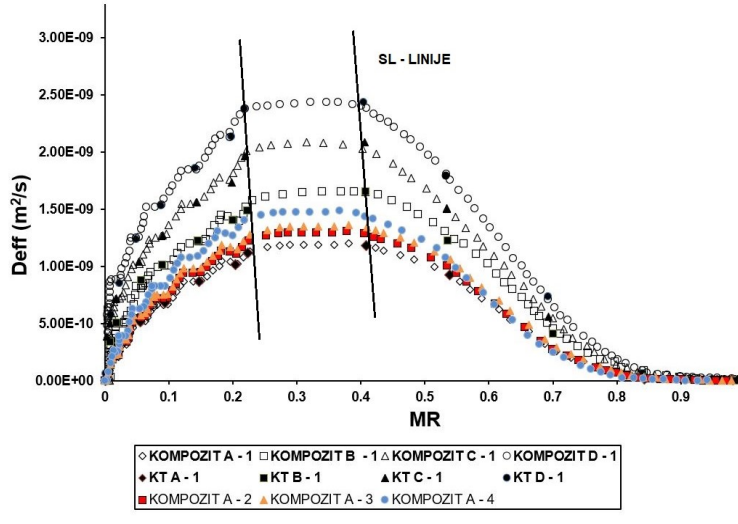
Količina vode za plastičnu obradu	21,87
Koeficijent plastičnosti po Feferkornu	27.95
Kriterijum plastičnosti	Dobro plastična
Bigo kriva – Skupljanje kod K_t	[%]
Bigo kriva – Gubitak vode kod K_t	[%]
Bigo kategorija osetljivosti	Osetljiva u sušenju

Tabela 2. Plastičnost i osetljivost na sušenje kompozita A

Deff – MR krive su konstruisane na način detaljno opisan u referencama [3-4]. Na krivama su zatim identifikovani svi mogući mehanizmi transporta vlage i njihovi prelazi iz jednog u drugi tokom konstantne i opadajuće faze sušenja. Detaljno opisan postupak konstrukcije SL linija na Deff-MR krivama koji omogućava pronalaženje karakterističnih

tačaka čak i za uzorke koji su ispucali tokom sušenja prikazan je u referenci [4]. Uslovi sušenja definisani u eksperimentu 4 su bili preoštri što je rezultiralo pojavom pukotina na uzorcima.

Rezultati su prikazani na slici 1. Jasno se uočava da je svih četri Deff – MR krive prikazano samo za slučaj kompozita 1. Za ostale kompozite (2 - 4) prikazana je samo početna kriva snimljena na temperaturi od 25°C. Na taj način je slika 1 formalno rasterećena prisustava SL linija za kompozite 2 – 4. Upravo ovakav način prikazivanja podataka omogućava čitaocu da se usredsredi na sveobuhvatno i potpuno razumevanje istorije transporta vlage tokom sušenja na primeru kompozita 1. Prethodna istraživanja su pokazala da se najkritičniji segment sušenja za sirovine u kojima su dominantne illitsko - smektitske gline nalazi na početku sušenja u blizini karakteristične tačke B [13]. Maseni transport registrovan u eksperimentima 1-B, 1-C i 1- D je do karakteristične tačke C nešto niži od onog registrovanog u eksperimentu 1-A. Evidentno je da su naprezanja kod kompozita B-D u ovom segmentu smanjena. Drugim rečima, podložnost formiranja pukotina tokom sušenja je prisustvom PASa smanjena naročito u početnoj (najkritičnijoj) fazi sušenja.

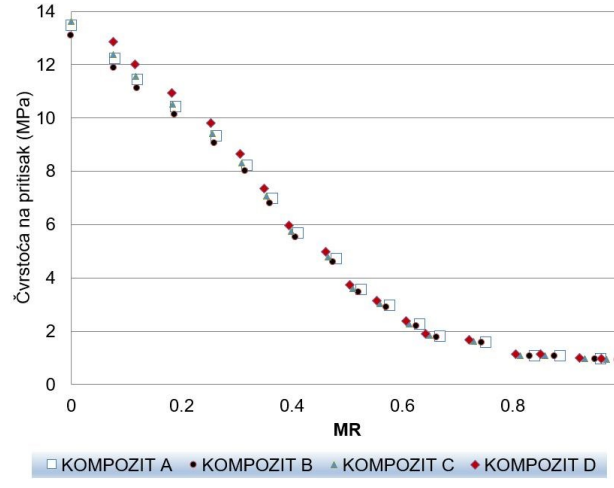


Slika 1. Deff – MR krive

Ukupni transport vlage registrovan u eksperimentima 1-B, 1-C i 1-D nakon karateristične tačke C je viši od onog registrovanog u proizvodima formiranim bez dodatka PASa. Ovo je direktna potvrda da prisustvo PASa poboljšava unutrašnji transport vlage. Važno je napomenuti da je opisani obrazac ponašanja tokom sušenja potvrđen i prilikom sušenja kompozita B – D. Ako se pojedinačne Deff – MR krive, na primer iz eksperimenta 1, uporede, može se videti da je maksimalno povećanje ukupnog transporta vlage tokom sušenja registrovano u eksperimentu D - 4. To znači da koncentracija PASa koja je malo iznad ili blizu CMC koncentracije obezbeđuje maksimalnu adsorpciju na površini gline.

Izračunate MS - MR krive su prikazane na slici 2 i 3. Oblik krivih na slici 2 je isti kao onaj registrovan u referencama [13] i [16]. Bez obzira na prisustvo ili odsustvo PASa, čvrstoća materijala je mala tokom sušenja sve dok se ne dostigne MR od 0.83. Ova

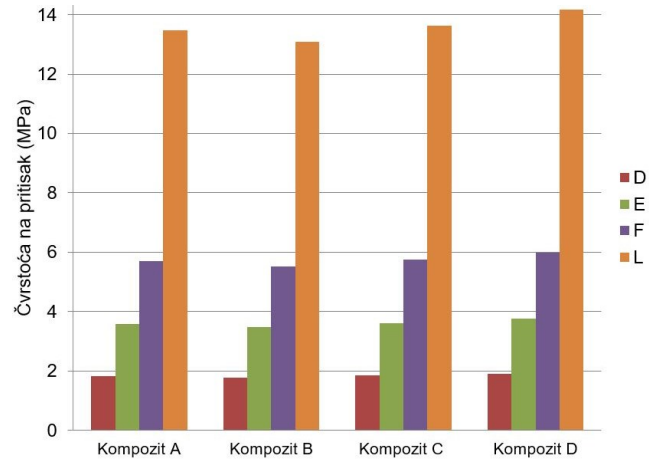
MR vrednost odgovara sadržaju vlage od 14.75%. Ovo je još jedna potvrda da je ovaj tip sirovine osetljiv na sušenje, posebno na početku. Dodavanje ARKOPALa N100 u sirovinu očigledno utiče na čvrstoću materijala. Ovaj efekat je direktno povezan sa koncentracijom PASa. Sa slike 3 se jasno uočava da su najveće čvrstoće materijala registrovane za koncentraciju PASa ispod ili blizu CMa. Na osnovu rezultata je utvrđeno da ja optimalna koncetracija AKROPALa N 100 iznosi 0,007%.



Slika 2. MS – MR krive

Adsorpcija PASa obavlja se preko vodoničnih ili Van der Valsovih sila dipol dipol sila. Ako je vodonična veza odgovorna za adsorpciju, veza koja se formira između funkcionalnih grupa PASa i minerala gline mora biti jača od veze koja se formira između minerala i medjuslojnih molekula vode [14]. Generalno reakcija PASa nejonskog tipa sa glinenim mineralima opada u nizu montmorionint, ilit kaolin. Razlog ovakvog ponašanja leži u činjenici da su strukturalno kaolinske gline sastavljene od jednog tetraedarskog sloja „silicijuma“ i jednog oktaedarskog sloja „aluminijuma“ dok se kod ilita i montmorionita kombinuju dva tetraedarska i jedan oktaedarski sloj. Pored toga važno je istaći da je razmak između strukturnih jedinaca u slučaju ilita i montmorionita različit kao i da se u prostoru izmedju slojeva kod ilita nalaze molekuli kalijuma a kod montmorionita molekuli kalcijuma ili magnezijuma. Izomorfne supstitucije i afinitet prema jonskoj izmeni je takođe različit. Banzak je utvrdio da se sa povećanim sadržajem minerala gline povećava i adsorpcija PASa nejonskog tipa [16].

Hidrofilni monomeri ARKOPAL N100 uspostavljaju vezu sa mineralima gline uglavnom kroz vodonične veze. To će dovesti do orijentacije molekula PASa na način da je njegov hidrofobni rep usmeren ka unutrašnjosti pora. Prisutnost PAS repova unutar pora će umanjiti interakcije između molekula vode i „poroznog matriksa“ tokom procesa sušenja što pored smanjenja naprezanja utiče i na ubrzavanje masen protoka tečne vode iz unutrašnjosti ka površini uzorka.



Slika 3. MS – MR krive prikazane za karakteristične tačke D-F i L

4. ZAKLJUČAK

Ovo istraživanje predstavlja obećavajuću strategiju za minimizaciju oštećenja koja nastaju tokom sušenja. U tu svrhu nejonska površinski aktivna supstanca ARKOPAL N100 je dodata u sirovinu tokom procesa homogenizacije. Rezultati su pokazali da koncentracije površinski aktivne supstance blizu ili malo iznad kritične micelarne koncentracije značajno poboljšavaju kretanje vlage iz unutrašnjosti ka površini. Pomenuta površinski aktivna supstanca je omogućila ujednačeniji transport vlage i redukciju napona koji se generišu tokm sušenja. Deff-MR i MS-MR krive potvrdile su da je ARKOPAL N 100 uticao na smanjenje napona u najkritičnijoj početnoj fazi sušenja. Pored toga, upotreba ove površinski aktivne supstance je optimizovala raspodelu vlage što je uticalo na povećanje čvrstoće materijala i smanjenje skupljanja. Predložena metoda nudi praktično rešenje za proizvođače koji žele da unaprede kvalitet proizvoda i smanje gubitke u proizvodnji tokom procesa brzog sušenja. Buduća istraživanja mogla bi da istraže uticaj drugih površinski aktivnih supstanci na različitim sirovinskim kompozitima sa ciljem prikupljanja podataka na osnovu kojih bi se tražilo optimalnije rešenje za širu industrijsku primenu.

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

Vladimir Radovanović[1]

https://doi.org/29DIMK.186R
Stručni rad

KATODNA ZAŠTITA ARMIRANOG BETONA

Rezime: *Armirano betonske konstrukcije se projektuju da traju 50 godina i duže, što je realno ako nema korozije armature. Međutim usled prirode betona kao poroznog materijala i kontaminacije betona hloridima i/ili ugljen-dioksidom iz atmosfere, dolazi do korozije armature, što dovodi do oštećenja objekta ili čak kolapsa konstrukcije pre isteka projektovanog radnog veka. Korozija uzrokuje ogromne ekonomske štete u vidu rekonstrukcije objekta, izmena ili zatvaranja saobraćaja. Rešenje je u primeni katodne zaštite. Primenom katodne zaštite se potpuno sprečava pojava korozije na novim objektima, odnosno zaustavlja se korozija na postojećim objektima dok god su ispunjeni kriterijumi standarda SRPS EN ISO 12696 čime se višestruko produžava radni vek objekta uz smanjenje troškova održavanja i očuvanje stabilnosti objekta.*
Ključne reči*: katodna zaštita, korozija, armatura, beton.*

CATHODIC PROTECTION OF REINFOCED CONCRETE

Summary: *Reinforced concrete structures are designed to last 50 years and longer, which is realistic if there is no corrosion of the reinforcement. However, due to the nature of concrete as a porous material and contamination of concrete with chlorides and/or carbon dioxide from the atmosphere, corrosion occurs, leading to damage to the structure and even collapse of the structure before the designed service life. Corrosion causes enormous economic damage in the form of reconstruction of the facility, change, or closure of traffic. The solution is in the application of cathodic protection. The application of cathodic protection completely prevents the occurrence of corrosion on new structures, and stops corrosion on existing structures as long as the criteria of SRPS EN ISO 12696 are met, which multiplies the service life of the structure while reducing maintenance costs and preserving the stability of the structure.*
Keywords*: cathodic protection, corrosion, reinforcement, concrete.*

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1. UVOD

Korozija armature u betonu je elektrohemijski proces. Usled različitih uticaja dolazi do razlike električnog potencijala između tačaka na čeliku koji je u kontaktu sa elektroprovodnom sredinom – elektrolitom, u ovom slučaju betonom. Formira se koroziona ćelija koja sadrži: katodu, anodu, metalnu vezu između katode i anode i elektrolit potreban za tok jona. Katoda je deo armature koji je elektro-pozitivniji od drugog dela koji predstavlja anodu u korozionoj ćeliji, koja je u suštini električno kolo. Deo aramture koji spaja anodnu i katodnu zonu je metalna veza za transfer elektrona, a okolni beton je elektrolit kroz koji teče struja u obliku jona.

Ima više pristupa kojima se može sprečiti korozija armature. Svi se baziraju na ukidanju nekog od elemenata korozione ćelije. Upotrebom membrana i premaza, sprečava se prodor vlage što dovodi do povećanja električnog otpora za korozinu struju i smanjuje se količina kiseonika koji je neophodan za hemjske reakcije. Dodavanje aditiva za vodonepropusnost deluje slično. Problem sa armirano betonskim konstrukcijama je što je nemoguće potpuno sprečiti prodor kontaminenata do armature, poput hlorida iz soli za odmrzavanje i CO_2 iz atmosfere u kombinaciji sa vlagom. Sve betonske konstrukcije su u nekom delu izložene silama zatezanja i deformacijama što neminovno dovodi do pojava mikro prslina zaštitnog sloja betona, što je dovoljno da armatura bude izložena koroziji. Obzirom da je osnovna uloga armature da preuzme sile zatezanja, utoliko je veći značaj zaštite od korozije armature u zoni zatezanja. Katodna zaštita je jedina metoda koja može da se primeni bez obzira na prisustvo vlage, hlorida ili karbonizaciju betona usled CO_2. Princip katodne zaštite je da se u masu betona ugrade namenske anode, tako da se koroziona reakcija izmesti sa armature na namensku anodu, koja je i predviđena da izdrži anodne reakcije. Dok god se koroziona reakcija odvija na namenskoj anodi, čelična armatura predstavlja katodu i ne korodira. Sistemi katodne zaštite se mogu projektovati da traju i duže od 100 godina. Ako nema korozije armature, nema posledičnog oštećenja betona, objekat zadržava prvobitnu stabilnost, a troškovi održavanja objekta su smanjeni.

2. ZAŠTITA ARMIRANOG BETONA OD KOROZIJE METODOM KATODNE ZAŠTITE

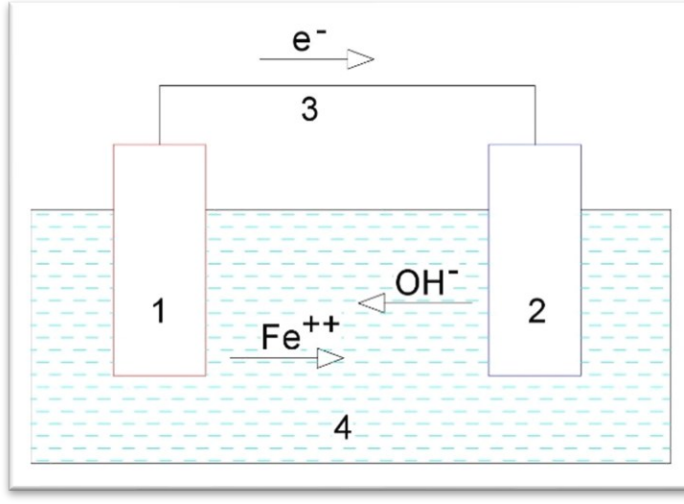
2.1. Korozija i pasivizacija čelične armature

Izloženi čelik će korodirati u vlažnoj atmosferi zbog razlika u električnom potencijalu na površini čelika, formirajući anodna i katodna mesta. Metal oksidira na anodi gde se javlja korozija prema sledećoj hemijskoj reakciji:

$Fe\ (metal) \rightarrow Fe^{2+}\ (aq.) + 2e^-$

Istovremeno, na katodnim mestima dolazi do redukcije. Tipični katodni procesi su:

$$\tfrac{1}{2}O_2 + H_20 + 2e^-\ (metal) \rightarrow 2OH^-(aq.)$$
$$2H^+(aq.) + 2e^-\ (metal) \rightarrow H_2(gas)$$



Slika 1. Osnovna šema korozione ćelije: 1 – anoda (elektro negativniji metal), 2 – katoda (elektro pozitivniji metal), 3 – metalna veza između anode i katode, 4 – elektrolit;
Source: (Izvor, TECHART katalog)

Elektroni proizvedeni tokom ovog procesa provode se kroz metal, dok se formirani joni transportuju kroz vodu iz pora betona, koja deluje kao elektrolit. Kvalitetan beton za čeličnu armaturu je okruženje visoke alkalnosti zbog prisustva hidroksida natrijuma, kalijuma i kalcijuma koji nastaju tokom reakcija hidratacije. Većina okolnog betona deluje kao fizička prepreka mnogim agresorima čelika. U takvom okruženju čelik je pasivan i svaki mali prekid njegovog zaštitnog oksidnog filma se ubrzo popravlja. Međutim, ako se alkalnost okoline smanji, na primer neutralizacijom atmosferskim ugljen-dioksidom, ili u slučaju da depasivizirajući anjoni kao što je hlorid dođu do čelika, onda može doći do smanjenja pH i ozbiljne korozije armature. Ovo može dovesti do mrlja od rđe na betonu i delaminacije betona zbog povećanja zapremine korodirane armature, povezanog sa konverzijom gvožđa u gvožđe oksid.

Faktori koji određuju brzinu korozije čelika u betonu su: prisustvo jonski provodljive vodene faze u kontaktu sa čelikom (tj. voda u porama betona), postojanje anodnih i katodnih mesta na metalu u kontaktu sa ovim elektrolitom i dostupnost kiseonika da bi se omogućilo odvijanje reakcija.

Propustljivost betona je važna u određivanju stepena do kojeg spoljašnje agresivne supstance mogu da ugroze čelik. Debeli betonski pokrivač niske propustljivosti će verovatno sprečiti jone hlorida iz spoljašnjeg izvora da dođu do čelika i izazovu depasivizaciju zaštitnog sloja armature.

Tamo gde je adekvatnu debljinu zaštitnog sloja betona teško postići zbog dizajna ili gde se očekuje agresivno okruženje, kao što su betonske površine mostova, tunela ili parkinga, može biti potrebna dodatna zaštita za ugrađeni čelik. Zaštita čelične armature može se izvesti na više načina.

Sama čelična armatura se može učiniti otpornijom na koroziju obezbeđivanjem zaštitnog premaza na bazi epoksidne smole ili nanošenjem prevlake od nerđajućeg čelika

ili cinkovanjem. U ekstremnim okolnostima, može se koristiti armatura od nerđajućeg čelika. Ograničavajući faktor upotrebe armature otporne na koroziju je visoka cena.

Nema sumnje da je najefikasniji način zaštite čelika koji je ugrađen u beton adekvatna debljina zaštitnog sloja betona visoke čvrstoće i niske propustljivosti bez depasivirajućih jona kao što su hloridi. Međutim, u praksi, beton se ugrađuje u velikim količinama u svim vremenskim uslovima i okruženjima, izložen industrijskoj atmosferi, solima za odmrzavanje i morskoj vodi. Kontaminirane materijale i propuste u ugradnji je teško u potpunosti izbeći, ali razumevanjem često složenih hemijskih i elektrohemijskih uslova koji mogu postojati, moguće je primeniti metod za izgradnju struktura koje će trajati i duže od 100 godina.

2.2. Mehanizam propadanja betona

Većina armirano betonskih konstrukcija širom sveta vrši svoju funkciju i nije značajno ugrožena od korozije. Manji broj struktura je oštećen usled dejstva agresivnih komponenti iz spoljašnje sredine ili zbog nekompatibilnosti sastojaka betona. Problemi mogu nastati kao rezultat lošeg dizajna, loše specificiranog betona, loše izrade i niza drugih faktora.

Mehanizmi propadanja betona su prvenstveno hemijsko-fizičke prirode (tj. hemijska reakcija sa formiranjem proizvoda veće zapremine od reaktanata koji proizvode fizičke efekte kao što su pucanje i delaminacija) i odvijaju se u tri faze:

Faza 1: Inicijacija (t_0) – Koncentracija agresivnih vrsta je nedovoljna za pokretanje bilo kakve hemijske reakcije ili se hemijska reakcija odvija veoma sporo. Nije došlo do fizičkog oštećenja betona. Trajanje t_0 može varirati od nekoliko minuta do projektovanog veka trajanja konstrukcije;

Faza 2: Propagacija (t_1) – Hemijske reakcije počinju ili se nastavljaju, može doći do fizičkog oštećenja betona, ali nije dovoljno da ugrozi konstrukciju. Ubrzanje procesa propadanja obično se dešava tokom ove faze zbog povećane dostupnosti agresivnih jona ili modifikacije betonskog okruženja;

Faza 3: Raspadanje (t_2) – Brzo raspadanje konstrukcije. Kombinovani efekti fizičkih i hemijskih procesa su dovoljno izraženi da konstrukcija više nije bezbedna i potrebni su obimni radovi na sanaciji ili u ekstremnim slučajevima rušenje.

2.3. Faktori koji utiču na stopu korozije

Okruženje koje obezbeđuje kvalitetan beton za ugrađenu čeličnu armaturu je okruženje visoke alkalnosti (uglavnom >pH 13), koje stvaraju hidroksidi natrijuma, kalijuma i kalcijuma koji se oslobađaju tokom različitih reakcija hidratacije. Pored toga, najveći deo okolnog betona deluje kao fizička barijera za većinu supstanci koje mogu dovesti do degradacije armature.

Pod uslovom da se ovo okruženje održava, čelik ostaje pasivan i svi mali prekidi u stabilnom zaštitnom oksidnom filmu se ubrzo popravljaju. Međutim, ako se alkalnost okoline smanji, na primer reakcijom sa atmosferskim ugljen-dioksidom (karbonizacija), ili ako su depasivirajući hloridni joni prisutni na površini čelika, može doći do korozije, što rezultira gubitkom poprečnog preseka armature i delaminacijom betona.

2.3.1. Debljina zaštitnog sloja

Neadekvatni zaštitni sloj betona je uvek povezan sa područjima visokog rizika od korozije zbog karbonizacije i prodiranja hlorida. Merenjem debljine zaštitnog sloja, područja visokog rizika mogu se lako identifikovati.

2.3.2. Pukotine u zaštitnom sloju betona

Generalno, dva oblika pukotina su od interesa kada se ocenjuje stanje armiranobetonske konstrukcije; one prisutne pre početka korozije koje bi mogle da inteziviraju procese korozije (npr. usled skupljanja ili pomeranja konstrukcije) i one koje su nastale kao direktna posledica korozije gde ekspanzivni proizvodi korozije dovode do pucanja i lomljenja betona.

2.3.3. Prisustvo hloridnih jona

Hloridni joni mogu ući u beton na dva načina:

- mogu se dodati tokom mešanja, namerno kao aditiv ili kao kontaminent u originalnim sastojcima, ili
- mogu ući u beton iz spoljašnjeg izvora kao što su morska voda ili so za odmrzavanje;

Kada hloridni joni stignu do armature u dovoljnim količinama, oni će depasivirati ugrađeni čelik razbijanjem zaštitnog oksidnog sloja koji se normalno održava u alkalnom okruženju.

Koncentracija hloridnih jona potrebnih za pokretanje i održavanje korozije zavisi od alkalnosti i pokazano je da postoji skoro linearna veza između koncentracije hidroksil jona i odgovarajućeg graničnog nivoa hlorida. Merenje koncentracije hlorida je definisano u standardu SRPS EN 14629.

2.3.4. Karbonazacija

Ugljen-dioksid prisutan u atmosferi kombinuje se sa vlagom u betonu i formira ugljenu kiselinu. Ova kiselina reaguje sa kalcijum hidroksidom i drugim alkalnim hidroksidima u vodi u porama betona, što dovodi do smanjenja alkalnosti betona. Na brzinu kojom dolazi do ove neutralizacije utiču faktori kao što su nivoi vlage i kvalitet betona. Dubina karbonizacije u konstrukciji može se prilično lako utvrditi upotrebom fenolftaleinskog indikatora na sveže izloženom materijalu u skaldu sa SRPS EN 14630.

2.3.5. Uticaj okoline

Mikroklima kojoj je armiranobetonski element izložen direktno utiče na verovatnoću i obim korozije armature. Faktori kao što su nivoi hloridnih jona i pH su već razmatrani, ali najvažniji aspekt lokalne sredine je nivo vlage. Karbonizacija, ulazak hloridnih jona, otpornost i brzina korozije su sve pod velikim uticajem stepena zasićenja vlage.

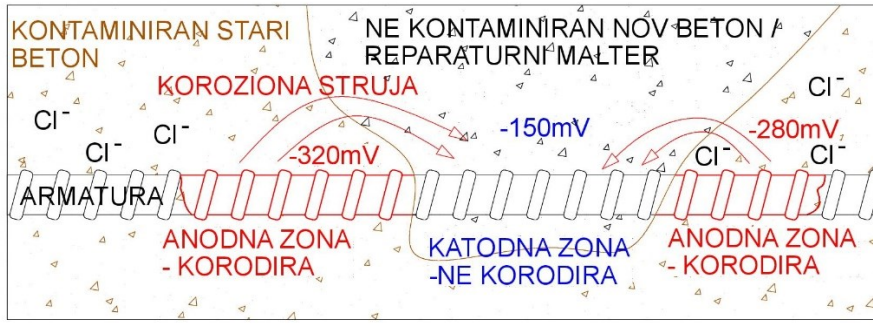


Slika 2. Karakteristično oštećenje konstrukcije od korozije;
Source: (Izvor, Istockphoto)

2.4. Konvencionalne popravke betona i korozija

Popravka postojećih armiranobetonskih konstrukcija gde je korozija već aktivna i gde su prisutne pukotine i odvaljivanje delova betona, obično podrazumeva ukanjanje odvojenog betona i ugradnju novog betona/reparaturnog maltera.

Sa aspekta korozije ovakva popravka predstavlja armaturu u dve susedne hemijski različite sredine, gde je novi beton bez kontaminacije (hloridi i/ili CO2), a stari okolni beton je kontaminiran, ali još uvek bez vidljivih oštećenja. Posledica ove razlike u elektrolitu je pojava razlike potencijala i izazivanje korozione struje, gde se armatura u starom betonu ponaša kao anoda (korodira), a armatura u novom betonu se ponaša kao katoda (ne korodira). Posle svega par godina pojavljuju se oštećenja na starom betonu oko popravke, što se naziva „prstenastom“ anodom. Uklanjanje i zamena svog kontaminiranog betona na konstrukciji zahvaćenog karbonizacijom ili kontaminacijom holiridima bi sprečilo pojavu „prstenaste anode“ ali to je često previše skupo rešenje ili teško izvodljivo sa apekta organizacije radova. Optimalno rešenje za ovaj problem je primena katodne zaštite samo za „zakrpe“ betona bez obzira što je okolni beton zahvaćen karbonizacijom ili kontaminiran.



Slika 3. Primer korozione ćelije kod konvencionalnih popravki betona; Source: (Izvor, TECHART katalog)



Slika 4. Primer „prstenaste“ anode – korozija se nastavlja oko popravke betona; Source: (Izvor, The Principles and Practice of Galvanic Cathodic Protection for Reinforced Concrete Structures Technical Note 6, G Sergi, J Broomfield and N Davison)

2.5. Katodna zaštita

Katodna zaštita podrazumeva da se u sistem uvodi namenska veštačka anoda. Jednosmerna struja se prenosi sa veštačke anode kroz elektrolit - beton na armaturni čelik koji treba da se zaštiti. Struja prolazi kao tok jona kroz vodu u porama betona do armature. Efekti struje katodne zaštite su:

- odbija sve hloridne jone od površine čelika;

- stvara hidroksilne jone koji popravljaju bilo kakvo oštećenje zaštitnog sloja pasivnog oksida na čeliku i povećavaju alkalnost u betonu na površini čelika;
- smanjuje potencijal anodnih (korodirajućih) oblasti na čeliku tako da struja više ne teče iz njih i korozija prestaje;

Sistemi katodne zaštite se dele na:

- Sistem katodne zaštite sa nametnutom strujom i
- Sistem katodne zaštite sa galvanskim anodama.

Princip delovanja je isti, ali se razlikuju namenske anode koje koriste, oblast primene i magnitude zaštitne struje. Katodna zaštita sa nametnutom strujom je prvi put isprobana na armirano-betonskom mostu kasnih 1950-ih u SAD. Sada se primenjuje na milionima kvadratnih metara betonske površine širom sveta. Galvanska katodna zaštita je prvi put primenjena na armiranom betonu 1990-ih na Floridi, SAD.

2.5.1. Sistem katodne zaštite sa nametnutom strujom

Katodna zaštita sa nametnutom strujom se bazira na izvoru jednosmerne struje koji obezbeđuje zaštitnu struju od namenske anode kroz elektrolit (vodu u porama betona) ka zaštićenoj metalnoj površini, da bi se promenio smer električnih struja povezanih sa procesom korozije. Anoda mora biti raspoređena po području koje treba zaštititi. Materijal za anode mora biti dovoljno robustan da izdrži hemijske reakcije i uslove eksploatacije.

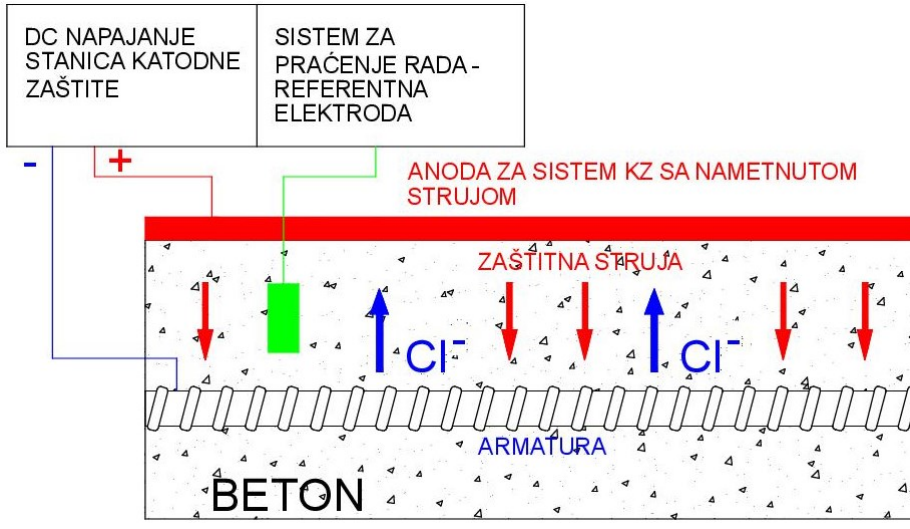
Postoji više tipova anoda koji mogu da se koriste u zavisnosti od konkretne strukture, uslova eksploatacije i dužine radnog veka.

Na raspolaganju su prvenstveno četiri osnovna tipa anodnih sistema, a to su:

- Provodni premazi;
- MMO titanijumska anoda u formi mreže ili trake;
- Provodni cementni sloj;
- Diskretne anode.

Pre nego što se instalira sistem katodne zaštite sa nametnutom strujom, potrebno je uzeti u obzir sledeće faktore za projektovanje:

- Obim oštećenja betona i prirodu i obim prethodnih popravki;
- Rezultate istraživanja: dubinu karbonizacije i profil hloridnih jona;
- Debljinu zaštitnog sloja betona;
- Električnu otpornost betona i postojećih popravki;
- Raspored armature i površinu iste;
- Električni kontinuitet armature;
- Metalne elementi i čelične konstrukcije u kontaktu sa betonom štićenog objekta;
- Moguću lokaciju za sistem električnih kablova;
- Dodatnu nosivost konstrukcije;
- Prisustvo čelika za prednaprezanje radi izbegavanja ili kontrole rizika od vodonične krtosti.



Slika 5. Model sistema katodne zaštite sa nametnutom strujom;
Source: (Izvor, TECHART katalog)

Sistemi katodne zaštite sa nametnutom strujom zahtevaju praćenje rada, obično u intervalima od šest meseci. Upotreba referentnih elektroda ugrađenih u beton omogućava daljinsko praćenje i kontrolu, bilo na stanici katodne zašite ili daljinski.

Korisni efekti katodne zaštite sa nametnutom strujom su:

PRIMARNI

- Negativna polarizacija čelika (električni potencijal je postao negativniji u odnosu na vrednost pre nametanja zaštitne struje);
- Sve lokalno generisane korozione ćelije su nadjačane;

SEKUNDARNI

- Uklanjanje hloridnih jona putem jonske migracije;
- Povećanje koncentracije hidroksilnih jona na čeličnoj armaturi i povećanje pH okolnog betona;

2.5.2. Sistem katodne zaštite sa galvanskim anodama

Sistem se sastoji od anode na bazi legure cinka koja je u električnom kontaktu sa armaturom. Usled prirodne razlike potencijala između cinka i čelika, struja samostalno teče od galvanske anode ka čeliku i izaziva sledećenje efekte:

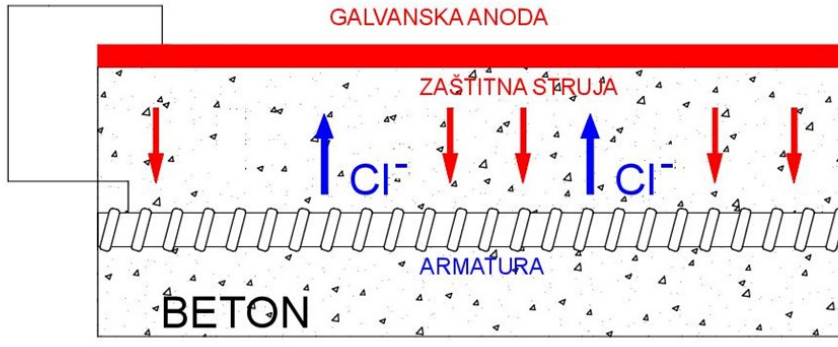
- Negativnu polarizaciju čelika;
- Odbijanje jona hlorida od čelika;
- Realkalizaciju betona oko aramature.

Postoji više tipova galvanskih anoda za atmosferski izložene betonske konstrukcije, poput:

- Termički prskani cink ili aluminijum-cink-indijum;
- Cink u specijalno formulisanim malterima;
- Legure cinka u obliku mrežice, fiksirane na beton i prekrivene malterom;

- Legura cinka u formi tankog lima pričvršćena za konstrukciju provodljivim lepkom;
- Diskretne cinkane anode ugrađene u masu betona.

Galvanske anode korodiraju jer su elektronegativnije od čelika. Princip rada je sličan pocinkovanju čelične konstrukcije. Kako ne postoji napajanje električnom energijom, ne postoji način kontrole ili podešavanja sistema galvanske katodne zaštite, osim dodavanjem više anoda. Galvanska katodna zaštita je stoga jednostavnija od katodne zaštite sa nametnutom strujom, ali daje niži nivo kontrole korozije, manje zaštitne struje i ima kraći radni vek.



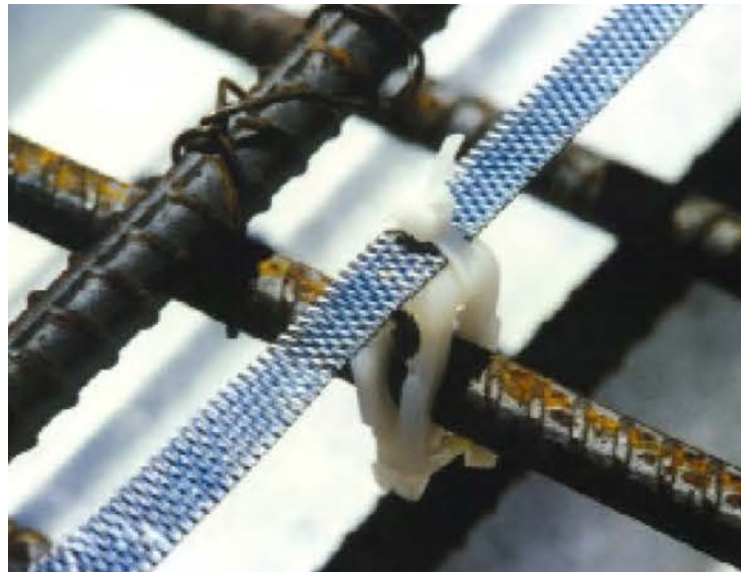
Slika 6. Model sistema katodne zaštite sa galvanskim anodama
Source: (Izvor, TECHART katalog)

2.6. Primena katodne zaštite na nove konstrukcije u fazi izgradnje

Katodna zaštita novih armiranobetonskih konstrukcija (koja se ponekad naziva i katodna prevencija, jer je to metoda koja se koristi za sprečavanje nastanka korozije) se koristi u korozivnim sredinama, kao i za konstrukcije u manje korozivnim sredinama, ali gde su dugovečnost i garantovana izdržljivost od posebne važnosti. Korozivne sredine se obično definišu kao one u kojima armirani beton može biti izložen rutinskom izlaganju hloridima tokom radnog veka konstrukcije, na primer, morske konstrukcije, konstrukcije autoputa ili tunela podvrgnute solima za odleđivanje i konstrukcije posebne industrijske primene. Katodna zaštita može sprečiti koroziju armature na svim nivoima kontaminacije hloridima, čineći tako armiranobetonske konstrukcije trajnim u agresivnom okruženju za relativno dug životni vek (obično preko 100 godina). Katodna zaštita primenjena na armiranobetonske elemente pre početka korozije zahteva približno jednu desetinu energije za sprečavanje korozije od one potrebne za zaustavljanje i sprečavanje korozije nakon što je korozija započela. Tako, sistemi katodne zaštite za nove konstrukcije predstavljaju niže troškove instalacije i rada, koriste manje materijala zbog niže potrebne zaštitne struje i ekološki su prihvatljiviji u poređenju sa popravkama oštećenja betona ili naknadno ugrađenom katodnom zaštitom tokom radnog veka konstrukcije.

Troškovi katodne zaštite pri izgradnji objekta su u proseku reda veličine oko 5% objekta koji se štiti. Cena zavisi od veličine štićene betonske površine. Generalno manja je cena po m2 površine koja se štiti ako je ukupna štićena površina veća jer se troškovi stanice katodne zaštite raspodeljuju na veću površinu. Pri proceni troškova treba imati u

vidu da se može uraditi optimizacija troškova, odnosno odabir površina betona koje su više ugrožene korozijom. U tom slučaju katodna zaštita se primenuje samo na odabrane površine betonske konstrukcije što utiče na smanjenje ukupunog troška.



Slika 7. MMO trakasta anoda postavljena na armaturu za sistem KZ sa nametnutom strujom pre ugradnje betona
Source: (Izvor, CPCL katalog)

2.7. Zahtevi za ugradnju katodne zaštite

Katodna zaštita zahteva da armatura (tj. katoda) bude električno povezna tj. kontinualna, što se postiže pomoću veznih žica neophodnih za konstrukciju armaturnog kaveza, a gde je to prikladno, one se mogu dopuniti zavarenim spojevima. Obezbeđivanje odgovarajućeg električnog kontinuiteta može se lako proveriti pre postavljanja betona. Najčešći oblik anode koji se koristi za novogradnju izloženu atmosferi su titanijumske trake obložene metalnim oksidom (MMO). One se ugrađuju u zaštitni sloj betona pomoću nemetalnih pričvršćenja. Da bi se sprečili kratki spojevi, važno je osigurati da trake ostanu električno izolovane od armature tokom ugradnje i postavljanja betona. Razvijene su standardne metode fiksiranja koje su omogućile da se na ovaj način ugradi više hiljada kilometara anodnih traka. Anode, kablovi i referentne elektrode se postavljaju na armaturu pre izlivanja betona. Posle provera spojeva, vrši se ugradnja betona. Neophodno je obezbediti da tipovi izolacije kablova, materijal anode i sve pomoćne komponente imaju potreban projektni vek. Ubrzane metode ispitivanja, na primer NACE TM 02945, dostupne su da bi se demonstrirao životni vek anodnih materijala. Kablovi se obično završavaju u razvodnim kutijama na površini beotna. Sistemi katodne zaštite obično imaju automatsko upravljanje, daljinski nadzor i kontrolu.

2.8. Ispitivanje korozije postojećih betonskih konstrukcija

Ispitivanje korozije na betonskim konstrukcijama se sve više koristi za praćenje degradacije objekta, kao i za definisanje područja kojima je potrebna popravka. Korozija armature počinje kao lokalizovan problem.

Utvrđivanje rizika od korozije ne može se postići samo nasumičnim uzorkovanjem hlorida i vizuelnim/delaminacionim pregledima. Testiranje treba da bude pravilno planirano od strane kompetentnih stručnjaka i prilagođeno specifičnim potrebama svake strukture.

Početni uzrok korozije je obično rezultat niza međusobno povezanih promenljivih:

- promene u permeabilnosti betona uzrokovane lokalnim razlikama u sastavu, zbijanju i očvršćavanju;
- greške u konstrukciji koje dovode do područja sa tankim betonskim slojem preko armature;
- varijacije u okruženju;

što dovodi do područja sa povišenim sadržajem hloridnih jona ili velikom dubinom karbonizacije. Prvi slučajevi iniciranja korozije često će biti u oblastima gde se na određenom elementu poklapaju propusni beton, tanak zaštitni sloj betona i teži uslovi sa aspekta uticaja okoline. Takođe, često postoji kašnjenje od nekoliko godina između trenutka ugrađivanja čelika koji postaje korozivno aktivan i razvoja korozije koja dovodi do vidljivih oštećenja.

Konstrukcije se često mogu podeliti na različite kategorije izloženosti i ugroženosti od korozije. Na primeru mosta se razlikuju zone koje su izložene kvašenju i hloridima iz soli za odmrzavanje (diletacije, poluspojevi, drenaža i dr.) i površine koje su zaštićene oblikom mosta od padavina te su izložene samo karbonizaciji.

Cilj sveobuhvatnog ispitivanja korozije treba da bude razumevanje mehanizma korozije, utvrđivanje rizika i razvoj strategije popravke i održavanja koja obezbeđuje najbolje tehničko i komercijalno rešenje za konstrukciju.

Najčešće korišćene metode za utvrđivanje rizika od korozije su:

1. Vizuelni pregled;
2. Ispitivanje delaminacija betona;
3. Merenje električnog potencijala i mapiranje.

Nalazi 1, 2 i 3 treba da se koriste za odabir reprezentativnih lokacija za:

- Detaljno merenje i mapiranje električnog potencijala;
- Izradu profila hlorida;
- Ispitivanje dubine karbonizacije;
- Praćenje brzine korozije;
- Ispitivanje električne otpornosti betona.

2.9. Katodna zaštita za postojeće konstrukcije

Katodna zaštita se može ugraditi i na postojećim objektima. Primenom katodne zaštite korozija se zaustavlja i poprečni presek aramture se više neće smanjivati. Obično se katodna zaštita ugrađuje u okviru rekonstrukcije i popravki arimirano betonskih konstrukcija. Neophodno je izvršiti poveru i omogućiti električni kontinuitet armature. U zavisnosti od odabranog tipa anoda vrši se priprema površine betona ili se prosecaju kanali za trakaste anode ili se buše rupe za diskretne anode;



Slika 8. MMO trakasta anoda postavljena u posečen žljeb pre ugradnje maltera
Source: (Izvor, https://www.vector-corrosion.com/lida-mmo-titanium-anodes)



Slika 9. Izgled zida posle ugradnje distribuiranih diskretnih anoda
Source: (Izvor, The Principles and Practice of Galvanic Cathodic Protection for Reinforced Concrete Structures Technical Note 6, G Sergi, J Broomfield and N Davison)

Troškovi primene katodne zaštite na postojećim objektima gde je korozija već prisutna su veći nego u slučaju kada se katodna zaštita postavi pri izgradnji objekta. Razlog je veća potreba za zaštitnom strujom što iziskuje veće količine materijala katodne zaštite a i usled pratećih troškova za građevinske radove i izradu pristupa konstrukciji.

Prateći građevinski radovi obuhvataju uklanjanje odvojenog betona, pripremu betonske površine, postavljanje skela, ugradnju novog betona, itd. Racionalno je ugraditi sistem katodne zaštite prilikom rekonstrukcije objekta. Primer troškova katodne zaštite mosta se može videti u literaturi [5].

3. ZAKLJUČAK

Većina armiranobetonskih konstrukcija pokazuje odličnu izdržljivost i dobre performanse tokom svog projektnog veka, međutim agresivno okruženje ili loša građevinska izvedba mogu dovesti do korozije armaturnog čelika u betonu.

Glavni mehanizmi za koroziju su ulazak ugljen-dioksida iz atmosfere (karbonizacija) i napad hlorida od ulivenih ili difuznih hlorida. Obzirom da se saobraćajnice tretiraju solju protiv zamrzavanja i izložene su povećanom zagađenju atmosfere, kao takve su posebno ugrožene od korozije.

Najčeći uzrok oštećenja betona je korozija armature, tj. porast zapremine korozionih naslaga usled stvaranja oksida gvožđa armature što stvara sile dovoljne za pucanje betona. Mehanizmi korozije i propadanja su u suštini isti i za karbonizaciju i za napad hloridima.

Primena katodne zaštite je najefikasnija metoda za sprečavanje i zaustavljanje korozije armature bez obzira na prisustvo vlage, karbonizacije, jona hlorida. Katodna zaštita se može primeniti na nove i na postojeće konstrukcije. Katodna zaštita sprečava koroziju dok god je ispunjen kriteriujm standarda SRPS EN ISO 12696. Primenom katodne zaštite radni vek konstrukcije se višestruko produžava, a troškovi održavanja se smanjuju jer nema više oštećenja betona i posledičnih popravki. Primenom katodne zaštite se čuva životna sredina jer se čuvaju resursi inače potrebni za popravke betona i sprečava se zagađenje koje se stvara pri popravkama;

Zahvale

Zahvaljujemo se kompaniji TECHART doo na pomoći pri izradi ovog rada.

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

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PREGLED STANJA U PODRUČJU RECIKLIRANJA EPS-A I ZAHTJEVI ZA PRIMJENU EPS-A KOD PLOČA ZA PODNO GRIJANJE I HLAĐENJE

Rezime: *Unatoč naglašavanju održivosti, umjesto recikliranja, dominantan način zbrinjavanja otpada u regiji je pohrana u tlo. Analizom dosadašnjih istraživanja recikliranja EPS-a, uočeni su nekonzistentni zaključci o degradaciji recikliranog EPS-a. Kao jedna od mogućih primjena recikliranog EPS-a, predlažu se ploče za podno grijanje i hlađenje. Dan je pregled minimalnih traženih zahtjeva za iste, postojećih sustava podnog grijanja te analiza prednosti i nedostataka tehničkih rješenja za ugradnju cijevi u ploče. Cilj članka je ukazati na rezultate dosadašnjih istraživanja, nedostatak spoznaja, te potaknuti daljnja istraživanja u području recikliranja EPS-a i njegove buduće primjene.*
Ključne riječi: *reciklirani EPS, izolacijske ploče za podno grijanje i hlađenje, varijabilnost svojstava, smanjenje potrošnje resursa, razvoj kružnih sustava*

STATE OF THE ART REVIEW IN THE FILED OF EPS RECYCLING AND REQUIREMENTS FOR THE APPLICATION OF EPS IN UNDERFLOOR HEATING AND COOLING BOARDS

Summary: *Despite emphasizing sustainability, the dominant method of waste management in the region is still landfill, rather than recycling. An analysis of previous EPS recycling research reveals inconsistent conclusions regarding the degradation of recycled EPS. One proposed application of recycled EPS is in the production of boards for underfloor heating and cooling. An overview of the minimal requirements for such boards, existing underfloor heating systems, and an analysis of the advantages and disadvantages of technical solutions for pipe installation in the boards are provided. The aim of the article is to highlight the results of previous research, the lack of knowledge, and to encourage further research into EPS recycling and its future applications.*
Keywords: *recycled EPS, insulation boards for underfloor heating and cooling, variability of properties, reduction of resource consumption, development of circular systems*

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1. UVOD

Globalno društvo suočava se s ozbiljnim izazovima gospodarenja otpadom i prelaskom na kružno gospodarstvo. Dominantan način zbrinjavanja otpada u regiji još je uvijek pohrana u tlo, izuzev Slovenije koja je jedna od zemalja s najmanjom količinom otpada na odlagalištima u Europskoj uniji, zahvaljujući visokim stopama recikliranja, kompostiranja i termičke obrade [1].

Ekspandirani polistiren (EPS), poznatiji kao stiropor, razvijen 1940-ih, sastoji se od 98% zraka i 2% polistirena [2]. Ovaj svestrani materijal pronašao je primjenu u raznim industrijama, uključujući pakiranje, robu široke potrošnje, građevinske materijale, poljoprivredu itd. Kao i svaki materijal, na kraju svog životnog vijeka postane otpad. Zbog niske biorazgradivosti, u okolišu može ostati više od stotinu godina [3]. Iako dobar kandidat za recikliranje, njegova niska gustoća otežava isplativost transporta, što može ograničiti mogućnosti recikliranja [8]. O opravdanosti recikliranja je moguće govoriti, prije svega, ako je lokacija recikliranja u blizini lokacije prikupljanja materijala.

U Europi se plastični otpad najčešće obrađuje spaljivanjem, dok recikliranje zauzima drugo mjesto. Ipak, velika količina plastike i dalje završava pod zemljom [4]. Oko polovice prikupljenog otpada za recikliranje izvozi se zbog nedostatka lokalnih kapaciteta [5]. Osim toga, globalna proizvodnja i spaljivanje plastike doprinose značajnim emisijama stakleničkih plinova, što je kritičan problem.

Nedostatak kapaciteta, tehnologije i sredstava za lokalnu obradu plastičnog otpada, kao što je ranije spomenuto, predstavlja prepreku u prelasku na kružno gospodarstvo, kao i nedostatak istraživanja u području recikliranja plastičnog otpada, konkretno EPS-a, na što će ovaj rad nastojati ukazati u redovima koji slijede.

2. O RECIKLIRANJU

Ekspandirani polistiren (EPS) zbog svojih toplinskih svojstava, male gustoće i otpornosti na vlagu široko je primjenjivan u građevinarstvu i ambalaži, no njegova trajnost i niska biorazgradivost čine ga ekološki izazovnim materijalom. U kontekstu zahtjeva za održivošću i kružnim gospodarenjem otpadom, recikliranje EPS-a sve više dobiva na važnosti. Međutim, prepreke poput niske svijesti potrošača, nedostatne infrastrukture te visokih troškova prikupljanja i obrade često rezultiraju njegovim odlaganjem ili energetskom oporabom, umjesto recikliranjem. Osim toga, područje recikliranja EPS-a još je uvijek slabo istraženo – osobito kada je riječ o svojstvima i potencijalnoj primjeni recikliranog materijala. Buduća vrijednost recikliranog EPS-a uvelike ovisi o korištenoj metodi recikliranja, prethodnoj namjeni te kvaliteti obrade i čišćenja post-potrošačkog materijala.

2.1. Metode recikliranja

Recikliranje ekspandiranog polistirena (EPS) moguće je provesti trima osnovnim metodama: mehaničkom, kemijskom i termičkom. Mehaničko recikliranje (sekundarno) podrazumijeva prikupljanje otpada, razdvajanje, čišćenje, drobljenje, granuliranje za potrebnu konverziju i ponovnu preradu, što rezultira proizvodnjom novih proizvoda bez promjene kemijskog sastava materijala (očuvanje polimernog lanca) [6]. Jedan od

najvažnijih preduvjeta za provedbu procesa mehaničkog recikliranja je uklanjanje nečistoća, tijekom kojeg mogu nastati značajne promjene u sastavu polimera. Veća kontaminacija otpada vodi većoj složenosti postupka. Tijekom procesa drobljenja, dolazi do fizičkog oštećenja kuglica, čime se gubi optimalan oblik istih što uzrokuje pad konačne gustoće. Svako mehaničko recikliranje narušava u određenoj mjeri svojstva EPS-a i dovodi do heterogenosti materijala, no ne postoje istraživanja u kojima je kvantificirana degradacija svojstava.[7] [8] [9]

Polimerni lanac moguće također je očuvati suvremenim metodama recikliranja pomoću otapala. D-limonen je otapalo prehrambenog podrijetla koje je ekološki prihvatljivije od drugih otapala poput toluena, unatoč skupom postupku. Druga prirodna ulja, uključujući ulje zvjezdanog anisa, eukaliptusa, timijana i kamilice, mogu se koristiti kao alternativa d-limonenu za učinkovito smanjivanje volumena EPS-a bez smanjenja molekularnih masa. Međutim, kod ovih metoda je potrebno uzeti u obzir utjecaj na okoliš i financijske aspekte zbog potrebe za velikim količinama otapala. [10]

Kemijsko (tercijarno) recikliranje depolimerizira EPS u stiren-monomer katalitičkim procesima [11], mijenjajući kemijsku strukturu polimera. Uključuje upotrebu različitih kemikalija za redukciju EPS-a. Proizvodi su manje molekule tekućine ili plinova koji se obično koriste k za dobivanje novih goriva i kemikalija [12]. Prednost su niski energetski zahtjevi, a glavni nedostatak složenost procesa. Većina kemijskih metoda zahtijeva stručnost i iskustvo istraživanja. Također, većina kemikalija koje se koriste u kemijskom recikliranju su iznimno skupe i nisu ekološki prihvatljive. [10]

Piroliza i uplinjavanje, iako ponekad svrstane u termičko recikliranje [10], [13], su termo-kemijski procesi depolimerizacije. Piroliza može obraditi kontaminiranu plastiku [16] i pretvara otpad u korisne ugljikovodike [12], no ima visoke energetske zahtjeve i dugo vrijeme reakcije. Uplinjavanje pretvara otpad u energiju taljenjem na visokim temperaturama (1200-1500 °C) [3], smanjujući odlaganje i troškove spaljivanja. Međutim, nedostaci su visoki energetski zahtjevi, emisije stakleničkih plinova i utjecaj dušika iz zraka Zbog navedenih nedostataka, kemijsko recikliranje se trenutno ne primjenjuje u praksi za EPS otpad.

Termičko recikliranje ili energetska oporaba se može provoditi uplinjavanjem i izgaranjem [3] učinkovito smanjujući volumen otpada uz oporabu energije u modernim postrojenjima kružnog gospodarstva. Iako generira značajnu energiju iz polimera, nije ekološki prihvatljivo zbog rizika od toksičnih tvari u zraku [14] te se stoga uglavnom ne smatra oblikom recikliranja.

2.2. Prethodna namjena i obrada post-potrošačkog EPS-a

Reciklabilni EPS potječe iz proizvodnje, građevinskih ostataka, ambalaže (elektronika, ribarstvo, poljoprivreda) i potrošačkog otpada, koji je tijekom upotrebe mogao biti izložen UV zračenju, oksidaciji ili toplinskoj degradaciji što negativno utječe na svojstva materijala. [4]

EPS iz ETICS sustava rijetko se reciklira zbog dugog vijeka trajanja (50 godina). EPS proizveden prije 2015. sadrži toksični HBCDD, čije uklanjanje otapalima komplicira i poskupljuje proces [15]

Obrada jako onečišćenog EPS otpada zahtijeva složene postupke čišćenja, posebno kod ambalaže iz prehrambene industrije. EPS u elektroindustriji je relativno čišći i lakši za obradu. Prije recikliranja ključno je sortiranje, obrada i čišćenje radi uklanjanja nečistoća

[4]. Prije samog procesa čišćenja reciklata, potrebno se pobrinuti da je reciklat odvojen od ostatka nepoželjnog otpada kako ne bi smanjivao kvalitetu reciklata, te uz to povećavao troškove uklanjanja nečistoća.

Čišćenje se provodi mehanički ili kemijski [4] , a uključuje degasifikaciju, filtraciju, pranje (s vodom i kemikalijama) uz specijaliziranu opremu i zbrinjavanje otpadnih voda. Uklanjanje ostataka hrane i mirisa je izazovno [7], stoga se prehrambena ambalaža rijetko reciklira. Alternativno je suho čišćenje (mehaničko zrakom, struganjem ili pjeskarenjem) za smanjenje troškova i potrošnje vode [4].

2.3. Svojstva recikliranog EPS-a

Hornberger i ostali su 1997. [16], [17] istraživali statička i dinamička svojstva recikliranog EPS-a iz ambalaže elektroničke opreme, analizirajući gustoću, tlačnu čvrstoću, puzanje, ublažavanje udara i oblikovanje s udjelima reciklata: 0%, 25%, 50%, 75% i 100%. Ustanovili su da mehanička svojstva recikliranih uzoraka uglavnom odgovaraju djevičanskom materijalu, s iznimkom 25% reciklata koji je pokazao 30% veću tlačnu čvrstoću i 50% manju deformaciju pri puzanju. Gustoća recikliranog EPS-a bila je veća od djevičanskog, no bez pravilnog obrasca. Mikroskopska struktura recikliranih kuglica (rastresita i šupljikava) nije odgovarala njihovim dobrim rezultatima čvrstoće i puzanja. Poboljšana svojstva 25% reciklata mogu biti anomalija zbog veće gustoće, neujednačenosti uzoraka ili razlike u obradi. Autori pretpostavljaju vezu između tlačne čvrstoće i gustoće, no to nije dosljedno dokazano. Različita priprema mješavine s 25% reciklata (miješanje prije ekspandiranja) mogla je utjecati na čvrstoću. Općenito, rezultati su ohrabrujući za recikliranje EPS-a u zaštitnoj ambalaži, s čvrstoćom recikliranih materijala unutar 95% djevičanskog EPS-a (izuzev 25% reciklata s većom čvrstoćom i boljim puzanjem). Dinamička ispitivanja pokazala su slična svojstva ublažavanja udara, no ponovljeni udarci više degradiraju materijale s većim udjelom reciklata. Materijal s 25% reciklata pokazao se najboljim kandidatom za zamjenu djevičanskog EPS-a u ambalaži, s blagim smanjenjem učinkovitosti ublažavanja koje se može kompenzirati debljim slojevima ambalaže.

U radu iz 2010. [18] Acierno, Carotrnuto i Pecce istraživali su utjecaj kvalitete i količine recikliranog EPS-a (0%, 20%, 40%) na masu, gustoću, tlačnu čvrstoću i toplinsku vodljivost. Povećanje udjela reciklata smanjuje gustoću, no male promjene procesnih parametara mogu to usporiti. Tlačna čvrstoća i toplinska vodljivost ovise samo o gustoći, neovisno o udjelu reciklata. Reciklirani EPS dobiven je mehaničkim recikliranjem djevičanskog EPS-a visoke (23 kg/m³) i niske (18 kg/m³) nasipne gustoće. Ograničenje je što reciklat nije bio prethodno korišten, pa njegova svojstva nisu degradirana, što ne odgovara stvarnom EPS otpadu. Ipak, rad daje smjernice za buduća istraživanja s post-potrošačkim otpadom. Eksperimenti s mješavinama djevičanskog EPS-a veće i recikliranog EPS-a niže gustoće pokazuju da reciklirani EPS smanjuje masu i gustoću proporcionalno razlici u nasipnoj gustoći, no parametri obrade (zazor kalupa) mogu kompenzirati te promjene. Tlačna čvrstoća i toplinska vodljivost ovise o gustoći do 40% udjela reciklata. Zaključno, do 40% recikliranog EPS-a proizvedenog u kontroliranim uvjetima može se koristiti bez značajnog gubitka svojstava, što je više od trenutne industrijske primjene.

Rad [19] istražuje mehanička i toplinska svojstva recikliranog EPS-a, ističući da se reciklirani EPS uglavnom proučavao u kompozitima. Naglašava potrebu ispitivanja

mješavina djevičanskog i post-potrošačkog EPS-a s ostacima hrane. U istraživanju, post-potrošačke kutije EPS-a iz prehrambene industrije su oprane industrijskim deterdžentom, sušene i mehanički reciklirane. Ispitivane su serije s 0%, 10%, 15% i 20% recikliranog sadržaja pomoću infracrvene spektroskopije, tlačnog ispitivanja, ispitivanja savojne čvrstoće, toplinske vodljivosti i otpornosti na udarce. Infracrvena spektroskopija pokazala je da kemijska struktura recikliranog EPS-a nije značajno promijenjena. Prisutnost recikliranog materijala čini mješavine krućima i lomljivijima. Mehanička svojstva definiraju materijale kao pjenaste polimere otporne na tlak i udarce, ali slabe na savijanje. Mjerena toplinska difuzivnost bila je oko $10^{-9} m^2/s$ za sve uzorke, uključujući referentni, što ukazuje da je reciklirani EPS slab difuzor topline.

Članak [20] istražuje recikliranje ambalažnog EPS otpada miješanjem s gumom radi poboljšanja žilavosti i mehaničkih svojstava. Razmatrane su tri metode povećanja gustoće (densifikacije) termo-mehaničkom obradom ekstrudiranjem uz dodatak gume. EPS otpad potjecao je iz ambalaže električnih i elektroničkih proizvoda. Metode 1 i 2 bile su neučinkovite zbog slabog mljevenja i dugotrajne naknadne densifikacije zagrijavanjem. Metoda 3, predgrijavanje (150°C, 20 min) praćeno mljevenjem (40 min), olakšala je oslobađanje zraka i poboljšala učinkovitost mljevenja te je odabrana za obradu. Densificirani materijal ekstrudiran je na temperaturama 200, 190 i 150°C u kuglice. Zagrijavanje je smanjilo volumen za više od 90%, što je značajno za logistiku transporta. Mljevenje nakon zagrijavanja trajalo je 20 minuta, za razliku od 2 sata kod ostalih metoda. Metoda 3 je najprikladnija zbog kraćeg vremena, učinkovitog odplinjavanja i smanjenja veličine. Nakon obrade, zrak je uklonjen, a EPS pretvoren u PS. Ispitivana su vlačna svojstva, udarna otpornost i MFR. Reciklirani EPS je krhak s niskim istezanjem i udarnom otpornošću, bez značajnih razlika u mehaničkim svojstvima s obzirom na metodu obrade. Vlačna čvrstoća recikliranog EPS-a (oko 16 MPa) niža je od djevičanskog PS-a zbog degradacije polimera. Učinkovita densifikacija je ključna za recikliranje EPS-a, a predgrijavanje uz mljevenje značajno smanjuje volumen, olakšavajući transport.

3. PLOČE ZA PODNO GRIJANJE I HLAĐENJE

Podno grijanje, s obzirom na toplinsku ugodnost, energetsku učinkovitost, iskoristivost prostora i ekonomičnost, predstavlja optimalno rješenje grijanja u stambenom prostoru. Za razliku od sustava koji zahtijevaju 55-90 °C, podno grijanje je niskotemperaturni sustav (35-45 °C), što rezultira manjim gubicima energije i nižom potrošnjom. Ugradnja niskotemperaturnih sustava ključna je za smanjenje emisija CO2 u zgradarstvu te je kompatibilna s dizalicama topline i obnovljivim izvorima energije. Većina sustava podnog grijanja može i grijati i hladiti, osiguravajući ravnomjernu temperaturu tijekom cijele godine. Prednost je i diskretan izgled bez vidljivih komponenti. Nedostaci uključuju skuplje popravke i održavanje zbog težeg pristupa te neprikladnost za prostore gdje je potrebna brza promjena temperature. [21]

3.1. Sustavi podnog grijanja

Prema principu zračenja topline, postoje tri osnovne vrste podnog grijanja: zračno, električno i toplovodno (hidraulično) [22].

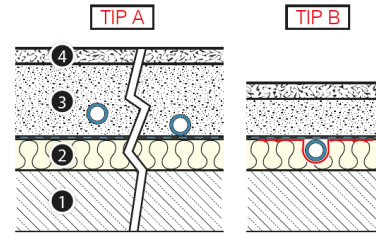
Zračno podno grijanje koristi zrak kao prijenosni medij, no zbog niskog toplinskog kapaciteta zraka nije isplativo za stambene objekte i rijetko se ugrađuje. Kombinacija sa solarnim zračnim grijanjem ima nedostatak proizvodnje topline samo danju, a noćno grijanje konvencionalnim sustavom je neučinkovito zbog prevelikih gubitaka.

Električno podno grijanje koristi grijaće kabele ili mreže ispod završnog sloja poda i preporučuje se za renovacije ili proširenja postojećih sustava zbog jednostavne ugradnje i male visine (do 5 mm).

Toplovodni (hidraulični) sustavi su najpopularniji i najisplativiji, koriste toplu vodu koja cirkulira kroz cijevi u podu. Cijena ugradnje ovisi o različitim faktorima, a preporučuju se za novogradnje zbog bolje integracije i visoke energetske učinkovitosti uz dug vijek trajanja.

Metode ugradnje električnih i toplovodnih sustava podnog grijanja su slične i dijele se na mokru i suhu ugradnju [22] Mokra ugradnja je najstariji oblik modernih sustava podnog grijanja. Podrazumijeva polaganje cijevi/kabela u betonsku ploču ili češće u cementni estrih/lagani beton iznad drvene podloge, no ima sporu toplinsku reakciju. Suha ugradnja uključuje postavljanje cijevi između OSB ploča ili ispod suhog cementnog estriha ili završnog sloja poda, omogućujući bržu reakciju sustava, ali često s većim toplinskim gubicima. Suhi estrisi od gipsanih vlakana ograničeni su na maksimalnu temperaturu od 45°C.

1. Međukatna AB konstrukcija
2. Toplinska izolacija
3. Cementni estrih
4. Završna podna obloga.



Slika 1. Podjela sustava podnog grijanja prema položaju ugradnje grijaćih cijevi [23]

Prema položaju ugradnje, sustav za podno grijanje dijeli se na dva tipa [23]: tip A – grijaće cijevi se nalaze na toplinskoj izolaciji ili iznad nje, u sloju cementnog estriha i tip B – grijaće cijevi se nalaze u gornjem dijelu sloja toplinske izolacije (*Slika 1*), pri čemu su slojevi poda:

Na tržištu je dostupno nekoliko metoda pričvršćenja cijevi podnog grijanja, a objašnjena su u nastavku prema proizvođaču KAN-therm [23].

Tacker metoda koristi plastične kopče zabijene u izolaciju Tacker alatom, pogodna za mokru ugradnju na različitim izolacijama (uključujući EPS), uz brzu i fleksibilnu ugradnju, no visina cijevi je ograničena duljinom kopči, a nepravilna uporaba može oštetiti cijevi.

Rail sustav koristi plastične ili metalne vodilice pričvršćene na izolaciju klinovima ili ljepljivom trakom, s utorima za sigurno držanje cijevi na određenim razmacima, primjenjiv za mokru i suhu ugradnju.

Ploče s utorima/čepovima (npr. KAN-therm Profil Sistem) su EPS ploče s izbočinama između kojih se utiskuju cijevi bez dodatnih pričvrsnica, uz mogućnost postavljanja aluminijske ploče za bolju distribuciju topline i recikliranje ploča na kraju vijeka.

TBS sustav je suha ugradnja s posebno profiliranim EPS pločama za cijevi, prekrivenim suhim estrihom i PE folijom, idealan za renovacije i lagane konstrukcije s brzom montažom.

NET sistem koristi žičanu mrežu za pričvršćivanje cijevi plastičnim spojnicama na različitim površinama, često se koristi u termoaktivnim stropovima i vanjskom grijanju.

Uponor Klett sustav koristi PEX cijevi s čičak kukicama koje se lijepe na izolacijske ploče s omčicama, čime se izbjegava probijanje izolacije i omogućuje brza instalacija, pogodan za mokru ugradnju. [24]

3.2. Minimalni traženi zahtjevi kod ploča za podno grijanje i hlađenje

Toplinsko-izolacijske EPS ploče ključne su u sustavima podnog grijanja i hlađenja jer smanjuju toplinske gubitke, osiguravaju mehaničku stabilnost i služe kao podloga za cijevi. Kako bi ploče mogle ispravno i dugoročno funkcionirati u sustavima podnog grijanja i hlađenja, moraju ispunjavati određene minimalne tehničke zahtjeve koji su definirani u relevantnim normama i tehničkim smjernicama.

Norma EN 13163 [25] definira specifikacije tvornički izrađenih proizvoda od EPS-a za građevinske primjene, te navodi minimalne tražene tehničke zahtjeve koje EPS proizvodi trebaju zadovoljiti (ovisno o specifičnoj namjeni). Uz zahtjeve iz norme, u obzir su uzete specifikacije EPS-a za podno grijanje i hlađenje dostupnog na tržištu, te je u skladu s istima dan preporučeni razred EPS-a za pojedino svojstvo.

Za podno grijanje i hlađenje preporučuju se EPS razredi EPS 100 ili EPS 150 (tlačna čvrstoća pri 10% deformacije), što zahtijeva minimalnu gustoću od 15–20 kg/m³ radi dovoljne tlačne čvrstoće i trajnosti. Stabilnost je bitna za sprječavanje deformacija i pukotina u estrihu, stoga se traži DS(N)5 ili bolji u normalnim uvjetima te DS(70,-)5 ili DS(70,90)5 pri povišenim temperaturama i vlazi (promjena dimenzija ≤ 0,5%). Čvrstoća okomita na ravninu ploče (TRi) mora biti ≥ 20 kPa. Ne postoji minimalni zahtjev za paropropusnost (μ-vrijednost treba biti deklarirana), no preporučuje se niska apsorpcija vode, WL(T) ≤ 3% (dugotrajno uranjanje). Toplinska vodljivost EPS-a trebala bi biti ≤ 0.038 W/m·K.

Norma [26], definira ispitivanje deformabilnosti i stlačivosti za plivajuće podove (akustične performanse).

Norma EN 1264 [27], [28] odnosi se na vodene sustave površinskog grijanja i hlađenja, a prema [27], minimalna toplinska otpornost izolacijskog sloja $R_{\lambda,ins}$ mora biti najmanje u skladu sa zahtjevom iz norme [28]. Prikazana je u *Tablica 1.*

	grijana prostorija ispod ili u blizini	negrijane ili povremeno grijane prostorije ispod, u blizini ili izravno na tlu[a]	vanjska temperatura zraka ispod ili u blizini		
			vanjska projektirana temperatura		
			$\theta_d \geq 0°C$	$0°C > \theta_d \geq -5°C$	$-5°C > \theta_d \geq -15°C$
otpor provođenja topline $R_{\lambda, ins}$ $(m^2K)/W$	0,75	1,25	1,25	1,50	2,00
[a] s razinom podzemne vode ≤ 5 m ispod potporne baze, vrijednost treba povećati.					

Tablica 1. Minimalni otpor provođenja topline izolacijskih slojeva ispod cijevi sustava grijanja/hlađenja [27]

Glede otpornosti na požar, toplinsko i zvučno-izolacijski sloj koji se najčešće primjenjuje u podnim konstrukcijama jest u razredu reakcije na požar E. Taj sloj u pravilu je obložen najmanje s 5 cm materijala u razredu A1 ili A2 (npr. cementni estrih + keramičke pločice). Uz negorivi zaštitni sloj iznad toplinsko-izolacijskog sloja i ispravno brtvljenje spojeva, ovakav zahtjev je pretjeran jer gorivi materijali ne mogu biti u kontaktu s plamenom, a i proces gorenja u prostoru bez izravnog dotoka kisika nije realan. [29]

4. ZAKLJUČAK

Ovaj pregledni stanja u području recikliranja EPS-a pružio je uvid u problematiku recikliranja ekspandiranog polistirena, s posebnim naglaskom na još uvijek dominantnu praksu odlaganja otpada u tlo unatoč globalnim nastojanjima za održivošću. Analizom postojećih znanstvenih radova o recikliranju EPS-a, utvrđen je, prije svega, značajan nedostatak istraživanja u ovom području te nekonzistentnost u zaključcima vezanim uz degradaciju recikliranog materijala. Zbog malog broja relevantnih radova, nije moguće donijeti zaključke o ponašanju recikliranog EPS-a ako se isti reciklira mehaničkim postupkom koji se trenutno pokazao kao najisplativiji i ekološki najprihvatljiviji postupak recikliranja. Potrebno je posvetiti posebnu pažnju području mehaničkog recikliranja EPS-a i provoditi daljnja istraživanja.

Rad je obuhvatio pregled izvora recikliranog sadržaja, načina njihove obrade i čišćenja, te različitih metoda recikliranja. Također je dan pregled postojećih sustava podnog grijanja i hlađenja te minimalnih zahtjeva koji se postavljaju pred EPS kao materijal u pločama za podno grijanje i hlađenje. Znanstvenih radova koji se bave takvom primjenom recikliranog EPS-a nema.

Cilj ovog rada bio je ukazati na značajan nedostatak spoznaja i detaljnih istraživanja u području recikliranja EPS-a, te ukazati na potencijal njegove primjene u sustavima podnog grijanja i hlađenja. Stoga je ključno potaknuti daljnja, opsežnija i metodološki rigoroznija istraživanja kako bi se premostio postojeći znanstveni jaz, razjasnile nejasnoće vezane uz degradaciju recikliranog EPS-a te utvrdio njegov puni potencijal i dugoročna održivost u spomenutoj primjeni.

Zahvale

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55 YEARS MERRISON RULES FOR THE EXPLAINING OF THE COLLAPSES OF THE STEEL BRIDGES WITH BOX SECTION

***Summary**:Machine and particularly car makers can run a pilot series to remedy defects before going into production, but the one-off nature and size of building structures make it impossible to test their loadbearing capacity and remove defects before construction. The aim of this paper is to help prevent the repetition of disastrous mistakes in the design and construction of load-bearing capacity of steel bridges with box sections, and contribute to engineering practice in this field. The cases describes there, show that the main causes of catastrophe are bad workmanship and a failure to recognize and think through accident scenarios. This theme put the questions whether our engineering students are getting the education they need for their future work. In the days before engineering science made it possible to predict the behaviour of load-bearing structures, failures were the main source of education and progress for builders. In the same way today, load-bearing structures must prove themselves in practice. The paper shows the significant influence of the Merrison Rules for the explanation of the collapses of the steel bridges with box section.*
***Keywords**: steel bridges, box sections, load-bearing capacity, disastrous mistakes.*

55 GODINA MERISONOVIH PRAVILA ZA OBJAŠNJAVANJE KOLAPSA ČELIČNIH MOSTOVA SA KUTIJASTIM PRESEKOM

***Rezime:** Proizvođači mašina, a posebno automobila, mogu da sprovedu pilot serije kako bi otklonili greške pre početka serijske proizvodnje. Nasuprot tome, jednokratna priroda i veličina građevinskih konstrukcija onemogućavaju testiranje njihove nosivosti i otklanjanje grešaka pre izgradnje. Cilj ovog rada je da pomogne u sprečavanju ponavljanja katastrofalnih grešaka u projektovanju i izgradnji čeličnih mostova sa kutijastim presekom, i da doprinese inženjerskoj praksi u ovoj oblasti. Prikazani slučajevi pokazuju da su glavni uzroci katastrofa loše izvođenje radova i neuspeh u prepoznavanju i razmatranju mogućih scenarija nesreća. Ova tema postavlja pitanje da li naši studenti inženjerstva dobijaju obrazovanje koje im je potrebno za njihov budući rad. U vreme kada inženjerska nauka još nije omogućila predviđanje ponašanja nosivih konstrukcija, upravo su kolapsi bili glavni izvor učenja i napretka za graditelje. Na sličan način, i danas nosive konstrukcije moraju dokazati svoju vrednost u praksi. Rad pokazuje značajan uticaj Merrisonovih pravila na objašnjavanje kolapsa čeličnih mostova sa kutijastim presekom.*
***Ključne reči:** čelični mostovi, kutijasti presek, nosivost, katastrofalne greške.*

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1. INTRODUCTION

When analyzing the causes of structural failures many investigators find, that there is hardly any case which could have been prevented by more detailed calculation, according to [1]. Other authors (J. Scheer [1], D.W. Smith [2], and O. M. Hahn [3], Fr. Faltus, M. Škaloud[19] and J. Pechar[20]) have looked into this problem and came to the same conclusion. The basic cause of most of catastrophic failures was one of the following: -possibilities of failure were never even considered during design; - conditions were not thoroughly investigated or recklessness during design or construction (rashness or even foolishness). On some occasions, successful structures have also been the cause of failure in later structures when seemingly unimportant changes, such as in size or slenderness, turned secondary factors into major influences [4]. According to [1], a collection of structural accidents might easily lead to the conclusion that construction work very frequently involves failure. There is no doubt that accidents happen too often and it cannot be denied that certain catastrophes could have been avoided. Nevertheless, the number of failure occurrences must be measured against the huge number of structures. When discussing the individual cases of bridge failure, the authors have mostly deliberately refrained from repeating well-documented investigations. We merely make reference to them, except in cases where our own deliberations have caused us to reach different conclusions[1]. As engineers continuously produce technical innovations with increasingly challenging load-bearing structures such as bridges with wider spans, of lighter design and new materials, it can happen that due to the limitations of their standard of knowledge, they fail to identify hitherto unknown phenomena and dangers. Many bridge collapses have been prevented only by the fact that the circumstances were fortunate and we can often learn as much from these cases as from the major disasters.

2. CASE STUDY

Progress in welding technology enabled the steelwork industry to make economic use of the torsional stiffness of the box girder combined with its aesthetic and maintenance advantages. Eccentric loads were largely absorbed by the two main girder walls and it was possible to build cable-stayed bridges suspended only in the middle. There were further advantages in trapezoidal box girders because their narrow lower width was good for the piers while their wide upper width allowed a generously proportioned road deck. The bearing of flange compressive forces in the wide, mostly longitudinally stiffened bottom flanges of the box girders was something new. The buckling theory had developed beyond the current codes of practice at the time but it was too little known, or even recognized, that the post-critical reserves, which are generally available in webs, only apply to flanges under certain circumstances and above all that imperfections there are of far greater significance. The result was that compressed and stiffened flanges were designed and constructed, in most cases unknowingly, with smaller failure probabilities than are usual in bridge building.

2.1. The 4th Danube Bridge in Vienna

The series of incidents with large steel box girder bridges began in 1969 with the 4th Vienna Danube Bridge. *The failure was initiated by temperature actions that had not been taken into consideration and an additional factor was oversimplified assumptions*

regarding the distribution of dead weight, which led to underestimation of the stresses at the points of failure(Fig.1a,b,c). However, the bridge, with individual spans of: 120m - 210m - 82m did not collapse because, as a continuous beam over 3 spans, it retained its load-bearing capacity as an almost statically determinate system even without significant bending moments in the two buckled areas of the compressed bottom flanges.

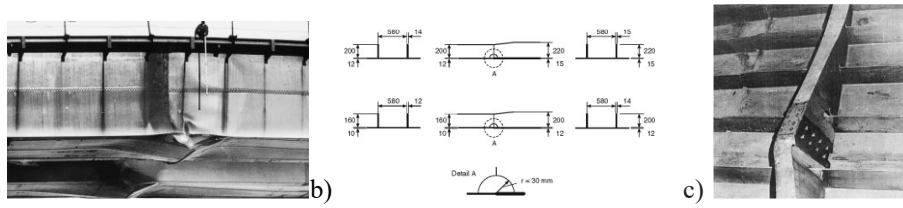


Figure 1,a,b,c. 4th Danube Bridge, Vienna, 1969 [1], a) Failure of stiffened, compressed flanges of box girders, b) Weld seam of longitudinal stiffeners-configuration in buckled areas, c) Similar seam in a less severely buckled area

The approx. 7.5m wide compressed bottom flanges of the two adjacent box girders were strengthened here with flat steel longitudinal stiffeners 580mm apart (Fig. 1c). In the area of one of the two bulges their cross section changed from 160 x 12 for the 10mm thick floor plate on the one side of the joint to 200 x 14 for the 12mm floor plate on the other. At the other failure point the 12mm floor plate was strengthened with 200 x 14 stiffeners and the adjacent 15mm plate with 200 x 15. The joints are shown in Fig. 1b, and one of them can be seen on the left hand side of the photograph 1c. In the area of the 10mm floor plate, the single spans had a load bearing stress of approx. 73% yield stress with w/d = 580 and obviously buckled before the failure of the entire span. Fig. 2a shows the buckling of the single span in close proximity to the main damage.

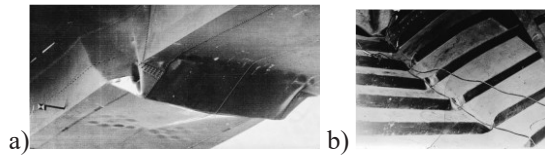


Figure 2a,b. 2th Danube Bridge, Vienna, 1969 [1] a) Buckling areas near failure point, b) Sharp fold in seam

The joint led to a displacement of the axis of gravity by approx. 1 cm. It is surprising that, to our knowledge, none of the investigations remarks on the cut-outs to be seen on the left-hand side of Fig. 1c. The value r = 30mm is derived from this photograph and is probably accurate. The cut-out weakened the smaller 160 x 12 profile and, calculated according to the German code DIN 18 800/2, Element 301, gave the floor plate an effective area of approx. 413 x 10, approx. 95%. This localized weak spot due to the cut-out and the displacement of the axis of gravity corresponds to the failure profile (Fig. 2b) with the sharp fold at the joint location. In addition to this, the relatively slender flat steel stiffeners with w/d up to approx. 13 had already been deformed laterally by welding of the joint (4 to 6mm according to [5]), which further limited their stiffening effect (see detailed investigation in [5]).

2.2. Case Study: Cleddau Bridge in Milford Haven, Wales

Nine months after the failure in Austria, the Cleddau Bridge in Milford Haven, Wales, collapsed during erection. The 819m bridge with individual spans of 77m - 77m - 77m - 149m - 213m - 149m - 77m, was built with trapezoid box girders 6.1m high, 12.5m wide at the top and 6.7m at the bottom. The road deck projected at both sides and was 20.3m wide. During cantilever erection, the ninth element was to be attached to a section already projecting 59m to reach the second pier. While this 17m long section, weighing approx. 100 t, was being extended, the transverse girder of the bearing over the first pier failed. The cantilever arm broke off and fell to the ground (according to [5]). The cause of the collapse was false assessment of the loading capacity of the large bearing plate and with this the failure to appreciate the importance of the eccentricity of its stiffeners, together with the unusual force flux resulting from the inclination of the webs. The fact that the structure was severely under-dimensioned becomes apparent in that the highest support force in later service life of the bridge was to be 55% higher than at the time of failure. Although the failure mode might suggest failure of the box girder (Fig. 3), author in[1] emphasize that the damage in fact had nothing to do this. In [1, 19] authors finds this important because this accident is frequently grouped with those of other box girders, whose failure was due to the failure of stiffened plate flanges.



Figure 3. Cleddau Bridge in Milford Haven, state after failure. 1970 [1]

2.3. Case West Gate Bridge, Melbourne

Only about four months later, the West Gate Bridge in Melbourne, Australia, collapsed during its erection (Fig. 4a).

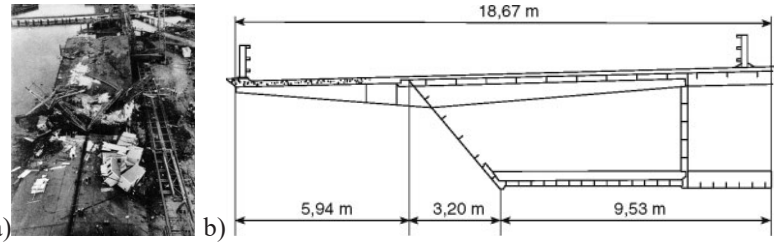


Figure 4,a,b. West Gate Bridge, Melbourne, collapsed span, half cross section [1]

The 848m bridge with span lengths of 112m - 144m - 336m - 144m - 112m had a three-cell, trapezoid cross section (Fig. 4b). The bridge beam was to be cable-stayed in the three inner spans. The unusual mode of erection, the inappropriate measures taken to remedy difficulties caused by faulty procedure and finally the collapse itself are described in [6, 7]. Figure 5a shows that the superstructure was first erected in two sections and Figure5b shows the large and varied deformations to the longitudinal joint, which had to be removed to enable the parts to be connected. The superstructure collapsed when this attempt was made.

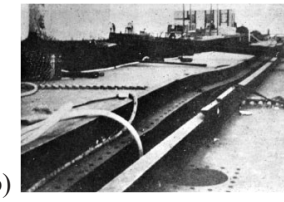
a)  b) 

Figure 5a,b. West Gate Bridge, Melbourne, longitudinal joint. 1970 [1], a) View from below before joining; b) State during joining

The accident had nothing at all to do with the buckling problem of box girder flanges. One of the reasons for the failure was overload of the box girder during attempts to align the camber by ballasting the box girder, which was being erected in halves and thus had a considerably reduced load-bearing capacity. The publications about the collapse [8, 19, 20] and about the reconstruction [9,19] contain further sources and information about the accident and give details of the defects in the original design of the bridge.

2.4. Case study: The Koblenz-Horchheim bridge over the Rhine in Germany

The Rhine Bridge in Koblenz, Germany **(Fig.6a,b.c)** is another typical example for the failure of stiffened compression flanges too. The Koblenz-Horchheim bridge over the Rhine in Germany collapsed during erection in 1971, exactly one year after the accident in Melbourne, and once again the bridge that failed had a large trapezoidal box girder. The structure was designed as a three-span bridge with a total length of 442 m. The 103m side spans were to be constructed first followed by the middle span which was to be assembled cantilevering from both sides without temporary support. Using an approx. 100 t derrick positioned at the tip of the cantilever on the left bank side, 104m of the middle span had been constructed when the bridge girder collapsed approx. 50m from the pier in the left side of the Rhine.

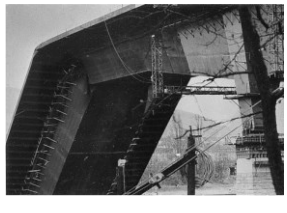
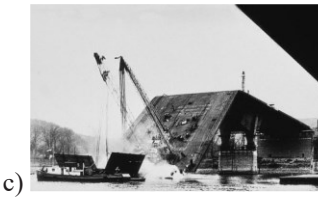
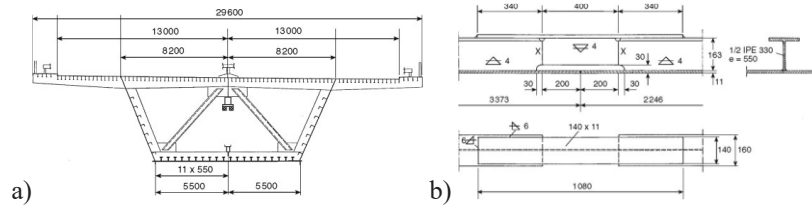
a)  b)  c) 

Figure 6,a,b,c. Rhine Bridge, Koblenz. 1971 [1] a) Bridge after collapse, b) Bridge before collapse; b) Bridge during collapse

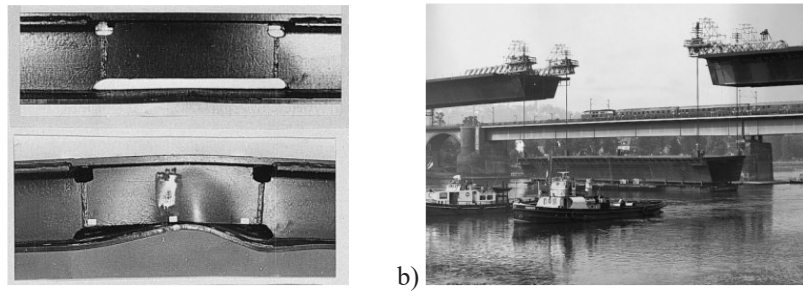
This occurred during lifting of the first part of the last section on the left side of the Rhine and resulted from exhaustion of the bearing capacity of the longitudinally stiffened plate bottom flange (Fig. 7a,b).



a) b)

Figure 7a,b. Rhine Bridge, Koblenz, construction in failure area [1] a) Cross section; b) Erection joint in bottom plate

The failure was mainly due to the detail of the bottom flange joint at the point of failure [10]. Fig. 7a shows the cross section of the bridge and Fig. 7b shows the joint in the floor plate with the longitudinal stiffeners. As the initial plan was to weld the cross joint in the floor plate automatically, a 400mm gap had been left between the ends of the longitudinal stiffeners 1/2 IPE 330. This gap was closed in the area of the stiffening webs by a fitting piece not connected to the bottom plate and by a flange splice rib welded to it. There remained a 30 + 400 + 30 = 460mm *long window without support in the bottom plate, which was 11mm thick in the failed joint.* Welding deformations contributed significantly to the weakness of the joint cross section with a slenderness ratio between 460/(0.289) = 145 for the plate member with hinged supports and 145/2 = 72 for the plate member with fixed ends. Although the plan to use an automatic welding machine was later abandoned, the joint configuration designed to suit it was not changed. Tests carried out as part of the expert report into the collapse [1] show how the joint cross section failed (Fig. 8a,b). When construction of the bridge recommenced, the extreme cantilever portion of more than 100m was avoided by lifting the 600 t middle section of the main span from the now much shorter cantilever arms (Fig. 8b). The course of the accident is described in [8,19] and also in [11, 12, 6,19].



a) b)

Figure 8a,b. Rhine Bridge, Koblenz [1], a) Failure in welding seam (experiment), b) Rebuilding with shorter cantilever

2.5. Case Study: Zeulenroda road bridge over Weida valley in Thuringia

The accident occurred during a cantilever erection in the second span, after two sections had already been assembled to form a 31.5m cantilever (Fig. 9b) and the third, approx. 14m long section, which was to be placed on temporary supports, was being prepared for positioning. The engineering world allegedly only heard many years later about the collapse of the Zeulenroda road bridge over Weida valley in Thuringia, East Germany during its erection in 1973, according to [1], H. P. Ekardt, provided a detailed report on this incident combined with generally valid observations about the responsibility of engineers [13] (1998), according to H. Elze who had dealt with this failure in a lecture in 1996 [14]. After completion of the bridge, the valley was to be flooded to serve as accumulation. The 6-span, 362m long bridge with 55m side spans and 63m middle spans had a trapezoidal box girder 2.15m high with an upper width of 5.4m and 4.0m lower width. The bridge deck with sidewalks was 11m wide and projected 3m at each side. The bottom flange was stiffened with 5 125 x 20 flat steel longitudinal stiffeners and with 150 x 10 flat steel stiffeners between the cross frames (Fig.9a). The box girder failed due to exhaustion of the compressive strength of the bottom flange in the second span directly in front of the pier (Fig. 9c).

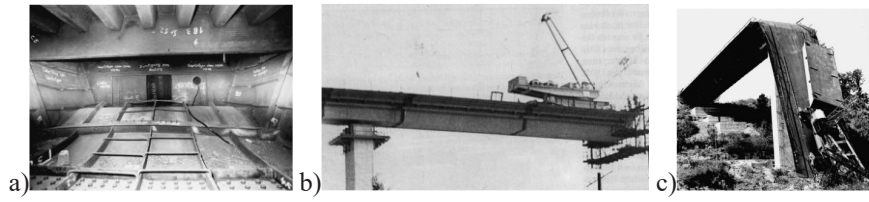


Figure 9a,b,c. Bridge, Zeulenroda reservoir,1973 [1], a) View of stiffened bottom plate, b) Before the failure, c) Broken cantilever

It must, however, be pointed out right from the beginning that in spite of the errors described in the assessment of the load bearing capacity of the stiffened flanges, defects of constructional details were always the primary cause of failure. K. Roik [9] therefore correctly identified the cause of the occurrences in Vienna, Milford Haven, Melbourne und Koblenz, with the words **"In all cases the main cause lay in defects in constructional detailing".** The first four of the five incidents have been reported, sometimes as summaries, in publications: [7, 19] and [5, 20]. For this reason they are only briefly described here while placing emphasis on general aspects of the cases.

3. CONSIDERATION ABOUT MENTIONED FAILURE OF THE BRIDGE CONSTRUCTION

According to [1] the authors of the Zeulenroda Bridge have been unable to obtain for aforementioned failure of the Bridges any details of the forces and moments at the failed cross section, any measurements, such as the panel thickness or frame spacing, or the configuration of the high-tensile bolt joints. All he can do is to summarize the details contained in [13] and [14] as follows: the bridge had been designed in 1968, i.e. prior to the failure of the 2nd Danube bridge in Vienna. In accordance with building regulations, a global safety factor of only 1.15 had been determined. For this purpose the load-bearing

capacity of the stiffened flange under compression was obviously overestimated by equating it with the ideal buckling load. The pressure on the project schedule also seems significant. Although the Koblenz failure two years earlier should have served as a warning to check all assumptions used for the structural analysis, time was running out and this was omitted. The total separation of erection planning from structural design also played a role. Witnesses have reported – off the record – that discussions in professional circles often mention the suspicion that the failure might have been due to defects in the steel supplied from the USSR according to the GOST standards. Another suggestion was that unusually strong sunshine on the day of the accident might also have caused an extreme difference in temperature between the top and bottom flange. As in Koblenz, the load on the end of the cantilever arm was avoided during reconstruction by use of a mobile crane standing on the floor of the valley. The incidents in Vienna, Milford Haven, Melbourne, Koblenz and Zeulenroda have often been quoted as grounds for experimental and theoretical research into the buckling problem. The failures occurred in longitudinally stiffened steel box girders, but this does not mean that the accidents are to be assigned fully to the breakdown of the buckling theory, in spite of the errors that occur in practice. The accidents can be assigned to the following causes[1]: The 4th Danube Bridge in Vienna: temperature effects, simplified assumptions on dead weight distribution, construction: flat stiffeners, joint eccentricities, weakening at joint, welding deformations; Cleddau Bridge in Milford Haven, Wales: Failure of bearing cross girder due to under-dimensioning; Case West Gate Bridge, Melbourne: Uncontrolled erection procedure; The Koblenz-Horchheim bridge over the Rhine in Germany: Design detailing: weak joint configuration; Zeulenroda road bridge over Weida valley in Thuringia: Insufficient safety margin during erection and false assessment of load bearing capacity of the stiffened bottom flange of the box girder.The statement “nevertheless… in the end even the slightest sign of low buckling stability, which would again be dealt with during erection, contributed to the disaster” is certainly correct according to [1] U. Krüger´s assessment [15]. In spite of information being available in engineering literature, there was little knowledge of when post-critical reserve strength was available to allow these formally slight buckling stabilities relating to the ideal buckling load to be justified or, on the other hand, when ideal buckling loads greater than failure loads made it necessary to base the assessment on other factors and include appropriate safety margins. Failure of structures almost always results in injury and loss of life. Structural safety is therefore of primary importance right from the beginning of conceptual design and should not be influenced or affected by other objectives such as cost, aesthetic shaping etc. Structural failures – and many partial failures – are often caused by one careless mistake arising because a vital problem has not been recognized, has been underestimated or carelessly treated. A bridge is never designed and built by only one engineer. A competent team is a fundamental requirement for the successful completion of a structure, according to [1]. At the same time every person involved must be prepared to support his colleagues even beyond his own particular field of action. Standardized safety is the basis of design. It must cope with shortcomings in design and construction, the development of the live load, the material strength and finally, the loss of resistance due to potential corrosion. Standards are established for “normal“ cases and sometimes they are not suitable for a particular project. Clearly, the safety requirements must be different for a simple, well-protected structure from those for a sensitive, exposed structure. For example, essential safety may be carefully modified for the assessment of an existing construction provided that any new parameters are considered and the condition of the construction is

thoroughly inspected. Any structure in which local failure could lead to a total collapse must be subjected to a higher degree of design safety. The examples of failures in this article, due to lost of load-bearing, does not aim to lecture the people involved in their design. For this reason, the incidents here are described without names of the concerned parties, in order to be fair to the affected colleagues, while at the same time learning from their mistakes.

4. COLLAPSES FAILURES AND MERRISON RULES

According [18] the UK Merrison Committee [17,19,20] was set up after the fatal 1970 collapses at Milford Haven Bridge, (Fig. 2a,b,c) when 4 people died, and West Gate (Yarra) Bridge when 34 people died. Key members were Prof Horne and Dr Flint who were charged with examining the reasons for failure, setting up major programmes of theoretical and experimental research on box girder behaviour and rapidly producing the detailed interim design and workmanship rules (IDWR) [18]. Prof Horne and Dr Flint, aided by the transport authorities, BSI Bridges Committee chaired by Dr. Kerensky [18], the steel research, consultancy and contractor community, produced radical new design and assessment rules IDWR in 2 years. This was remarkable and contrasts with the slow fragmented process for the Eurocodes. On Milford Haven, Dr Merrison drawing on, the now largely inaccessible, reports by Shirley-Smith, Pugsley, Jones of RAE and BAC states that *"The primary case of the collapse of the Milford Haven Bridge was a failure of the diaphragm over Pier 6, and that this failure was certainly not primarily due to manufacture of the bridge or its components. The diaphragm as designed simply was not strong enough to resist the severe compressive forces it was subject to during erection."* There was a full public investigation [19,20] of the failure of the West Gate Bridge. Collapse was attributed to two inadequacies: -the designers failed to make the slender stiffened plates and the details of joints which were strong enough; -the contractor adopted an unsuitable method of erection which triggered the collapse. Reading the full report should be mandatory for all involved in the design or construction of mega-structures, so that they understand the complexity of interacting technical, contractual and human factors which combined to cause the fatal collapse. Following Milford Haven collapse *full height stiffeners had been added to the diaphragms over the bearings at West Gate, but the failure was elsewhere near mid span at a transverse joint between the half boxes as they were being forced together.* The shortcomings of the original design were highlighted by the extent of strengthening required for both the erection process and for service requirements. One unusual feature of West Gate Bridge was the bolted assembly introducing eccentricities in connections of prefabricated stiffened panels. This choice was to avoid site welding after a failure to communicate the requirements for fabrication of the 1962 King Street Bridge [18] led to the brittle fracture of its girders in service. Others had problems with box girders. Research on box girders became a priority worldwide and international cooperation facilitated the progress.

5. INFLUENCE OF THE MERRISON COMMITTEE AND INTERNATIONAL RESEARCH ON THE DEVELOPMENT OF STEEL BOX GIRDERS IN THE WORLD

According to [18] the results of this UK work and similar another from around the world[19-24] is reported in many conference proceedings. The Proceedings of the 1976

Conference on “Steel Plated Structures“at Imperial Collage provided updated and test and analytical results. Further developments were covered at the 1977 “ECCS/IABSE Liege” conference on the stability of steel structures. Other relevant research includes. Proceedings of the 1980 Cardiff conference ‘New Code for the Design of Steel Bridges’ set out the UK state of the art before the issue of BS5400-3 1982 “Steel Bridges”. The simplifications in the BS5400-3 meant that the opportunities for more detailed analysis and advanced design based on the research programmes and the rapid evolution of finite element analysis were inhibited. In the UK box girders were shunned for 30 years after the collapses, with Dartford (1991) and Second Severn main span (1996) being plate girders. In Germany in 1978 the DASt-Ri 012 „Plate buckling“ was published. Around the world box girders flourished including Germany (e.g. K.-Adenauer-Bridge 1972), Czech Republic (1974)[19-24] and Bulgaria (1976 and 1986). The “Asparuhov Bridge” in Varna (Fig. 10a,b,c), Orthotropic bridge in Sofia((Fig.11a,b,c,d), Viaduct “Hemus 2”(Fig.12) and the “Tyne Bridges” (Fig.13), shows what can be achieved by exploiting the strengths **and torsional rigidity of box girders.**

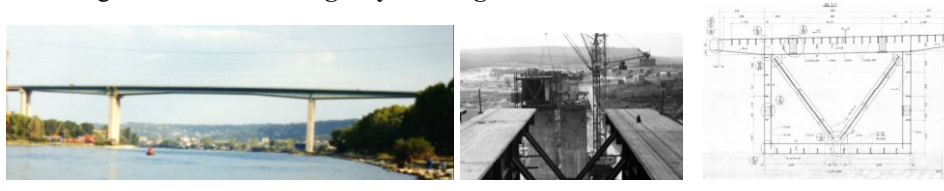


*Figure 10 a, b, c. a) View of the Asp. bridge,1976 b) Erection of the blocks c) Cross sectio*n

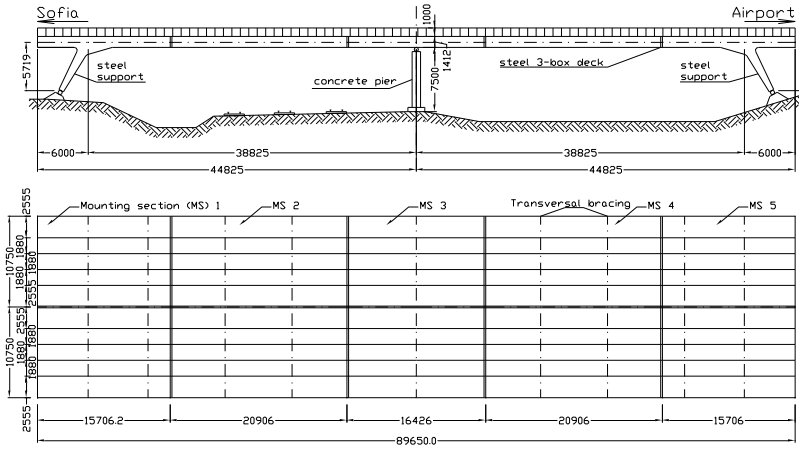


Figure 11a. Longitudinal cross-section and top view of the separate sections

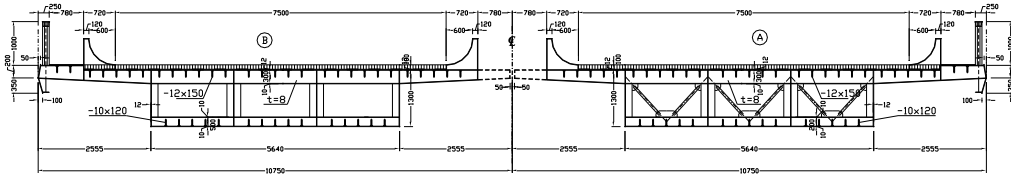


Figure 11b. Cross-section of bridge

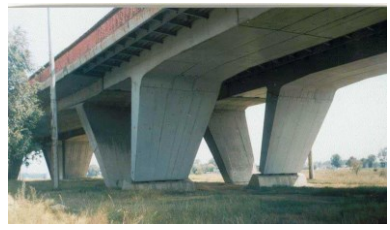


Figure 11c. A view of the orth. bridge



Figure 11d. A view on the orth. bridge



Figure 12. Viaduct 2 "Hemus" highway

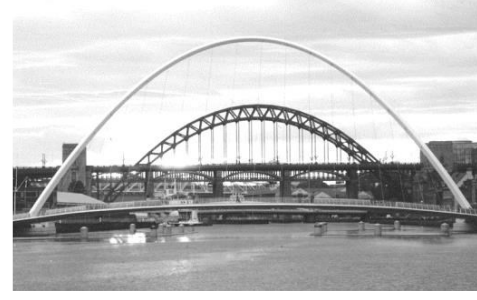


Figure 13. Tyne bridges 1850 to 2000

6. CONCLUSION

According to [1,19,20,21,22,23 and 24] not one bridge collapse, as reported in this article, was due to inaccurate structural safety analysis. It is also doubtful whether the safety theory based on a probability approach, which is now the basis for all new standards internationally, is likely to reduce the incidence of failure and collapse of structures. This is because the causes are not statistically distributed, but are rather gross errors that do not fit into any probability calculation. Such concepts are perhaps better suited for appraising the serviceability of our structures. They are often forced to extrapolate and to accept the risk this entails [16]. Here the scientific progress in structural engineering has not brought about any radical changes, and hence we agree with H. P. Ekardt [13],"Construction is in a state of continuous development, progressing through emergency situations and constantly breaking new ground in actual projects, creating something new, and in this respect is removed from state or legal control – the area to be controlled is in itself insufficiently objectified and defined for the requirements of law. What is needed is professional self-control based on knowledge, experience, balanced judgment and responsibility. Control and self-control are the two poles between which the practice of designing innovative load-bearing structures moves, particularly when the area of technology involved is in a state of rapid development.." The foreword began with a quotation from George Frost [17] and in this sense these descriptions are intended to contribute to the science of engineering. According to J. Scheer´s [1] experience: "building work is often linked with failure; this has always been the case and always will be". Yet if we learn from the circumstances and mistakes leading up to disasters in the past, we may possibly help to reduce the number of failure occurrences, collapses and catastrophes in the future [1].

Acknowledgement

This article is dedicated to Dr. Eng. Dr. h. c. Radomir Folić on the occasion of his 85-th birthday.

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XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
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Radomir Folić[1], Miloš Čokić[2],
Nadja Kurtović Folić[3]

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SEISMIC VULNERABILITY AND NONLINEAR RESPONSE OF UNREINFORCED MASONRY BUILDINGS

Summary: *Unreinforced masonry (URM) buiding behaive nonlinear and are vulnerable under seismic actions. URM buildings constitute one of the oldest structural systems built without considering modern seismic design criteria. Masonry is an anisotropic and heterogeneous material and difficalt for modeling. In this paper review of literature and analysis, single building, nonlinear behaviour using both nonlinear static (NSA) and incremental dynamic analisys (INDA). Relavant value of deformation obtain with both aforemention methods of analsys are compared. Seismic fragility analysis of buildings structures also analysed. In the final notes, appropriate conclusions and recommendations are formulated with reference to Eurocode 6 of the 2nd generation, which refers to masonry structures.*
Keywords: *Unreinforced masonry (URM) buildings, modelling, homogenization, assessment, seismic vulnerability, limit value, damage states*

SEIZMIČKA POVREDLJIVOST I NELINEARNI ODGOVOR NEARMIRANIH ZIDANIH ZGRADA

Rezime: *Nearmirane zidane konstrukcije (URM) se ponašaju nelinearno i povredljive su na seizmička dejstva. Zgrade URM-a predstavljaju jedan od najstarijih konstruktivnih sistema izgrađenih bez uvažavanja savremenih seizmičkih kriterijuma projektovanja. Zidarija je anizotropan i heterogen materijal i težak za modelovanje. U ovom radu je dat pregled literature i analiza nelinearnog ponašanja, jedne zgrade, korišćenjem nelinearne statičke i inkrementalne dinamičke analize. Upoređene su relevantne vrednosti deformacija dobijene obema metodama analize. Analizirana je i seizmička povredljivost ovih konstrukcija. U završnim napomenama formulisani su odgovarajući zaključci i preporuke sa osvrtom na Evrokod 6, druge. generacije, koji se odnosi na zidane konstrukcije.*
Ključne reči: *Nearmirane zidane zgrade, modeliranje, homogenizacija,procena, seizmička povredljivost,, granične vrednosti,* stanja oštećenja

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1. INTRODUCTION AND MODELLING

One of the oldest types, and most widely used, construction is unreinforced masonry (URM). This construction type still constitutes a significant percent in our and many region. Masonry is an anisotropic and heterogeneous material with a combination of masonry units and mortar. Depending on the level of accuracy and the simplicity different approaches are available for the numerical analysis with elastic or inelastic (non-liner) material behavior under seismic actions. Masonry buildings are rigid and heavy structures; hence, the load-bearing masonry walls attract higher levels of earthq-uake loads when compared to flexible frame URM have poor and brittle behavior [13].

Beside Eurocodes [6], and [7], in [14] masonry building design in seismic areas through recent experiences and prospects from a Euro Code point is discussed. In paper [24] discussed behavior factor (introduced in Eurocode 8) estimation for seismic design of unreinforced masonry buildings. Slovenian experience in damage as a measure for earthquake-resistant design of masonry structures described in [22]. In [21] seismic fragility analysis of low-rice unreinforced masonry buildings subjected to ground motion is described. A simplified micro-modeling approach considering expanded units and defining the constitutive relationships for unit and interface. The macro-modeling approach constitutive relationship is used as a homogenized material. The macro-modeling approach is often associated with the equivalent frame modeling. the building stability verification can be performed by equivalent lateral force or response spectrum analyses, in which the seismic demand is reduced by a behavior factor, if some ductility is expected for the analyzed building, in the case of plastic deformation Figure 1.

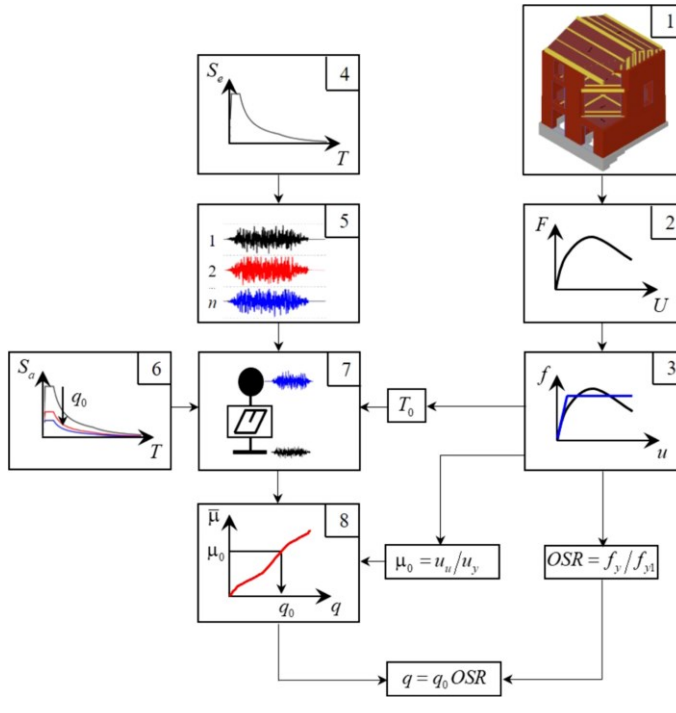


Figure 1. Flowchart of the proposed numerical procedure to determine the behaviour factor [25]

Seismic vulnerability of masonry buildings most frequently has been evaluated using empirical methods based on damage observation after earthquake. Most of them assume only in-plane failure behavior, which is non-conservative, as it overlooks out-of-plane behaviour. It is also unjustifiably in current EC 8 considered only in-plane behaviour, controlled by flexure or by shear, and limit drifts for the performance criteria. Comparison with experimental evidence shows that these limits are usually underestimated [4]. Different modeling approach considered in [23].

The vulnerability of masonry buildings was discussed in [2], [4], [9], [12], [15] and [21]. General aspect of vulnerability and methodologies of risk analysis is subject of ref. [10] and [16]. Nonlinear modelling of seismic response of masonry structures consider in [1], [5], [11]. Finite element and equivalent frame modelling approaches for analysis of unreinforced masonry building are described in [18]. A simplified approach for the seismic analysis of masonry structures is subject of paper [18].

A state-of-the- art (review) of simplified analytical methods for seismic vulnerability assessment of unreinforced masonry buildings is given in [20]. Review of performance-based design is subject of ref. [24]. Masonry behaviors and modeling is discussed in paper [3], and global structural parameters for unreinforced masonry buildings subjected to simulated ground motions in [13]. In most of the works, the behavior and analysis of the walls of UR masonry buildings is considered Out-of plane behaviour of U-shaped unreinforced masonry structures discussed in [23].

Predicting compressive strength due to its nonhomogeneous and anisotropic nature, compounded by variations in the properties of the constituent bricks and mortar is difficult. Impact of factors like the slenderness ratio and mortar-to-brick joint thickness ratio on masonry compressive strength highlights the need for more precise compressive strength predictions.

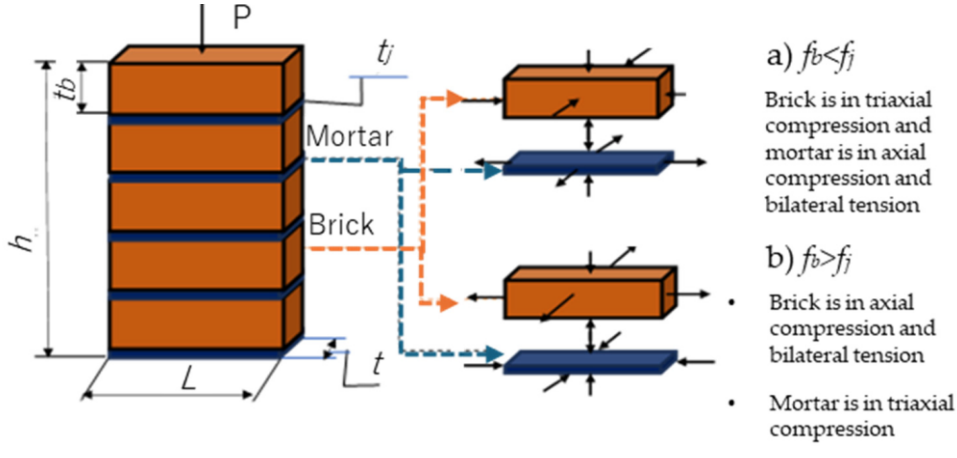


Figure 2. Stress condition of brick and mortar under uniaxial compression, after [17].

Depending on the level of accuracy and the simplicity desired, different approaches are available for the numerical analysis of masonry structures with elastic or inelastic material behavior [24]. These methods can be applied to simulate the structure at a macro or micro level and present the stage of damage using different ways and constitutive laws [25].

Methodologies for the nonlinear modeling of URM structures, which can be categorized into 1) the discrete element method, 2) the continuum homogenous method, and 3) the equivalent frame method (EFM). The continuum homogeneous method is typically utilized to model URM structures with complex architectures. Nonlinear static (pushover) analysis (NSA) is the most popular method. Incremental dynamic analysis (IDA) is the most advanced type of detailed analysis, in which accelerograms are applied to the building model, and their intensity increased until the collapse occurs [20].

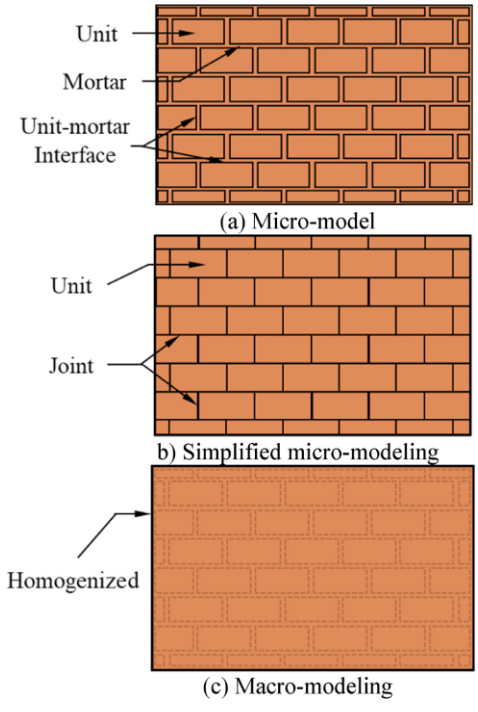


Figure 3. Diferent modeling approach [23]

The out-of-plane behavior of masonry walls is an important aspect of their response, particularly in seismic regions. While previous studies have examined various geometries and materials, there exists a dearth of research on U-shaped walls, specifically with diverse geometric configurations. Moreover, the influence of axial load on out-of-plane behavior remains largely unexplored, limiting practical applications.

Equivalent frame (EF) modeling of unreinforced masonry structures (URM) is that there is no interaction between in plane and out of plane action of the wall. In [23] out-of plane behaviour of U-shaped URM.

Diferent modelling aproach are

- macro-modelling; - simplified micro-modelling; -micro modelling (Figure 3).

Depending on the level of accuracy and the simplicity desired, the representations are show in Figure 4. Examples for homogination and illustrative structural component with beam elements or macro-blocks shows in Figure 4.

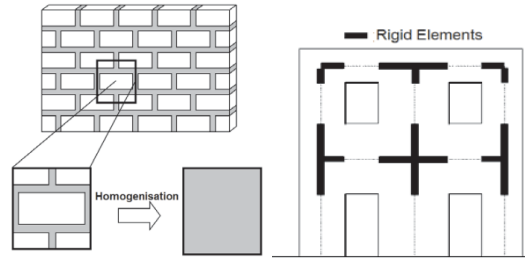


Figure 4. Homogenization of masonry and illustrative structural component [3]

The structural elements are directly provided in terms of internal forces such as shear force or bending moment (and related generalized displacements), instead of stresses and strains. One example for two story building with perforation showen in Figure 5.

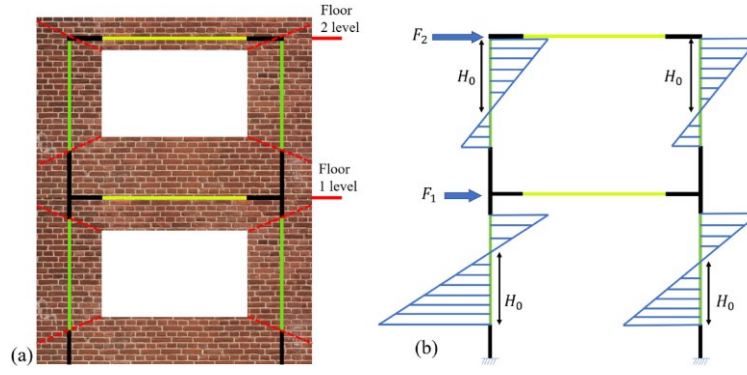


Figure 5. (a) A perforated two-story URM wall; (b) the moment diagram of the wall subjected to lateral loadings and definition of Ho, after [21]

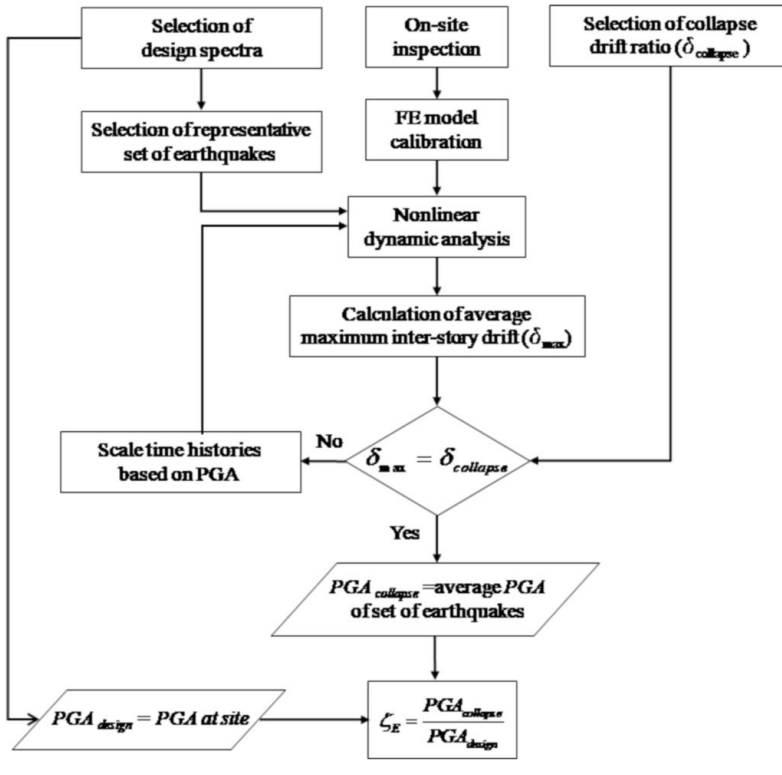


Figure 6. Flowchart to compute vulnerability index, after [15]

If the behavior factor provided for in EC8 is used to assess the load capacity, with which the overload factor (*OSR*) is switched on it, is recommended [14] and [25], where: $q = q^{*} \cdot OSR$ (unreinforced masonry buildings q = 2.0 αu/α1; single-storey unreinforced masonry buildings αu /α1 = 1,4; two- or more storey unreinforced masonry buildings αu /α1 = 1,8.

The seismic vulnerability of an existing building depends on different conditions such as aging, poor maintenance, outdated design, materials characteristics, the construction place, and natural events. The seismic **vulnerability index** for an existing building is defined by a new parameter ζ_E that is the ratio between the maximum bearable seismic action of the existing structure and that one required to design a new one on the same site with the same dynamic properties [15], or: $\zeta_E = PGA_{collapse} / PGA_{design}$; $\zeta_E = S_a / S_{a,d}$.

The Peak Ground Acceleration (*PGA*), or spectral acceleration S_a, is used to define this index. Figure 6. showes Flowchart to compute vulnerability index. It is similarly expressed through the aforementioned acceleration spectra, and the third way is related to non-linear static analysis (relationship of bearings) [15].

2. CASE STUDY ANALYSIS

2.1. Geometric and material properties of the structure

In this paper, the fragility and vulnerability assessments are compared, based on the methods used for the analysis and the response of the structure in two main directions. Nonlinear static (NSA) and nonlinear dynamic analysis (NDA) methods are used for the calculation of the building's response under seismic action. Results of NSA and the detailed calculation procedure and results are given in paper [9]. The subject of the analysis is a unreinforced masonry (URM) building (Fig. 7) with 3 levels (ground floor+2 stories). The plan view and the 3D model of the structure are shown in Figure 7.

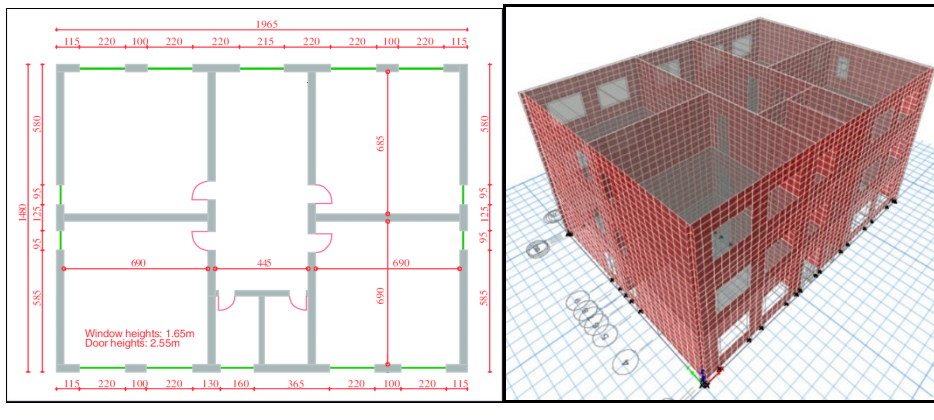


Figure 7. a) Building plan; b) Numerical model, after [9]

The length of the spans in both directions is displayed in Figure 7, left. The height of each story is 3.6 which make the total height of the building 10.8 m. In order to simplify the modelling and calculation process, soil-structure interaction is not included

in the calculation and design. Material properties of URM (f'_m = 10.34 Mpa, f_t = 2.00 MPa, E = 9307.92 MPa) for structural walls and concrete C12/15 (f_c = 12.00 Mpa, f_t = 2.16 MPa, E = 27000 MPa) have been adopted for model analysis [8]. Thickness of URM walls is 36 cm and the adopted thickness of the floor structures is equal to 12 cm and it modelled as a concrete (C12/15) plate.

2.2. Loads and actions

The loads acting on the structure are as follows: permanent loads (G_i) – self-weight of structural elements and an additional permanent load; the variable-live load (Q_i). The adopted value of the permanent constant load is g_{pl} = 3.0 kN/m^2 and the load intensity of the variable-live load amounts q = 3.0 kN/m^2 on all floors except the roof, where they amount g_R = 1.0 kN/m^2 and q_R = 1.0 kN/m^2.

2.3. Structural Model (assumptions and simplifications

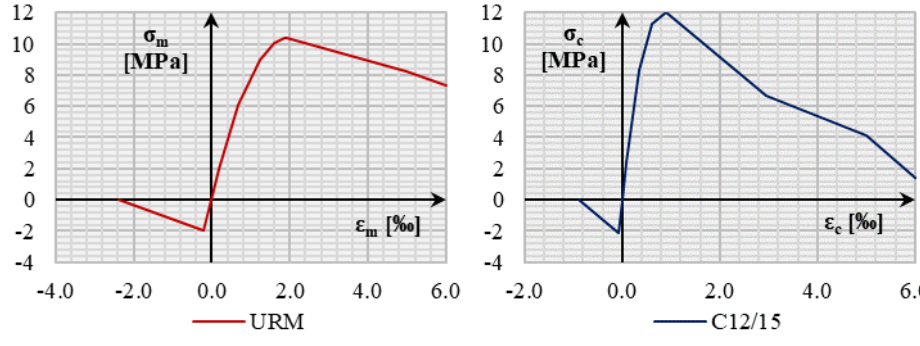


Figure 8. Material properties of URM (left) and concrete C12/15 (right), after [8]

In models for post-elastic analysis of structural response to the removal of individual vertical elements, the following assumptions and simplifications were used:

- The calculation includes the effects of second order logic (*P-Δ*);
- To describe the nonlinear behaviour of the material, the nonlinear properties of the material were used to describe the behaviour of the URM walls (Fig. 8, left) [8];
- Floor structures are modelled as rigid diaphragms;
- The UMR walls are modelled as the multi-layer shell elements, with nonlinear behaviour.

2.4. Nonlinear Dynamic Analysis

NDA is conducted by using ten particular accelerograms [26], [27]. The accelerograms (Figure 9) are scaled according to elastic response spectra for the intensity level of 10% possibility of exceedance in 50 years.

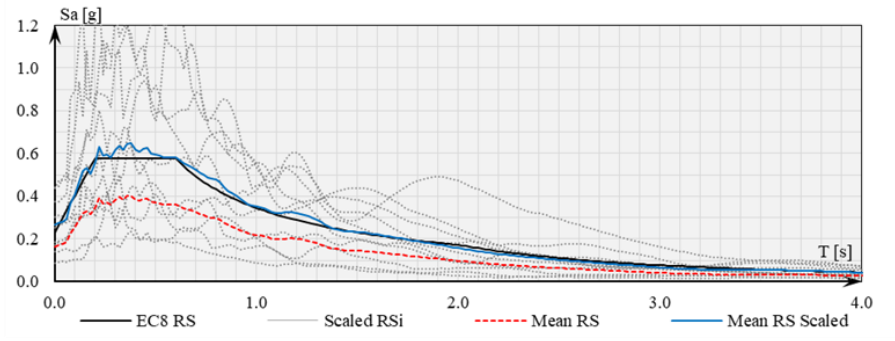


Figure 9. Response spectra and TH data, after [27]

2.5. Calculation of Fragility Curves and Vulnerability analysis

Damage of a structural system may be quantified through threshold performance points (small damage – *SD*; moderate damage – *MD*; extensive damage – *ED*; complete damage – *CD*), which are determined according to [10] for seismic fragility analysis using NDA method. The values and the results are shown in Table.

DS	***SD***	***MD***	***ED***	***CD***
***IDR* [-], [10]**	0.0016	0.0032	0.008	0.0187
$\mu_{DS,i}^{IDR}$ **[-]**	0.018	0.038	0.102	0.254
$\sigma_{LN,DS,i}$ **[-]**	0.254	0.385	0.558	0.719

Table 1. Seismic structural DS threshold values, according to [10] and calculated values

Linear regression method was used to analyze the data and calculate fragility curves, using the analytical cumulative distribution function (CDF) for the lognormal distribution:

$$P_{DS_i|IM}\left(\mu_{LN|DS_i}^{IM}, \sigma_{LN|DS_i}^{IM}\right) = \Phi\left(\frac{\ln IM - \mu_{LN|DS_i}^{IM}}{\sigma_{LN|DS_i}^{IM}}\right) \quad (1)$$

where Φ is the cumulative distribution function of the standard normal distribution, $\mu_{LN|DS_i}^{IM}$ and $\sigma_{LN|DS_i}^{IM}$ are the mean and standard deviation in log-log space of the probability density function (PDF). This procedure is explained in detail in papers [26], [27]. Based on the obtained results, fragility curves and PDF are calculated. PDF for the occurrence of different damage states were calculated according to the equations, described in paper [27].

Calculation process of vulnerability curves was performed according to the method described in [28]. Calculation of the vulnerability curves, based on the fragility results was done according to the equation and process described in the paper [27]:

$$E(C|IM) = \sum_{i=0}^{n} E(C|DS_i) \cdot P(DS_i|IM) \quad (2)$$

where n is the number of considered DS (DS_i), $P(DS_i|IM)$ is the damage probability; $E(C|DS_i)$and $E(C|IM)$ are the cumulative distribution of cost (or loss) according to [28]. The values of $E(C|DS_i)$ are adopted from [28].

Damage Scale	$E(C\|DS_i)$			
Damage State	Slight (SD)	Moderate (MD)	Extensive (ED)	Complete (CD)
	2%	10%	50%	100%

Table 2. Damage factor functions of building typology according to [28]

3. DISCUSSION OF THE RESULTS

3.1. Structural response

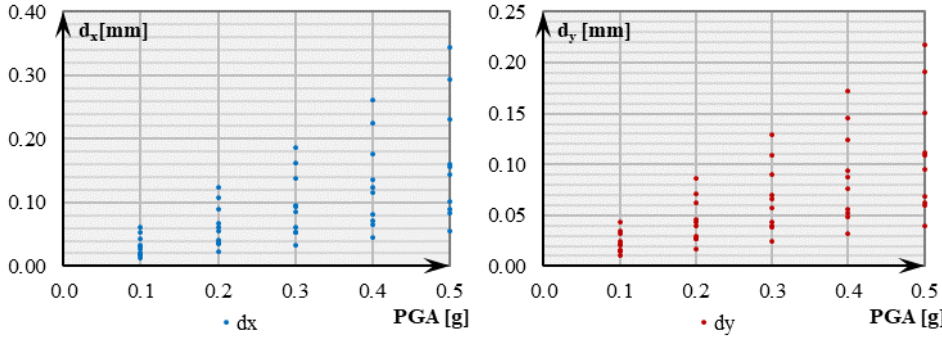


Figure 10. NDA – max displacement results in X (left) and Y direction (right)

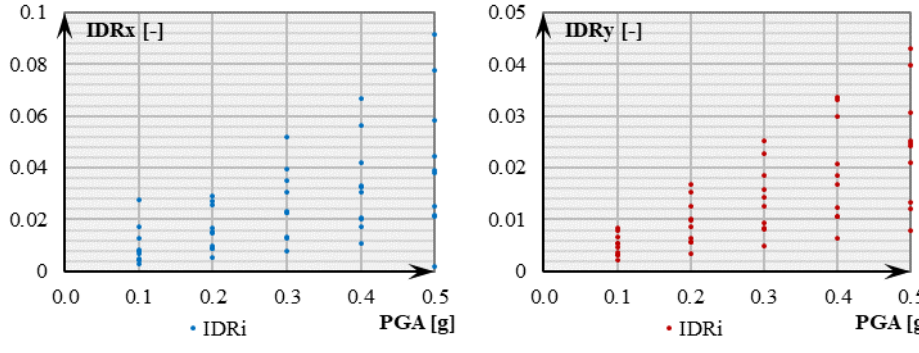


Figure 11. NDA – max IDR results in X (left) and Y direction (right)

The results obtained from the NDA, fragility and vulnerability assessments provide understanding of the seismic behavior of the case study unreinforced masonry (URM) structure. The displacement and inter-story drift (IDR) results, fragility curves, and vulnerability functions show the building's performance for increasing levels of PGA.

The maximum displacement values (d_{max}) in both *X* and *Y* directions (Figure 10) increase with the peak ground acceleration (PGA). The displacements (Figure 10) and the drift values (Figure 11) in X-direction are higher compared to the Y-direction. This indicates a greater vulnerability and lower stiffness in the X-direction, probably due to the structural configuration. The results show different seismic response of the structure in main directions, due to the geometry of the structure.

Fragility curves (Figure 12) were calculated using the lognormal cumulative distribution function for four damage states: Slight (SD), Moderate (MD), Extensive (ED), and Complete Damage (CD).

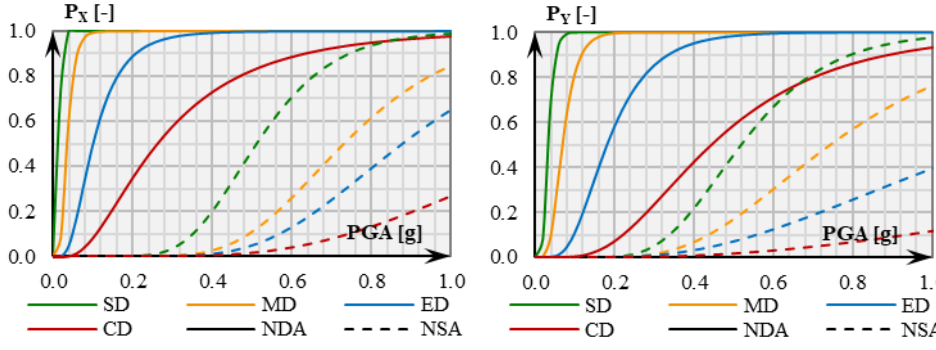


Figure 12. Fragility curves for X (left) and Y direction (right)

The Y-direction fragility functions show better seismic response, compared to the ones in the X-direction. In the X-direction, there is a 50% probability of exceeding extensive damage at a PGA of approximately 0.2*g*, where in the Y-direction, this threshold is reached at approximately 0.3*g*. This difference shows the higher chance of the structure to obtain the damage in the X-direction. The complete damage state (CD) in the X-direction reaches near certainty (97%) at a PGA of around 1.0g, while in the Y-direction the value remains slightly lower, at approximately 93%.

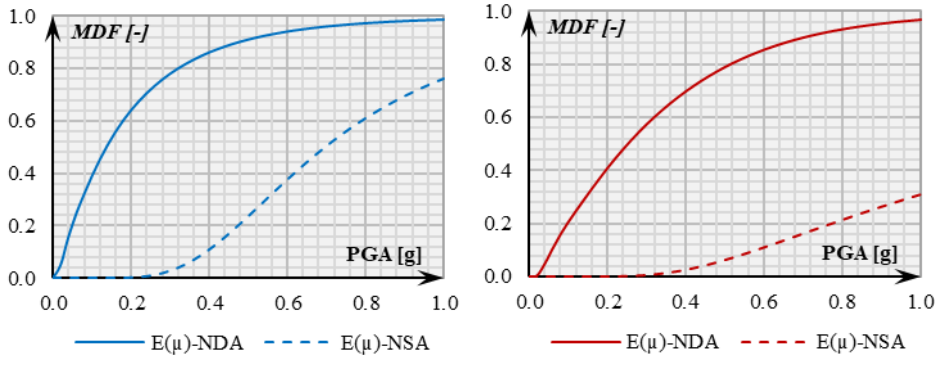


Figure 13. Vulnerabilty curves for X (left) and Y direction (right)

The vulnerability curves (Figure 13) show the expected damage ratio (loss) as a function of PGA. As well, they indicate to the similar conclusions as the fragility analysis results. In the X-direction, the expected loss increases more (surpassing 90% at PGA =

0.5*g*), compared to the Y-direction, which has expected loss reaching approximately 79% at the same intensity.

3.2. Comparison of NSA and NDA Fragility and Vulnerability Results

Fragility curves describe the probability of reaching or exceeding a specified damage state as a function of seismic intensity—in this case, PGA. The comparison between fragility curves derived from NSA and NDA shows significant differences in the assessment of structural vulnerability.

The comparison between NDA and NSA fragility curves shows significant differences in the prediction of structural damage for all damage states—Slight (SD), Moderate (MD), Extensive (ED), and Complete (CD)—and in both main directions (X and Y). In general, NDA shows more realistic results of structural behavior under seismic excitation, compared to NSA.

In the X-direction, NDA shows early exceedance of damage states, with the probability of SD reaching 100% at a PGA of 0.06g. NSA does not predict any significant damage until approximately 0.20g, indicating a clear underestimation of vulnerability. For MD, NDA reaches 50% exceedance around 0.10g–0.12g, while NSA does not reach the same probability until approximately 0.54g. This gap is more pronounced in higher damage states. For ED, NDA reaches 50% exceedance at about 0.22 g, and NSA does not reach it until nearly 0.90g. The same pattern remains for CD, with NDA reaching 50% exceedance around 0.38g, while NSA achieves that probability only at 1.00g. This big difference in the results shows how NSA, in this case, significantly underestimates the potential for high-level damage and collapse, especially in more vulnerable directions.

Similar trends can be observed in the Y-direction, where NDA consistently shows higher probabilities of exceedance at lower PGA. NDA shows 50% probability for ED at approximately 0.20g, compared to approximately 0.96g for NSA. For CD, the 50% exceedance level is reached at 0.42g in NDA and again only at 1.00g in NSA. This highlights, again, the limited capacity of NSA, in this case, in the assessment of the severe damage progression, due to its lack of modeling of dynamic interaction.

In terms of the vulnerability analysis, the Mean Damage Factor (MDF)—a normalized measure of expected damage ranging from 0 (no damage) to 1 (total collapse)—was used to evaluate and compare the cumulative effects of damage predicted by NDA in both directions. In the X-direction, MDF reaches 0.25 at 0.06g, 0.64 at 0.20g, and 0.91 at 0.50g, and approaches 0.99 at 1.00g. This shows quick damage progression and high vulnerability in this direction. In the Y-direction, MDF increases more gradually, with values of 0.10 at 0.06g, 0.41 at 0.20g, 0.79 at 0.50g, and 0.97 at 1.00g. These results suggest that the structure is somewhat more robust in the Y-direction, with delayed onset and slower accumulation of damage.

4. FINAL REMARKS AND CONCLUSIONS

Recent earthquakes revealed the high vulnerability of unreinforced masonry (URM) buildings to seismic loads. Material of URM is brittle type with low-tensile

strength and large mass with high susceptibility to earthquake events. The brittle material type with low-tensile strength and large mass with high susceptibility to earthquake events.

Among the structural parameters, the strength ratio seems to be the most dominant one regarding the seismic performance of URM models. Period and ductility seem to have relatively moderate effects on seismic performance; Damage state probability plots of URM models verify that masonry buildings have limited ductility capacity and a narrow margin of safety between the initial state of inelastic behavior and the ultimate (collapse) state. In paper [13] authors conclude that using only two limit states (onset of damage and ultimate) for URM buildings are appropriate.

2nd of Eurocode 6 published by CEN in April 2022. Major changes are: Masonry units with innovative geometric properties allowed; Verification under combined loading; Capacity reduction factor for slenderness and eccentricity; Addition of coefficient of friction for out-of-plane shear. In Part 2: Detailing of the spacing of movement joints has been revised; Tolerance specifications for masonry to be used with thin layer mortar have been developed; Pointing of masonry has been substantially reviewed.

For the case study analysis, the comparison of NDA and NSA fragility curves clearly shows the more accurate and comprehensive damage prediction capabilities of NDA in comparison to NSA. While NSA tends to underestimate damage in both directions, particularly for higher damage states such as Extensive and Complete Damage, NDA offers a more realistic assessment of structural vulnerability. The key difference lies in NDA's ability to account for dynamic effects, nonlinear behavior, and progressive damage, which are essential in accurately capturing the true seismic response of a structure.

The vulnerability analysis further shows the quick progression of damage in the X-direction, where the structure is found to be more vulnerable. The Mean Damage Factor (MDF) shows a quick rise in damage in the X-direction, particularly at lower levels of PGA, indicating that this direction is more vulnerable to seismic actions. The Y-direction shows a more gradual increase in vulnerability, which means that it has better performance under seismic actions in that direction.

The results confirm that NDA should be preferred for seismic performance assessments, as it provides more accurate damage predictions and vulnerability estimates compared to NSA.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
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XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

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Andrija Zorić[3]

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Stručni rad

RECONSTRUCTION OF THE LESKOVAC FAIR HALL

Summary: *The paper shows the reconstruction of the supporting structure of the main hall of the Leskovac Fair. The specifics of the construction with a roof structure in the form of a network of orthogonal cables were analyzed. The significance of the building is reflected in the innovative system applied to the roof and in the architectural achievement of Yugoslav architecture in the mid-20th century. In this period, only a few buildings with this design were built in the world. In terms of historical heritage, the Fair building is the embodiment of former economic development and a witness to the industrial and cultural life of Leskovac and its surroundings. The paper presents the measures aimed at rehabilitating the building. The reconstruction design envisaged the addition of two floors with commercial function along with the conversion of the building into a shopping center. The newly designed part of the structure is subtly incorporated into the existing reinforced concrete structure. The design provided for maximum preservation of the authenticity of the external appearance of the hall.*
Keywords: *cable structure, roof structure, RC arch, Leskovac fair hall*

REKONSTRUKCIJA HALE LESKOVAČKOG SAJMA

Rezime: *U radu je prikazan postupak rekonstrukcije noseće konstrukcije glavne hale "Leskovačkog sajma". Analizirane su specifičnosti gradnje i oblikovanja konstrukcije sa strukturom krova u vidu mreže ortogonalnih kablova. Značaj objekta ogleda se u inovativnom sistemu primenjenom za obradu krova i u arhitektonskom dostignuću Jugoslovenske arhitekture sredinom XX veka. U svetu je u ovom periodu bilo izgrađeno još samo par objekata sa ovakvim rešenjem. U pogledu istorijskog nasleđa, objekat Sajma je oličenje nekadašnjeg privrednog razvoja i svedok industrijskog i kulturnog života Leskovca i okoline. U radu su prezentovane mere u cilju sanacije objekta. Projektom rekonstrukcije predviđena je dogradnja dve etaže komercijalnog sadržaja uz konverziju objekta u tržni centar. Novoprojektovani deo konstrukcije je suptilno inkorporiran u postojeću armiranobetonsku konstrukciju. Projektom je bilo predviđeno maksimalno očuvanje autentičnosti spoljašnjeg izgleda hale.*
Ključne reči: *kablovska konstukcija, krovna konstrukcija, AB luk, Leskovački sajam*

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1. HISTORY OF THE FAIR COMPLEX

The main hall of the Leskovac Fair was designed in 1958, and was built in the period from 1959 to 1961 for the purpose of holding the international fair of textiles and textile machinery, as well as sports events. According to the design specifications, the hall was designed as a multi-purpose one, but this idea was never fully realized. The technical documentation was prepared by the design team of the Belgrade design bureau "RAD" headed by engineer Nikola Stakić [7,10]. The architectural part of the design was signed by arch. Milorad Cvetić, and the construction of the central roof by Eng. Edmund Balgač. Over the years, with the decline in economic activity in Leskovac, the hall fell into disrepair, and its regular and investment maintenance was lacking. So, half a century after its construction, it was in a very poor general condition (Fig. 1). In 2007, the company "Delta" bought the Leskovac Fair, which consisted of five halls with about 8,000 m^2, with the intention of demolishing it. According to the design, a residential and commercial complex was to be built, with a hotel and an underground garage. The hall itself did not fit into the design. Renovating the hall and maintaining it was considered too expensive [5].



Figure 1. The round pavilion of the Leskovac Fair 1959. and 2019.

The initiative of the professional public to preserve the round pavilion was supported by the local government in 2015. The main factor in preserving the hall from demolition was played by its previous protection. The round pavilion of the Leskovac fair is classified as an engineering-architectural structure of special cultural importance, which was declared an immovable cultural property by the corresponding Act of the Institute for the Protection of Cultural Monuments in Niš. In 2017, the company "Metla Komerc" from Leskovac bought the entire property of the Fair from the group "Delta Real Estate" with the plan to build a residential and business complex in its place, but with the preservation of hall number three.

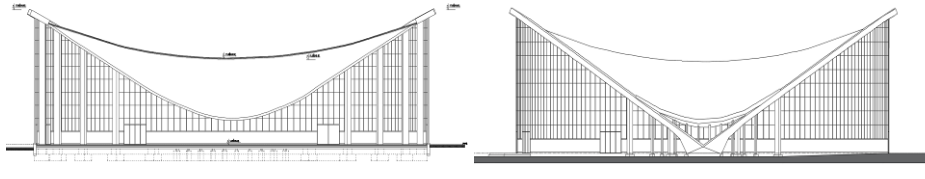


Figure 2. Cross-section and facade of the round pavilion pavilion in Leskovac [7]

2. CONSTRUCTION SYSTEM OF THE HALL

The building of the "Leskovački Sajam" hall was designed as a reinforced concrete (RC) structure with an approximately circular layout, with a mean diameter of about 65m. The roof structure is a plates and shell support supported on the peripheral main RC supports. The main supports are arched and inclined at an angle of approximately 29°. The static system of the main supports is an arch restrained on both sides, and the dimensions of the cross section are 300/70cm. At the points where the arches are restrained into the foundation supports, the height of the cross section of the arches increases sharply. The shape of the arches is parabolic but not mathematically defined. The shape of the supports is determined so that their gravity line coincides with the support line for the roof's own weight when the auxiliary cables are taut. The main beams receive all the forces of the roof that are transmitted to them via the supporting cables as well as the prestressing force from the auxiliary cables. Reactions from the supporting cables act outside the plane in which the main beams lie, and tend to lift the beams. This effect is counteracted by the weight of the main supports, the remaining weight of which is supported by twelve peripheral columns. The peripheral columns have a rectangular cross-section, dimensions 45/90cm. These columns receive the effect of the wind on the facade surfaces and transfer them to the perimeter supports and to the foundations. The columns are placed radially and in addition to receiving the load from the main arch supports, they also carry a glass facade that rests on them via a steel substructure. The pillars are founded on single foundation footings with dimensions 1.4/2.2m, and their foundation depth is 2.0m. All the peripheral foundation footings are connected by an RC ring with dimensions 40/150cm.

Inside the hall, independently of its main supporting structure, a gallery was originally designed and built at a height of 4.3m above the floor of the hall. The gallery was in the shape of a circular ring with a width of 8.6 m. Auxiliary rooms, dressing rooms and toilets were located in the basement, positioned below the gallery and approached by two spiral staircases. The new design did not provide for the existing gallery and auxiliary rooms. The structures of the basement and the gallery are independent of the other structures of the hall, so the work on their removal can be done independently of the other works.



Figure 3. Condition of the supporting cable wires (photo by P.Petronijević)

The roof structure of the hall is designed as a plate and shell support supported on the main arched inclined supports. The supporting structure of the plate and shell support is represented by the suspended roof. The catenary shaped load-bearing cables support the roof covering, and the forces from the cables are received by the peripheral arch supports. Two massive peripheral arched supports that are laid at an angle of approximately 29° carry a series of cables composed of 8 wires of ϕ5mm high-quality steel (cold-drawn and subsequently heat-treated) (Fig. 3). The cables are placed with a certain angle in the formwork of the perimeter support during its reinforcement and are concreted into them. The cables are used for supporting concrete roof plates (carrying cables) and are placed at a distance of 80 cm. A structure consisting only of these two structural elements (peripheral beams and supporting cables) would be unstable under asymmetric loading, suction wind effect and/or horizontal forces [1,2]. Such a structure would suffer significant deformations outside its plan at load changes. The reduction and equalization of deformations as well as the stiffening of the structure for horizontal actions is achieved by the introduction of auxiliary cables passed over the supporting ones. The effect of any asymmetric load is also reduced by the effect of the tensioning forces of the auxiliary cables.

The position of the supporting cables is such that in each cross-section (section perpendicular to the horizontal projection of the supporting cables), the height position of the supporting cables towards the center of the hall increases. As a result, in that cross-section, the auxiliary cable passed over the supporting cables has a downward concave shape and, when taut, exerts pressure on all the supporting cables under it.



Figure 4. Measurement of the frequency of the wire in the supporting ropes with a frequency meteri (left), Determining the breaking force and strength in the wire (right), (photo by P.Petronijević)

During the construction of the roof structure, only the main peripheral beams girders were concreted in formwork and scaffolding, while the construction of the surface girder was done without scaffolding and formwork. After concreting the main peripheral supports, auxiliary cables were placed. Their distance in the layout from the base is also 80 cm, as with the supporting cables (Fig. 4). They are composed of 3ϕ5mm wires each and serve to stiffen the roof structure. Stiffening was achieved by introducing a prestressing force using special threaded spindles with which they are tensioned. By tightening the auxiliary cables, the pressure force was transferred to the supporting cables. With that, a tension force was introduced into them, they lengthened and assumed a new position. In this way, a rigid network of cables was obtained, in the grid fields of which, formed with two supporting and two transverse cables, the prefab plates were to be placed [3,4]. By loading the supporting cables with a constant load (from the own

weight of the concrete slabs, thermal insulation and waterproofing), the auxiliary cables are partly relaxed. This partially reduced the stiffness of the roof surface. In order for the roof structure to maintain its spatial stability under the suction effect of the wind and to avoid unwanted deformations of the roof surface when the roof is not symmetrically loaded, a force of such magnitude was introduced into the auxiliary cables during their prestressing, that a part of it remains in the cables even when the roof structure is loaded with a full load.

Given that the technical data and mechanical characteristics of prestressing cables are not known, average values between 60 and 70% of the characteristic strength of the rope were adopted for the initial force in the cable. The tensile strength of high-grade steels can be in a wide range of 1200-1800MPa. The permanent force in the cables depends on the size of the losses and potential degradation of the cross-section due to corrosion (due to the age of the building) and loosening of the anchors. The total losses of the initial prestress force usually amount to 10-20%, but they can be higher if the relaxation of the steel is high. In the period of the 60s, the metallurgical technology for the production of "low relaxation" steel was not widely distributed, so for normal relaxation steels a relaxation measure of 8% was adopted. The total relaxation was adopted according to the recommendations as a triple value of the relaxation measure. So the adopted permanent prestressing force in the auxiliary cables is 76% of the initial force. Due to the way of introducing the force, the absence of the frictional force along the pipe and the drop of force due to shrinkage and flow of concrete (the cables are free), other losses of the drop of the initial force are neglected. The nominal strength of high-grade steel is adopted as 1600MPa. The calculated permanent prestress force is ≈50kN per one stabilizing cable. For the assumed values of the mechanical characteristics of the cables and the computationally obtained force in the most loaded cable for the operational load, the value of the safety coefficient $\nu=S_{ul.}/S_{d.}=175kN/84kN=2.08$ is obtained. The required safety coefficient values for different structures are adopted from 2.5 to 5.0. In the literature, the usual values of the minimum safety coefficient for roof structures are $\nu=2.5\text{-}3.0$. Values $\nu>3.0$ are adopted for the reason of increasing the stiffness of the form and not because of increasing the carrying capacity of the rope. The relatively small value of the obtained safety coefficient is another argument in favor of the necessity of testing the roof structure and determining the actual force in the supporting cables. It should be pointed out that the safety of the cable construction can be higher than the calculation one reduced to the cross-sectional area of the cable, because the dependence between the load and the force in the cables is not linear.



Figure 5. Cavern on the roof shell (left), a sample of repair mortar (right), (photo by P.P.)

According to the original project, the prefabricated "concrete" plates were supposed to be 3 cm thick, and the other dimensions and shapes should be such that they fit into the grid fields formed by two supporting and two auxiliary cables. As the roof surface has a double curve of the variable radius of curvature, the plates had to be cast in molds of variable geometry. The prefab plates should have been reinforced with ϕ3mm wire at a distance between the wires of 10 to 15 cm. The casting of the assembly joints of the plates should have been done after applying the initial load on the roof, which in terms of intensity is equal to the calculated load from snow and wind ≈1kN/m^2. The transition from the main arched beams to the prefab plates should have been done using specially shaped plates set in mortar. An inspection of the ceiling found that it was cast on the spot with lightweight vermiculite concrete of a rigid consistency, where wire mesh and paper were used for the lost formwork (Fig. 5). The shell is reinforced with a reinforcing mesh on which paper and fabric are suspended. The thickness of the shell varies and ranges from 4 to 6 cm. The cause of such a deviation is the geometric deviation of the formwork, ie. Formation of depressions between cables. This way of making the shell excluded the previous introduction of forces by the initial ballast. The result of this is a very low stiffness of the shell and an oscillation frequency of the structure of f=1.76 Hz [6].

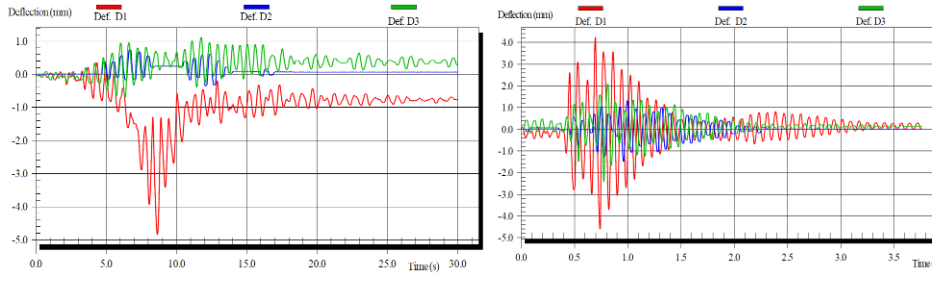


Figure 6. Dynamic deflection caused by walking, dynamic deflection caused by a half-span jump



Figure 7. Roof covering replacement and thermal rehabilitation (left), damage repair (right), (photo by P.P.)

The main drawback of the modified solution of making the shell were the partially deactivated cable wires. Several methods of rehabilitation were considered, of which shotcreting with light aggregate shotcrete from the lower side was the most logical,

taking into account the originally applied materials. The positive effects of the application of lightweight aggregate shotcrete for shell rehabilitation would be: increased stiffness, improved thermal properties, a significant reduction in noise by 6.4dB and an increase in the non-flammability class to A1. For the needs of this project, he developed several mix designs of shotcrete with the use of liapor aggregates (Table 1). The implementation of this solution was abandoned due to the cost and technological complexity of the procedure. A deterrent factor was the minimum thickness of 20 mm in which shotcrete mortar can be applied.

shotcrete mortar composition	mortar sample I	mortar sample II	Estricht
Liapor granulate 1-4mm	1.05 m^3	1.05 m^3	0.75 m^3
Cement	150kg CEM I 52.5R	185kg CEM II 42.5R	280 kg CEM 42.5R
Cementol	7.5 kg OMEGA F	9.25 kg OMEGA F	1.2 kg SMB
Water	70 kg	85 kg	125 kg
Polyethylene fibers Fiber X19	0.6 kg	0.6 kg	
Polypropylene fibers 16mm	0.4 kg	0.4 kg	1.0 kg
Density of fresh concrete mass	726 kg/m^3	785 kg/m^3	910 kg/m^3
Density of dry concrete mass	665 kg/m^3	685 kg/m^3	850 kg/m^3
Compressive strength, 28 days	2.8 MPa	2.2 MPa	5.0 MPa

Table 1. Recipe for making shotcrete (jet) lightweight aggregate mortar

In the part where the roof surface is separated from the perimeter supports, the roof construction rests on a special monolithic construction supported on a series of columns. The original roof structure was insulated with a "portoplex" system. The dedicated coatings consisted of: cold coating with bitumen emulsion, two layers of tarpaper, two layers of jute and four hot coatings with bitumen. Later, during the operation of the building, the roof covering was replaced with sheet metal. The tin cover has been partially repaired over the years. Hardboard boards 13 mm thick were hung from the underside of the roof. The found condition of the ceiling covering was bad and it fell off in several places due to the roof leaking. After removing the ceiling, traces of a fire that occurred at some point during operation were recorded on a significant part of the roof [9].

The foundation of the main supports is at a depth of D_f=-5.9m in a layer of coarse gravel. The calculated bearing capacity of the soil according to the data from the original calculation of the structure at the foundation depth of the abutments is 500kN/m^2. Investigative works in the soil as well as the geomechanical study were not performed at the time of the conceptual design. The size of the horizontal displacement of the foundation depends, in addition to the reactions of the supports, on the ratio of the coefficient of the bed and the sides of the foundation pit and the coefficient of friction of the foundation in the ground. Due to the relatively large foundation depth and significant lateral surface of the foundation, friction on the sides should not be neglected. In the original calculation, a horizontal displacement of 2 cm was adopted for each foundation. With this, the designer wanted to include other unwanted effects such as concrete

shrinkage and possible inaccuracies in the theory according to which the force in the cables was calculated. The foundations of the perimeter supports accept a significant load. According to the original calculation, the total reaction is over 11000kN. To make the foundations as light as possible, they were designed as a combination of RC slabs and walls. Underground water appears already at a depth of 2.3m, so the foundations were made by driving the pile barriers and concreting a buffer layer of about 1.0m inserted with a funnel (concreting under water). After hardening of the buffer layer and pumping of water, the foundation supports were made by concreting in dry conditions.

3. CONDITION AFTER RECONSTRUCTION

The reconstruction design, used the existing hall exclusively as a shell, i.e. as the carrier of the facade and roof covering. Inside the building, a newly designed modern RC structure of two way concrete slab floor with drop panels has been inserted (Fig. 8). Two floors with commercial content were designed and a shopping center was created. The specificity of the design solution was reflected in the complete expansion of the new and existing construction of the hall. The derived expansion width is different and ranges from 5-15cm. Special attention was paid to the undisturbed deformation of the roof shell as the most flexible part of the building. The facade construction made of sandwich panels is the only part that transfers the load between the new and the existing construction (Fig. 10). All glass surfaces, except for the entrance portals, are solely supported on the existing structure. The newly designed structure is a successful example of a structural design. The symbiosis of two RC constructions, totally different in their conception, structural design and static systems, was achieved [8].



Figure 8. Inserted construction, flat slab with drop panels (photo by P.Petronijević)



Figure 9. Implementation of the new construction (left), finishing works (right), (photo by P.P.)



Figure 10. Conversion of the fair hall into a commercial space, (photo by P.Petronijević)

4. CONCLUSION

On the example of the Fair complexes, one can see the paradigm shift of spatial planning in the Republic of Serbia in the last half a century. Currently, there is a period in which urban policy is dominantly based on the real estate market. Financial policy has imposed the mantra that the construction of residential and commercial square footage is in fact urban development. The last couple of decades stand out as a period where the public interest and the public good were put behind the interests of big capital. This phenomenon is currently culminating with the intention of demolishing most of the halls of the Belgrade Fair.

The example of the round pavilion of the fair in Leskovac shows that the implementation of commercial function has completely ruined one of the most impressive hall spaces in Serbia. Damage was done to the architecture of the building itself, but the urban degradation of that part of Leskovac was partially avoided. The area of the round pavilion of the Leskovac Fair is large and could incorporate various contents. But buildings of this type cannot be seen as shells but as a space that has its own architectural integrity. The repurposing of the round pavilion completely degraded its classification as an engineering-architectural structure of special cultural importance, which was declared an immovable cultural property by the corresponding Act of the

Institute for the Protection of Cultural Monuments in Niš. The building was originally designed with a completely clear form and purpose, with the function of something useful for the community. The only positive thing is that the building was physically saved from demolition. The original intention to demolish something that has cultural property value as an aspect of public interest was extremely malicious towards the wider community. By converting the round pavilion into a shopping center, the chance to preserve it with its original purpose was irretrievably missed.

Acknowledgement

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

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Stručni rad

ELASTIČNI SPEKTAR ODGOVORA U NOVOM EVROKODU 8

Rezime: *Obavezna primena evrokodova u Republici Srbiji uvedena je relativno skoro, što je u pojedinim slučajevima nametnulo izazove, jer su razlike u odnosu na dosadašnju praksu znatne. Priprema nove generacije evrokodova trenutno je u završnoj fazi. Kako će u bliskoj budućnosti nova generacija evrokodova biti objavljena i obavezujuća u zemljama Evropske unije, to se očekuje i u domaćoj praksi. Implementacija nove generacije standarda u praksi je zahtevan i kompleksan proces, koji pre svega temelji na blagovremenom upoznavanju stručne javnosti sa svim relevantnim pojedinostima. Osnovna ideja ovog rada je pružanje informacija o novinama koje se odnose na elastični spektar odgovora u delu 1-1 novog Evrokoda 8. Budući da temu spektra nije moguće izolovati od ostalih odredbi standarda koje na njega utiču, kroz rad je dat odgovarajući osvrt i na niz drugih odredbi koje su od značaja.*
Ključne reči: *Evrokod 8, elastični spektar odgovora, seizmički hazard, pseudoubrzanje.*

ELASTIC RESPONSE SPECTRUM IN THE NEW EUROCODE 8

Summary: *Mandatory application of Eurocodes in the Republic of Serbia was introduced relatively recently, which in some cases imposed challenges since the differences compared to the previous practice are significant. The preparation of the new generation of Eurocodes is currently in its final phase. Since in the near future the new generation of Eurocodes will be published and obligatory in the European Union countries, this is also expected in domestic practice. The implementation of the new generation of standards in practice is a demanding and complex process, which is primarily based on the timely familiarization of the professional public with all relevant details. The main idea of this paper is to provide information about the novelties related to the elastic response spectrum in part 1-1 of the new Eurocode 8. Since the topic of the spectrum cannot be isolated from the other provisions of the standard that affect it, the paper provides an appropriate overview of a number of other important provisions.*
Keywords: *Eurocode 8, elastic response spectrum, seismic hazard, pseudo-acceleration.*

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1. UVOD

Trenutno je aktuelna završna faza pripreme nove (druge) generacije evrokodova. Ovo se naravno odnosi i na Evrokod 8, koji je kako formalno tako i suštinski pretrpeo određene izmene. Deo 1-1 novog Evrokoda 8 odnosi se na opšta pravila i seizmičko dejstvo; deo 1-2 pokriva zgrade; u delu 2 obrađeni su mostovi; procena stanja i ojačanja zgrada i mostova predmet su dela 3; silosi, rezervoari, cevovodi, tornjevi, jarboli i dimnjaci razmatrani su u delu 4; a deo 5 posvećen je geotehničkim aspektima, temeljima, potpornim i podzemnim konstrukcijama.

Obavezna primena evrokodova u Republici Srbiji uvedena je relativno skoro, što je u slučajevima pojedinih konstrukcija (npr. čelične i drvene) nametnulo izazove budući da su razlike u odnosu na dosadašnju praksu znatne. Uzimajući u obzir činjenicu da će u bliskoj budućnosti nova generacija evrokodova biti objavljena i obavezujuća u zemljama Evropske unije, za očekivati je da će se to desiti i u domaćoj praksi. Implementacija nove generacije standarda u praksi je zahtevan i kompleksan proces, koji pre svega temelji na blagovremenom upoznavanju stručne javnosti sa svim relevantnim pojedinostima. Stoga, glavna ideja ovog rada je da se učini prvi korak u pružanju informacija o tome šta donosi novi Evrokod 8 u delu 1-1 [5], i to po pitanju osnovnog prikaza seizmičkog dejstva, odnosno, horizontalnog elastičnog spektra odgovora. Naravno, priču o samom spektru nije moguće izolovati od ostalih odredbi standarda koje na njega utiču, te će kroz rad biti dat odgovarajući osvrt i na niz drugih odredbi koje su od značaja.

Rad je organizovan na sledeći način: u Poglavlju 2 date su osnove projektovanja koje su od značaja za glavnu temu rada; horizontalni elastični spektar pseudoubrzanja i njemu odgovarajući elastični spektar pomeranja prikazani su u Poglavlju 3, gde su dati i izrazi za određivanje maksimalnih projektnih vrednosti kretanja tla; numerički primer prikazan je u Poglavlju 4; a Poglavlje 5 sadrži zaključke.

2. OSNOVE PROJEKTOVANJA

Konstrukcije treba da budu projektovane tako da u slučaju dejstva zemljotresa, sa odgovarajućim stepenom pouzdanosti, ljudski životi budu zaštićeni, oštećenja budu ograničena, a objekti od značaja za civilnu zaštitu ostanu funkcionalni. Ovako zahtevane performanse moguće je ostvariti proverom neprekoračenja određenih graničnih stanja pri propisanim seizmičkim dejstvima, u probabilističkom smislu. Sve navedeno se načelno odnosi i na nekonstrukcijske, odnosno pomoćne (eng. „ancillary“) elemente.

Za razliku od trenutno važeće, u novoj generaciji Evrokoda 8 definisana su četiri granična stanja (LS): blizu rušenja (NC), značajno oštećenje (SD), ograničenje oštećenja (DL), i potpuno funkcionalno (OP). Prva tri već postoje u aktuelnoj verziji Evrokoda 8, konkretno u delu 3 [3] (koji na tu temu, kao što je poznato, nije u potpunoj saglasnosti sa delom 1 [2]), dok četvrto granično stanje (OP) predstavlja novinu. U slučaju stanja OP, konstrukcija je neznatno oštećena, dozvoljen je neprekidan rad sistema koji se u njoj nalaze, i njena sanacija je ekonomična. Stanja NC i SD smatraju se graničnim stanjima nosivosti, dok se stanja DL i OP smatraju graničnim stanjima upotrebljivosti.

Seizmička dejstva treba da budu određena u smislu povratnih perioda. Dostizanje zahteva po performansama treba ostvariti odabirom odgovarajućih povratnih perioda, $T_{\mathrm{LS,CC}}$, u zavisnosti od pojedinih graničnih stanja LS, i klase posledica konstrukcije CC. Za referentni povratni period, T_{ref}, usvojena je vrednosti od 475 godina (ona može biti

izmenjena u Nacionalnom prilogu), i ona odgovara graničnom stanju SD i klasi posledica CC2, tj. periodu $T_{SD,CC2}$. Dakle, seizmičko dejstvo vezano za $T_{SD,CC2}$ označeno je kao „referentno seizmičko dejstvo“. Faktori performansi, $\gamma_{LS,CC}$, koji se primenjuju kao faktori multiplikacije amplitude seizmičkog dejstva, mogu se koristiti kao alternativa povratnim periodima, pri čemu je iz gore navedenog jasno da je $\gamma_{SD,CC2} = 1{,}0$. Ova alternativa je prilično korisna budući da primena povratnog perioda može biti izvodljiva samo ukoliko je dostupna karta seizmičkog hazarda za razmatrani povratni period. Klase posledica označene su sa CC1, CC2, CC3-a i CC3-b. Kada su u pitanju zgrade, kojima je ovaj rad i posvećen, njihove definicije slične su definicijama klasa značaja u delu 1 trenutno važećeg Evrokoda 8 [2]. Dakle, na primer, obične zgrade svrstane su u klasu CC2, škole pripadaju klasi CC3-a, a bolnice klasi CC3-b.

Iz svega navedenog jasno je da za povratni period nekog graničnog stanja i klasu posledica postoji odgovarajuća vrednost faktora performansi. Kada su u pitanju zgrade, ti podaci su definisani u delu 1-2 novog Evrokoda 8 [6], i ovde su dati u **Tabeli 1**, pri čemu se napominje da su vrednosti izvan zagrada povratni periodi izraženi u godinama, a vrednosti u zagradama su faktori performansi.

Granično stanje	**Klasa posledica**			
	CC1	**CC2**	**CC3-a**	**CC3-b**
NC	600 (1,10)	1600 (1,50)	2500 (1,75)	5000 (2,20)
SD	275 (0,80)	475 (1,00)	600 (1,10)	900 (1,25)
DL	100 (0,60)	115 (0,60)	125 (0,65)	140 (0,65)

Tabela 1. Povratni periodi $T_{LS,CC}$ izraženi u godinama i faktori performansi $\gamma_{LS,CC}$ za zgrade

Kada je neophodno izvršiti proveru graničnog stanja OP za zgrade iz klase CC3 i njihovu opremu, načelno se može usvojiti da su povratni periodi i faktori performansi za konstrukciju isti kao i za stanje DL, a da za opremu iznose 475 godina i 1,0, respektivno.

Seizmički projektni slučajevi treba da budu kategorisani u klase seizmičkog dejstva u skladu s **Tabelom 2**, a u zavisnosti od vrednosti indeksa seizmičkog dejstva, S_δ, koji je definisan ispod u **(1)**, pri čemu je δ koeficijent koji zavisi od klase posledica CC, F_α je faktor amplifikacije tla, F_T je faktor amplifikacije usled topografije, a $S_{\alpha,475}$ je referentno maksimalno spektralno ubrzanje za povratni period od 475 godina.

$$S_\delta = \delta F_\alpha F_T S_{\alpha,475} \tag{1}$$

Klasa seizmičkog dejstva	**veoma niska**	**niska**	**umerena**	**visoka**
Indeks seizmičkog dejstva (m/s/s)	$S_\delta < 1{,}30$	$1{,}30 \le S_\delta < 3{,}25$	$3{,}25 \le S_\delta < 6{,}50$	$S_\delta \ge 6{,}50$

Tabela 2. Definicija klasa seizmičkog dejstva u zavisnosti od indeksa seizmičkog dejstva S_δ

Prema [6], vrednosti koeficijenta δ za zgrade koje pripadaju klasama CC1, CC2, CC3-a i CC3-b iznose 0,6, 1,0, 1,25 i 1,6, respektivno. Opsezi vrednosti indeksa S_δ u **Tabeli 2** kalibrisane su na osnovu nivoa seizmičnosti datih u **Tabeli 3** za zgrade CC2, uz pretpostavku da zbirni efekti faktora F_α i F_T iznose 1,3.

Nivo seizmičnosti	veoma nizak	nizak	umeren	visok
Referentno maks. spektralno ubrzanje (m/s/s)	$S_{\alpha,475} < 1{,}0$	$1{,}0 \leq S_{\alpha,475} < 2{,}5$	$2{,}5 \leq S_{\alpha,475} < 5{,}0$	$S_{\alpha,475} \geq 5{,}0$

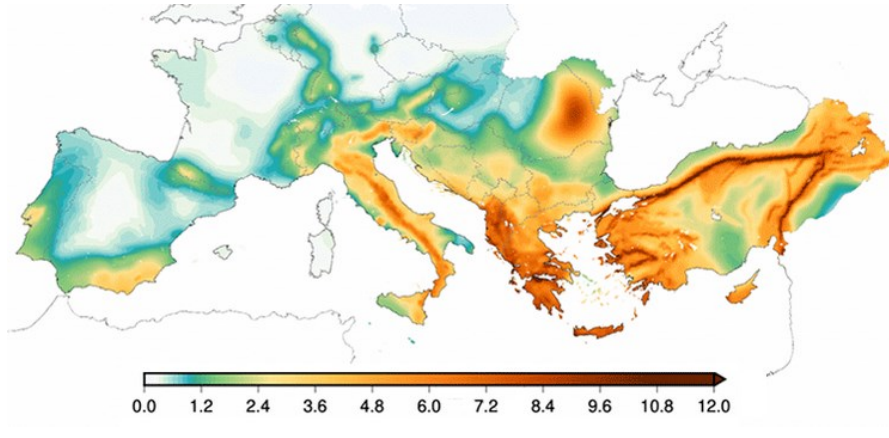
Tabela 3. Definicija nivoa seizmičnosti u zavisnosti od $S_{\alpha,475}$

Za primenu Evrokoda 8 potrebno je da teritorije budu mapirane u zavisnosti od lokalnog seizmičkog hazarda, i takav prikaz podrazumeva da su referentni spektralni parametri dati u Nacionalnom prilogu, kao i da oni treba da budu grupisani u seizmičke zone koje su međusobno razgraničene, i u kojima je seizmičnost kvalifikovana u skladu s **Tabelom 3**. Seizmički hazard treba da bude opisan sa dva referentna spektralna ubrzanja, tj. sa već pomenutim $S_{\alpha,475}$, i sa $S_{\beta,475}$, gde $S_{\alpha,475}$ odgovara vrednosti pseudoubrzanja koja definiše plato horizontalnog elastičnog spektra, dok $S_{\beta,475}$ odgovara spektralnoj vrednosti za period od 1 s, označen sa T_β. U oba slučaja vrednosti odgovaraju prigušenju od 5% i kategoriji tla A, definisanom kao standardna stena. Vrednosti $S_{\alpha,475}$ određuju se analizom seizmičkog hazarda, što se istovremeno može uraditi i za određivanje vrednosti $S_{\beta,475}$, ali se one mogu dobiti i kao:

$$S_{\beta,475} = f_h S_{\alpha,475} \tag{2}$$

Vrednosti f_h iznose 0,2, 0,3 i 0,4 za veoma niske i niske, umerene, i visoke nivoe seizmičnosti (**Tabela 3**), respektivno.

U informativnom Aneksu A dela 1-1 [5] date su mape sa vrednostima $S_{\alpha,475}$ i $S_{\beta,475}$, i one su zasnovane na evropskom modelu seizmičkog hazarda, eng. „European Seismic Hazard Model“ (ESHM20), videti i [4,1]. Na **Slici 1** dat je isečak mape, preuzet iz [4], sa prikazom vrednosti $S_{\alpha,475}$, izraženih u m/s/s.



Slika 1. Prikaz $S_{\alpha,475}$ zasnovan na ESHM20 (skala u m/s/s), preuzet iz [4]

Određivanje faktora amplifikacije tla F_α, koji se po svojoj prirodi odnosi na kratke periode, moguće je na dva načina. Prvi je zasnovan na primeni izraza u kojima figurišu dubina stenske formacije na kojoj je brzina smičućeg talasa v_s > 800 m/s, H_{800}, i ekvivalentne vrednosti v_s do referentne dubine H, $v_{s,H}$, i njegova upotreba podrazumeva

da su vrednosti H_{800} i $v_{s,H}$ poznate, što u domaćoj praksi nije čest slučaj. Drugi način, koji je u nastavku i razmatran, temelji na izrazima datim u **Tabeli 4**. Iz tabele je jasno da na tlu kategorije A važi da je $F_\alpha = 1{,}0$, što je i očekivano budući da u steni amplifikacije nema. Sve navedeno odnosi se i na faktor amplifikacije tla F_β, koji je po svojoj prirodi vezan za srednje periode. On je neophodan za određivanje elastičnog spektra odgovora razmatranog u narednom poglavlju, a zbog preglednosti predstavljen je na ovom mestu. Kategorije tla date u **Tabeli 4** neće biti detaljno razmatrane, ali je njihov pojednostavljen opis načelno sličan opisu u trenutno važećem Evrokodu 8.

Kategorija tla	F_α	F_β
A	1,0	1,0
B	$1{,}3(1 - 0{,}1S_{\alpha,RP}/g)$	$1{,}6(1 - 0{,}2S_{\beta,RP}/g)$
C	$1{,}6(1 - 0{,}2S_{\alpha,RP}/g)$	$2{,}3(1 - 0{,}3S_{\beta,RP}/g)$
D	$1{,}8(1 - 0{,}3S_{\alpha,RP}/g)$	$3{,}2(1 - S_{\beta,RP}/g)$
E	$2{,}2(1 - 0{,}5S_{\alpha,RP}/g)$	$3{,}2(1 - S_{\beta,RP}/g)$
F	$1{,}7(1 - 0{,}3S_{\alpha,RP}/g)$	$4{,}0(1 - S_{\beta,RP}/g)$

Tabela 4. Faktori amplifikacije tla F_α i F_β za standardne kategorije tla – podrazumevane (eng. „default“) vrednosti

Za veoma nisku i nisku klasu seizmičkog dejstva (prema **Tabeli 2**) moguća su pojednostavljenja, i to u vidu usvajanja $F_\alpha = 2{,}20$ i $F_\beta = 4{,}0$ za bilo koje stabilne uslove tla, te proširenja tla kategorije D, E i F na veoma meka stabilna tla sa 100 m/s $\leq v_{s,H} <$ 150 m/s. Prvo pojednostavljenje je konzervativno, i za neke vrednosti $S_{\alpha,475}$ može da se desi da njegovo usvajanje poveća klasu seizmičkog dejstva u odnosnu na realnu.

Iz **Tabele 4** se vidi da u izrazima za određivanje F_α i F_β figurišu $S_{\alpha,RP}$ i $S_{\beta,RP}$ (izraženi u g), respektivno. Njihove vrednosti mogu biti određene iz odgovarajućih mapa za razmatrane povratne periode, ili alternativno, što je u nastavku i razmatrano, korišćenjem iznad datih (**Tabela 1**) faktora performansi $\gamma_{LS,CC}$:

$$S_{\alpha,RP} = \gamma_{LS,CC} S_{\alpha,475} \tag{3}$$

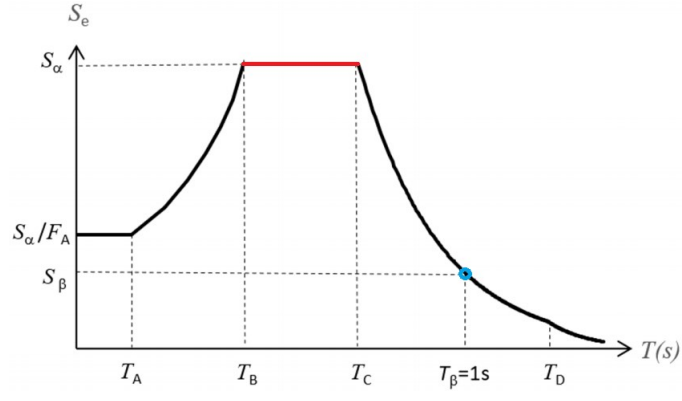
$$S_{\beta,RP} = \gamma_{LS,CC} S_{\beta,475} \tag{4}$$

Kada je u pitanju faktor amplifikacije usled topografije F_T, u slučaju da postoje topografske neregularnosti po pitanju visine veće od 30 m i po pitanju prosečnog nagiba terena većeg od 15º, treba primeniti odgovarajuće vrednosti $F_T > 1{,}0$, i to samo u slučaju tla kategorije A i B. U svim ostalim slučajevima treba usvojiti $F_T = 1{,}0$. U standardu su date tri situacije u kojima je potrebno usvojiti $F_T > 1{,}0$, pri čemu je za dve propisano da je $F_T = 1{,}2$, a za treću da je $F_T = 1{,}4$ (za detalje videti [5]).

Na kraju ovog poglavlja treba napomenuti da klasa seizmičkog dejstva direktno utiče na projektovanje konstrukcije. U slučaju veoma niske klase, odredbe Evrokoda 8 mogu biti zanemarene, dok u slučaju niske klase i za pojedine kategorije konstrukcija (definisane od strane relevantnih organa ili u Nacionalnom prilogu), zahtevi po pitanju performansi mogu biti zadovoljeni kroz primenu jednostavnijih pravila od onih koja su data u relevantnim delovima Evrokoda 8.

3. ELASTIČNI SPEKTRI ODGOVORA I MAKSIMALNE PROJEKTNE VREDNOSTI KRETANJA TLA

U odnosu na horizontalni elastični spektar pseudoubrzanja u trenutno važećem Evrokodu 8, za koji su predložena dva osnovna tipa, definicija spektra u novoj generaciji standarda značajno je izmenjena (**Slika 2**), i bez podele na tipove. Pre svega, umesto primene jednog parametra za određivanje pojedinih delova spektra, za koji je u trenutno važećem Evrokodu 8 izabrano maksimalno ubrzanje tla, za novi spektar primenjena su dva: S_α koji definiše plato spektra, i S_β koji definiše spektralnu vrednost u $T_\beta = 1$. Druga bitna razlika odnosi se na oblast veoma kratkih perioda, gde je sada u spektru usvojena horizontalna grana između $T = 0$ s i $T = T_A$. Ova izmena je potpuno u skladu sa poznatom činjenicom da za veoma kratke periode praktično ne dolazi do amplifikacije. Posledično, umesto dosadašnje četiri, spektar sada ima pet grana. Kao što je ispod **Slike 2** navedeno, horizontalna osa spektra je, za razliku od dosadašnje prakse, prikazana u logaritamskoj razmeri, što poboljšava vidljivost za periode koji su najčešće zastupljeni.



Slika 2. Horizontalni elastični spektar pseudoubrzanja (horizontalna osa u logaritamskoj razmeri), preuzeto iz [5] i dopunjeno prikazom značenja parametara S_α i S_β

Određivanje spektralnih vrednosti $S_e(T)$ za pojedine grane spektra vrši se prema sledećim izrazima:

$$0\ \text{s} \le T < T_A: \quad S_e(T) = \frac{S_\alpha}{F_A} \tag{5}$$

$$T_A \le T < T_B: \quad S_e(T) = \frac{S_\alpha}{T_B - T_A}\left[\eta\left(T - T_A\right) + \frac{T_B - T}{F_A}\right] \tag{6}$$

$$T_B \le T < T_C: \quad S_e(T) = \eta S_\alpha \tag{7}$$

$$T_C \le T < T_D: \quad S_e(T) = \eta \frac{S_\beta T_\beta}{T} \tag{8}$$

$$T \ge T_D: \quad S_e(T) = \eta T_D \frac{S_\beta T_\beta}{T^2} \tag{9}$$

Spektralna ubrzanja S_α i S_β treba odrediti kao:

$$S_\alpha = F_\alpha F_T S_{\alpha,RP} = F_\alpha F_T \gamma_{LS,CC} S_{\alpha,475} \tag{10}$$

$$S_\beta = F_\beta F_T S_{\beta,RP} = F_\beta F_T \gamma_{LS,CC} S_{\beta,475} \tag{11}$$

Vrednost perioda T_C i T_B treba odrediti iz izraza:

$$T_C = \frac{S_\beta T_\beta}{S_\alpha} = \frac{S_\beta}{S_\alpha} \ (T_\beta = 1) \tag{12}$$

$$T_B = \begin{cases} \dfrac{T_C}{\chi}, & \text{za } 0{,}05 \text{ s} \le \dfrac{T_C}{\chi} \le 0{,}10 \text{ s} \\ 0{,}05 \text{ s}, & \text{za } \dfrac{T_C}{\chi} < 0{,}05 \text{ s} \\ 0{,}10 \text{ s}, & \text{za } \dfrac{T_C}{\chi} > 0{,}10 \text{ s} \end{cases} \tag{13}$$

Ukoliko se ispostavi da je vrednost $T_C \ge 1$, potrebno je sprovesti specifičnu studiju u cilju definisanja horizontalnog elastičnog spektra.

Kada su u pitanju vrednosti T_A, F_A i χ, u odsustvu specifičnih studija, za njih može biti usvojeno 0,02 s, 2,5 i 4, respektivno. Isto važi i za vrednost T_D, koja može biti određena (u sekundama) kao:

$$T_D = \begin{cases} 2, & \text{za } S_{\beta,RP} = \gamma_{LS,CC} S_{\beta,475} \le 1 \text{ m/s/s} \\ 1 + S_{\beta,RP} = 1 + \gamma_{LS,CC} S_{\beta,475} (S_\beta \text{ u m/s/s}), & \text{za } S_{\beta,RP} = \gamma_{LS,CC} S_{\beta,475} > 1 \text{ m/s/s} \end{cases} \tag{14}$$

Što se tiče faktora korekcije prigušenja η, kao i do sada, za prigušenje $\xi = 5\%$ on iznosi 1,0, ali je za razliku od trenutno važećeg Evrokoda 8 njegova definicija u novoj generaciji standarda, koja važi za periode T do 4 s, data u odnosu na periode T_A i T_B:

$$\eta = \left\{ \begin{array}{ll} 1, & \text{za } T \le T_A \\ \sqrt{\left(10 + \left(\dfrac{T_B - T}{T_B - T_A} \right)^3 (\xi - 5) \right) / (5 + \xi)}, & \text{za } T_A < T < T_B \\ \sqrt{10 / (5 + \xi)}, & \text{za } T \ge T_B \end{array} \right\} \ge 0{,}55 \tag{15}$$

Kao što je poznato, u trenutno važećem Evrokodu 8 izraz dat u trećem redu **(15)** primenjuje se u celom opsegu perioda. Donja granica od 0,55 ostala je nepromenjena.

Vrednosti elastičnog spektra pomeranja, $S_{De}(T)$, primenjivog na konstrukcije sa dugim periodima, treba odrediti kao:

$$T \le T_E: \qquad S_{De}(T) = S_e(T) \left(\frac{T}{2\pi} \right)^2 \tag{16}$$

$$T_E < T \le T_F: \quad S_{De}(T) = S_{De}(T_E)\left[1 + \left(\frac{F_L}{F_\beta} - 1\right)\frac{T - T_E}{T_F - T_E}\right] \tag{17}$$

$$T > T_F: \quad S_{De}(T) = S_{De}(T_E)\frac{F_L}{F_\beta} \tag{18}$$

Za T_E treba usvojiti 6 s ukoliko je $T_D < 6$ s, u suprotnom se usvaja da je $T_E = T_D$, a podrazumevana vrednost za T_F je 10 s. F_L je faktor amplifikacije tla koji se po svojoj prirodi odnosi na duge periode. Ukoliko vrednost $v_{s,H}$ nije poznata, važi sledeće: $F_L = F_\beta$ za kategoriju tla A; $F_L = 0{,}90F_\beta$ za kategorije tla B ili E; $F_L = 0{,}75F_\beta$ za kategorije tla C ili F; i $F_L = 0{,}60F_\beta$ za kategoriju tla D. Za poznato $v_{s,H}$ važi da je

$$F_L = \left(\frac{v_{s,H}}{800}\right)^{-0,4} \tag{19}$$

Konačno, maksimalne projektne vrednosti ubrzanja tla (PGA_e u m/s/s), brzine tla (PGV_e u m/s) i pomeranja tla (PGD_e u m) mogu biti određene kao:

$$PGA_e = \frac{S_\alpha}{F_A} \tag{20}$$

$$PGV_e = 0{,}06\left(S_\alpha S_\beta\right)^{0,55} \tag{21}$$

$$PGD_e = S_{De}(T_F) = 0{,}025\eta T_\beta T_D F_L F_T S_{\beta,RP} \tag{22}$$

U izraze iznad S_α i S_β treba uneti u m/s/s, a T_β, T_D i T_F u sekundama.

4. NUMERIČKI PRIMER

U okviru ovog poglavlja prikazan je numerički primer koji temelji na proizvoljno izabranoj vrednosti $S_{\alpha,475} = 2{,}50$ m/s/s $\approx 0{,}25$ g (umeren nivo seizmičnosti prema **Tabeli 3**, i $S_{\beta,475} = 0{,}3 \cdot 2{,}50 = 0{,}75$ m/s/s $\approx 0{,}075$ g prema **(2)**), klasi posledica CC2 i graničnom stanju SD (koeficijent $\delta = 1{,}0$, povratni period $T_{SD,CC2} = 475$ godina, faktor performansi $\gamma_{SD,CC2} = 1{,}0$), i pretpostavci faktora amplifikacije usled topografije $F_T = 1{,}0$. Shodno **Tabeli 4**, određene su odgovarajuće vrednosti faktora amplifikacije tla F_α i F_β, i rezultati su dati u **Tabeli 5**. U tabeli su takođe date vrednosti indeksa seizmičkog dejstva S_δ, koje su određene iz **(1)**, kao i klasifikacija seizmičkog dejstva, prema **Tabeli 2**.

Iz **Tabele 5** je jasno da je za tlo kategorija A i B dobijena niska klasa seizmičkog dejstva, što znači da bi u praksi za pojedine kategorije konstrukcija zahtevi po pitanju performansi mogli biti zadovoljeni kroz primenu jednostavnijih pravila od onih koja su data u relevantnim delovima Evrokoda 8.

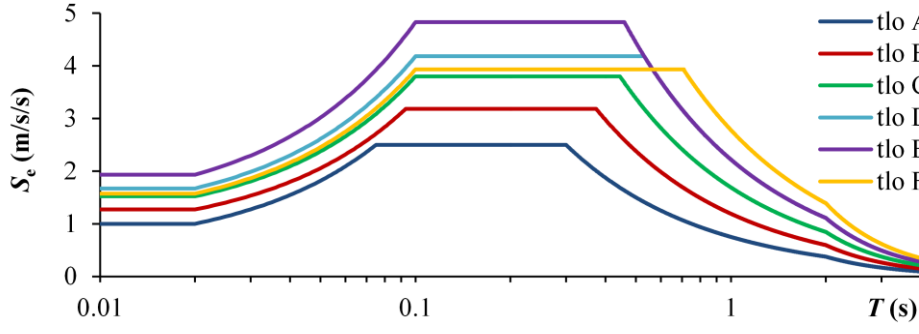
Vrednosti spektralnih ubrzanja S_α i S_β, određene iz **(10)** i **(11)**, i perioda T_C, T_B i T_D, određene iz **(12)-(14)**, respektivno, date su u **Tabeli 6**, a za T_A, F_A i χ usvojene su podrazumevane vrednosti. Za prigušenje ξ usvojena je vrednost od 5%, iz čega sledi da je faktor korekcije prigušenja $\eta = 1{,}0$. Horizontalni elastični spektri pseudoubrzanja dobijeni za razmatrane ulazne podatke prikazani su na **Slici 3**.

Kategorija tla	F_α	F_β	S_δ (m/s/s); klasa seizmičkog dejstva
A	1,0	1,0	2,50; niska
B	1,27	1,58	3,18; niska
C	1,52	2,25	3,80; umerena
D	1,67	2,96	4,18; umerena
E	1,93	2,96	4,83; umerena
F	1,57	3,70	3,93; umerena

Tabela 5. Vrednosti F_α, F_β, S_δ i klase seizmičkog dejstva za različite kategorije tla

Kategorija tla	S_α (m/s/s)	S_β (m/s/s)	T_C (s)	T_B (s)	T_D (s)
A	2,50	0,75	0,3	0,075	2,0
B	3,18	1,19	0,37	0,093	
C	3,80	1,69	0,44	0,10	
D	4,18	2,22	0,53		
E	4,83	2,22	0,46		
F	3,93	2,78	0,71		

Tabela 6. Vrednosti S_α, S_β, T_C, T_B i T_D za različite kategorije tla



Slika 3. Horizontalni elastični spektri pseudoubrzanja za različite kategorije tla i prigušenje od 5%

5. ZAKLJUČAK

Priprema nove (druge) generacije evrokodova trenutno je u završnoj fazi. Evrokod 8 je i formalno i suštinski pretrpeo određene izmene, i njegova implementacija u praksi predstavljaće izazov. Prvi korak u prevazilaženju istog je blagovremeno upoznavanje stručne javnosti sa svim relevantnim pojedinostima. Glavna ideja ovog rada je pružanje informacija o novinama koje se odnose na elastični spektar odgovora u delu 1-1 novog Evrokoda 8.

U radu su razmatrane osnove projektovanja, prvenstveno vezane za zahteve po performansama, granična stanja, određivanje seizmičkih dejstava, klase seizmičkog dejstva, nivoe seizmičnosti, i opis seizmičkog hazarda. One su od ključnog značaja za određivanje horizontalnih elastičnih spektara odgovora koji su detaljno razmatrani u radu, i nakon kojih su dati izrazi za određivanje maksimalnih projektnih vrednosti seizmičkog kretanja tla. Kroz odgovarajući numerički primer, pokazano je određivanje spektara pseudoubrzanja za umeren nivo seizmičnosti i šest kategorija tla.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
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Prethodno saopštenje

COMPARATIVE ANALYSIS OF CONCRETE WITH PORTLAND COMPOSITE CEMENTS WITH REDUCED CLINKER CONTENT

Summary: *The paper presents the results of experimental testing of concrete mixtures made with Portland composite cements with reduced clinker content (PC 50M). The aim of the study was to examine the justification for using this type of cement (defined by the document SDO 15 001:2021) in concretes commonly used in construction practice. The compressive strength reached values ranging from 48.0 to 55.8 MPa, while all other parameters: splitting tensile strength, water impermeability, frost resistance, and thermogravimetric analysis, also showed very good performance. The results support the use of this type of cement in the construction industry.*
Keywords: *Portland composite cements, concrete, mechanical properties, durability.*

UPOREDNA ANALIZA BETONA SA PORTLAND KOMPOZITNIM CEMENTIMA SA SMANJENIM SADRŽAJEM KLINKERA

Rezime: *U radu su prikazani rezultati eksperimentalnih ispitivanja betonskih mešavina spravljanih sa portland-kompozitnim cementima sa smanjenim sadržajem klinkera (PC 50M). Cilj rada je bio da se ispita opravdanost upotrebe ovog tipa cementa (definisan dokumentom SDO 15 001:2021) u betonima koji se uobičajeno koriste u građevinskoj praksi. Čvrstoće pri pritisku, dostizale su vrednosti od 48.0 do 55.8 MPa, dok su i svi ostali parametri: čvrstoća pri zatezanju cepanjem, vodonepropustljivosi, otpornosti na dejstvo mraza i termogravimetrijska analiza pokazali jako dobre vrednosti. Dobijeni rezultati podržavaju mogućnost upotrebe ovog tipa cementa u građevinskoj industriji.*
Ključne reči: *portland-kompozitni cementi, beton, mehaničke karakteristike, trajnost.*

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1. INTRODUCTION

Human society has always had a need for construction materials, initially natural ones such as stone, wood, and earth, and later artificial materials developed through the processing of natural resources. The development of construction materials represents one of the key components in the advancement of human civilization.

The beginning of organized construction material production dates back to ancient Egypt (around 3000 BCE), where bricks and mortars were used. The discovery of Portland cement in 1844 marked the beginning of the "concrete era", with concrete now being the second most widely used material in the world, right after water [1].

During the production of Portland cement, complex physicochemical reactions occur, including the decarbonation of limestone (release of CO_2), the formation of new mineral phases, and the creation of clinker—a synthetic rock composed mainly of minerals such as alite (C_3S), belite (C_2S), aluminate (C_3A), and ferrite (C_4AF).

The entire production process consists of eight main stages: raw material extraction, preparation (grinding, proportioning and blending) and storage, kiln firing (clinker formation) and cooling, grinding, and the storage and packaging of the final cement product. Each stage requires precise management and control to ensure the high quality of the final product, in accordance with applicable industrial standards.

However, intensive cement production also brings certain challenges, especially in terms of environmental protection. The cement industry is responsible for approximately 5-8% of total global carbon dioxide (CO_2) emissions, which is largely attributed to the production process of Portland cement clinker [1].

In the context of modern climate change and environmental challenges, the goal of the construction industry is to develop materials that are not only functional and reliable but also sustainable. Portland-composite cements with reduced clinker content represent a response to these demands, as they enable a significant reduction in CO_2 emissions without compromising concrete performance.

The optimization of clinker content in cement has been achieved by replacing a certain percentage of Portland cement clinker with various types of materials. The most common substitute materials, including ground granulated blast furnace slag, fly ash, and silica fume, are by-products of coal combustion in power plants, iron production in blast furnaces, and the production of silicon and aluminum alloys.

The use of these industrial by-products has multiple environmental benefits. In addition to the pozzolanic effect (the formation of products similar to hardened cement stone, in the presence of water and calcium hydroxide), their use in cement production contributes to reducing the amount of waste that would otherwise be disposed of in landfills, which directly affects the reduction of soil, air and water pollution. The disposal of such materials often poses the risk of releasing harmful gases, dust, and heavy metals into the environment.

However, when industrial by-products are added to concrete, these components are permanently bonded into the solid structure of hardened concrete. Studies have shown that heavy metals in cement composites remain stably immobilized within the concrete and mortar matrix, without significant leaching or emission into the environment. In this way, not only is the environmental impact reduced, but the circular economy is also supported through the reuse of materials from other industrial processes [2].

In addition to the obvious environmental benefits, the inclusion of by-products in the composition of cement also has technical advantages - it improves the durability of concrete, resistance to chemical influences and the lifespan of structures [3].

For all these benefits to be effectively implemented in practice, standardization in the field of cement is essential. It ensures the production and use of materials with predictable and reliable properties. The European standard EN 197-1 has laid the foundation for the classification and quality control of cement within the EU. By implementing this standard through national regulations, namely SRPS EN 197-1 [4] and SDO 15 001:2021 [5], Serbia is following the European direction in the development of construction materials. These standards enable the use of a wide range of supplementary materials (such as slag, fly ash, pozzolans, and limestone), which significantly reduce clinker content, enhance concrete resistance, and make it more suitable for specific exploitation conditions.

For all the above reasons, the future direction of improvement the cement industry is oriented toward the development and application of Portland-composite cements with a higher proportion of mineral additives, as well as the increasing use of alternative raw materials and fuels. Accordingly, for this reason, and for the purposes of the analyses presented in this paper, three concrete mixtures with cements from different manufacturers in Serbia, type PC 50M, were prepared and tested.

Portland-composite cements with reduced clinker content (PC 50M), used in concrete mixtures, are defined by the Serbian Document for the Assessment of Portland-Composite Cements with Reduced Clinker Content (SDO 15 001:2021). These cements are considered ordinary cements that are not covered by the scope and application of the Serbian standard that adopted the harmonized standard SRPS EN 197-1:2013, nor by the existing technical regulation *Rulebook on the Quality of Cement* (Official Gazette of RS, Nos. 34/2013 and 44/2014).

Since these types of cements have only recently been placed on the market, it can be said that the professional community still has limited experience with the properties of concrete made with them. For this reason, the study presents some of the basic properties of hardened concrete made with the following cements:

- PC 50M (S-V-L) 42.5N, manufacturer “A”: Portland cement clinker with an addition of ground granulated blast furnace slag, fly ash, and limestone (in a proportion of 36-50%), and gypsum and mineral fillers (in a proportion of 0-5%);
- PC 50M (V-L) 42.5N, manufacturer “B”: Portland cement clinker with an addition of fly ash and limestone (in a proportion of 36-50%, with the limestone content limited to 6-20%), and gypsum and mineral fillers (in a proportion of 0-5%);
- PC 50M (V-L) 42.5N, manufacturer “C”: Portland cement clinker with an addition of fly ash and limestone (in a proportion of 36-50%, with the limestone content limited to 6-20%), and gypsum and mineral fillers (in a proportion of 0.5%).

2. COMPONENET MATERIALS AND TESTING METHODS

As previously mentioned, three concrete mixtures were prepared using identical quantities of constituent materials, with the only difference being the type of cement applied. The mixtures are labeled according to the origin of the cement with the symbols A, B and C. For the concrete production, a natural river three-fraction aggregate (D = 16 mm) with density of 1528 kg/m^3 was used. Also, tap water from the municipal supply system was used.

The adopted concrete mix design is shown in Table 1.

After mixing and placing, the concrete was cured in accordance with the SRPS EN 12390-2:2019 [6] standard until the day of testing.

The following properties of the hardened concrete samples were tested: bulk density (in accordance with 12390-7:2019 [8]), compressive strength (in accordance with 12390-3:2019 [8]), splitting tensile strength (in accordance with 12390-6:2019 [9]), water impermeability (in accordance with 12390-8:2019 [10]), frost resistance (in accordance with SRPS U.M1.206:2023 [11]), and thermogravimetric analysis (TG/DTA).

Component		Quantity (kg/m^3)	
Aggregate	Fraction I (0-4 mm)	870	1740
	Fraction II (4-8 mm)	383	
	Fraction III (8-16 mm)	487	
Cement		375	
Water		195	

Table 1. Quantities of constituent materials for 1 m³ of concrete

3. TEST RESULTS

3.1. Bulk density and compressive strenght

The compressive strength of the concrete was tested in accordance with the SRPS EN 12390-3:2019 [8] standard, using a gradual loading method on 15 cm edge concrete cubes until failure. The tests were conducted on three samples at the ages of 7, 28, and 60 days. The obtained compressive strengths are shown in Figure 1. Bulk density was determined using the same samples, as for the compressive strength, at the age of 28 days. The bulk density values were similar for all types of concrete reaching 2354 kg/m^3 for concrete A, 2311 kg/m^3 for concrete B and 2332 kg/m^3 for concrete C.

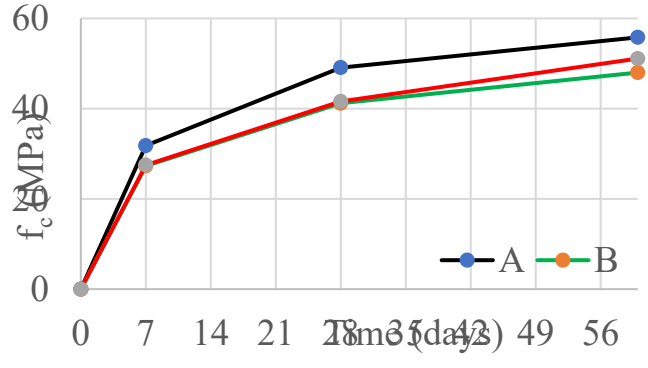


Figure 1. Compressive strength of tested mixtures

From the displayed results, it can be observed that mixture A, compared to the other two, has approximately 15% higher compressive strength after 7 and 28 days. These results can be associated with the presence of ground granulated blast furnace slag in the cement used for preparing concrete mixture A.

In comparison, at 60 days of age, mixture A shows 14% and 8% higher compressive strength compared to mixtures B and C, respectively. This indicates the activation of pozzolanic properties in the composite cements of mixtures B and C, whose effects may become more prominent over a longer period.

In any case, from the perspective of the achieved compressive strengths, it can be concluded that the subject cements face no difficulties in reaching high strength classes with the use of 370 kg/m³ of cement and a water-cement ratio of 0.52.

3.2. Tensile Strength by Splitting

Tensile strength determined by the splitting method of a cylindrical sample under linear loading along the axis represents a practical and reliable alternative to direct tensile testing. The obtained value is, on average, 15-20% higher than the one obtained through direct tensile testing.

The testing was conducted in accordance with the standard SRPS EN 12390-6:2024[8], on cylindrical samples with a diameter and height of 150/150 mm at an age of 39 days.

The results of the tensile strength by splitting for each type of concrete were obtained as the average values of three tested samples. The results of the tests are presented in Table 2.

Concrete mix	Concrete age	Tensile strength by splitting
	t (days)	f_{ct} (MPa)
A	39	2.7
B	39	3.3
C	39	2.8

Table 2. Results of tensile strength by splitting

The calculated values of tensile strength by splitting range from 2.7 to 3.3 MPa, which are expected given the compressive strengths obtained. The differences between the concrete mixes are not significant.

3.3. Water impermeability

The water impermeability of concrete represents its ability to resist water penetration under a pre-defined pressure. This property is closely related to the concrete structure, primarily its porosity and the arrangement of capillary voids in the cement paste. The lower the amount and connectivity of capillary pores, the higher the water impermeability of the concrete, meaning that the concrete demonstrates better resistance to water infiltration through its mass.

The water impermeability of concrete was tested according to the standard SRPS EN 12390-8:2019 [10] on cylindrical samples with a diameter and height of 150/150 mm at an age of 36 days.

The samples were exposed to a pressure of 5 bar for a duration of 72 hours. Afterward, the samples were broken along the cylinder's axis, and the maximum water penetration depth of each sample was measured. The results of the tests are shown in Figure 2.

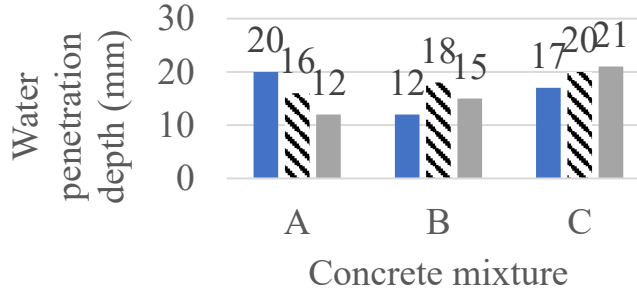


Figure 2. Results of the water penetration depth of tested concrete mixtures

Based on the presented results, it can be concluded that all three concrete mixtures had relatively low water penetration, ranging from 12 to 21 mm. In accordance with the SRPS U.M1.206:2023 standard [11], all three concrete mixtures can be classified as waterproof, with water impermeability classes V-3 (h_{max}=20 mm) and V-2 (h_{max}=30 mm). The obtained results confirm that the use of Portland-composite cements with reduced clinker content (PC 50M) is technically justified in terms of water impermeability.

3.4. Frost Resistance

The primary cause of concrete destruction due to frost action are the internal stresses that occur when water freezes in the pores or cracks present in the concrete. These internal stresses are a result of the expansion of ice, which has a 9% greater volume than the water from which it was formed. The compactness of the concrete, i.e., low capillary porosity, is one of the main factors in the concrete's resistance to frost, and this can be achieved through a sufficiently low water-cement ratio and proper concrete placement.

The frost resistance of the concrete was tested in accordance with the SRPS U.M1.206:2023 Annex D standard [10] on cubic samples (15 cm edge). The samples were subjected to alternating cycles of freezing and thawing for a total of 50 cycles. The

compressive strength of the reference samples and those exposed to freezing and thawing cycles were tested according to the standard. The average compressive strength values for both groups are shown in Table 3.

Based on the results presented, it can be concluded that all samples met the frost resistance criteria for 50 cycles, as the decrease in compressive strength between reference samples and samples exposed to freezing and thawing cycles did not exceed 25%.

Concrete mix	Sample type	Compressive strength	Strength loss
		f_c *(MPa)*	Δf_c *(%)*
A	Reference	55.8	8.1
	Frozen samples	51.3	
B	Reference	48.0	6.5
	Frozen samples	44.9	
C	Reference	51.1	12.1
	Frozen samples	44.9	

Table 3. Results of concrete frost resistance testing

3.5. Termogravimetric analysis

Thermogravimetric analysis (TG/DTA) was performed on powdered concrete samples at the age of 28 days, in the laboratory of the University of Belgrade Innovation Center, Faculty of Technology and Metallurgy. According to this method, the material was subjected to heating with a gradual increase in temperature in the range from 0 to 1000°C, while continuously measuring the sample's mass loss over time, referred to as the loss on ignition.

The results show the percentage mass loss as a function of temperature and time, presented in the form of a TG curve. The second curve, DTG, represents its first derivative, i.e., it is defined by tangents to the TG curve. The maximum values (peaks) of the DTG curve indicate the points at which the greatest mass change occurs, showing the temperature at which the decomposition of a specific component within the sample takes place. The analysis is based on the knowledge of the temperatures at which specific compounds decompose.

The results of the thermogravimetric analysis are shown in Figure 3.

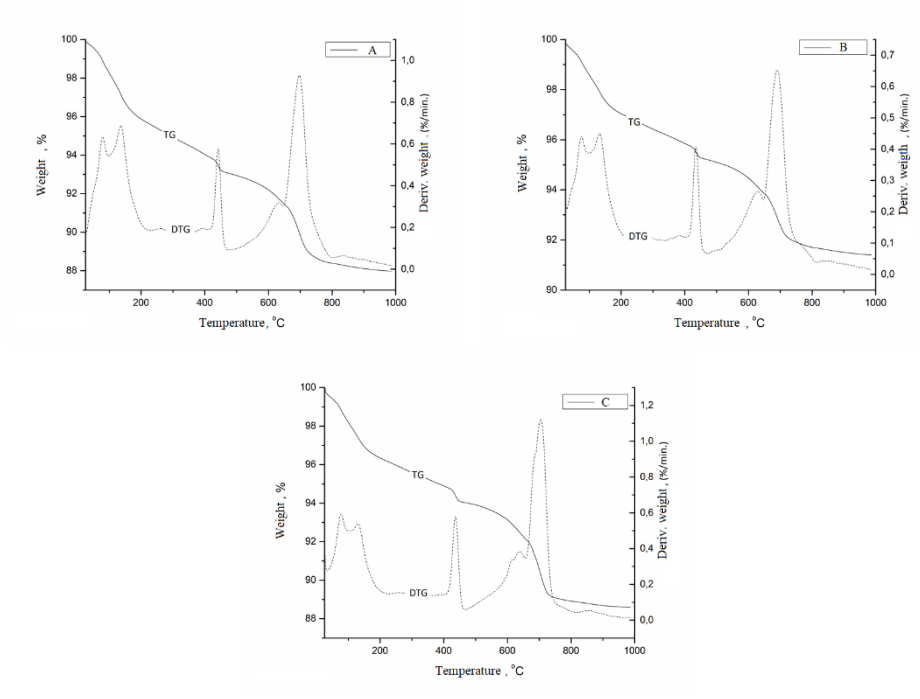


Figure 3. Results of Thermogravimetric Analysis of Concrete Samples A, B, and C

From the obtained results, it can be concluded that the first major loss on the DTG curve was recorded for sample A at a temperature between 100-200°C, where chemically bound water, associated with the decomposition of cement hydration products, is lost.

The next significant loss occurs at a temperature just below 500°C, where chemically bound water from calcium hydroxide ($Ca(OH)_2$) is lost. In those concretes where this peak is the highest, it can be expected that additional pozzolanic activity occurs. Where it is absent, it is likely that $Ca(OH)_2$ has already been utilized, and there will not be enough left to contribute to further strength gain at later ages of the concrete.

Considering that the compressive strength increase from 28 to 60 days was 13.8% for mix A, 16.6% for mix B, and 23.0% for mix C, it was expected that mixes with higher strength gains would show a lower peak on the DTG curve at 500°C. This was confirmed for mix C, which indicates that this mix can be expected to experience a later strength gain. However, final conclusions cannot be drawn without additional tests on carbonation and compressive strength at higher concrete ages.

4. CONCLUSION

The paper presents the results of testing concrete mixes prepared with Portland-composite cements with reduced clinker content (PC 50M), as defined by the Serbian Assessment Document for Portland-composite cements with reduced clinker content (SDO 15 001:2021). Since this type of cement has been produced in Serbia only since 2021 and

is still not widely used in practice, the conducted tests aimed to determine the justification for its use in concrete production.

The tests on hardened concrete led to the following conclusions:

- In terms of achieved compressive strength, it can be concluded that these cements have no difficulty in reaching high strength classes of concrete;
- The compressive strength gain is clearly expressed, even at ages greater than 28 days;
- The splitting tensile strengths ranged between 2.7 and 3.3 MPa, which is consistent with the corresponding compressive strengths;
- All three concrete mixes exhibited relatively low water penetration depths, ranging from 12 to 21 mm, and in most cases met even the requirements for the water impermeability class V-3;
- All three concrete mixes met the criteria for frost resistance after 50 freeze-thaw cycles.

In general, all obtained test results are encouraging, both from the aspect of mechanical characteristics of concrete, and from the aspect of durability of concrete, and show that the use of more environmentally friendly cements is possible in construction practice.

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[8] SRPS EN 12390-3:2019 Testing hardened concrete - Part 3: Compressive strength of test specimens

[9] SRPS EN 12390-6:2024 Testing hardened concrete - Part 6: Tensile splitting strength of test specimens

[10] SRPS EN 12390-8:2019 Testing hardened concrete - Part 8: Depth of penetration of water under pressure

[11] SRPS U.M1.206:2023 Concrete — Guidance and rules for national technical requirements for production of concrete applied in concrete, reinforced concrete and prestressed concrete structures

SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

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Prethodno saopštenje

RECYCLING POSSIBILITIES OF WOOD-PLASTIC COMPOSITES (WPC)

Summary: *Nowadays, the main goal is to minimize the negative impact of a product or service on the environment. Wood-plastic composites (WPC) can be produced in a sustainable way, with minimal waste. WPC composites can be produced from natural raw materials. They can also be produced as a result of recycling, where either wood or plastic come from the recyclate. In WPC production a large percentage of waste can be reused. WPC does not contain formaldehyde or aggressive organic compounds, it is usable product (it can be grinded and reused), as waste it is not hazardous and can be disposed of with other waste.*
Key words*: wood-plastic composite (WPC); WPC recycling; waste; wood and plastic.*

MOGUĆNOSTI RECIKLIRANJA DRVNO-PLASTIČNIH KOMPOZITA (WPC)

Rezime: *U današnje vreme glavni cilj je minimizirati negativan uticaj proizvoda ili usluge na životnu sredinu. Drvno-plastični kompoziti (WPC) se mogu proizvesti na održiv način, uz minimalnu količinu otpada. WPC kompoziti se mogu proizvoditi od prirodnih sirovina. Mogu se dobiti i kao rezultat recikliranja, gde iz recikliranog materijala dolazi ili drvo ili plastika. U proizvodnji WPC veliki procenat otpada se može ponovo koristiti. WPC ne sadrži formaldehid niti agresivna organska jedinjenja, upotrebljiv je proizvod (može se samleti i ponovo koristiti), kao otpad nije opasan i može se odložiti sa drugim otpadom.*
Ključne reči*: drvno-plastični kompozit (WPC); recikliranje WPC; otpad; drvo i plastika.*

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1. INTRODUCTION

A composite is a material consisting at least two components that form separate phases and do not mix with each other. The final composite contains better or new properties compared to the properties of the initial components [1]. The goal of processing composites is to improve the structural, thermal, chemical or some other characteristics of the material. The structure of the composite consists the matrix which surrounds and holds together the groups of fibers or parts of the reinforcement, and the reinforcer, which gives the composite strength and hardness [2]. Composites with wood fillers can be competitive with inorganic filler materials. Wood is a renewable raw material which is more environmentally friendly and less energy-intensive compared to many other building materials [3;4].

Wood-plastic composites (WPC) are one example of modern materials, which are basically obtained by combining two substances that are most often recycled products (plastic and wood). WPC is an ideal alternative to traditional materials and due to its numerous advantages, it is the most common substitute for wood, and has been a trend in the world of architecture, design and construction for many years. They are used in the automotive, construction and road industries, e.g. outdoor deck floors, railings, fences, landscaping timbers, cladding and siding, park benches, moulding and trim, window and door frames, and indoor furniture as well as for agriculture or the packaging industry. In the production of some profiles for windows, doors, fences, benches, decks and floors, WPC are environmentally friendly, durable, stable, and easy to maintain [5]. They can be used for indoor and outdoor applications.

The development of WPC materials aims to obtain products with desired characteristics. Nowdays researchers are focused on continuous improvement and precise definition of influential parameters so that the final product satisfies its function and has the desired properties [6]. The new generation of WPC enabled the rapid development of composites with good mechanical properties, high dimensional stability, and the ability to be formed into complex shapes. The most common are wood-plastic profiles for flooring, coated with pure PVC material on the surface [7].

WPC composites are products that do not require further processing. They are resistant to weather conditions, water and mold and can be used outdoors, where wood alone would be unsuitable [5].

2. ADVANTAGES OF USING WOOD-PLASTIC COMPOSITES (WPC)

WPC have many advantages: they use cheap and available raw materials, with price are competitive to wood and plastomers, they are easy to produce using existing production techniques, products can be in various sizes and shapes, do not require surface protection, have lower maintenance costs compared to wood, can be connected with nails. WPC can have very good properties in various conditions of use, which are dominantly influenced by wood materials (fillers). The size and shape of wood component have esencial influence on the final composite properties. In addition to wood, other non-wood lignocellulose materials are also used as fillers. The organic origin of the fillers determines their properties, and finally the properties of the manufactured WPC. In order to reduce natural disadventages, various pre-treatments and chemical

modifications of organic fillers are performed. As a result of the modifications, composites with better application properties are obtained.

Wood or wood-based additives in wood plastic composites (WPC) may be present as reinforcement and as fillers. The reinforcement can be made of wood in the form of fibres or grains, i.e. wood flour [8;9]. Usually, fibres with small dimensions are used, which allows even distribution of fibres in the matrix and also provide a larger specific surface area and better adhesion [10]. On the other hand, lignocellulose plastics in the form of particles act as fillers when their dimensions are approximately equal in all directions [11].

Wood flour is commonly used in WPC production due to its availability and easy incorporation into conventional plastic manufacturing processes [12]. Wood flour particles (100 μm – 500 μm in size) are shorter than 1 mm in length and have a wide distribution of length to diameter ratios (aspect ratio or L/D ratio) [13]. Wood fibers differ from wood particles because they have a higher length to width ratio. Particle size characteristics, and in particular the aspect ratio, significantly affect the strength and stiffness of the composite [12;13]. Wood flour compared to wood particles acts more as a filler, improving stiffness but not strength.

Wood filler is a widely available reinforcement which lowers the price of composite material [14]. Wood as filler is environmentally friendly due to the absence of toxic substances and its recyclability and biological degradation. Because of its renewability, this product can compete with inorganic fillers. Wood fillers are lightweight materials, which means that the composites obtained with them have lower density compared to composites obtained with inorganic fibres. They are also resistant to abrasion. The main disadvantages of wood fillers include their flammability and non-resistance to moisture [14].

Wood-plastic composite materials have better performance characteristics and properties when manufacturing complex profiles compared to the use of individual classic materials such as wood or plastic [15]. The most important factors influencing the construction and technological characteristics of WPC profiles besides the mechanical characteristics of the material, are also profile design, mass and production technology. These factors directly affect the cost of production and the possibility of using WPC materials in construction and many other industries. This gives manufacturers the opportunity to replace some conventional profiles with this composite material as it is more suitable and cheaper as well as environmentally friendly.

3. USE OF RECYCLED MATERIALS IN PRODUCTION OF WOOD-PLASTIC COMPOSITES (WPC)

Sustainability has become a major concern for buyers when purchasing or choosing products. Many people nowdays have a goal to save the planet and for this reason they increased use of green, sustainable and recyclable products, including wood products for their homes. Industrial wood manufacturers, in response to that, try to fulfill high demands and therefore invented the wood plastic composites (WPC) as an alternative to different wood products because they are made of recyclable materials such as scrap wood flour and wasted plastics.

Wood-plastic composites can be obtained from new raw materials. On the other hand, the production of such composites is possible when secondary raw materials

including recycled materials available in market are used. Recycled matrix or a reinforcement/fill can be used [8].

Products made of plastics can be recycled to obtain new materials or polymer products. Recycled plastics can be obtained from different sources subjected to a different storage and reprocessing conditions, and therefore recycled plastics can have different properties depending of their degree of degradation. Plastic waste can contain different colours and contaminants, leading to a different results when these plastics are combined with wood flour / fillers and affecting the properties of final WPC [8].

The growing amount of wood waste implies the necessity to find ways of managing with it considering the environmental aspects and the principles of sustainable waste management. It is necessary to find ways of using the production waste instead of sending it to the landfill sites. One way is to use them as reinforcement in wood-plastic composites (WPC). Wood products from woodworking plants can be easily recycled and contribute to lower greenhouse gas emissions compared to non-renewable materials [16].

In the production of wood-based composites it is possible to use wood fuel (biomass) as combustion energy. Such fuel usually consists wood residues (wood and bark) from the wood production. Use of wood as a fuel and energy source plays an important role and has a beneficial effect on the environment. Emissions of biogenic CO_2 during wood combustion often contributes to neutral effect on global warming. However, the use of petroleum-based adhesives (amino resins) for the production of wood-based composites has a significant impact on the environment [17]. The biggest problem is wood-based plastics, which in addition to wood also contain adhesives or other additives. In that case, it is not possible to thermally transform waste and obtain wood fuel because combustion will generate pollution.

Mineral wool is a waste fraction that is currently considered un-recyclable. Väntsi and Kärki, 2014 presented the mechanical and moisture resistance properties of wood plastic composites (WPC) utilizing recycled mineral wool as filler in their stady [18]. According to the findings, the addition of recycled mineral wool increased the moisture resistance properties of the composites significatly, but some mechanical properties decreased. Mineral wool is usually used as a building insulation material and makes about 60% of the total market for insulation products [18]. Therefore, it is a material fraction usually found in the waste considered difficult to recycle and is usually sent to landfill.

4. CHALLENGES IN RECYCLING WPC - REUSE OF PRODUCT

WPC can be recycled after they reach their useful service life. The thermoplastic nature of WPC makes recovering possible [19]. However, there are a few challenges when doing that. Recycling activities of the polymer in WPC shortens the useful service life of the product compared to the virgin polymer-based product and causing not only lower UV and biological resistances, but also lowers down mechanical performance [19]. Therefore, WPC manufacturers have to consider this problem when they decide on how to obtain the raw materials: whether by buying wastes from secondary wood manufacturers or recycling the existing products.

The reyclin capacity also depends of how is the WPC treated during their useful service life. WPC are mostly used for the exterior application depending of degradation of the material performance and recycling potential [19]. Winandy et al., 2004 exposed

WPC to decay on its surface area and found mycelium cluster in the interface between the wood flour and thermoplastics [19]. In the exterior, it would look like surface erosion. They also noted that WPC with higher wood content is more susceptible to this kind of decay compared to WPC with less wood content.

UV-degradation is considered as an environmental issue because the surface of WPC slowly degrades when is exposed to UV light. The oxygenated functional group in the thermoplastic can lead to further photooxidation when exposed to UV light [19].

WPC recycling has not been sufficiently researched and only a few publications are available. One limitation of WPC recycling is wood component, which begins to degrade and emit volatile substances during repeated processing [20]. Due to low thermal stability of wood, processing temperatures must be lower than the degradation temperature of wood, which is around 200 °C. This is one of the reasons why thermoplastics with a processing temperature up to 200 °C are used for the production of wood-plastic composites (WPC). Polyethylene, polypropylene and polyvinyl chloride are used as type of thermoplastics, while polyester, epoxy and phenol resins are used as duroplasts [21]. Acrylonitrile butadiene styrene (ABS plastic) and polyamides can also be used as a matrix for wood-plastic composites.

Another limitation is the degradation of the polymer. Recycling causes thermal and oxidative degradation, i.e. chain fission and molecular weight reduction [22;23]. The resulting change in viscosity may require modification of further production processes.

5. CONCLUSIONS

Wood-plastic composites (WPC) are materials with expanding applications worldwide that contribute to sustainable economic development. WPC offer a lot of advantages to the prospective consumers by being durable, easy to install, easy to maintenance, biodegradable, UV-resistant, flexible in design, can be recycled and many more. WPC products should be designed so that can be treated as a resource at the end of their useful service life, not just as waste.

WPC has biodegradation potential and potential to use recyclable materials as a raw material, e.g. wood and plastic waste. Wood-based waste can be used in production of WPC because it is renewable, relatively cheap, fully or partially recyclable and biodegradable.

Use of biomass and more efficient use of WPC by-products could improve sustainability in construction sector.

WPC can be recycled after reach their useful service life, but there are some challenges in recycling process that could affect the properties of the composites. Based on research, recycled products would be less UV-resistant compared to virgin polymer WPC. Therefore, the recycling activity could potentially reduce the useful service life of the WPC installed into the object. However, continuous exposure to direct UV-light can cause development of decay, that will reduce the aesthetic value since the photooxidation reaction would alter the color of the composites. Possibility of recycling the finished composite is also adventage of WPC.

WPC for recycling can come from postconsumer sources or from own waste generated during production. On-site recycling helps save resources and reduces production costs.

Therefore, it seems that such material may belong to the group of sustainable building materials. Wide range of their applications confirm the necessity of further research and improvement of WPC with the aim of generally increasing their use in various areas of modern life.

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Sokobanja, May 21 - 23, 2025.

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Prethodno saopštenje

VIBRATION ANALYSIS OF STEEL BEAMS WITH A WEB OPENING

Summary: *The subject of this paper is the vibration analysis of steel beams with hexagonal web openings. In addition to verifying the limit states of existing structures where new web openings are introduced, beams with web openings are also used in newly planned structures. This paper presents a comparative analysis of serviceability limit state calculation methods according to the methods provided in prEN 1993-1-13. A comparative calculation of natural frequencies of oscillation is analyzed, as this represents the main parameter for assessing structural vibrations.*
Keywords: *castellated beam, web openings, limit states, natural frequencies, vibrations.*

ANALIZA VIBRACIJA ČELIČNIH NOSAČA SA OTVORIMA U REBRU

Rezime: *Predmet ovog rada je analiza vibracija čeličnih nosača sa otvorom u rebru šestougaonog oblika. Pored dokaza graničnih stanja izvedenih konstrukcija, kod kojih se izvode novi otvori u rebru nosača, nosači sa otvorima u rebru nalaze primenu i kod novoprojektovanih konstrukcija. U radu je data uporedna analiza metoda proračuna graničnih stanja upotrebljivosti prema metodama datim u prEN 1993-1-13. Analiziran je uporedni proračun sopstvenih frekvencija oscilovanja, koji predstavlja glavni parametar za ocenu vibracija konstrukcija.*
Ključne reči: *saćasti nosači, otvori u rebru, granična stanja, sopstvena frekvencija, vibracije.*

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1. INTRODUCTION

Due to the rolling technology, hot-rolled profiles have a significantly greater web thickness compared to analogous welded profiles, as it is dictated by the manufacturing process. Thus, solid steel girders created using welded construction have lower weight and material consumption. However, solid steel girders have a somewhat higher unit price than rolled ones, which is a consequence of the greater scope of work required for their production (cutting, trimming, welding, etc.). One of the advantages of solid steel girders is the ability to optimally select the dimensions of the cross-sectional elements, thereby ensuring optimal utilization of load-bearing capacity, stability, and deformation characteristics.

In order to find the optimal solution, various types of girders have been designed, such as:

- Rolled profiles;
- Solid-steel girders;
- Girders with web openings;
- Truss girders, et cetera.

Girders with web openings play a significant role in shaping the modern concept of load-bearing structures. The openings in the web allow for easy integration of cable trays, ventilation ducts, and other routed installations. Depending on the shape of the openings, they are classified as follows:

- honeycomb girders with hexagonal openings (*castellated beams*), Figure 1a
- cellular girders with circular openings (*cellular beams*), Figure 1b
- girders with sinusoidal openings (*Angelina beams*), Figure 1c, and others.

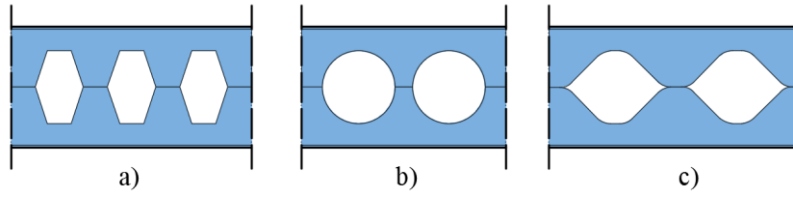


Figure 1. Structural design solutions for web opening shapes

In this paper, castellated beams with regular hexagonal openings, made from hot-rolled profiles, are analyzed. This way, "I" beams with openings in the web are obtained, with a height approximately 1.50 times greater than the height of the original "I" profile, while the weight remains unchanged. A comparison of the weight of castellated beams with equivalent welded beams shows a difference in weight of about 25% [3].

Beams with web openings are formed through a series of processes: longitudinal cutting of the profile along defined lines, separating the formed parts of the profile (T-elements) in the direction of the section height by half the height of the opening, longitudinal shifting of the separated parts in opposite directions until the adjacent axial flat parts of the cutting line come into contact, and forming welded joints along the contact lines.

The paper analyzes the results obtained using the effective stiffness method, the Vierendeel beam method, and finite element analysis (FEA). Key serviceability limit state parameters (deflections and natural oscillation frequency) were compared, and the results are presented in tabular form.

2. THEORETICAL BACKGROUND

The design of beams with web openings will be addressed in the new standard prEN 1993-1-13 [2], which is currently in draft form. This standard defines additional rules that extend the application of standards EN 1993-1-1 and EN 1993-1-5 to the design of hot-rolled and welded beams with unreinforced openings, as well as those with longitudinal and/or transverse stiffeners. The global analysis of load-bearing structures consisting of members with one or more web openings should account for the influence of openings on the flexural stiffness of the elements. For beams with openings whose maximum dimension does not exceed 50% of the section height, this influence can be neglected in the global analysis [2]. In beams that satisfy the condition $L/h \geq 18$, where the openings are regularly spaced, the reduction in shear stiffness may also be neglected in the global analysis. The effective moment of inertia for the cross-section of a beam with multiple openings may be determined as [2]:

$$I_{eff,a,o} = I_a - n_o \left(\frac{a_{eff}}{l}\right) \frac{{h_0}^3 t_w}{12} \tag{1}$$

When designing beams where the ratio $L/h < 18$, additional deformations must be considered, i.e., the increased flexibility of the beam due to significant shear effects in the area of regularly spaced circular or hexagonal openings. The effective flexural stiffness of the beam may be reduced, and the corresponding effective moment of inertia of the beam's cross-section can be determined as follows [2]:

$$I_{eff,a,o} = \frac{I_a}{1+\frac{n_0}{12}\cdot\frac{a_{eff}}{L}\cdot\frac{{h_0}^3 t_w}{I_a}+0.80\cdot n_0\cdot\left(\frac{a_{eff}}{L}\right)^3\cdot\frac{I_a}{I_{bT}\cdot I_{tT}}} \tag{2}$$

Expressions (1) and (2) use the following symbols:
I_a- moment of inertia of the cross-section in the part of the beam with a solid web;
I_{bT}- moment of inertia of the bottom T-element;
I_{tT}- moment of inertia of the top T-element;
n_o- number of evenly-spaced openings;
a_{eff}- effective length of the opening;
L- beam span;
t_w- web thickness;
h_0- height of the opening.

Advanced calculation methods can be used for the global analysis of beams with irregularly spaced openings and beams with more complex load distributions and regularly spaced openings, where the beam is modeled as a Vierendeel beam: see Figure 2.

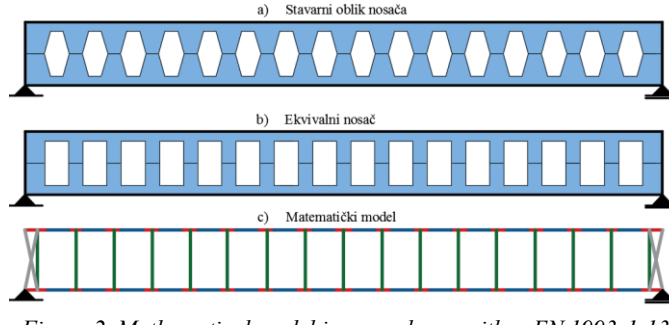


Figure 2. Mathematical model in accordance with prEN 1993-1-13

To form the model, the opening must be replaced with an equivalent rectangular shape of length $a\_eq = b_0 + 0.25h_0$ and height $h\_eq = h_0$, where a_eq is the equivalent opening length and h_eq is the equivalent opening height. The flange in the opening zone is of T-section, corresponding to the T-section of a beam with rectangular openings.

The flange in the non-reduced zone is modeled as a linear element, with characteristics corresponding to the T-section formed by the flange and half the height of the web. The verticals in the beam's web between the openings are modeled as linear elements placed along the centroidal axis, with characteristics corresponding to the cross-sectional area of the web between the equivalent rectangular openings. The web section far from the openings can be modeled as a truss with diagonal elements, where the area of these elements is 50% of the full web area. The bearing capacity of the end vertical of the beam's web at the support, or directly adjacent to the connection with another element, should be checked for the force values acting at that location.

The distribution of shear forces showed that the moments in the halves of the verticals and the flange are approximately zero, so hinges can be placed there, eliminating the internal static indeterminacy of the beam. This approach allows for understanding of the force flow and enables the calculation of axial forces in the parts of the beam above and below the openings, which are the result of the bending moments in the beam, as well as the calculation of bending moments caused by the transverse force acting in the part of the beam between the openings. The limiting dimensions are given in Table 1.

opening shape	max h_0	max a_0	max s_0	minimal height of T element	minimal height of T element in tension
hexagonal	0.75h	$1.50h_0$	$0.25h_0$	max ($t_f + r$+10; $t_f + 30\ mm$)	max ($t_f + r$+10 mm; $t_f + 30\ mm$)
h - height of the profile with web openings; ho - height of the opening; t_f - flange thickness of the hot-rolled profile; r - fillet radius of the hot-rolled profile at the web-to-flange junction					

Table 1. Limiting dimensions of openings according to prEN 1993-1-13

According to [5] and [7] it is necessary to meet the following as well:

- $d_1 = d_2 \geq 0{,}10h_w$,
- $a_o \leq h_o$,
- $b_o = s_o$,
- $0{,}25a_o \leq b_o \leq 0{,}50a_o$.

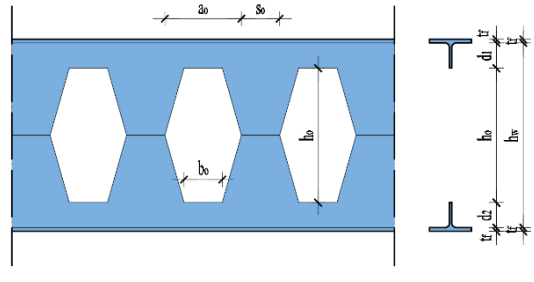


Figure 3. Basic dimensions of the honeycomb girder

Based on the provided recommendations, it can be concluded that prEN 1993-1-13 does not specify a limitation for the angle θ, nor minimum values for $b_o = s_o$. In [6], additional recommendations are given for girders with circular and rectangular openings, where a minimum web width is required. The angle limitation is provided in [1] and ranges between 45–60°, with a permissible deviation of ∓2°.

For the calculation of global bending resistance, the cross-sections of the girder must be classified at the location of the web opening and the location of the web's vertical. The classification is carried out by EN 1993-1-1, where the web at the opening is treated as a cantilevered part. The stress distribution is assumed to be uniform. In addition to the checks required by EN 1993-1-1 and EN 1993-1-5, additional checks are also necessary:

- check of global shear resistance;
- check of global bending resistance and stability of the compressed T-element;
- check of combined resistance to Vierendeel bending of the T-element, reduced due to axial force and shear force;
- check of shear resistance of the T-element due to Vierendeel bending;
- check of web buckling resistance in the area of widely-spaced openings;
- check of web buckling, shear, and bending resistance between openings;
- analysis of the effect of stiffeners on web buckling between openings;
- check of resistance due to web buckling and shear at the end of the beam with closely-spaced openings near joints;
- check of lateral-torsional buckling stability;
- check of weld resistance at the web joint of the two profiles;
- increased deformability of beams due to web openings must be taken into account during the Serviceability Limit State (SLS) check.

The rules provided in prEN 1993-1-13 apply to beams where the axial force is less than $0.02 \cdot N_o,pl,Rd$, where N_o,pl,Rd is the plastic design axial resistance of the cross-section. Beams subjected to axial compression must have Class 1 or 2 cross-sections at the location of the largest opening. Axial forces should be distributed between the T-elements in proportion to the areas of their cross-sections and combined with additional forces resulting from bending and shear when checking lateral-torsional buckling and the resistance of the T-element.

According to [6], the possible failure modes are:

- web bending;
- web buckling;
- shear buckling;
- horizontal web shear;
- vertical shear;
- overall bending resistance of the beam;
- Vierendeel bending of the T-element.

From the above, it can be observed that the standards define different failure models. The most common failure mode in beams with hexagonal openings is the Vierendeel mechanism, which occurs due to the interaction between bending moments and shear forces.

Failure occurs because the shear force induces an additional, secondary bending moment, which, in the ultimate limit state, can lead to the formation of four plastic hinges at the points of geometric discontinuity. For beams that are not subjected to axial compression, the only global stability loss is lateral-torsional buckling [4].

3. MATERIALS AND METHODS

Since the material savings are around 15–25% according to [4] and [3], the justification for producing castellated (honeycomb) profiles is questionable, as they require additional processing. However, in pedestrian bridges, parking garages, long-span intermediate floor structures, and other constructions where stricter deflection and/or vibration limits are required, and the load-bearing capacity is not the governing factor in selecting the cross-section, the application of honeycomb girders is highly relevant. For this reason, a comparative analysis of deflection and natural frequencies using the Finite Element Method (FEM) and according to prEN 1993-1-13 is presented in this paper.

The functionality of a structure or part of it can be significantly compromised due to vibrations, which may negatively affect user comfort, both in building construction and bridge engineering. In building construction, the issue of vibrations primarily relates to vertical vibrations of intermediate floor structures caused by traffic loads or human movement, while in bridge engineering, vibration issues are prominent in pedestrian and railway bridges. Eurocode does not provide explicit criteria for verifying serviceability limit states caused by vibrations in building structures. The three main parameters that influence vibrations are:

- natural frequency of the construction *(f);*
- damping *(D);*
- modal mass (M_{mod}).

The natural frequency refers to the first oscillation mode of the intermediate floor structure in the vertical plane and can be determined based on the dynamic analysis of the structure. The effect of damping primarily depends on the material used and can be considered constant for steel structures. Modal mass depends on the structure's self-weight, while frequency is a function of the system's stiffness. The natural frequency is not the only parameter affecting the acceptability of vibrations, but it is one of the key indicators. For this reason, Eurocode allows the verification of the vibration serviceability limit state for pedestrian bridges to be considered satisfied if the natural frequency is less than 5 Hz. For intermediate floor structures, EN 1990/NA provides values, as shown in Table 2. A requirement for both pedestrian bridges and intermediate floor structures of office buildings is that the natural frequency should be greater than 5 Hz.

Object purpose	Generally satisfactory	Often unsatisfactory behavior	Limit of vertical acceleration (% relative to g)
Sports halls, public spaces	$> 10H_z$	$< 6H_z$	10%
Residential buildings	$> 8H_z$	$< 5H_z$	0.10%
Business buildings	$> 8H_z$	$< 5H_z$	0.20%

Table 2. Empirical values for acceptable neutral frequency values of building structures

Large openings in the web of the girder cause additional deflections due to bending and shear, which should be calculated using the appropriate method based on the principles of elastic theory, applying expressions 1 and 2. However, approximations are suitable for standard cases, such as uniformly loaded simple beams with multiple evenly distributed circular or hexagonal openings. For other cases, the contribution of each opening to the total deflection increase should be determined using the following expression [1]:

$$\frac{w_{add}}{w_b} = 1{,}60\left(\frac{{h_o}^3 t_w}{I_a}\right)\left(1-\frac{x_0}{L}\right)\left(\frac{x_0}{L}\right)^2\left(\frac{a_{eff}}{L}\right) + 29k_o\left(\frac{I_a}{h^3 t_w}\right)\left(1-\frac{2x_0}{L}\right)\left(\frac{a_{eff}}{h-h_o}\right)^3\left(\frac{h}{L}\right)^2 \quad (3)$$

The values are summed for each opening individually. In this way, the deflections due to the combination of additional bending and Vierendeel bending are obtained.
For long openings where $a_{eff} \geq 2h_o$, the equations [1] are used:

$$w_{V_{ier,add}} = \frac{V_{ser,Ed} \cdot {a_{eff}}^3}{12E(I_{tT}+I_{bT})} \quad (4)$$

$$w_{V_{ier,max}} = \frac{a_{eff}}{200} \quad (5)$$

If the conditions for the limit dimensions of the openings are satisfied and the beam is subjected to quasi-static loading, simplified rules can be used to calculate the additional deflections using equations [1]:

$$\frac{w_{add}}{w_b} = 10n_o\left(\frac{h_o}{h}\right)^3\left(\frac{h}{L}\right)^2 \; za \; s_o \; ili \; b_o > 0{,}35h_o, \quad (6)$$

$$\frac{w_{add}}{w_b} = 3{,}50n_o\left(\frac{h_o}{h}\right)^3\left(\frac{h_o}{s_o}\right)\left(\frac{h}{L}\right)^2 \; za \; s_o \; ili \; b_o \leq 0{,}35h_o. \quad (7)$$

In expressions (3), (4), (5), (6), and (7) the following notations have been introduced:

- I_a- moment of inertia of the cross-section at the part of the beam with a full web;
- I_{bT}- moment of inertia of the lower T-element;
- I_{tT}- moment of inertia of the upper T-element;
- $V_{ser,Ed}$- shear force at the centerline of the opening for the serviceability limit state;
- w_{add}- deflection due to the combination of additional bending and shear bending;
- w_b- deflection due to pure bending at the midpoint of the beam span, determined using the moment of inertia of the cross-section at the part of the beam with a full web I_a;
- x_0- the position of the opening relative to the nearest support;
- k_o- coefficient dependent on the stiffness;
- n_o- number of evenly-distributed openings;
- a_{eff}- effective opening length;
- L- beam span
- t_w- web thickness
- h_0- height of the opening

4. RESULTS AND DISCUSSION

4.1. Results

The aim of the analysis is to provide a comparative presentation of the methods given in the standard prEN 1993-1-13, in order to assess the accuracy for calculating the serviceability limit states.

The adopted static system is a simple beam made of hot-rolled profiles of steel strength class S355. The permanent load represents a linearly uniformly distributed load with an intensity of 9 kN/m, and the live load is 20 kN/m. Spans ranging from 11 to 20 meters were analyzed, with a step of 0.50 meters. To carry out the analysis, four groups of models were analyzed.

Group 1 includes models for which, under the given load, an optimal selection of full hot-rolled profiles was made to satisfy both the ultimate and serviceability limit states. The elements were modeled using linear (beam) elements, and the design was carried out by EN 1993-1-1.

Groups 2, 3, and 4 represent models in which cellular beams are modeled in different ways. Group 2 includes models where the elements are modeled using shell elements with the actual shape of the openings. Based on the conducted analysis, the geometry of the cellular beams was adopted.

Group 3 includes models in which cellular beams are modeled using a single beam element with an effective moment of inertia, to assess the applicability of the effective moment of inertia method in the global analysis of structures. The effective moment of inertia was determined according to expression (2).

Group 4 represents models where cellular beams are modeled as Vierendeel beams using linear elements with appropriate geometric characteristics of the cross-section. The results of the deflection and natural frequency analyses for Groups 2, 3, and 4 are presented in Table 3.

Range	Group 2		Group 3		Group 4	
L	Deflection	Natural Frequency	Deflection	Natural Frequency	Deflection	Natural Frequency
[m]	[mm]	[Hz]	[mm]	[Hz]	[mm]	[Hz]
20	37.0	5.32	29.33	5.88	43.63	4.78
19.50	32.4	5.68	26.84	6.14	48.10	4.55
19	41.1	5.03	33.62	5.49	47.33	4.59
18.50	38.70	5.19	30.87	5.73	57.30	4.17
18	34.70	5.48	27.64	6.06	51.77	4.38
17.50	31.5	5.76	24.82	6.39	50.26	4.45
17	38.8	5.19	32.92	5.55	34.51	5.38
16.50	34.4	5.51	29.20	5.90	30.79	5.69
16	30.70	5.83	26.06	6.24	29.08	5.86
15.5	27.2	6.20	22.95	6.65	25.74	6.22
15	38.1	5.23	29.93	5.82	40.35	4.98
14.5	34	5.55	26.39	6.20	38.22	5.11
14	29.7	5.93	22.93	6.65	33.50	5.46
13.5	34.8	5.48	28.16	6.0	30.43	5.73
13	40.6	5.07	33.76	5.48	40.72	4.96
12.5	36.6	5.34	29.60	5.85	28.02	5.97

12	36.5	5.35	29.99	5.81	27.69	6.00
11.50	37.8	5.26	31.21	5.70	28.27	5.95
11	35.8	5.40	30.03	5.81	30.97	5.68

Table 3. Presentation of deflection and natural frequencies

4.2. Discussion

The effective stiffness method was used to perform the global analysis. The simplicity of applying this method represents an advantage of this analysis, which is particularly applicable for everyday practical calculations. From the analysis of the obtained results shown in Table 3, the following was concluded: The average deviation of the results in the deflection analysis is 18.5%, while the maximum deviation is 22.79%. The results for natural vibration frequencies show somewhat smaller deviations. The average deviation is 9.20%, while the maximum deviation is 12.14%. It is important to note that the effective stiffness method results in greater element stiffness, which leads to lower deflection values than the actual ones. From the attached results, it can be seen that the deviations are approximately constant across all analyzed spans.

By applying the method of the Vierendeel girder, the average deviations are significantly smaller than those obtained using the effective stiffness method, amounting to 7.3% for deflections and 3.66% for natural vibration frequencies. However, the application of the Vierendeel girder method resulted in significant deviations in some individual results, reaching up to 59.55%.

5. CONCLUSIONS

Based on the provided information, it can be concluded that the effective stiffness method is more suitable for use when assessing serviceability limit states, but the accuracy of the method should be considered during its application.

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

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Stručni rad

IZRADA DEKLARACIJA PROIZVODA O ZAŠTITI ŽIVOTNE SREDINE (EPD) ZA GRAĐEVINSKE PROIZVODE OD GLINE U SKLADU SA EN 15804

Rezime: *Deklaracije proizvoda o zaštiti životne sredine (Environmental Product Declarations-EPDs) za blokove od gline i crepove su od suštinske važnosti za postizanje održivosti u građevinskoj industriji. Ocenjivanje životnog ciklusa (Life Cycle Assessment-LCA) ovih proizvoda obuhvata sve faze njihovog životnog ciklusa, pružajući informacije o njihovom uticaju na životnu sredinu. Ispunjavanje zahteva i smernica standarda EN 15804, opštih programskih uputstava (General Programme Instructions-GPI) za Međunarodni EPD Sistem (International EPD System) i pravila za kategorizaciju proizvoda (Product Category Rules-PCR) za opekarske proizvode osigurava validnost i uporedivost EPD deklaracija. Upotrebom naprednih alata u specijalizovanom programu kao što je „One Click LCA", deo platforme EPD Hub (Irska), unapređuje se proces kreiranja EPD deklaracija za građevinske proizvode od gline.*
Ključne reči: *Deklaracije proizvoda o zaštiti životne sredine (EPDs), blokovi od gline, crepovi od gline, ocenjivanje životnog ciklusa proizvoda (LCA), održiva gradnja, EN 15804.*

CREATING ENVIRONMENTAL PRODUCT DECLARATIONS (EPDs) FOR CLAY CONSTRUCTION PRODUCTS IN ACCORDANCE WITH EN 15804

Summary: *Environmental Product Declarations (EPD) for clay blocks and roof tiles are essential for achieving sustainability in the construction industry. The Life Cycle Assessment (LCA) of these products includes all stages of their life cycle, providing information on their impact on the environment. Meeting the requirements and guidelines of the EN 15804 standard, the General Program Instructions (GPI) for the International EPD System and the Product Category Rules (PCR) for clay construction products ensures the validity and comparability of the EPD declarations. By using advanced tools in a specialized software such as "One Click LCA", part of the EPD Hub (Ireland) platform, the process of creating EPD declarations for clay construction products is improved.*
Key words: *Environmental Product Declatations (EPDs), Clay blocks, Clay roof tiles, Life Cycle Assessment (LCA), Sustainable construction, EN 15804.*

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1. UVOD

Deklaracija proizvoda o zaštiti životne sredine (*Environmental Product Declaration-EPD*) je standardizovani način kvantifikacije uticaja različitih proizvoda ili sistema na životnu sredinu. EPD dokument se kreira i verifikuje u skladu sa međunarodnim standardima ISO 14025 i EN 15804 [1,5].

Evropski standard EN 15804 sadrži osnovna pravila kategorizacije proizvoda za sve vrste građevinskih proizvoda i usluga. Cilj ovog standarda je da obezbedi da su sve Deklaracije proizvoda o zaštiti životne sredine (EPD) koje se odnose na građevinske proizvode, usluge i kompletne procese u građevinarstvu budu verifikovane, utvrđene i predstavljene na jedinstven način.

Ocenjivanje životnog ciklusa (*Life Cycle Assessment-LCA*) analizira sve ili više faza životnog ciklusa građevinskog proizvoda, uzima u obzir različite uticaje tih faza na životnu sredinu, ocenjuje, analizira i tumači rezultate u skladu sa standardima ISO 14040 i ISO 14044.

2. LCA METODOLOGIJA

Prema standardu ISO 14040, ocenjivanje životnog ciklusa svih proizvoda, uključujući i građevinske proizvode od gline se vrši u četiri (4) uzastopne i međuzavisne faze: (1) Faza definisanja ciljeva, oblasti primene, cilja i obima LCA; (2) Faza analize inventara životnog ciklusa (LCI); (3) Faza ocenjivanja životnog ciklusa (LCIA); (4) Faza interpretacije LCA rezultata [2].

Uobičajene metodologije za ocenjivanje životnog ciklusa proizvoda (LCIA) koje se koriste u EPD:

– *CML metodologiju* je razvilo CML – Odeljenje za industrijsku ekologiju Univerziteta u Lajdenu, Holandija;

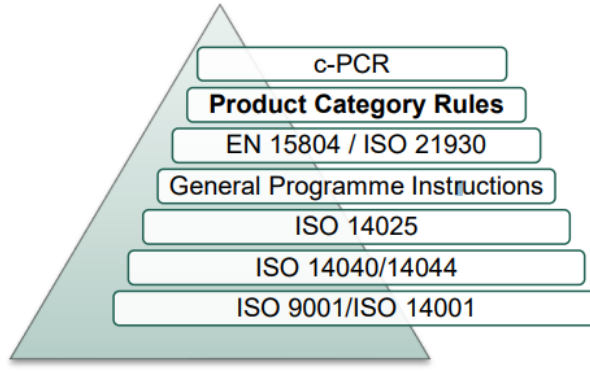
– *TRACI (TREJSI)* – alat za procenu uticaja na životnu sredinu razvijen je da predstavlja uslove u SAD, a to je u skladu sa politikom EPA (Environmental Protection Agency);

– *PEF* – (*Product Environemntal Footprint*) Evropska komisija je 2021.godine predložila otisak proizvoda na životnu sredinu (PEF) kao uobičajen način merenja ekološkog učinka. Alati, dokumenti i paketi koji se odnose na šemu uticaja na životnu sredinu su obezbeđeni u Evropskoj platformi na LCA|EPLCA.

3. USPOSTAVLJANJE PRAVILA O KATEGORIJI PROIZVODA (PCR)

Prilikom kreiranja EPD dokumenta neophodno je uspostaviti pravila o kategoriji proizvoda (*Product Category Rules-PCR*). PCR je grupa pravila, zahteva i smernica na osnovu kojih se formira Deklaracija proizvoda o zaštiti životne sredine (EPD). Pravila o kategoriji proizvoda (PCR) se koriste kao dopuna Opštim programskim uputstvima (*General Programme Instructions-GPI*), npr. u pogledu pravila proračuna, scenarija izrade i sadržaja EPD deklaracije. Oni obezbeđuju da se funkcionalno slični proizvodi procenjuju na isti način prilikom sprovođenja LCA i za poređenje proizvoda. Predstavljaju ključni deo standarda ISO 14025 jer omogućavaju transparentnost i

uporedivost između EPD deklaracija. Na Slici 1. je dat grafički prikaz relevantnih standarda za uspostavljanje PCR proizvoda [1,6].



Slika 1. Relevantni standardi za uspostavljanje PCR proizvoda, izvor: General Programme Instructions (GPI) version 5.0.1

4. EPD STANDARDI I PRAVILA

EPD obično važi pet godina i generiše se u skladu sa relevantnim standardima. Građevinski EPD su zasnovani na standardima ISO 14040/14044, ISO 14025, EN 15804 ili ISO 21930.U Tabeli 1. su prikazani opšti i specifični standardi koji se koriste za kreiranje LCA i EPD za građevinske proizvode.

Vrsta	Standard/pravilo	Upotreba
Opšti	ISO 14040/44	Standard za LCA studije
	ISO 14025	Standard za EPD
Specifični	ISO 21930	Standard za građevinski sektor
	EN 15804	Pravila kategorije proizvoda (PCR) za evropski građevinski sektor
	C-PCR	Posebna pravila za kategoriju proizvoda u evropskom građevinskom sektoru

Tabela 1. Opšti i specifični standardi i pravila koji se koriste za kreiranje LCA i EPD za građevinske proizvode u Evropi, izvor: Ecochain, https://ecochain.com/

Referentni standardi za izradu EPD dokumenta za građevinske proizvode koji se koriste na nacionalnom nivou su:

- SRPS ISO 14025: Oznake i deklaracije o zaštiti životne sredine - Deklaracija zaštiti životne sredine tipa III - Principi i procedure;
- SRPS EN 15804: Održivost objekata - Deklaracije proizvoda o zaštiti životne sredine - Osnovna pravila za kategorizaciju građevinskih proizvoda;
- SRPS ISO 14040: Upravljanje zaštitom životne sredine - Ocenjivanje životnog ciklusa - Principi i okvir;

4.1. LCA građevinskih proizvoda od gline

LCA studije su sveobuhvatne ocene uticaja životnog ciklusa proizvoda na životnu sredinu i sprovode se u skladu sa standardima ISO 14040/44. EPD je dokument koji transparentno izveštava o uticaju proizvoda ili materijala na životnu sredinu, na osnovu ocene životnog ciklusa proizvoda [2,3].

LCA identifikuje tokove materijala, energije i otpada proizvoda tokom celog životnog ciklusa tako da se uticaji na životnu sredinu mogu odrediti na standardizovan način. U EPD deklaracijama za građevinske proizvode od gline se ocenjuje uticaj životnog ciklusa proizvoda (*Life cycle assessment-LCA*) na vazduh, vodu i zemljište, preko informacionih modula (A,B,C,D):

- (A1 - A3) - faza proizvodnje;
- (A4 - A5) - faza ugradnje proizvoda u objekat;
- (B1 - B7) - faza upotrebe;
- (C1 - C4) - kraj životnog ciklusa;
- D - faza u kojoj je opisana prednost ponovne upotrebe otpada i recikliranja.

Važno je napomenuti da svi ulazni podaci (potrošnja sirovina, energije i vode) i izlazni podaci (emisije u vazduh, vodu, zemljište, stvaranje otpada) moraju biti iz istog vremenskog perioda da bi bili uporedivi. Na Slici 2. su prikazane faze i moduli životnog ciklusa građevinskih proizvoda od gline prema PCR za građevinske proizvode i građevinske usluge - podkategorija - PCR-D Cigle, blokovi, crepovi, tvrde ploče na bazi gline i silikatne zemlje [6,7].

Faze i moduli životnog ciklusa (X = uključeni u LCA; MND = modul nije deklarisan; MNR = modul nije relevantan)																	
Faza proizvodnje			Faza ugradnje		Faza upotrebe/korišćenja							Faza kraja životnog ciklusa					
Snabdevanje sirovinama	Transport	Proizvodnja	Transport	Proces ugradnje	Upotreba	Održavanje	Popravka	Zamena	Obnavljanje	Snabdevanje potrebnom energijom	Snabdevanje vodom	Demontiranje,rušenje	Transport	Prerada otpada	Odlaganje	Mogućnost ponovne upotrebe, rekuperacije, i/ili recikliranja	
A1	A2	A3	A4	A5	B1	B2	B3	B4	B5	B6	B7	C1	C2	C3	C4	D	
X	X	X	X	X	MNR	MNR	MNR	MNR	MNR	MNR	MNR	MNR	X	X	X	X	

Slika 2. Faze i moduli životnog ciklusa građevinskih proizvoda od gline

4.2. EPD prema revidiranom standardu EN 15804+A2:2019

Glavni ISO standard za održivost u građevinskom sektoru je ISO 21930. Ovaj ISO je više u skladu sa EN 15804 od 2017. godine i pokazuje važnost EPD deklaracija u održivoj budućnosti građevinskog tržišta.

Rezultati ocenjivanja uticaja životnog ciklusa građevinskih proizvoda na životnu sredinu (LCIA) predstavljeni su u tri (3) različite kategorije: (1) Uticaji na životnu sredinu; (2) Korišćenje resursa; (3) Stvaranje otpada.

Standard EN 15804+A2 je najčešće korišćeni EPD standard. Pomenuti standard je do jula 2022.godine zahtevao izveštavanje o šest (6) kategorija uticaja na životnu sredinu u EPD dokumentima [4,5].

Indikatori uticaja na životnu sredinu do 2022.godine prema EN 15804:2012+A1:2013 [4]: (1) Potencijal globalnog zagrevanja (GWP), (2) Zakišeljavanje (AP); (3) Eutrofikacija (EP); (4) Stratosferski potencijal oštećenja ozona (ODP); (5) Fotohemijski potencijal stvaranja ozona (POCP); (6) Abiotsko iscrpljivanje: uključujući oskudne hemijske elemente (ADPE) i fosilna goriva (ADPF).

Izmenjeni standard EN 15804:2012+A2:2019 sada zahteva izveštavanje o trinaest (13) kategorija uticaja sa šest (6) dodatnih kategorija uticaja na životnu sredinu. U Tabeli 2. su prikazani indikatori koji opisuju korišćenje resursa koji će biti uključeni u svaki modul deklarisan u EPD, dok su u Tabeli 3. prikazani dodatni indikatori uticaja na životnu sredinu a koji se odnose na zdravlje ljudi [5].

Parametar	Jedinica
Korišćenje obnovljivih primarnih energetskih izvora kao energije	[MJ]
Korišćenje obnovljivih primarnih energetskih resursa kao sirovina	[MJ]
Ukupna upotreba obnovljivih primarnih energetskih resursa	[MJ]
Korišćenje neobnovljive primarne energije isključujući neobnovljive izvore primarne energije koji se koriste kao sirovine	[MJ]
Korišćenje neobnovljivih primarnih energetskih resursa koji se koriste kao sirovine	[MJ]
Ukupna upotreba neobnovljive primarne energije	[MJ]
Upotreba sekundarnih materijala	[kg]
Korišćenje obnovljivih sekundarnih goriva	[MJ]
Korišćenje neobnovljivih sekundarnih goriva	[MJ]
Upotreba sveže vode	[m^3]

Tabela 2. Parametri koji opisuju korišćenje resursa, izvor:EN 15804:2012+A2:2019

Kategorija uticaja	Indikator	Jedinica
Učestalost emisija čestica	Potencijalna incidenca bolesti zbog emisije PM (PM)	-
Jonizujuće zračenje, zdravlje ljudi	Potencijalna efikasnost izlaganja ljudi u odnosu na U235 (IRP)	[kBq U-235eq]
Eko toksičnost (sveža voda)	Potencijalna uporedna toksična jedinica za ekosisteme (ETP-fw)	[CTUe]
Toksičnost za ljude, efekti kancera	Potencijalna komparativna toksična jedinica za ljude (HTP-c)	[CTUh]
Ljudska toksičnost, ne-kancerogena dejstva	Potencijalna komparativna toksična jedinica za ljude (HTP-nc)	[CTUh]
Uticaji vezani za korišćenje zemljišta/kvalitet zemljišta	Indeks potencijalnog kvaliteta zemljišta (SQP)	-

Tabela 3. Dodatni indikatori uticaja na životnu sredinu, izvor:EN 15804:2012+A2:2019

Jedna od najvećih promena u EN 15804+A2 tiče se biogenog ugljenika u svim oblicima. Prema revidiranom EN 15804+A2 standardu pet (5) indikatora potencijala globalnog zagrevanja (GWP) je proglašeno: (1) GWP-fosil; (2) GWP-biogeni; (3) GWP-luluc (korišćenje zemljišta i promena namene zemljišta); (4) GWP-ukupni (zbir

prethodna tri GWP indikatora); (5) GWP-GHG (jednak GWP-ukupnom osim što je faktor karakterizacije za biogeni CO_2 postavljen na nulu) [5].

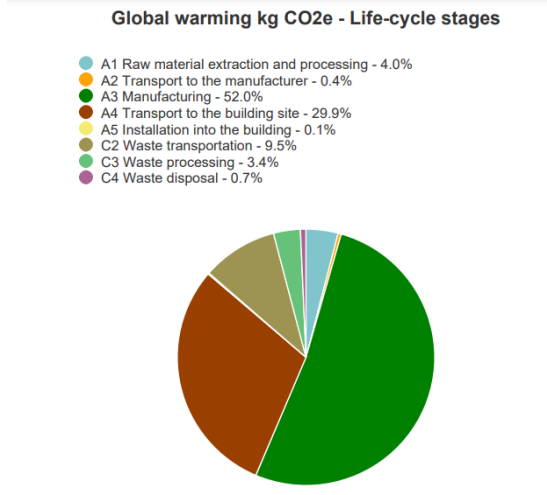
Bitno je naglasiti da EPD kreirani prema standardu EN 15804+A1 neće biti uporedivi sa EPD deklaracijama koje su kreirane prema revidiranom EN 15804+A2 standardu zbog promene načina izračunavanja indikatora. Na Slici 3. je dat prikaz rezultata na životnu sredinu po fazama životnog ciklusa za 1 tonu crepova od gline prema revidiranom standardu EN 15804:2012+A2:2019 sa povećanim brojem kategorija uticaja.



Slika 3. Rezultati uticaja na životnu sredinu po fazama životnog ciklusa za 1 tonu crepova od gline, izvor: EPD-IMS-2023-03-EN, Institut IMS a.d. Beograd, 2023.godine

4.3. Ugljenični otisak proizvoda (CFP)

Ugljenični otisak proizvoda (*Carbon Footprint of Products-CFP*) se definiše kao emisija gasova staklene bašte (GHG) proizvoda kroz faze njegovog životnog ciklusa, uključujući nabavku materijala, proizvodni proces, distribuciju, upotrebu i upravljanje otpadom na kraju njegovog životnog veka, kao i relevantan transport u svakoj fazi proizvoda. Na Slici 4. je prikazan (GWP) po fazama životnog ciklusa za 1 tonu blokova od gline [10].



Slika 4. GWP po fazama životnog ciklusa za 1 tonu blokova od gline, izvor: EPD-IMS-2022-01-EN, Institut IMS a.d. Beograd, 2022.godine

4.4. Tip EPD deklaracije u odnosu na LCA proizvoda

Na osnovu uključivanja ili isključivanja faza životnog ciklusa proizvoda, razlikujemo pet (5) različitih tipova EPD, i to [5]:

od kolevke do kapije sa modulima C1–C4 i modulom D (A1–A3, C i D). Ove faze su minimalne da se deklarišu za podrazumevani tip EPD-a. Oni će biti zasnovani na deklarisanoj jedinici;

- *od kolevke do kapije sa opcijama, moduli C1–C4 i moduli D* (A1–A3, C i D) i dodatni moduli. Dodatni moduli mogu biti A4 i/ili A5 i/ili B1–B7). Ova vrsta EPD će se zasnivati na funkcionalnoj jedinici ili deklarisanoj jedinici. Ako B-moduli i scenariji upotrebe nisu deklarisani, EPD će biti na osnovu deklarisane jedinice;
- *od kolevke do groba i modul D (A, B, C i D).* Ova deklaracija će se zasnivati na funkcionalnoj jedinici ili deklarisanoj jedinici;
- *od kolevke do kapije (A1–A3).* Ove faze su minimum koji treba deklarisati za sve građevinske proizvode koji su izuzeti od deklarisanja modula C i D i zasnivaju se na deklarisanoj jedinici. Ova vrsta EPD nije dozvoljen za proizvode koji sadrže biogeni ugljenik;
- *od kolevke do kapije sa opcijama (A1–A3) i dodatni moduli*. Dodatni moduli mogu biti A4 i/ili A5). Ova vrsta EPD je moguća samo za građevinske proizvode koji su izuzeti deklarisanja modula C i D. Ovaj tip EPD će biti zasnovan na funkcionalnoj ili deklarisanoj jedinici. Ova vrsta EPD nije dozvoljen za proizvode koji sadrže biogeni ugljenik;

4.5. Specijalizovani LCA softveri za kreiranje EPD

LCA softveri su alati koji se koristi za efikasnije i efektivnije sprovođenje ocenjivanja životnog ciklusa proizvoda (LCA). LCA softveri omogućavaju da se pojednostavi složen zadatak prikupljanja i analize podataka tokom celog životnog ciklusa proizvoda.

Najčešće korišćeni LCA softveri za kreiranje EPD su [izvor: https://lca-software.org/]:

- Sphera (former GaBi), Sphera Solutions, Inc., Chicago, USA;
- Sima Pro, PRé Sustainability, Holandija
- Umberto, iPoint-systems gmbh, Nemačka;
- Mobius, Ecochain Technologies B.V., Holandija;
- One Click LCA, One Click Lca Ltd, Finska;
- Open LCA, GreenDelta, Nemačka;

Institut IMS a.d. Beograd pruža usluge izrade EPD deklaracija tipa III za građevinske proizvode od 2019. godine u skladu sa standardima ISO 14025 i EN 15804. U procesu izrade EPD dokumenata, Institut IMS koristi „One Click LCA“, specijalizovan program razvijen od strane kompanije One Click LCA Ltd, Finska, kao i Ecoinvent bazu podataka, koju je razvila Ecoinvent Asocijacija, Švajcarska. Za unošenje podataka u program „*One Click LCA*“ se koriste specijalizovani alati (*EPD on-line tools*). Licencirana baza podataka Ecoinvent se stalno ažurira u skladu sa zahtevima međunarodnih standarda [8].

4.6 . Verifikacija EPD

U skladu sa zahtevima standarda EN 15804, eksternu verifikaciju EPD moraju uraditi nezavisna stručna lica ili ovlašćene institucije koje su kompetentne za datu oblast i nisu učestvovale u izradi predmetnog EPD.

EPD je validan pet (5) godina od datuma izdavanja, nakon čega se ponovo pregleda i verifikuje, osim u slučaju da je tehnologija proizvodnje značajno izmenjena. Postoje dve (2) vrste nezavisnih verifikatora koji mogu da sprovedu ovaj proces: (1) Akreditovana tela za sertifikaciju i (2) Odobreni pojedinačni verifikatori [1,5].

Po završetku procesa verifikacije, EPD deklaracija se sastoji od tri (3) ključna dokumenta: (1) Pozadinski LCA izveštaj, sistematski i sveobuhvatni rezime LCA projekta za podršku verifikatoru treće strane prilikom verifikacije EPD-a. Ovaj dokument je poverljiv; (2) Izveštaj o verifikaciji potpisan od strane eksternog verifikatora; (3) Javni EPD dokument koji pruža LCA rezultate i drugi EPD sadržaj.

4.7. ECO platforma i EPD operateri programa

ECO platforma je krovna organizacija različitih nacionalnih operatera EPD programa u Evropi i podržava stvaranje Evropskog jezgra EPD sistema zasnovanog na EN 15804 evropskom standardu. Osnovana je 2013. godine od strane operatera EPD programa, kao i članova podrške iz industrije, organizacija zelene gradnje, LCA praktičara i operatera za korišćenje specijalizovanih softverskih alata. Trenutno ima 112 članova uključujući osnivače. Ostali članovi ECO platforme su: LCA konsultanti, verifikatori, dobavljači softvera i softverskih alata, proizvođači i drugi akteri. Uključeni

evropski programski operateri su dogovorili određene minimalne standarde u vezi sa upravljanjem kvalitetom i procedurom verifikacije, na čije poštovanje su se obavezali [13].

Međunarodni EPD sistem, član ECO platforme je globalni program operater za deklaracije proizvoda o životnoj sredini zasnovan na ISO 14025 i EN 15804. Kompanija EPD International AB iz Švedske je programski operater i ima ukupnu odgovornost za upravljanje i rad Međunarodnog EPD sistema. Baza podataka Međunarodnog EPD sistema trenutno sadrži više od 10.000 EPD deklaracija za širok spektar kategorija proizvoda od strane organizacija u 45 zemalja [11].

4.8. Budućnost EPD u vezi sa Uredbom o građevinskim proizvodima (CPR)

Uredba o građevinskim proizvodima (*Construction Products Regulation-CPR*) postavlja standardizovane zahteve za bezbednost, performanse i uticaj na životnu sredinu za građevinske proizvode širom EU. Buduće izmene Uredbe o građevinskim proizvodima (CPR) naglašavaju (eventualnu) obaveznu integraciju ekoloških deklaracija, uključujući EPD, u Deklaraciju o performansama (DoP) za građevinske proizvode u Evropi [14].

Ova integracija će početi sredinom 2026. godine i počeće sa indikatorom potencijala globalnog zagrevanja (GWP) iz EN 15804+A2 PCR. Istovremeno, sve grupe građevinskih proizvoda moraju da pregledaju svoje harmonizovane standarde putem zahteva za standardizaciju. Kada se finalizira, ove informacije moraju biti unešene u Deklaraciju o performansama (DoP)[14].

Do 2028. godine, ove informacije o građevinskim proizvodima biće dodate u Digitalni pasoš proizvoda (DPP). Generalno, ova uredba ima za cilj da isporuči održivije (građevinske) proizvode na evropska tržišta.

5. ZAKLJUČAK

Na osnovu svega navedenog možemo izvesti sledeće zaključke:

– Uporedivost EPD za građevinske proizvode je moguća samo u okviru iste grupe proizvoda i u okviru istog programskog operatera.

– EPD izdati prema stanadardu EN 15804+A1:2013 se ne mogu upoređivati sa EPD izdatim prema revidiranom EN 15804+A2:2019 stanadardu zbog povećanog broja indikatora uticaja na životnu sredinu, pogotovo u odnosu na potencijal globalnog zagrevanja (GWP).

– Uporedivost EPD za građevinske proizvode od gline je uslovljena i izborom specijalizovanog softvera u kojem se modeluje LCA proizvoda, kao i odabirom specifičnih ili generisanih podataka iz dostupnih baza podataka za svaku fazu životnog ciklusa proizvoda.

– Odabir programskog operatera, kao i specijalizovanog softvera sa naprednim alatima za modelovanje LCA proizvoda, unapređuje se proces kreiranja EPD deklaracija za građevinske proizvode od gline u cilju postizanja održivosti u građevinskoj industriji.

Zahvalnost:

Zahvaljujemo se Ministarstvu nauke, tehnološkog razvoja i inovacija Republike Srbije na pomoći i podršci koju nam pruža za sprovođenje istraživačkog rada (Ugovor br. 451-03-136/2025-03/200012).

6. REFERENCE

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[4] SRPS EN 15804:2020 (identičan sa EN 15804:2012+A1:2013) - Održivost objekata - Deklaracije proizvoda o zaštiti životne sredine - Osnovna pravila za kategorizaciju građevinskih proizvoda.

[5] SRPS EN 15804:2020 (identičan sa EN 15804:2012+A2:2019) - Održivost objekata - Deklaracije proizvoda o zaštiti životne sredine - Osnovna pravila za kategorizaciju građevinskih proizvoda.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

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AN EXPERIMENTAL STUDY ON THE INFLUENCE OF GGBS CONTENT AND GRINDING AIDS ON CEMENT PERFORMANCE

Summary: *The construction industry significantly contributes to global greenhouse gas emissions, with cement production responsible for 80–90% of this impact. To reduce emissions, the use of SCMs, such as ground granulated blast-furnace slag (GGBS), has gained attention due to its environmental benefits and enhanced durability. However, increasing GGBS content affects properties like setting time and strength development, and challenges grinding efficiency due to GGBS's harder texture. This paper investigates the effects of increasing GGBS content on cement performance and explores the role of grinding aids in improving production capacity and energy efficiency, aiming to optimize GGBS use without compromising cement quality.*
Key words: *Sustainable concrete, green cements, grinding aids, slag.*

EKSPERIMENTALNO ISPITIVANJE UTICAJA SADRŽAJA ZGURE IZ VISOKIH PEĆI I ADITIVA ZA MLEVENJE NA PERFORMANSE CEMENTA

Rezime: *Građevinska industrija značajno doprinosi globalnoj emisiji stakleničkih gasova, pri čemu je proizvodnja cementa odgovorna za 80–90% uticaja. Da bi se smanjile emisije, povećana je upotreba dodatka cementa (SCM), kao što je mlevena granulirana zgura iz visokih peći (GGBS), glavno zbog benefita na životnu sredinu i povećane trajnosti betona. Istovremeno, povećani sadržaj zgure utiče na vreme vezivanja i razvoja čvrstoće cementa, i otežava efikasnost mlevenja zbog tvrdoće zgure. Ovaj rad istražuje efekte povećanja sadržaja zgure na performanse cementa, kao i ulogu aditiva za mlevenje cementa na povećanje produktivnosti i energetske efikasnosti proizvodnje, sa ciljem da se optimizira sadržaj zgure bez da se smanji kvalitet cementa.*
Ključne reči: *Održljivi beton, zeleni cementi, aditivi za cement, zgura*

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1. INTRODUCTION

The construction industry contributes roughly 11% of global greenhouse gas (GHG) emissions [12], with an estimated 80–90% of this impact originating from cement production [9]. In response to this, efforts to reduce clinker content in cement have intensified, driven by the need to mitigate the carbon footprint of cement and concrete production. This push for more sustainable construction materials has led to increased interest in incorporating supplementary cementitious materials (SCMs) into cement formulations.

The hydraulic properties of ground granulated blast-furnace slag (GGBS) were first recognized in 1862 by Emil Langen in Germany. By 1865, commercial production of lime-activated GGBS had begun, and around 1880, it was first combined with Portland cement (PC) [8]. Since then, GGBS has become one of the most widely used SCMs, offering environmental benefits such as reduced CO_2 emissions and improved long-term durability.

Increasing the GGBS content in cement can significantly influence a wide range of properties, including setting time, strength development, and overall durability. However, one of the major challenges in producing high-slag cements lies in the grinding process. GGBS is generally harder to grind than clinker, which can negatively impact mill efficiency, reduce overall production capacity, and complicate the manufacturing process [8]. To address these challenges, the use of grinding aids has emerged as a viable strategy to improve the grindability of GGBS and maintain production efficiency.

Grinding aids, or grinding additives, are substances introduced into the mill during grinding that enhance the rate of size reduction. They work by improving particle dispersion and reducing agglomeration [10]. These additives typically function by adsorbing onto the surface of cement particles, reducing inter-particle attraction, and minimizing agglomeration [1]. This results in more efficient grinding, lower energy consumption, and improved particle size distribution, especially in systems involving hard-to-grind materials like GGBS. Furthermore, grinding aids can contribute to the enhanced overall performance of the final cement product, improving critical characteristics such as strength development and setting time.

The primary objective of this paper is to investigate the effects of progressively increasing GGBS content on key cement properties, including setting time and strength development. The secondary objective is to assess the effectiveness of ADING's grinding aids in improving cement production capacity while simultaneously reducing energy consumption during the manufacturing process. The results aim to provide valuable insights into optimizing slag utilization, thereby enhancing the sustainability of cement production, all while ensuring that performance standards are not compromised.

2. EXPERIMENTAL WORK

2.1. Materials and methods

For the preparation of the cement samples in this experiment, ordinary Portland cement (OPC) clinker was used, with a typical mineralogical composition of approximately 61.3% tricalcium silicate (C_3S), 14.7% dicalcium silicate (C_2S), 6.8% tricalcium aluminate (C_3A), and 7.9% tetracalcium aluminoferrite (C_4AF). Ground

granulated blast-furnace slag (GGBS) was incorporated as a supplementary cementitious material, and gypsum was added to regulate the setting time. Two different grinding aids, designated GA1 and GA2, were used during the grinding process. Both additives were supplied by ADING AD Skopje.

The raw materials—clinker, fly ash, limestone, and gypsum—were first prepared and sieved to ensure uniform granulometric composition, with a maximum particle size of 8 mm. Milling was conducted using a laboratory ball mill (DRUM MILL TYPE TTS50 - Siebtechnik GmbH) according to the regimen:

- 5kg of raw materials are loaded into the mill,
- The sample is ground for 30 minutes at a speed of 45 rotations per minute,
- The sample is removed from the mill and is ready for testing.

The composition of the cement samples used in the experiment is presented in Table 1.

	Ref.	C1/1	C2/1	C3/1	C4/1	C1/2	C2/2	C3/2	C4/2
Clincer [%]	75	73	71	69	67	73	71	69	67
Slag [%]	20	22	24	26	28	22	24	26	28
Gypsum [%]	5	5	5	5	5	5	5	5	5
GA1 [g/t]	/	300	400	500	600	/	/	/	/
GA2 [g/t]	/	/	/	/	/	300	400	500	600

Table 1. Composition of cement samples

The first sample served as a reference and was prepared without the use of any grinding aid. Subsequent samples were produced with a gradual increase in slag content, incremented by 2% for each composition. Each of these samples with increased slag content was prepared in parallel using the two different grinding aids, GA1 and GA2, to evaluate their respective effects on grinding efficiency and cement performance.

To evaluate the grindability of the cement produced under laboratory conditions, particle size distribution (PSD) was selected as the primary assessment parameter. PSD was tested using an air jet sieve following the method defined in EN 196-6 [2].

All of the samples were tested to determine the standard consistency and setting time following the method defined in EN 196-3 [3].

The samples were also tested to assess the effects of increasing slag content and the use of grinding aids on the mechanical properties of the cement. The mechanical properties were evaluated on both mortar and concrete specimens.

The mortar specimens were prepared and tested according to the procedures outlined in EN 196-1:2016 [4], which specifies the methods for determining the mechanical properties of cement-based mortars. These tests provided valuable data on the influence of slag content and grinding aids on the performance of the cement.

For the concrete specimens, the mix composition is presented in Table 2. The workability of the concrete specimens was evaluated in accordance with EN 12350-2:2009 [5], which provides the standardized method for conducting the slump test, a widely used procedure for determining the consistency and flowability of fresh concrete. In addition, the air content of the concrete was determined using an air entrainment meter according to EN 12350-7:2019 [6].

Constituent	kg/m^3
Cement	400
Sand 0-4mm	891
Agregate 4-8mm	284
Agregate 8-16mm	637
Water	172
Superfluid 21M EKO	2

Table 2. Concrete mix

The mechanical properties of the concrete specimens were determined following the guidelines outlined in EN 12390-3:2019 [7], which provides the procedure for testing hardened concrete to evaluate its compressive strength at specified curing ages. These tests were crucial for understanding how the addition of slag and the use of grinding aids affected the overall performance of the concrete, ensuring that the samples met the required strength standards.

2.2. Results and discussion

2.2.1. Cement and mortar specimens

The results from the Particle Size Distribution (PSD) tests are presented in Table 3.

Sieve size		0.90	0.45
Passing through sieve [%]	Pef.	98.56	69.99
	C1/1	99.13	70.49
	C2/1	98.34	69.91
	C3/1	98.94	70.17
	C4/1	99.02	70.50
	C1/2	99.40	71.23
	C2/2	98.04	70.91
	C3/2	98.87	70.84
	C4/2	98.78	71.12

Table 3. PSD of cement samples

The results indicate that the particle size distribution (PSD) of all samples is consistent with that of the reference cement. A noticeable difference is expected between the PSD of laboratory-milled samples and that of cement produced under industrial

conditions. However, the test confirms that despite the gradual increase in slag content, increased by 2% in each sample, the use of grinding aids enables the achievement of a comparable PSD across all mixtures. This demonstrates the effectiveness of grinding aids in maintaining grinding efficiency and uniform fineness, even in high-slag cement compositions.

The results from the tests for standard consistency and setting time of cement are shown in figure 1.

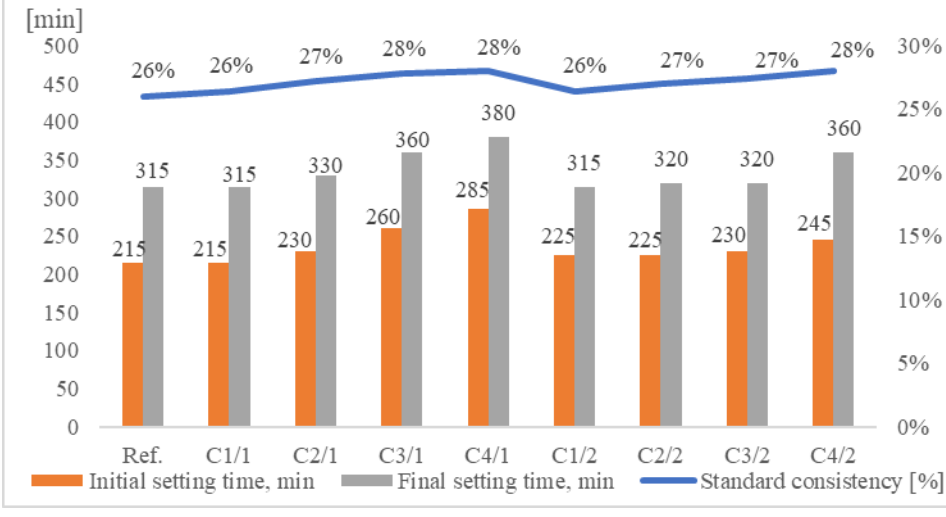


Figure 1. Standard consistency and setting time of cement samples

It can be concluded that, in this particular experiment, increasing the slag content results in a slight increase in the standard consistency and setting time of the cement samples.

As previously mentioned, the mortar specimens were prepared and tested in accordance with the procedure outlined in EN 196-1. Figures 2 and 3 present the mechanical properties of the mortars prepared with each cement sample. The flexural strength remains relatively consistent regardless of the slag content. However, a more noticeable variation is observed in the compressive strength across the different mortar compositions. The specimens prepared with C1/1, C1/2, C2/1, and C2/2 - where the slag content was increased by 2% and 4% compared to the reference sample - exhibited good mechanical performance, in some cases even surpassing that of the reference mortars. This improvement can be attributed to the effect of the grinding aid, demonstrating that it is possible to maintain the same strength class despite the higher slag content. That being said, a gradual decline in mechanical properties is clearly observed with further increases in slag content, becoming most pronounced in samples C4/1 and C4/2, where the slag content was increased by 8%. This outcome is expected, as slag hydrates at a significantly slower rate compared to clinker. However, with extended curing time, these samples have the potential to develop the same or even superior mechanical properties compared to the reference samples [11].

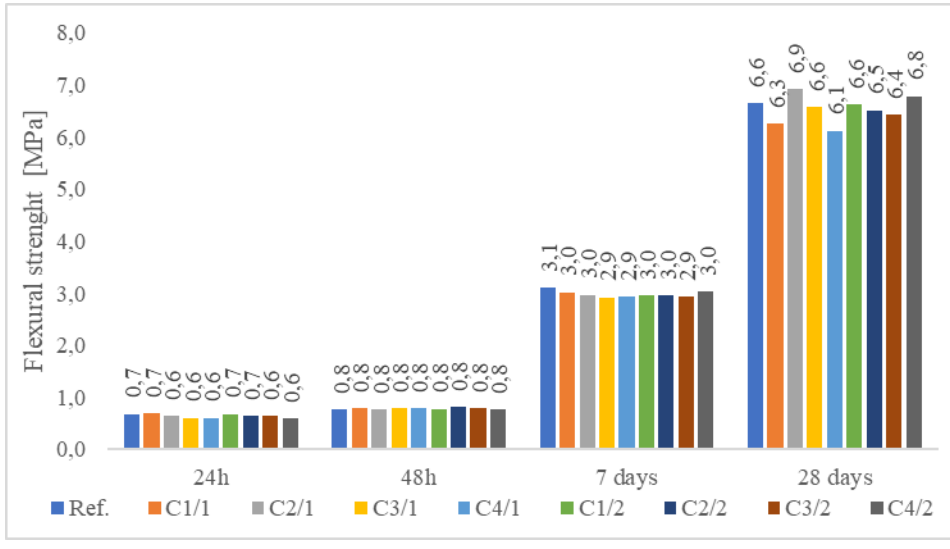


Figure 2. Flexural strength of mortar specimens

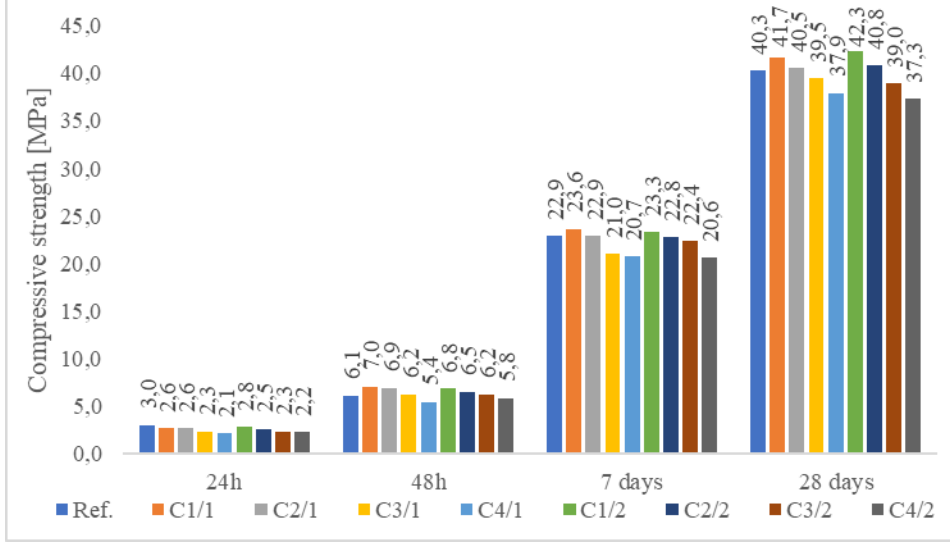


Figure 3. Compressive strength of mortar specimens

2.2.2. Concrete specimens

All concrete specimens were prepared according to the formulation outlined in Table 2. To ensure that cement type remained the sole variable throughout the tests, all specimens were produced with the same water-to-cement (W/C) ratio and identical dosage of superplasticizer.

The results from the tests on fresh concrete are presented in Table 4.

Constituent	Slump at 5min [mm]	Air content [%]	Density of fresh concrete [kg/m^3]
Ref.	150	3.2	2380
C1/1	155	3.0	2403
C2/1	120	3.5	2330
C3/1	110	3.4	2375
C4/1	70	3.2	2370
C1/2	150	3.5	2375
C2/2	125	3.5	2400
C3/2	100	3.4	2370
C4/2	60	3.5	2357

Table 4. Fresh properties of the concrete mixtures

The results indicate that the air content and density of the fresh concrete remain consistent across all specimens. However, a notable difference is observed in workability. Specifically, as the slag content increases, workability decreases. This trend aligns with the earlier findings on standard consistency of the cement samples (Figure 1), where an increase in slag content corresponded to higher standard consistency values.

The results from the compressive strength test are shown in Figure 4.

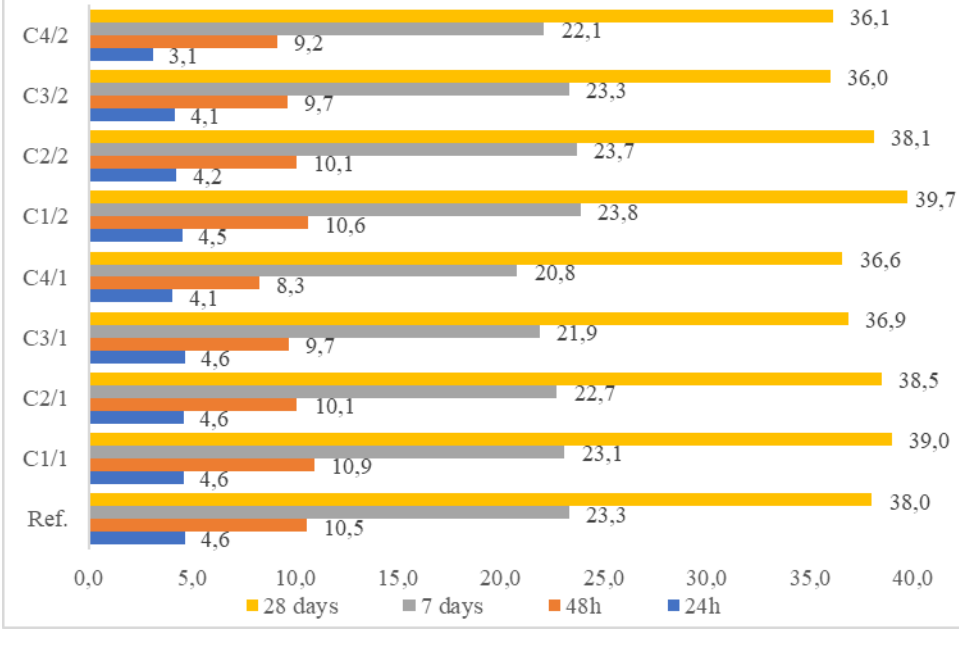


Figure 4. Compressive strength of concrete specimens

Upon examining the results, it can be concluded that, similar to the mortar specimens, the concrete specimens prepared with C1/1, C1/2, C2/1, and C2/2 exhibited good mechanical performance. However, a further increase in slag content results in a decline in mechanical properties.

3. CONCLUSION

This study investigated the effects of increasing slag content in cement on the fresh and hardened properties of mortar and concrete. The results demonstrated that moderate increases in slag content (2–4%) in samples such as C1/1, C1/2, C2/1, and C2/2 did not negatively impact mechanical performance; in fact, some exhibited improved strength compared to the reference mortar and concrete. This enhancement can be attributed to the action of the grinding aid, which helped maintain the required strength class despite the reduced clinker content.

However, as slag content increased to 8% (e.g., C4/1 and C4/2), a decline in compressive strength became evident. Nevertheless, since slag hydrates more slowly than clinker, these samples may reach equal or greater mechanical properties over time with prolonged curing. Additionally, increasing slag content negatively affected workability, consistent with earlier findings that showed higher standard consistency values for cement samples with greater slag content.

Overall, the findings suggest that with careful formulation and the use of grinding aids, partial replacement of clinker with slag can be an effective strategy to produce sustainable cement while maintaining acceptable performance, especially in long-term applications.

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

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Pregledni rad

IZVEDBA GRAĐEVINA OD ARHITEKTONSKOG BETONA

Rezime: *Arhitektonski beton (vidni beton, dekorativni beton, natur beton) je beton koji je vidljiv bez završne dekorativne obrade te sve češće krasi pročelja zgrada, armiranobetonskih elemenata trgova, interijera. Različitost izvedbe strukture i teksture betona danas daje za pravo različitosti vidljive površine betona. Uz beton veliki utjecaj na izgled površine betona ima vrsta oplate, vrsta sredstva za odvajanje oplate (oplatani), spojna sredstva oplate, te njegovanje betona. Također izvođač radova mora imati iskustva u izvedbi armiranobetonskih elemenata. U radu će se prikazati vrste arhitektonskih betona, detalji na koje je potrebno obratiti pozornost tijekom izvedbe, greške koje se pojavljuju u izvedbi i dat će se prikaz nekoliko objekata koji su izvedeni u Hrvatskoj.*
Ključne reči: *arhitektonski beton, vidljiva površina, izvedba, primjena*

CONSTRUCTION OF BUILDINGS WITH ARCHITECTURAL CONCRETE

Summary: *Architectural concrete (visible concrete, decorative concrete, natural concrete) is concrete that is visible without final decorative treatment and is increasingly adorning the facades of buildings, reinforced concrete elements of squares, and interiors. The diversity of the structure and texture of concrete today gives rise to the diversity of the visible surface of concrete. In addition to concrete, the type of formwork, the type of formwork release agent (formwork), the formwork bonding agents, and the care of the concrete have a great influence on the appearance of the concrete surface. The contractor must also have experience in the execution of reinforced concrete elements. This paper will present the various types of architectural concrete, highlight key details that require attention during execution, discuss common errors encountered in the process, and showcase several completed projects in Croatia.*
Keywords: *architectural concrete, visible surface, performance, application*

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1. UVOD

Arhitektonski beton, vidni beton, dekorativni beton, natur beton, je naziv za iste vrste betonske površine koje ostaju izložene i ne skrivaju se ispod drugih materijala, tj. nemaju završnu obradu nanošenjem drugog dekorativnog materijala. Takvi betoni su posve vidljivi i ističu svoju prirodnu boju ili su obojani pigmentima. Vrlo često površina ovih betona je glatka (iz oplate) ili joj je nametnuta tekstura s obzirom na različite vrste površinske obrade (brušenje, poliranje, štokanje, pjeskarenje i dr.). Primjenom ove vrste betona postiže se moderan i jedinstven izgled, a pritom armiranobetonski (AB) elementi zadržavaju zahtijevanu čvrstoću i trajnost. Beton ne mora biti samo siva, dosadna konstrukcijska struktura već se današnjim tehnologijama izvedbe i različitim vrstama betona može postići različitost od klasične izvedbe nanošenja žbuke, fasade na betonske površine.

2. PODJELA ARHITEKTONSKOG BETONA

Arhitektonski beton dobio je naziv na način da se ispune zahtjevi arhitekta (projektanata) za vizualnim izgledom površine betona od boje, teksture ili završne obrade. Osim zahtjeva za izgledom, beton mora zadovoljiti prije svega mehaničku otpornost i stabilnost svakog projektiranog AB elementa a najzahtjevniji segment je da betonska površina zadovolji trajnost AB konstrukcije, što je zapravo i glavni zadatak betona, a to je da štiti armaturu. Svaki nedostatak u izvedbi na površini betona izazvat će otvoreni put ka degradaciji konstrukcije, te estetski izgled bit će narušen. Vrlo važan detalj trajnosti betona prilikom izvedbe arhitektonskih betona je primjena hidrofobnih premaza, koji su prozirni, penetriraju u površinsku strukturu betona i na taj način čine površinu betona dugotrajnijom, ali ne i stalno zaštićenom (potrebna je opetovana primjena hidrofobne impregnacije tijekom vremena).

2.1. Podjela arhitektonskih betona, prema načinu proizvodnje

- Predgotovljeni betonski elementi koji se proizvode u pogonima te se naknadno montiraju na gradilištu (npr fasadni elementi, montažna stepeništa, gazišta). Ova vrsta betona ima trajnosna svojstva duža nego monolitno izvedena konstrukcija, jer sami elementi proizvode se u pogonu u kontroliranim uvjetima okoline. [1]
- Betonski elementi koji nastaju kao ukrasno-konstruktivni elementi izvedeni u oplati na licu mjesta (nosivi/pregradni zidovi, podne ploče). Prilikom izvedbe arhitektonskih monolitnih elemenata, najbitniji faktor je iskustvo izvođača. Izvođač na gradilištu mora paziti na vremenske uvjete izvođenja jer ako pada kiša može doći do ispiranja površine betona, ako puše vjetar i vani je toplo, moguća je pojava pukotina. Tijekom hladnog perioda hladnoća ima utjecaj na sredstva za premazivanje oplate, gdje je vrlo važno odabrati dobru vrstu sredstva za oplatu i način njegovanja betona.[1]

2.2. Podjela arhitektonskih betona prema boji betona

Prirodna boja betona ovisi o sastavnim materijalima betona, ona je najčešće siva, ali sa bijelim cementima i bijela. Obični klasični betoni koji ostaju arhitektonski vidljivi ovise o vrsti cementa, vrsti agregata, nijansa se može razlikovati i o v/c faktoru, tj. o količini vode. Osim sastavnih materijala veliki utjecaj ima vrsta oplate, oplatana, te njegovanje betona. Tijekom gradnje objekata od arhitektonskog betona ne smije doći do promjene u vrsti sastavnih materijala jer se estetski narušava izgled objekta. Slika 1. prikazuje kako je tijekom gradnje došlo do promjene vrste cementa.



Slika 1. Različitost u boji betona

Umjetno proizvedena boja betona dobije se dodavanjem inertnog dodatka pigmenata. Pigment je vrsta mineralnog dodatka koji se dodaje u beton u obliku sitnih čestica (praha, tekućine, granula), čija je namjena obojati vezivo. Boja pigmentiranih betona ovisi o nekoliko faktora [2]:

a) o v/c faktoru, što je veći v/c faktor beton je svjetliji

b) o vrsti cementa što je cement svjetliji beton je svjetliji

c) o količini pigmenta koji se dodaje na masu cementa, više pigmenta boja je intenzivnija, tamnija (uobičajeno je doziranje oko 5% na masu cementa)

Malena promjena u količini vode tijekom proizvodnje pigmentiranog betona može utjecati na nijansu boje u elementu.

2.3. Podjela arhitektonskih betona prema načinu površinske obrade

Glatka površina betona iz oplate koja se naknadno dorađuje, mehaničkim zahvatom poput brušenjem i poliranjem, pjeskarenjem i štokanjem, daje poseban izgled betonu. [3]

Brušenje površine betona sa poliranjem je postupak kojim se uklanja površinski sloj betona, te je vidljiva struktura betona. Primjenom finije brusne ploče, betonska površina može doseći visoki sjaj a taj postupak nazivamo poliranje. Izgled površine ovisi o debljini sloja koji se brusi, što je sloj manji, manje se vide veće frakcije agregata, što je sloj veći vidljiva je struktura betona s većim zrnima agregata (slika 2).



Slika 2. Brušena i polirana površina betona

Pjeskarenje površine betona je metoda završne obrade gdje pod pritiskom vode u kombinaciji s pijeskom dolazi do uklanjanja površine betona. Također izgled površine betona ovisi o dubini pjeskarenja (slika 3).



Slika 4. Pjeskareni pigmentirani beton

Štokana površina betona, često puta ovu vrstu izvođenja nazivamo „filigranska obrada“, jer ovisi o samom iskustvu radnika koji uklanja površinski sloj betona štokalicom (čekićem). Dubina uklanjanja betona, te alati kojima se površina betona obrađuje uveliko određuju njen krajnji izgled.



Slika 5. Površina betona izvedena štokanjem

Vrlo popularna izvedba arhitektonskih betona je izvedba betona s reljefnom površinom. Reljef na površini betona može biti utisnut u beton ili takav da beton ispunjava udubljenja u oplati pa nakon skidanja oplate ostaju nabori, ispupčenja i slično. Za ovu vrstu izvedbe koriste se i različiti materijali: drvene daske, poliuretanski umetci, guma, metalni kalupi i sl. Često puta montaža drvenih letvica/daščica (obično širina 8, 10, 12 cm ili njihova kombinacija) složenih u horizontalnom ili vertikalnom nizu. (slika 6) [3]



Slika 6. Reljefni beton od horizontalnih dašcica

Svojstvo upijanja drvene građe, izbor oplatnog ulja mora biti poznato (prethodnim probama utvrđeno) jer može dodatno otežavati izgled buduće površine.

3. IZVEDBA ARHITEKTONSKOG BETONA

Beton je nehomogen materijal, tijekom proizvodnje, transporta i ugradnje mogu nastati različite greške koje narušavaju estetski izgled površine betona. Uz beton veliki utjecaj na izgled površine betona ima vrsta oplate, vrsta sredstva za odvajanje oplate (oplatan), spojna sredstva oplate, njegovanje betona. Zahtjevima u projektu moraju se jasno definirati boja i tekstura površine betona. U projektu je potrebno napisati kakva se površina betona očekuje u smislu boje, teksture, oblika i sl., definirati način izvedbe, tehnička rješenja vezana uz oplatu i sl., propisati program kvalitete za materijale i izvedbu, te najaviti izrada probnih uzoraka i probe na gradilištu u vidu modela zida, poda i sl.. Puno je faktora koji utječu na kvalitetu izvedbe vidnog betona.

Beton kao glavni materijal za izvedbu vidnih betona ima ista svojstva koja su propisana specifikacijom prema normi HRN EN 206, Beton -- Specifikacija, svojstva, proizvodnja i sukladnost. [4]

Oplata tj. oplatna ploča je dio sustava koji je u direktnom kontaktu sa betonom. Oplatna ploča ili oplatno lice daje betonu oblik i ujedno sprječava curenje cementnog mlijeka i samog betona. [5] Zahtjevi u projektu moraju sadržavati detaljne specifikacije rastera oplatnih elemenata, teksturu konačne površine. Izvođač prije izvođenja mora u Planu kontrole izvedbe betonske konstrukcije opisati način montaže oplate tj. redoslijed sastavljanja ploča, detalje spojeva elemenata, obratiti pozornost na upijanje oplatne ploče (upojna ili neupojna), način ukrute ploča međusobno (distanceri oplate) i sl. [5]

Kada govorimo o oplatnim pločama, razlikujemo ploče sa neupijajućom površinom koje se izrađuju od čelika (slika 7.), plastike ili sličnih materijala (fiberglas, polisulfidne ploče, poliuretanske, polistirol) ili pak upijajuće oplate kao što su drvene oplatne ploče sa vodootpornim filmom. [6]



Slika 7. Čelična oplata [6]

Ploče sa upijajućom površinom su drvene ploče (slika 8.) od svježe piljene građe (daske, fosne), sušena drvena građa, laminirane drvene ploče furnirane (šper-ploče), OSB ploče, 3 slojne laminirane ploče („žute“ ploče) i sl. [6]



Slika 8. Drvene upijajuće ploče piljena drvena građa [6]

Ploče koje se u Hrvatskoj ipak najviše koriste su ploče od lameliranog drveta presvučene zaštitnim, vodootpornim filmom. Takve ploče mogu dugo trajati uz pravilno čišćenje i održavanje a i lako se zamjenjuju. Mana im je bubrenje drveta i slične deformacije uslijed prodiranja vlage u slojeve ispod zaštitnog filma. Također, primjenu se upojne folije koje se postavljaju na oplatne ploče.

Nedostaci drvenih upijajućih ploča je sposobnost upijanja viška cementnog mlijeka iz betona, te kod učestale uporabe dolazi do zasićenja drveta cementnim mlijekom pa se mijenjaju karakteristike oplate, te su moguće pojave promjena u boji betona. Također, oplata se manje kvalitetno odvaja od betonske površine, gubi se i struktura drveta za otisak u betonu. Osim toga, i vrijeme montaže može biti osjetno duže nego kod gotovih oplatnih sustava. [6] Drvenu građu treba i čuvati od isušivanja koje može dovesti do vitoperenja dasaka. Prednost ovakvih ploča je cijena/ekonomičnost jer su relativno jeftin materijal, pogotovo za uporabu na poslovima betoniranja manjih razmjera. Dobra tekstura površine daske koja u betonu ostavlja trag godova i nepravilnosti drveta, te je zato jedna od omiljenijih tekstura koju arhitekti žele potencirajući imitaciju drveta maksimalno prirodnog izgleda.

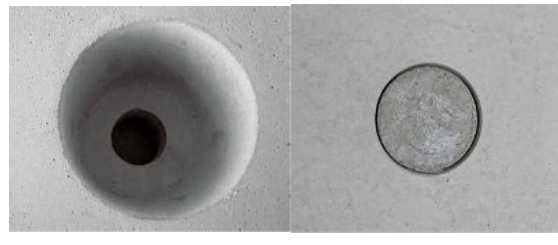
Pribor za međusobno spajanje oplata i drugi detalji koji mogu imati veliku ulogu u konačnom izgledu vidnog betona je spoj oplate. Kod montaže oplatnih sklopova,

osim vanjskih podupirača koji sprječavaju širenje oplate na koju djeluje pritisak svježeg betona, dvostrane oplate se međusobno podupiru tako da se povežu navojnim šipkama-distancerima za oplatu („juvidurke“). Modeli okvirnih oplata različitih proizvođača na oplatnim pločama imaju tvornički pripremljene rupe i dopuštaju bušenje novih rupa. Distanceri za oplatu (cijevi) mogu biti plastične ili od mikrobetona. Otvori se zatvaraju posebnim plastičnim čepovima. [6] (slika 9.).



Slika 9. Distanceri za oplatu od plastike i mikrobetona

Konusi moraju biti plastični jer nakon odvajanja oplate ostaje samo njihov trag u betonskoj površini. Nakon što se demontira oplata, rupe u betonu se zadržavaju ili zatvaraju betonskim čepovima [6]. (slika 10).



Slika 9. Betonski čepovi za vidne betone

Greške koje se mogu pojaviti na površini i uzroci mogu biti različiti: raster vidnih površina (radni taktovi), širina zida i veličina zaštitnog sloja betona, vrsta i receptura betona, loš odabir oplatnog sustava, nestručna izrada ili montaža oplate, brtvljenje, oplatna ulja, nestručna ugradnja, loše vibriranje, ugradnja u neprikladnim vremenskim uvjetima, nestručna demontaža oplate, loša ili nikakva njega betona. [3]

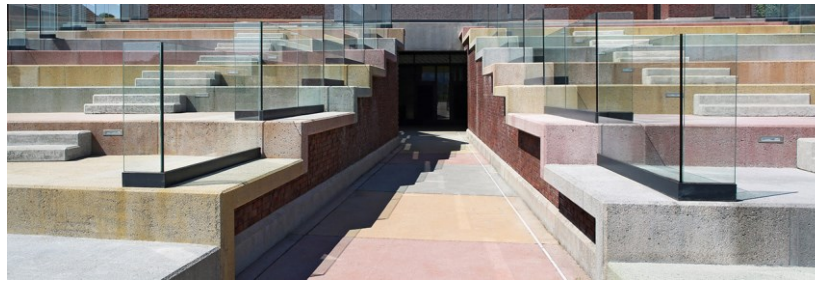
4. IZVEDENI OBJEKTI U HRVATSKOJ

U Hrvatskoj tijekom niza godina, izvedeno je puno objekata od arhitektonskog betona. Jedan od njih je City Island -„Gradski Otok“ nalazi se na južnom ulazu u Zagreb koji u svom kompleksu sadrži poslovne urede, kongresni centar, gastro sadržji, rekreacijski centar i dječji vrtić. Površine betona uglavnom su vidljive i nastale uklanjanjem oplate, a podne vanjske ploče i vanjski zidići su brušena i polirana površina betona. (slika 9.)



Slika 9. „Gradski Otok“ City Island

Poslovno-prezentacijska zgrada koja se nalazi u Pleternici, a čiji je trg i muzej posvećen bećarcu (Slavonska narodna pjesma), izvedena je 2019. godine. Vanjski okoliš trga i krov muzeja izvedeni su u 5 različitih obojenih betona (žuta, terakota, zelena, crvena i crna), a koje predstavljaju polja Slavonije. (slika 10)



Slika 10. „Trg bećarca“ u Pleternici [7]

Hotel „Park“ u Rovinju, kombinacija je pigmentiranog betona u okolišu, te natur betona koji je obradom štokanja vidljiv u sobama hotela. (slika 11)



Slika 11. Hotel Park u Rovinju [8]

5. ZAKLJUČAK

Beton je građevinski materijal, koji se primjenjuje za izvedbu nosivog dijela AB konstrukcije, sa najčešće izvedbom fasade kao završni izgled konstrukcije. Oblikovanje betona, laka dostupnost komponenti, svojstvo prilagodljivosti oblika, te njegova trajnost čine ga uvijek poželjnim materijalom u graditeljstvu. Sa druge strane njegov hladan i siv vanjski izgled čine ga estetski neprivlačnim. Različitim metodama izrade ili obrade površina betona može se površina učiniti dovoljno zanimljivom da ostane vidljiva. U posljednje vrijeme arhitektonski (vidni, natur) betoni postaju sve popularniji te svojom raznovrsnošću daju arhitektima mogućnost igre s bojom i teksturom betona.

Projektnom dokumentacijom jasno treba propisati očekivanja krajnjih rezultata u izvedbi arhitektonskog betona. Projekt treba sadržavati sve potrebne informacije i rješenja koja izvođač treba slijediti. Izvedba i krajnji izgled arhitektonskog betona ovisi o više čimbenika. Izvođač arhitektonskih betona mora imati iskustvo u izvedbi, te svi sudionici u izvedbi moraju biti upoznati sa svim rizicima uslijed loše izvedbe. Najveći nedostatak arhitektonskih betona je otežana mogućnost ili praktički nikakva mogućnost sanacije vidne površine kako bi se ona dovela u prvobitno stanje. Osim izvođača potrebna je dobra ujednačene kvaliteta betona, kvalitetna oprema za ugradnju betona i odgovarajući oplatni sustavi, te na kraju vrlo važno je njegovanje betona.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

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COMPARATIVE ANALYSIS OF DIFFERENT GLASS TYPES DURING THE STANDARD FIRE TEST

Summary: *The paper investigates the fire resistance of three types of glass samples exposed to standard fire conditions. The tested samples were: thermal insulation glass (4 + 16 + 4 mm), single glazing with 8 mm thickness, and single glazing with 6 mm thickness. All three samples were placed in the standard furnace and tested simultaneously. The objective of this testing is to evaluate the possibility of replacing single glazing with thermal insulation glass (double glazing: 4 + 16 + 4 mm) in applications where single glazing is currently planned. The aim of the testing was to determine when all three samples would lose their integrity, i.e., when the last piece of glass would break. Installing thermal insulation glass in places where single-glazed windows are installed has higher heat loss in winter, as well as better comfort inside the rooms in summer. Some buildings are equipped with single-pane windows, which are installed with the intention of allowing rapid breakage during a fire to facilitate the extraction of smoke and heat. The glazing type relevant in this case is one that will lose its integrity within 15 minutes of testing. The results indicate that it is feasible to install thermal insulation glass (4 + 16 + 4 mm double glazing) as a replacement for single-glazed windows.*
Key words: *thermal insulation glass, single glazing, standard fire test, integrity.*

KOMPARATIVNA ANALIZA TRI TIPA STAKALA IZLOŽENIH STANDARDNOM POŽARU

Rezime: *U radu se ispituju tri tipa stakala izloženih standardnom požaru. Ispitani su uzorci: termoizolaciono staklo (4 + 16 + 4) mm, jednostruko staklo debljine 8 mm i jednostruko staklo debljine 6 mm. Sva tri uzorka su stavljena u standardnu, ispitnu peć i testirana istovremeno. Cilj ispitivanja je dokazivanje mogućnosti ugradnje termoizolacionih stakala (dvostrukog stakla (4 + 16 + 4) mm) na mestima gde je predviđena ugradnja jednostrukih ostakljenja. Ugradnjom termoizolacionog stakla na mestima gde su ugrađena jednostruka stakla, pružaju se manji toplotni*

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[4] dipl.inž. maš, rukovodilac laboratorije, Insitut za ispitivanje materijala – IMS, Bulevar vojvode Mišića 43, dragisa.ivanisevic@institutims.rs

gubici u zimskom periodu, kao i bolji komfor unutar prostorija u letnjem periodu. Ispitivanjem dolazimo do podatka za koji vremenski period će sva tri uzorka izgubiti integritet, odnosno kada će pući i poslednje od tri ispitivana stakla. Na objektima postoje pozicije prozora kod kojih je predviđena ugradnja jednostrukih stakala koja u slučaju požara treba da puknu jer na tim mestima treba da dođe do ekstrakcije dima i toplote iz prostora. Tip zastakljenja koje je primenjivo u navedenom slučaju jeste ono staklo koje će izgubiti integritet pre isteka 15. minuta ispitivanja. Rezultati ispitivanja pokazuju da je izvodljivo ugraditi termoizolaciono staklo (dvostruko staklo (4 + 16 + 4) mm) umesto jednostrukog.
Ključne reči: *termoizolaciono staklo, jednostruko staklo, standardno požarno ispitivanje, integritet.*

1. INTRODUCTION

A renowned architect frequently emphasized that windows represent the most significant and creative element of a building façade [1]. They contribute substantially to the aesthetic character of the façade, while simultaneously ensuring adequate ventilation and natural lighting of interior spaces, thereby fostering a connection between occupants and the natural environment. Moreover, contemporary windows are required to meet a wide range of functional demands, including protection against cold, heat, noise, wind, precipitation, forced entry, and fire. These functional criteria for windows—and for façade components more broadly—must be established during the initial design phase. There are primary and additionally functions of the windows [1].

Primary functions of windows:

- Protection against rain, wind, and cold
- Transparency or translucency
- Provision of natural daylight
- Enabling communication with the surrounding environment
- Ventilation

Additional functions of windows:

- Thermal protection
- Environmental protection
- Sound insulation
- Sun protection
- Protection of the building and its occupants
- Fire protection
- Enabling the use of solar energy
- Ensuring a comfortable feeling within the space
- Providing design possibilities
- Suppressing electromagnetic influences

This imposes numerous additional responsibilities on window and façade designers, as they must consider a variety of legal frameworks, regulations, building codes, standards, and technical guidelines throughout the design process.

The fire resistance of glass construction is a critical area of research in civil engineering and building safety, particularly in the context of fire protection and energy efficiency optimization. Glass, as a widely used material in construction, must meet diverse functional requirements, including thermal insulation, aesthetic qualities, and

reliable performance under fire conditions. Traditionally, single glazing has been employed in certain applications due to its ability to break rapidly during a fire, facilitating the extraction of smoke and heat—an essential feature in buildings where swift evacuation is critical. However, single glazing presents significant drawbacks, such as high heat loss during winter and reduced indoor comfort during summer, leading to increased energy consumption for heating and cooling.

In contrast, thermal insulation glass (double glazing), such as the (4 + 16 + 4) mm configuration, offers superior thermal performance by reducing heat loss and enhancing the energy efficiency of buildings. Nevertheless, its application in scenarios requiring rapid glass failure during a fire remains underexplored. This raises the question of whether thermal insulation glass can effectively replace single glazing in applications where fire safety is a priority, while also maintaining or improving other functional characteristics.

This study aims to investigate the fire resistance of three types of glass samples—thermal insulation glass (4 + 16 + 4 mm), single glazing with a thickness of 8 mm, and single glazing with a thickness of 6 mm—exposed to standard fire conditions in a furnace. The primary objective of the research is to determine the time to loss of integrity for each sample, i.e., the moment when the glass breaks, in order to evaluate the feasibility of replacing single glazing with thermal insulation glass in buildings designed for rapid smoke ventilation. Particular emphasis is placed on assessing whether thermal insulation glass can meet the criterion of losing integrity within 15 minutes, which is relevant to the fire safety requirements of specific building types.

This research contributes to a better understanding of the performance of different glass types under fire conditions and provides a foundation for informed decision-making in construction projects.

2. TESTING METHODOLOGY

Three glass samples were tested in accordance with the SRPS EN 1363-1 standard [2], where the criterion for assessing fire resistance was based on the preservation of integrity—specifically, the appearance of sustained flames and the release of hot gases at locations where glass fragments had detached. The standard defines a time-temperature curve representing the development of a standard fire (Figure 1). This fire development is described by a logarithmic equation, where t [min] denotes time and T [°C] represents the temperature inside the furnace as a function of time t. The standard fire temperature curve is given by the following equation:

$$\Theta_g = 345 \log_{10}(8t + 1) + 20 .$$

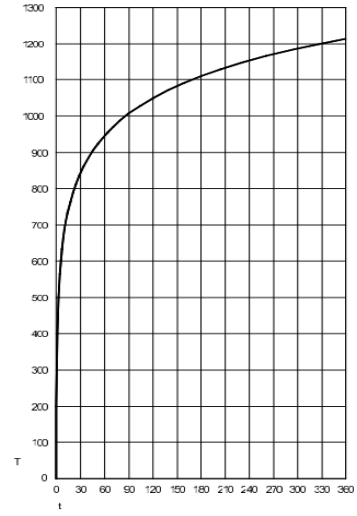


Figure 1. Time-temperature standard fire curve [1]

In addition to the appearance of flame on the sample, integrity can also be assessed using a cotton pad or cylindrical steel rods with diameters of 6 mm and 25 mm. The cotton pad is used when suspicious areas appear on the sample, indicating a potential for flame transfer from the fire-exposed side to the non-exposed side. Steel rods are employed when cracks develop on the sample. If either the 6 mm or 25 mm diameter rod penetrates the glass, the sample is considered to have lost its integrity. The loss of integrity marks the end of the test for that particular glass sample.

The standard test furnace is installed at the Laboratory for Thermal Techniques and Fire Resistance at the IMS Institute in Belgrade. The data acquisition system for monitoring standard fire tests in the vertical furnace was fully developed in-house, in accordance with the SRPS EN 1363-1 standard [2]. The main components of the test furnace include:

- a data acquisition system,
- two-stage burners for liquid fuel, manufactured by *Ecoflam* (Italy), type *Major P25 AB HS TL V.C.*, with a thermal output of 296 kW,
- differential pressure transmitters, manufactured by *TESTO* (Germany), model 6321, with a measurement range of ±100 Pa,
- type K thermocouples for temperature measurement inside the test furnace.

Three glass samples were mounted in pre-prepared openings within the mobile wall of the test furnace, constructed from gas concrete blocks with a thickness of 250 mm. Each opening had dimensions of 660 mm × 1095 mm. The samples were positioned in the lower section of the furnace as follows:

- A thermal insulation glass panel, with dimensions 640 mm × 1085 mm (width × height), type (4 + 16 + 4) mm, was installed in the lower left zone.
- A single glazing panel, 8 mm thick, with the same dimensions (640 mm × 1085 mm), was placed in the center of the lower zone.

- The third sample, a single glazing panel with a thickness of 6 mm and identical dimensions (640 mm × 1085 mm), was positioned in the lower right zone of the furnace (Figure 2).



Figure 2. The types of glass instaled in the test furnace

3. TEST RESULTS

After the installation of the three different glass samples inside the test furnace, testing commenced in accordance with the general testing conditions [2]. The first sample was thermal insulation double glazing (4 + 16 + 4) mm. The second sample was single glazing with a thickness of 8 mm, and the third sample was single glazing with a thickness of 6 mm.

One minute after the test began, both the third sample (6 mm thick single glazing) and the second sample (8 mm thick single glazing) broke. After two minutes, the first sample (thermal insulation glass) also failed. In the third minute, part of the glass from the third sample (6 mm thick single glazing) detached. This event marked the loss of integrity of sample number three (Figure 3). Testing of the first and second samples continued.



Figure 3. Integrity lost (sample number three)

In the 12th minute, the inner glass detached, and the outer glass began to crack. Flame penetration was observed through the sample, resulting in the loss of integrity of sample number one (Figure 4). Testing of sample number two (8 mm thick single glazing) continued.



Figure 4. Integrity lost (sample number one)

In the 23rd minute, an integrity check was performed using a Ø 6 mm rod. The loss of integrity of sample number two was confirmed (Figure 5). The test was concluded.



Figure 5. Integrity lost (sample number two)

4. CONCLUSION

This study investigated the fire resistance of three types of glass samples—thermal insulation glass (4 + 16 + 4 mm), single glazing with a thickness of 8 mm, and single glazing with a thickness of 6 mm—under standard fire conditions. The results demonstrate that thermal insulation glass can serve as a viable alternative to single glazing in applications where rapid breakage is necessary for smoke and heat ventilation during a fire. Specifically, the tested thermal insulation glass exhibited a loss of integrity within the critical 15-minute timeframe, aligning with fire safety requirements for specific building designs.

These findings suggest that double glazing not only meets the required fire performance criteria but also offers additional advantages, such as reduced heat loss during winter and improved indoor comfort during summer, thereby contributing to enhanced overall energy efficiency in buildings.

The successful performance of thermal insulation glass in this context opens new possibilities for its broader application in construction, particularly in buildings where single glazing is currently specified for fire safety purposes. However, further research is needed to investigate the behavior of thermal insulation glass under varying fire scenarios and across different structural configurations. Additionally, future studies should explore the long-term durability and cost-effectiveness of substituting single glazing with double glazing in fire-sensitive environments. These insights could support the development of optimized building designs that balance fire safety, energy efficiency, and occupant comfort.

The objective of the test was to evaluate the feasibility of installing thermally insulating glass (double glazing, 4 + 16 + 4 mm) in locations where single glazing is currently planned. In certain window positions within the building, single glazing is specified to break during a fire, enabling the extraction of smoke and heat from the space.

In such cases, the applicable type of glazing must be one that loses its integrity within the first 15 minutes of fire exposure.

Acknowledgement

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

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FROST RESISTANCE OF MASONRY MORTAR BLENDED WITH INDUSTRY AND AGRICULTURAL WASTE

Summary: *The use of waste materials in masonry mortars is not common, as traditional mortars primarily rely on cement and lime as binders. This study aims to investigate the effect of replacing cement with locally available industrial and agricultural waste materials: ceramic waste powder, corn cob ash, and fly as, on the frost resistance of mortar. As part of the experimental investigation, eight different mortar mixtures were prepared, varying the volume ratios of solid components (cement:lime:sand = 1:0.7:4.2 and 1:1:4), with cement being replaced by waste materials up to 50%. Since the mortars retained 75% of their initial compressive strength after 25 freeze-thaw cycles, it has been demonstrated that the selected materials can be successfully used in innovative and more environmentally friendly masonry mortars in low-temperature environments.*
Keywords: *Masonry mortars, ceramic powder, fly ash, biomass ash, frost action*

. OTPORNOST NA DEJSTVO MRAZA MALTERA ZA ZIDANJE SA INDUSTRIJSKIM I POLJOPRIVREDNIM OTPADOM

Rezime: *Upotreba otpadnih materijala u malterima za zidanje nije uobičajena, budući da se tradicionalni malteri primarno oslanjaju na cement i kreč kao veziva. Ova studija ima za cilj istraživanje efekta zamene cementa lokalno dostupnim otpadnim materijalima iz idustrije i poljoprivrede: keramičkim otpadnim prahom, pepelom kukuruznog oklaska i letećim pepelom, na otpornost maltera na dejstvo mraza. U okviru eksperimentalnog ispitivanja, pripremljeno je osam različitih malterskih mešavina, pri čemu su varirani zapreminski odnosi čvrstih komponenti (cement:kreč:pesak = 1:0.7:4.2 i 1:1:4), a cement je supstituisan do nivoa od 50%. S obzirom na to da su nakon 25 ciklusa zamrzavanja i odmrzavanja malteri zadržali 75% svoje prvobitne čvrstoće pri pritisku, pokazano je da se izabrani materijali mogu uspešno koristiti u inovativnim i ekološki prihvatljivijim hibridnim malterima za zidanje u sredinama u kojima vladaju niske temperature.*
Ključne reči: *Malteri za zidanje, keramički prah, leteći pepeo, biopepeo, dejstvo mraza.*

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1. INTRODUCTION

The construction industry ranks among the largest global consumers of natural resources and is a significant source of CO_2 emissions. This environmental impact has prompted an urgent shift toward sustainable practices, including the development and application of eco-friendly construction materials. A promising strategy involves the use of industrial and agricultural by-products as supplementary cementitious materials (SCMs), which not only conserves virgin resources but also reduces the carbon footprint associated with Portland cement production—particularly emissions from clinker manufacturing. Furthermore, repurposing locally available waste materials contributes to improved waste management and supports circular economy principles.

In alignment with these sustainability goals, this study investigates the potential of three locally sourced waste materials from Serbia—corn cob ash (CCA), fly ash (FA), and ceramic waste powder (CWP)—as SCMs in the production of environmentally friendly masonry mortars. The successful application of these materials hinges on their ability to meet durability requirements, particularly resistance to freeze-thaw cycles, which is a critical performance parameter in cold and temperate climates.

SCMs are known to influence frost resistance by enhancing the microstructure of cementitious composites. Pozzolanic materials such as fly ash, silica fume, and slag react with calcium hydroxide—a hydration by-product of cement—to form additional cementitious phases that densify the mortar matrix, reduce permeability, and minimize porosity. These changes hinder water ingress and the formation of internal ice crystals, thereby mitigating freeze-thaw damage and extending the material's service life [1]-[4].

However, the frost resistance imparted by SCMs varies significantly depending on their chemical composition, particle size distribution, water absorption capacity, and mix design. While extensively studied materials like fly ash and silica fume have demonstrated favorable results, less conventional SCMs such as biomass ash may exhibit limitations due to lower pozzolanic activity or higher porosity [5]-[8]. Therefore, a detailed understanding of each material's contribution to frost durability is essential for their effective application in construction.

This study aims to experimentally evaluate the frost resistance of masonry mortars incorporating CCA, FA, and CWP, thereby bridging the gap in current research, which has largely concentrated on cement mortars and concrete. To the best of our knowledge, this is the first comprehensive assessment of these specific waste materials—sourced from local industrial and agricultural processes in Serbia—as SCMs in masonry mortars. The findings provide valuable insights into the feasibility of utilizing such materials to develop sustainable, frost-resistant mortars suitable for use in modern construction practices.

2. MATERIALS AND METHODS

2.1. Materials

The Portland Cement (PC) used in this study was produced at the Lafarge cement plant in Vojvodina. It has a Blaine fineness of 4000 cm^2/g and a density of 3.1 g/cm^3.

FA, CCA, and CWP were utilized as SCMs in the cement-lime masonry mortars.

FA was provided by the thermal power plant Nikola Tesla B in Obrenovac, Serbia. It is characterized by the Blaine fineness of 6200 cm^2/g and a density of 2.05 g/cm^3.

CCA was sourced from ALMEX-IPOK in Zrenjanin, Serbia, a facility that utilizes biomass waste—specifically corn cobs—as an energy source, producing significant amounts of CCA as a by-product. To achieve the required fineness, the collected ash was ground using a laboratory ball mill. The processed CCA has a Blaine fineness of 6800 cm^2/g and a density of 2.50 g/cm^3.

CWP was derived from manufacturing waste, specifically damaged clay hollow blocks discarded at the NEXE Stražilovo production facility in Petrovaradin. The processed CWP has a Blaine fineness of 4800 cm^2/g and a density of 2.63 g/cm^3.

The following figure illustrates the appearance of the collected SCM samples.



(a) (b) (c)

Fig. 1. (a) Corn cob ash, (b) Fly ash, (c) Ceramic waste powder

The chemical composition of the SCMs is summarized in Table 1.

	FA	CCA	CWP
SiO_2, %	53.64	45.76	60.86
Al_2O_3, %	25.74	5.91	16.38
Fe_2O_3, %	7.36	3.37	6.81
Na_2O, %	0.30	0.00	0.77
K_2O, %	1.48	13.10	2.39
MgO, %	3.09	8.30	3.89
CaO, %	7.15	14.08	9.38
SO_3, %	2.75	1.26	0.80
P_2O_5, %	0.06	2.81	0.14
Content Cl, %	0.01	0.50	0.002
Reactive SiO_2, %	48.16	38.21	50.26

Tab. 1. Chemical compositions of SCMs

All tested SCMs demonstrated positive pozzolanic properties due to their favorable chemical composition. FA and CWP were classified as Class 10 pozzolanic materials, while corn CCA was classified as Class 5, in accordance with SRPS B.C1.018:2015 [9]. Furthermore, all SCMs met the activity index requirements specified in SRPS EN 450-1:2014 [10].

2.2. Methods

The chemical composition of the raw materials was determined in accordance with the standards SRPS EN 196-2 [11] and ISO 29581-2 [12].

The compressive strength of masonry mortar was evaluated in accordance with SRPS EN 998-2 [13] and SRPS EN 1015-11 [14]. After casting, the samples were cured in a humidity chamber at 95% relative humidity for seven days, then stored at 65% relative humidity for an additional 21 days until testing.

Since no standardized regulations or specific requirements exist for evaluating the frost resistance of masonry mortars, an internal testing method was adopted. After a 28-day curing period, standard mortar prisms were subjected to 25 freeze-thaw (f/t) cycles. Each cycle consisted of immersing the specimens in water at 20°C for two hours, followed by storage in a chamber at -20°C for an equal duration. After f/t exposure, the specimens were visually inspected for signs of damage before undergoing compressive strength test.

2.3. Mixing and proportioning of mortars

The experimental study involved eight distinct mortar mixtures. Reference cement-lime mortars were prepared using mixing ratios of 1:0.7:4.2 and 1:1:4 (cement/lime/sand) by volume. In the remaining mixtures, 50% of the cement content was replaced with fly ash, corn cob ash, or ceramic waste powder, following the corresponding volume-based mixing ratios. These ratios and substitution levels were determined based on prior laboratory experiments. The component proportions for each masonry mortar are detailed in Table 2.

To achieve the required workability, defined by a flow value of 175±10 mm in accordance with SRPS EN 1015-3 [15], the water content was adjusted accordingly.

Vol. ratio	Mortar	m_c (g)	m_l (g)	m_s (g)	m_{scm}(g)	w/b	m_w (g)
1:0.7:4.2	C2	193.7	59.2	1350	/	1.05	265.6
	FA2-50	96.9	59.2	1350	59.6	1.25	269.5
	CCA2-50	96.9	59.2	1350	68.7	1.18	265.2
	CWP2-50	96.9	59.2	1350	74.2	1.20	276.3
1:1:4	C3	201.8	92.5	1350	/	0.90	264.9
	FA3-50	100.9	92.5	1350	62.0	1.05	268.2
	CCA3-50	100.9	92.5	1350	71.6	1.00	265.0
	CWP3-50	100.9	92.5	1350	77.3	1.00	270.7

m_c - mass of cement; m_l-mass of lime; m_s - mass of sand; m_{scm} - mass of SCM; m_w - mass of water; w/b - water to binder ratio

Tab. 2. Labels and component materials quantities for designed masonry mortars

3. TEST RESULTS AND DISCUSSION

3.1. Compressive strength

Compressive strength of masonry mortar was tested at the ages of 28 and 90 days, and the results are shown in Figure 2.

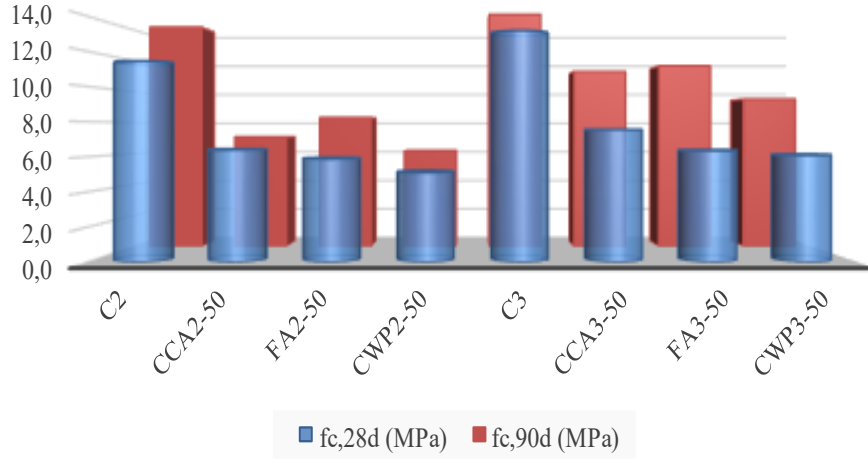


Fig. 2. The compressive strength of mortars after 28 day and 90 days

The results indicate that at 28 days, compressive strength decreases as the cement replacement level increases, regardless of the type of SCM used. Strength reductions ranged from 45–55% and 42–54% for mortars with volume ratios of 1:0.7:4.2 and 1:1:4, respectively, compared to the reference mortar. This decline is primarily attributed to the dilution effect, as SCMs exhibit lower initial hydraulic reactivity and hydrate more slowly than cement [2][16]. As a result, high-volume SCM-based mixtures produced fewer hydration products, leading to lower compressive strength compared to the reference blends at 28 days.

However, due to the delayed pozzolanic reaction, all SCM-blended mortars exhibited significant strength improvements at 90 days, with the most substantial gains observed in the 1:1:4 mortar series. The greatest strength increases were recorded in fly ash (FA)-blended masonry mortars, particularly FA2-50 and FA3-50, which achieved strength gains of 42% and 84%, respectively.

Despite the observed strength reduction, all blended mortars meet the compressive strength requirements for Class 5. This confirms that these environmentally friendly mortars are suitable for structural applications in masonry construction, in accordance with Eurocode 6 and Eurocode 8. Table 3 provides an overview of the compressive strengths and corresponding classifications for each masonry mortar.

Mortar	C2	CCA2-50	FA2-50	CWP2-50	C3	CCA3-50	CCA3-60	FA3-50	CWP3-50
fc, 28d (MPa)	11.46	6.41	5.89	5.10	13.18	7.55	6.41	6.35	6.09
Class	10	5	5	5	10	5	5	5	5

Tab. 3. Class of masonry mortars based on the achieved compressive strength

3.2. Frost resistance of masonry mortars

After f/t exposure, the specimens were visually examined for signs of damages. The appearance of specimens is given in Figure 3.

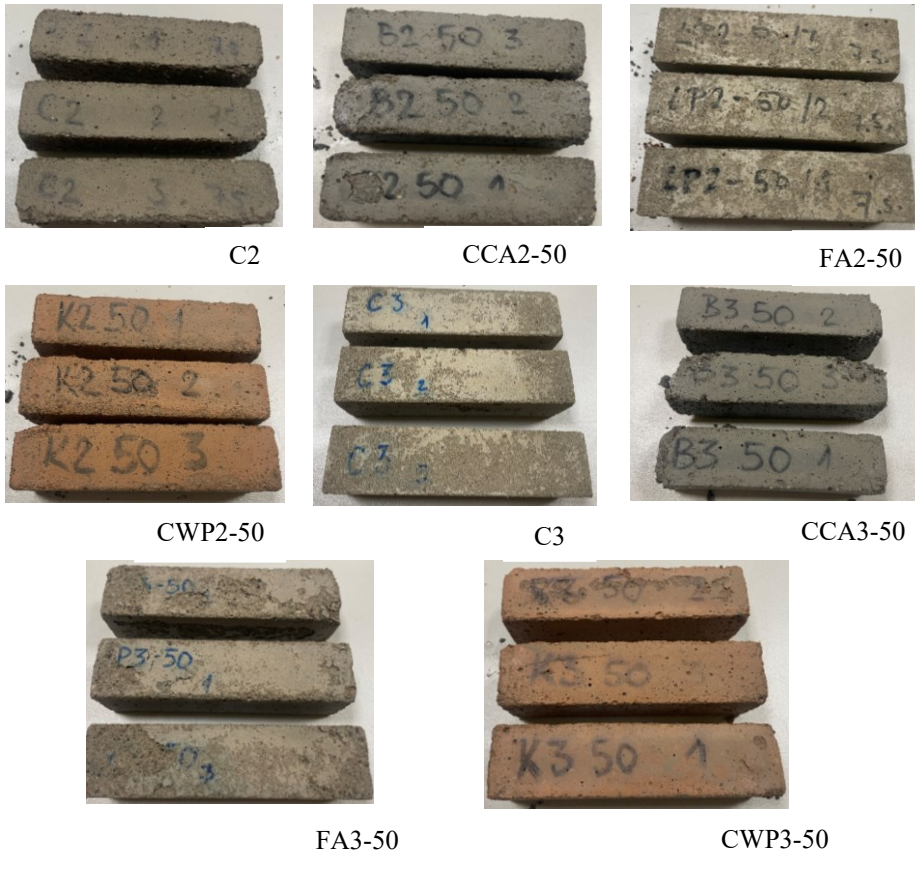


Fig. 3. Appearance of mortar specimens after 25 freeze-thaw cycles

Based on the extent of damage observed during exposure to freeze-thaw cycles, the mortars were classified into three degrees of damage. The types of damage and their corresponding classifications after freeze-thaw cycles are presented in Table 4.

Damage illustration	Damage type
	**SC – Scaling**: local flaking or peeling of a thin surface layer of hardened mortar, with no disruption of the bond between the binder phase and aggregate grains
	**SP – Surface spalling**: flaking or peeling of a hardened mortar layer over a larger surface area, with no disruption of the bond between the binder phase and aggregate grains.
	**D – Deep matrix damage**: significant deterioration of the mortar matrix, characterized by a disrupted bond between the binder phase and aggregate grains, along with delaminated and cracked sections within the cross-section.

Tab. 4. Types of damages on the specimens after f/t cycles

The most common form of damage seen in five out of eight mortar types was spalling. Mortars with a 1:0.7:4.2 volume ratio showed local to surface degradation, ranging from scaling to spalling. In contrast, mortars containing CCA with a 1:1:4 mixing ratio experienced significant structural damage, identified as deep matrix deterioration. These results suggest that this specific SCM is not suitable for producing frost-resistant masonry mortars with the chosen mixing ratio.

After conducting visual inspections and classifying the damage severity, the compressive strength of the specimens was assessed following exposure to freeze-thaw cycles. Figure 4 displays the compressive strength results before (fc,28d) and after (fc,ft) the freeze-thaw cycles, with the corresponding strength changes summarized in Table 5.

Mortar	C2	CCA2-50	FA2-50	CWP2-50	C3	CCA3-50	FA3-50	CWP3-50
Damage type	SP	SP	SC	SP	SC	D	SP	SP
fc,ft (MPa)	5,7	5,2	6,5	3,8	11,7	5,0	10,6	4,6
fc,ft/fc,e (%)	50	81	111	75	89	66	166	75
Class	-	+	+	+	+	-	+	+

Tab. 5. Damage type and achieved class after f/t cycles

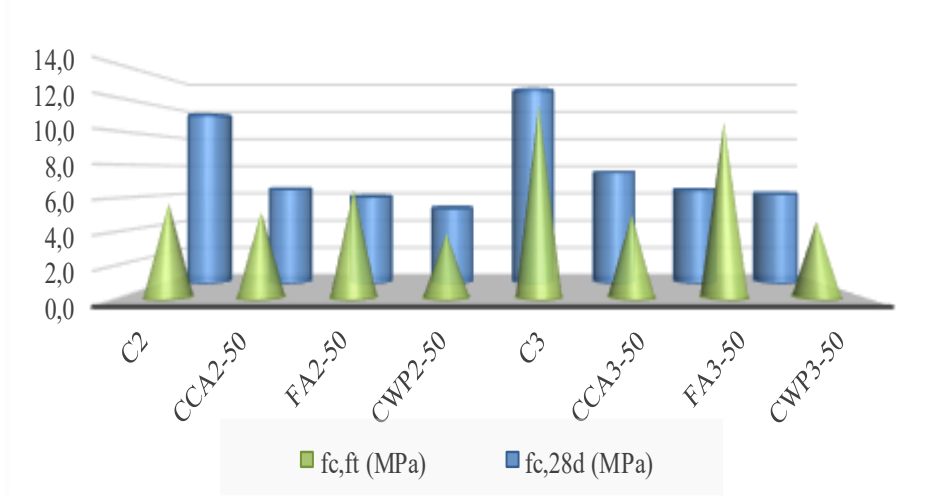


Fig. 4. The compressive strength of mortars at the age of 28 days and after 25 freeze-thaw cycles

Using the standard threshold for frost-induced strength reduction in cement-based composites—where a strength loss of less than 25% is considered acceptable—six out of eight mortars met this requirement. However, CCA-blended mortars with a 1:1:4 mixing ratio, which sustained significant damage, along with the reference mortar with a 1:0.7:4.2 ratio, failed to meet this criterion. Interestingly, FA-based mortars exhibited strength gains after freeze-thaw cycles, with the FA30-50 mortar showing the highest increase at 66%. This enhancement is likely attributed to the pozzolanic activity of FA, which reduces porosity in the cementitious matrix, thereby improving its elastic modulus and overall compressive strength [17][21]. Additionally, all CWP-based masonry mortars experienced a 25% strength reduction after freeze-thaw cycles, maintaining at least 75% of their original strength and meeting the required threshold.

4. CONCLUSIONS

This study investigated the durability of eco-friendly masonry mortars incorporating supplementary cementitious materials (SCMs)—specifically fly ash, ceramic waste powder, and biomass ash—with a focus on their resistance to freeze-thaw cycles. The experimental results demonstrated the strong potential of these alternative materials to contribute to sustainable construction practices by significantly reducing the environmental impact associated with conventional cement-based mortars.

The key findings of the study can be summarized as follows:

- Eco-friendly masonry mortars with up to 50% cement replacement using industrial and agricultural waste materials can be successfully produced while meeting the required mechanical performance standards for use in structural applications.
- In terms of frost resistance, the incorporation of SCMs was shown to improve the durability of mortars exposed to freeze-thaw conditions. Among the tested materials, fly ash proved particularly effective, enhancing frost resistance due to its pozzolanic activity, which contributes to matrix densification and reduced porosity. Some fly ash-based mixtures even exhibited strength gains after freeze-thaw exposure.

- While mortars containing corn cob ash showed comparatively lower performance under freezing conditions—likely due to lower pozzolanic reactivity and higher water absorption—the findings highlight the importance of SCM selection based on both material characteristics and environmental demands.

Overall, the findings underscore the feasibility of using locally available industrial and agricultural waste materials in the development of sustainable, frost-resistant masonry mortars. These results contribute to ongoing efforts in eco-conscious construction and offer a valuable reference for future research and practical implementation of green building materials.

Acknowledgments

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EXPERIMENTAL ANALYSIS OF EXPOSED CONCRETE SLABS THROUGH IMAGE ANALYSIS

Summary: *The paper presents a comparative analysis of eight concrete slabs prepared under laboratory conditions. The slabs were cast using formwork of varying quality, cured for 1 and 7 days, and compacted for 7s and 10 s. The comparison of properties, specifically the pore surface area of different diameters, was conducted using image analysis software. The results show that curing conditions had no significant effect on the surface characteristics of exposed concrete, while both the type of formwork and compaction duration demonstrated a more significant influence.*
Keywords: *exposed concrete, pore surface area, pore diameter, exposed concrete classes*

EKSPERIMENTALNA ANALIZA PLOČA OD NATUR BETONA KROZ ANALIZU SLIKA

Rezime: *U radu je prikazana uporedna analiza osam betonskih ploča pripremljenih u laboratorijskim uslovima. Ploče su formirane korišćenjem oplata različitog kvaliteta, negovane tokom 1 i 7 dana, i ugrađivane kompaktiranjem u trajanju od 7 i 10 sekundi. Poređenje svojstava natur betona sprovedeno je kroz analizu površine pora različitih prečnika, korišćenjem softvera za analizu slika. Rezultati pokazuju da period nege nije imao značajan uticaj na formiranje pora na površini betona, dok su tip oplate, kao i trajanje kompaktiranja imali izraženiji uticaj.*
Ključne reči: *natur beton, površina pora, prečnik pora, klase natur betona*

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1. INTRODUCTION

Although concrete is one of the most widely used building materials, it is not tipically considered as a material with aesthetic value. Nevertheless, advancements in concrete technology and construction techniques have led to the development of concrete structures that do not require an additional facade. In a broader sense, exposed concrete refers to any concrete surface that is not subsequently plastered or covered with another material. In a narrower sense, it refers to exposed concrete surfaces that meet specific criteria regarding color uniformity, surface roughness, surface damage, pores, stains, etc [1].

According to the DBV Guidelines for Exposed Concrete, published by the German Concrete and Construction Technology Association and the Association of German Cement Manufacturers [2], four classes of exposed concrete are defined, designated as SB1, SB2, SB3 and SB4. Concrete that falls under classes SB1 and SB2 is considered standard exposed concrete, while classes SB3 and SB4 represent higher-quality surfaces with stricter requirements in terms of placement precission, color consistency and surface appearance.

In order to determine which class of exposed concrete has been achieved, additional subclasses are introduced [2,3]. These subclasses refer to the following surface properties of concrete:

- texture (classes T1, T2 and T3),
- porosity (classes P1, P2, P3 and P4),
- color uniformity (classes FT1, FT2 and FT3),
- flatness (classes E1, E2 and E3),
- formwork type and quality (classes SHK1, SHK2 and SHK3),
- construction and formwork joints (AF1, AF2, AF3 and AF4).

As it can be concluded from these subclasses, the design and construction of exposed concrete surfaces is a complex task, influenced by many factors that can affect both the quality and final appearance of the concrete. These factors are related to component materials applied, types of formwork used, production, transportation and placement of fresh concrete, cover thickness, environmental conditions, long-term degradation processes and human factor [4,5].

Among all these variables, this research focuses on the influence of formwork and fresh concrete placement on the properties of exposed concrete slabs. For concrete surfaces classified as SB4, the highest-quality formwork (class SHK3) is required. Such formwork must be free from internal vibration damage, as well as from mortar or concrete residues. Additionally, the condition of the formwork material and its ability to absorb water are also very important. In exposed concrete applications, each formwork panel is typically limited to a single use, to ensure optimal surface quality.

The presented research focuses on the number and pore area on the concrete surface, as some of the measurable parameters used to define the exposed concrete class. After sample preparation, image analysis was used to evaluate and compare these parameters.

2. MATERIALS AND METHODS

The experimental part of the work consisted of producing concrete slabs from a single concrete mix, placed in two types of formwork with different levels of damage and stored under varying conditions. The experimental setup is presented in Table 1, while photos of the formwork before concrete placement are shown in Figure 1. Phase I refers to production of first group of 4 slabs, while phase II refers to production of second group of 4 slabs, produced using same formwork. Both formwork types were divided into two sections using an EPS plate. In this way, a total of eight slabs were produced and tested.

	Phase I	Phase II
Formwork I	Placing of concrete: compaction through vibrations in duration of 10 s. Duration of curing in humid conditions: 7 days	Placing of concrete: compaction through vibrations in duration of 10 s. Duration of curing in humid conditions: 1 day
Formwork II		Placing of concrete: compaction through vibrations in duration of 7 s. Duration of curing in humid conditions: 7 days

Table 1. Short overview of experimental setup

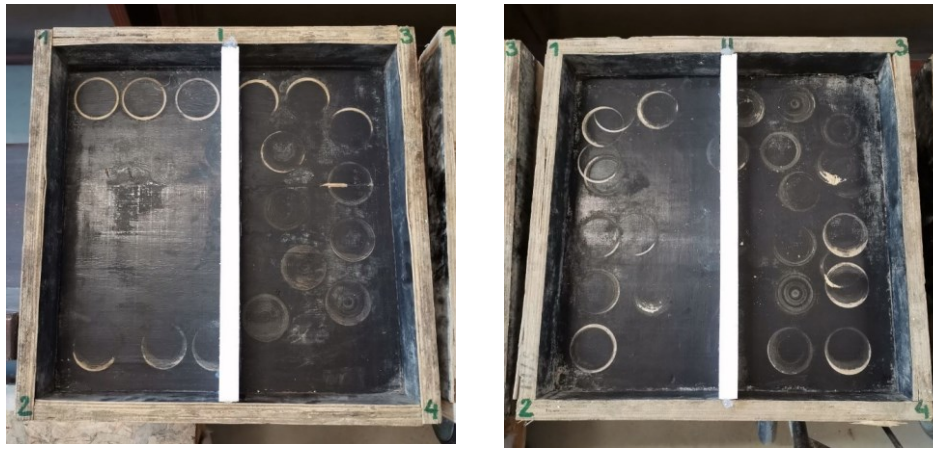


Figure 1. Formwork I (left) and formwork II (right)

The design of the concrete mixture used for slab production, per one cubic meter of concrete, is shown in Table 2. Cement CEM I 42,5R and river aggregate separated into three fractions were used. The compressive strength of the concrete was 34,4 MPa, calculated as the average value of two cube specimens, each measuring 10 cm in length.

Cement (kg/m^3)	Water (kg/m^3)	Aggregate (kg/m^3)		
		0/4 mm	4/8 mm	8/16 mm
360	190	850	435	400

Table 2. Design of concrete mix

Curing of the concrete slabs under humid conditions was carried out by covering them with wet material and plastic foil to prevent water evaporation. After the curing period (1 or 7 days), the slabs were stored under laboratory conditions (temperature 20±2 °C and relative humidity 50±10%) until reaching the age of 28 days. During this time, the concrete slabs were periodically photographed. Selected slabs were then cut into smaller samples in order to analyze surface damage and pores in more detailed manner. These samples were taken from the central part of each slab, as shown in Figure 2. The samples were then analyzed using an *Ash Omni digital microscope and measuring system* (Laboratory of materials, Faculty of Civil Engineering, University of Belgrade) along with *MIPAR image analysis software* [6] (Institute for Technology of Nuclear and Other Mineral Raw Materials). Photographs of the slabs were taken using a digital microscope under identical conditions (same lighting and magnification) and then analyzed, using *MIPAR* software, to assess the number and total pore area, which were categorized into three groups: pores with a diameter larger than 2 mm, pores with a diameter between 1 mm and 2 mm, and pores with a diameter smaller than 1 mm. Each photograph had a resolution of 1920x1080 pixels. Measurements revealed that a length of 10 mm in the concrete samples corresponded to between 124 and 127 pixels in each photograph. Since the variation between photographs was less than 3%, the same scale was applied for all measurements. The total area of each photograph was 13,486.56 mm^2.

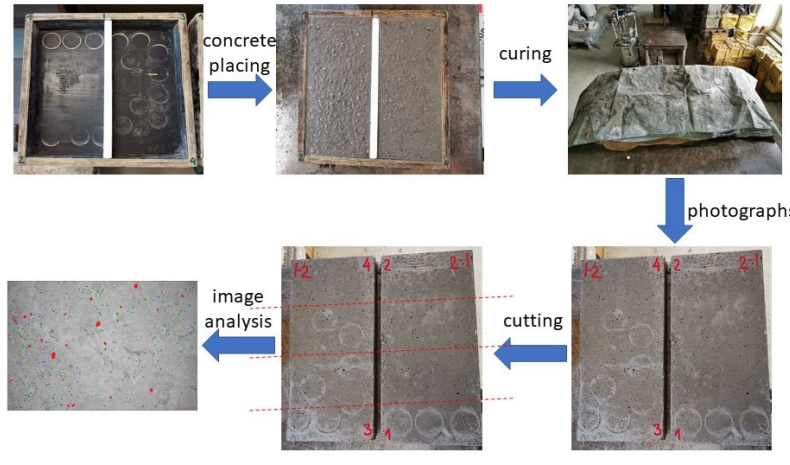


Figure 2. Preparation of samples for image analysis

3. RESULTS AND ANALYSIS

The slab produced using formwork I (left part) during phase I was considered the reference sample, as it was made with formwork that had minimal damage and was cured in humid conditions for 7 days. Photographs of this slab during the drying period are shown in Figure 3. As expected, color changes from darker to lighter shades of gray are observed, due to the drying process. The colors are uniformly distributed across the entire surface of the sample as it ages.

Images of the tested slabs after image analysis are shown in Figure 4. Red color indicates pores with diameters larger than 2 mm, green represents pores with diameter between 1 mm and 2 mm, and blue corresponds to pores with diameter smaller than 1 mm. The distribution of pores with different diameters, based on their number and total area within the observed surface, is presented in Figures 5 and 6. The data presented in these diagrams were obtained by summing the values measured from the two halves of the same slab.

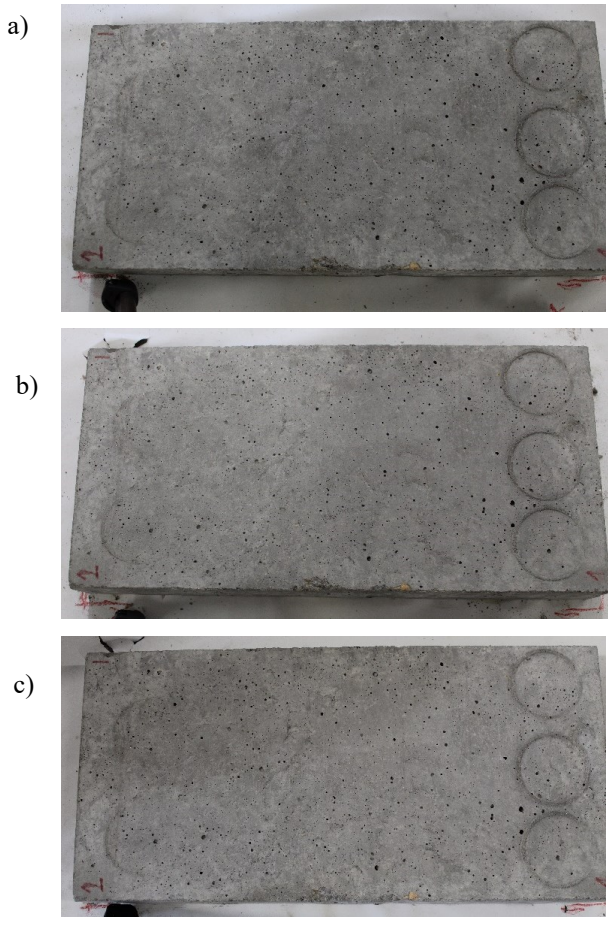


Figure 3. Slab designated as I-1: a) at the age of 8 days, b) at the age of 14 days and c) at the age of 22 days

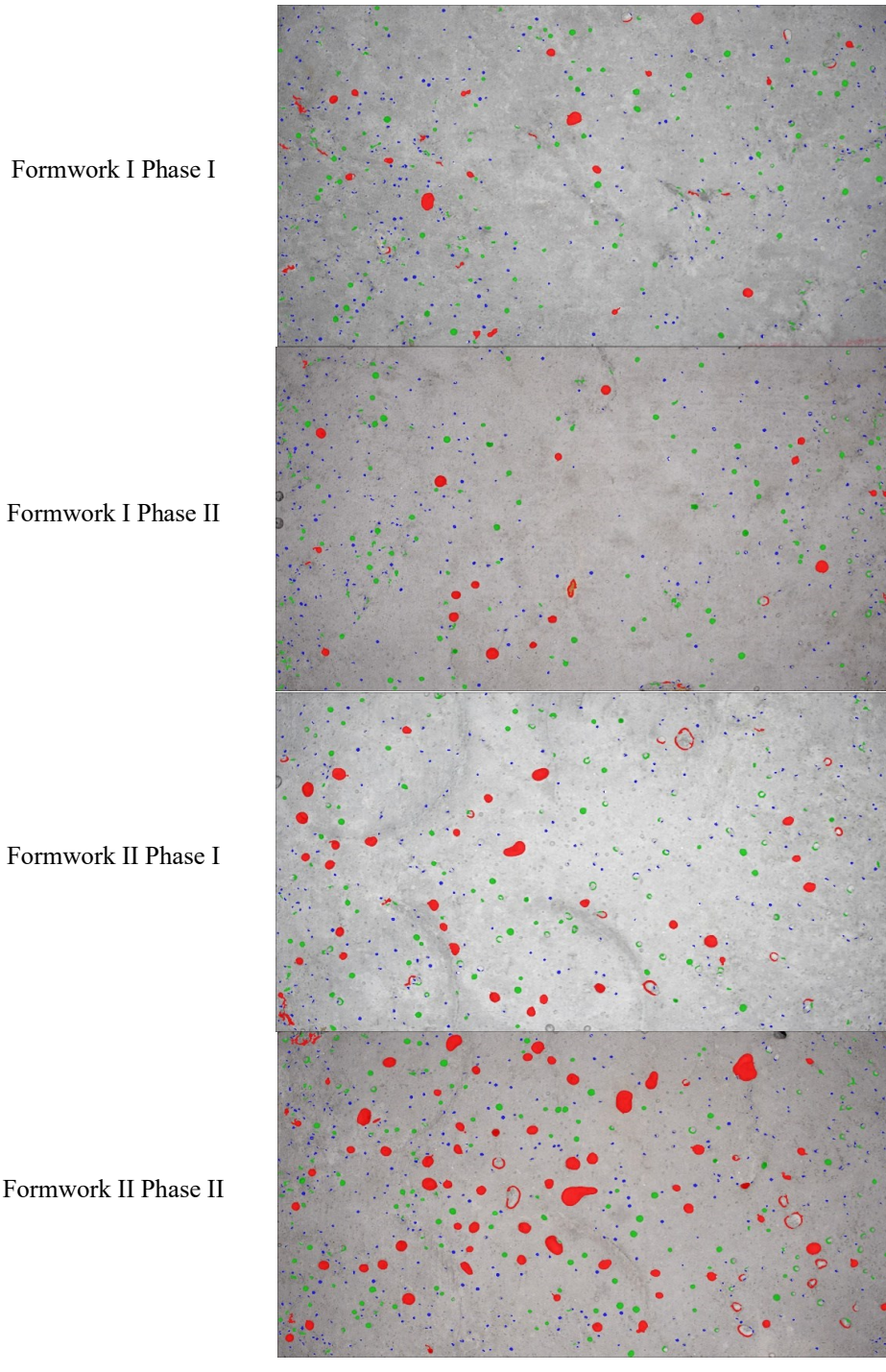


Figure 4. Image analysis of the tested slabs

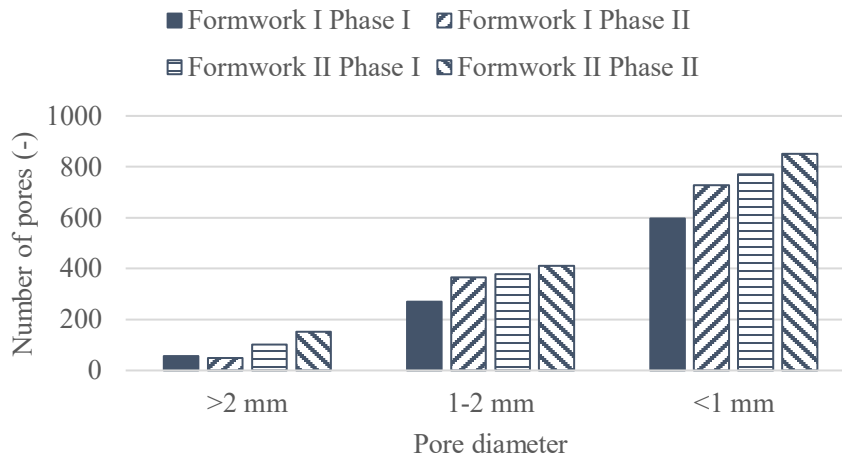


Figure 5. Distribution of pores according to their number

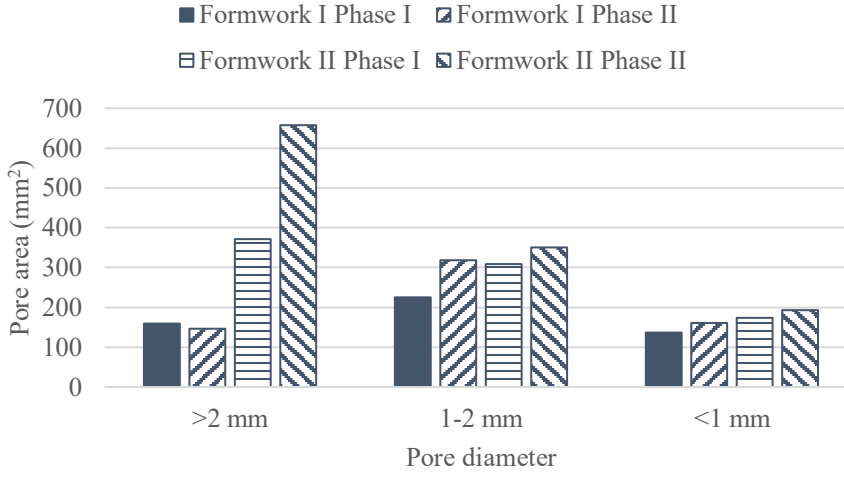


Figure 6. Distribution of pores according to their total area

The porosity class of exposed concrete is defined on a representative surface of 500 mm × 500 mm. Since it was not possible to analyze this entire surface area, the conditions regarding the surface area of pores larger than 2 mm were calculated based on the percentage of surface area they occupy. Therefore, the pore area for the sample placed in formwork I and cured for 7 days occupied 0.59% of the total surface area, while for the 1-day curing period, it occupied 0.54%. In both cases, it is classified as porosity class P3, since the surface area of pores with diameters larger than 2 mm is below 0.6%. For the

samples placed in formwork II, the pore surface area exceeded the allowed limit, even for the lowest porosity class P1, for both compaction durations.

When comparing the results within each group, it can be concluded that the curing period had little effect on the distribution of pores on the concrete surface. The number of pores larger than 2 mm was higher for the sample cured for 7 days, while the number of smaller pores was greater for the 1-day curing period. The pore area for each group of pores was similar in both cases, with the greatest difference observed in the pores with diameters between 1 mm and 2 mm.

The differences in compaction duration led to an increase in both the number and total area of pores in the concrete slabs placed in formwork II during phases I and II. The effect was most pronounced on pores larger than 2 mm, with their number increasing by 50% and their total area increasing by 75%.

When comparing concrete slabs from formwork II with those from formwork I, it is evident that the type of formwork significantly influences the surface porosity of exposed concrete. As already stated, although the samples were prepared under the same conditions and on the same day, the samples from formwork II in phase I showed a 77% increase in the number of pores larger than 2 mm, a 40% increase in the number of pores with diameter between 1 mm and 2 mm, and a 28.7% increase in the number of pores smaller than 1 mm.

As shown in Figure 4, the image analysis did not account for all of the pores, as some pores had open borders, and therefore their areas were not calculated. However, their number was significantly higher in the samples from formwork II, where larger pores were present in greater quantities.

4. CONCLUSION

The presented research focused on analyzing the influence of formwork, curing period and compaction quality on the properties of exposed concrete. Since the surface area of pores larger than 2 mm is one of the measurable parameters used to assess the quality of exposed concrete, image analysis of the pore distribution on the concrete surface was used to compare different samples.

The results show that the curing period had minimal impact on the surface area of larger pores. However, the condition of the formwork had a significant effect on pore distribution, as the number of pores and their total area increased by more than 50% when more damaged formwork (II) was used. As a result, the same concrete mix failed to fulfill the requirements for the lowest porosity subclass when placed in this type of formwork.
Image analysis has demonstrated good potential for determining the properties of exposed concrete, provided that the image preparation is done correctly. In the future, image analysis could also be used to detect discolorations on the concrete surface.

Acknowledgement

This research was funded by the Ministry of Science, Technological Development and Innovation of the Republic of Serbia and the University of Belgrade – Faculty of Civil Engineering (contract number 200092).

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
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XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

Slobodan Ranković[1]
Andrija Zorić[2]
Branko Ranković[3]
Mladen Milinković[4]

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Stručni rad

TESTING OF SPORT EQUIPMENT - SUSPENDET BASKETBALL BACKSTOP BY TRIAL LOAD

Summary: *The paper presents the results of testing a sports equipment structure (suspended basketball backstop) using trial loading. The structure was examined under the influence of static and dynamic loads in accordance with the standard SRPS U.M1.047, and following a testing protocol developed in line with the standard for playground equipment – basketball equipment (SRPS EN 1270). The tests on the foldable structure of the suspended backstop were conducted on an actual (installed) structure mounted in a sports hall in Kraljevo. During the trial loading, measurements of global deformations (deflections) and local deformations (strains) of the tested structure were carried out.*
Keywords: *suspended basketball backstop, testing, trial loading*

ISPITIVANJE SPORTSKOG REKVIZITA - VISEĆEG KOŠA PROBNIM OPTEREĆENJEM

Rezime: *U radu su prikazani rezultati ispitivanja konstrukcije sportskog rekvizita (visećeg koša) probnim opterećenjem. Konstrukcija koša ispitivana je na uticaje statičkog i dinamičkog opterećenja u skladu sa standardom SRPS U.M1.047, a prema programu ispitivanja usvojenim u skladu sa standardom za opremu za igrališta – oprema za košarku (SRPS EN 1270). Ispitivanja rasklopive konstrukcije visećeg koša sprovedena su na realnoj (montiranoj) konstrukciji ugrađenoj u sportskoj hali u Kraljevu. Tokom probnog opterećenja vršena su merenja globalnih deformacija (ugiba) i lokalnih deformacija (dilatacija) ispitivane konstrukcije.*
Ključne reči: *viseći koš, ispitivanje, probno opterećenje*

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1. INTRODUCTION

The suspended basketball backstop, manufactured by "Milinković Company" and designed for installation in school sports facilities, features a foldable mechanism that enables significantly improved space utilization.

The entire backstop structure (Figure 1) is made of hollow rectangular and square profiles, dimensioned according to static calculations. The backstop structure consists of a main and an auxiliary girder. The main girder comprises two steel profiles (1) that are hinge-connected at their upper ends to the roof structure of the sports hall. At the hinge connection, the profiles are spaced 3.5 meters apart. In the "working" position (i.e., during play), the girder is positioned at a 40-degree angle relative to the vertical axis. A vertical frame (5), to which the backboard and hoop are attached, is mounted at the lower end of the main girders. In the lower part, extending from the truss to the upper part of the girders, the steel profiles (1) are reinforced with diagonal braces (2) and struts (4). The two main steel girders are interconnected with a strut (3) located two-thirds of the span from the upper hinge. This strut includes hinge supports spaced 2.0 meters apart, forming the connection to the auxiliary girder. A lifting pulley is also mounted at the center of this strut. The auxiliary girder consists of four steel profiles (6, 6a) and two struts (7), forming a trapezoidal girder. At the upper end, the main profiles (6) are hinge-connected to the hall's roof structure (at a distance of 23 cm), while at the lower end, they are hinge-connected to the lower main girders (6a). A cable hook (8) is mounted at the midpoint of the strut (3). The structural analysis was performed using the "Tower" software, applying the standard EN 1270:2005 [3].



Figure 1. The suspended backstop structure in the unfolded ("working") position.

For the testing under trial loading conditions, the selected backstop structure was the one installed at the "Dimitrije Tucović" Elementary School in Kraljevo. The facility is a multipurpose standard-design hall with a span of 17 meters, featuring a structure composed of reinforced concrete arches and infill made of ferrocement elements, manufactured using the technology developed by "Milinković Company." The backstop structure and the mechanism for deploying it into the unfolded ("working") position are shown in Figure 2.

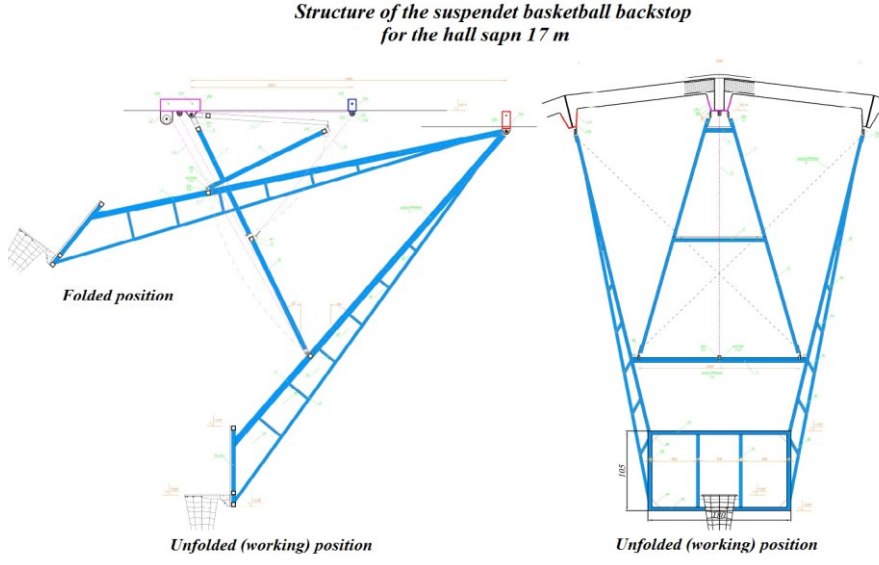


Figure 2. Structure of the suspended backstop and the mechanism for deploying it into the unfolded ("working") position.

2. TESTING OF THE SUSPENDED BACKSTOP STRUCTURE UNDER TRIAL LOADING CONDITIONS

2.1. Methodology of testing under trial loading conditions

The testing of the suspended basketball backstop structure, which represents basketball equipment, under trial loading conditions was carried out in accordance with the method outlined in SRPS U. M1. 047:1987. [2]. The testing program is adopted that the intensity and position of the trial load should be in compliance with the applicable standard addressing this issue, SRPS EN 1270:2009. [3].

The sports equipment – the suspended backstop structure – was tested to assess its rigidity and deformability. The rigidity test was conducted by applying a vertical force of 1050 N to the outer side of the hoop. The deformability test involved simultaneous application of a gravitational force of 3200 N at the root of the hoop, a horizontal lateral force of 900 N on the hoop, and a horizontal perpendicular force on the backboard of 900 N (Photo 1). The duration of the loading was longer than the prescribed 1 minute and 10 seconds.

For the application of trial static loads, weights with individual masses of 5 kg, 10 kg, 15 kg, 20 kg, and 25 kg were used, as well as a tensioner ("Flaschenzug" up to 1000 kg).

In the rigidity test, the vertical force was applied by "hanging" a person weighing 110 kg on the outer side of the hoop (Photo 7). For the deformability test, the vertical force was applied via a platform suspended by ropes at the root of the hoop, using weights totaling 320 kg. The horizontal lateral force was applied by attaching a cable to the hoop, which was then loaded with gravitational weights of 90 kg through a pulley fixed to the reinforced concrete arch of the hall. The horizontal force acting

perpendicularly to the backboard was applied by attaching a cable to the hoop and bracing it against a column on the opposite side of the court at the height of the hoop, using the "Flaschenzug." The intensity of this force was measured with an electronic dynamometer. The loads were applied in 4 (3) equal steps.

In addition to the static analysis, a dynamic analysis was also performed, which involved determining dynamic parameters (frequency and damping). The dynamic excitation was caused by suddenly releasing a person weighing 110 kg from the hoop (simulating a "dunk").

2.2. Conditions for Verifying the Rigidity and Stability of the Suspended Backstop Structure

The verification of the rigidity and stability of the suspended backstop is carried out in accordance with SRPS EN 1270: 2009. [3]. The loading scheme is shown in Figure 3, while the layout of the instruments used for determining the mechanical characteristics that define the rigidity and stability of the tested structure is presented in Figure 4.

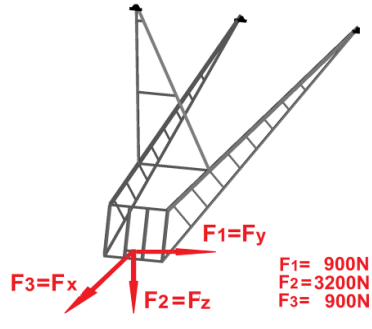


Figure 3. Trial Loading Scheme.

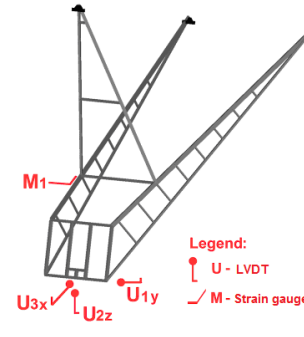
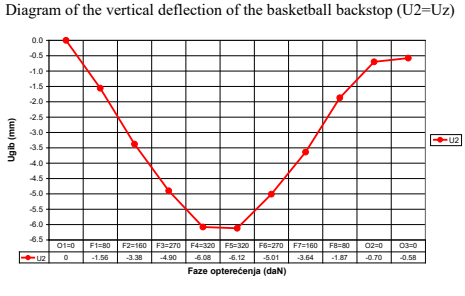
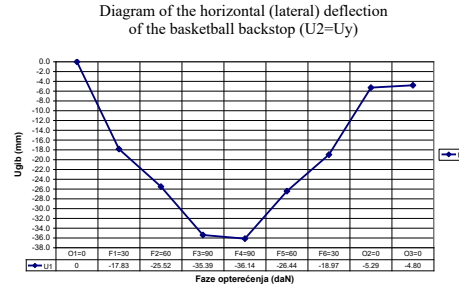


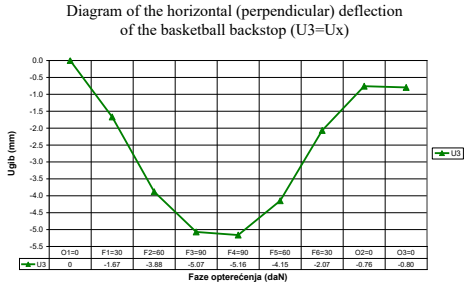
Figure 4. Measurement Instrument Layout.

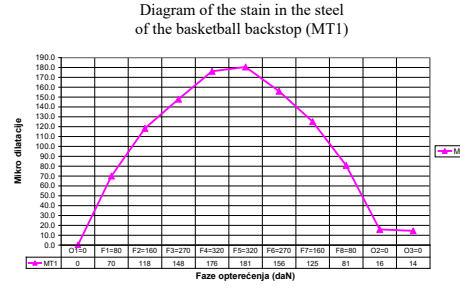
2.3. Results of Static Load Testing

The values of displacements in three orthogonal directions and the strains in the steel of the most heavily loaded structural member are presented as a function of the applied load through a diagram.





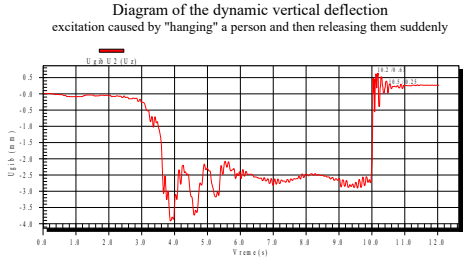


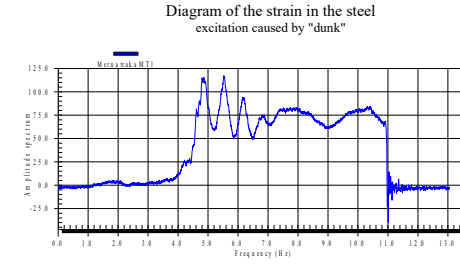


2.4. Results of Dynamic Load Testing

The dynamic characteristics of the suspended backstop structure were determined based on excitation caused by "hanging" a person weighing 110 kg and then releasing them suddenly, simulating a "dunk." The dynamic vertical deflection and strain in the structure are presented in the form of diagrams.

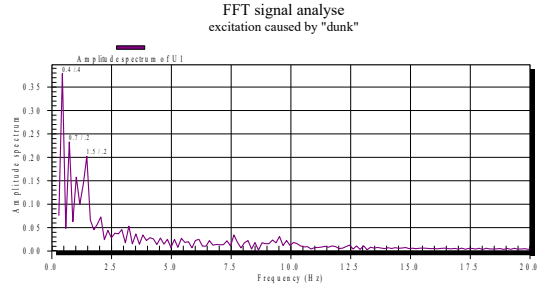
- Diagram Representation of Dynamic Deflection and Strain in the Structure





Logarithmic Decrement: $\delta = \frac{1}{n} \cdot \ln \frac{A_0}{A_n} = \frac{1}{3} \cdot \ln \frac{0,63}{0,25} = 0,308$ (1)

- FFT Signal Analysis from Vertical Deflection



Dominant frequencies: f_1=0,4 Hz; f_2=0,7 Hz; f_3=1,5 Hz;

2.5. Analysis of the Results of the Conducted Tests

2.5.1. Hoop Testing:

The basketball hoop was statically loaded with a force of F1 = 1050 N on the outer part for a duration of 5 seconds. During this time, the mechanism casing had a clearance of less than 8 mm, while the tilt of the hoop was less than 30°, as required by the standard [3].

2.5.2. Testing of the Structure's Rigidity:

For testing the rigidity of the structure in an enclosed space, in accordance with the same standard [3], forces F1 = 900 N and F3 = 900 N were applied for a duration of 1 minute and 10 seconds.

2.5.3. Testing of the Structure's Stability:

During the stability testing of the structure, forces were simultaneously applied in three orthogonal directions, and deformations (maximum and residual displacements) were measured. The results are presented in the following table:

Testing of the Structure's Stability			
Forces (N)	**Displacements (mm)**	**Residual displacements (<10 mm)**	**% residual displacements (<15%)**
Horizontal lateral F1=Fy=900 N,	U1=36,14 mm (Uy)	U1y=4,80 mm	13,3%
Vertical (gravitational) F2=Fz=3200 N	U2= 6,12 mm (Uz)	U2z=0,58 mm	9,4%
Horizontal perpendicular (on the backboard) F3=Fx=900 N	U3= 5,16 mm (Uy)	U3x=0,70 mm	13,5%

Table 1. Displacements during the stability testing of the structure.

2.5.4. Measured Local Deformations (Strains - Stresses):

The value of the maximum measured strain, or the induced stress in the steel of the truss structure, was $\varepsilon=181\cdot10^{-6}$ ($\sigma=380$ daN/cm^2) $< \sigma_{rač} < \sigma_{doz}$

The values of the dynamic parameters: (frequency f = 0,4-1,5 Hz; and logarithmic decrement δ = 0,31) were within the allowable limits.

Upon inspecting the structure before and after loading, no anomalies or damage were observed on the steel elements and connections due to the applied load.

3. CONCLUSIONS

The paper presents an example of experimental testing of the load-bearing capacity and usability of a specific structure, a sports equipment, the suspended basketball backstop. Through trial loading of the structure, it was necessary to determine the mechanical characteristics in terms of deformations (displacements) in three orthogonal directions and stresses in the most heavily loaded structural elements. Based on the obtained measurement results and their analysis, conclusions can be drawn regarding the state of the structure in terms of rigidity and stability. It was concluded that the specific structure can withstand the load prescribed by the standards, meaning that it is a technically correct structure.

4. PHOTO DOCUMENTATION



Photo 1. Trial Loading on the Tested Suspended Basketball Backstop Structure.

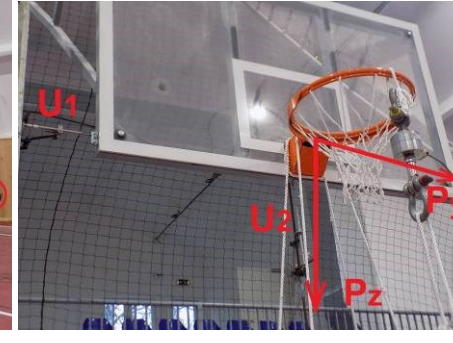


Photo 2. Position of the deflectometers U1 and U2 and directions of the applied trial load



Photo 3. Position of the deflectometer U3 and dynamometer, and directions of the applied trial load



Photo 4. Position of the strain guage M1 on the suspended basketball backstop structure.



Photo 5. Position of the deflectometer U1.



Photo 6. Position of the deflectometer U2





Photo 7. Load on the basketball hoop with a mass of Q = 110 kg (F1 = 1100 N).

Acknowledgements

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XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
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BENCHMARKING CIRCULARITY OF BUILDINGS ACROSS EUROPEAN COUNTRIES

Summary: *The transition towards a circular economy in the built environment requires effective benchmarking frameworks to evaluate and compare circularity performance of buildings. However, existing benchmarking systems for assessing circularity performance in buildings remain underdeveloped. This paper presents a brief literature review on benchmarking circularity of buildings across European countries at two different levels: (1) building materials and products, and (2) buildings and the built environment. The review identifies key gaps and challenges and highlights the need for harmonized and comprehensive benchmarking approaches, based on robust and interoperable datasets, to support informed decision-making in circular construction.*
Keywords: *circular economy, benchmarking, building materials, construction products, buildings, built environment, sustainability.*

EVALUACIJA CIRKULARNOSTI ZGRADA U EVROPSKIM ZEMLJAMA

Rezime: *Prelazak ka cirkularnoj ekonomiji u izgrađenom okruženju zahteva efikasne okvire za merenje i upoređivanje performansi cirkularnosti zgrada. Međutim, postojeći sistemi za evaluaciju (benčmarking) cirkularnosti u zgradama još uvek nisu dovoljno razvijeni. Ovaj rad predstavlja kratak pregled literature u oblasti procene i poređenja cirkularnosti zgrada u evropskim zemljama na dva različita nivoa: (1) građevinski materijali i proizvodi i (2) zgrade i izgrađeno okruženje. Pregled identifikuje ključne nedostatke i izazove, i ističe potrebu za razvojem usklađenih i sveobuhvatnih sistema evaluacije, baziranih na robusnim i interoperabilnim bazama podataka, koji će podržati donošenje informisanih odluka u cirkularnoj gradnji.*
Ključne reči: *cirkularna ekonomija, evaluacija i poređenje performansi (benčmarking), građevinski materijali i proizvodi, zgrade, izgrađeno okruženje, održivost.*

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1. INTRODUCTION

The construction sector is one of the largest consumers of energy and raw materials worldwide, and accounts for over 35% of the European Union's (EU) total waste generation, 50% of all raw materials consumed annually and 5-12% of total greenhouse gas (GHG) emissions. Across this sector, buildings are responsible for 42% of total energy consumption, 35% of energy-related GHG emissions, and 33% of all materials consumed annually [1], which highlights the urgent need for a transition towards a more sustainable and circular built environment.

The concept of a Circular Economy (CE) (Figure 1) offers a strategic framework to address these challenges, and the EU has recognized the leading role of CE in the construction industry in achieving sustainability goals.

Various CE definitions are being stated which sometimes leads to ambiguity when attempting to establish a clear pathway toward its primary objectives. *CE* and *Sustainability* are two overlapping concepts that share the same goal of preserving environmental capital, generating economic benefits, and enhancing social welfare [2]. However, the transition to a CE often involves trade-offs between these three dimensions of sustainability, potentially creating unintended burdens instead of delivering holistic benefits [3]. Consequently, the development of circularity assessment models requires a narrow scope which focuses mainly on CE progress in accordance with its fundamental principles.

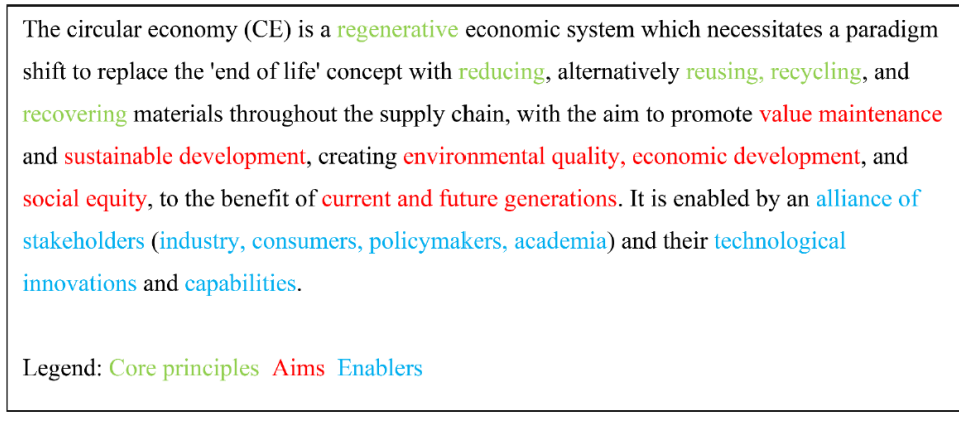


Figure 1. The concept of circular economy [4]

EU policies, such as the *European Green Deal* [5] and the *Circular Economy Action Plan* [6], set a high goal level – to establish a fully CE by 2050. The tool for achievement of this goal in construction sector is a set of regulatory measures: *EU Taxonomy* [7], *Construction Product Regulation* (CPR) [8], and *Energy Performance of Buildings Directive* (EPBD) [9]. However, there is still a long way to go to the set goal due to the lack of harmonized methodologies and standardized indicators for evaluation of circularity – key elements that are necessary to enable informed decision making within the construction industry. Furthermore, the ability to benchmark circularity of buildings remains underdeveloped, despite growing interest in CE principles.

This paper presents a brief literature review on benchmarking circularity of buildings across European countries at two different levels: (1) building materials and products, and (2) buildings and the built environment. The review identifies key gaps and

challenges and highlights the need for harmonized and comprehensive benchmarking approaches, based on robust and interoperable datasets, to support informed decision-making in circular construction.

2. BENCHMARKING CIRCULARITY OF BUILDINGS IN EUROPE

The transition towards a CE in the built environment requires effective benchmarking frameworks to evaluate and compare the circularity performance of buildings. The circularity of buildings can be assessed using a variety of indicators, defined as quantitative or qualitative factors or variables that provide a simple and reliable way to measure achievements, reflect changes related to implementation, or help assess the results achieved [10].

Circularity indicators are typically classified into three levels: micro, meso, and macro. These indicators support the evaluation of buildings and construction products across the three main life cycle phases – construction, use, and end-of-life. Although circularity indicators in the construction sector are still evolving and lack standardization, building materials and components can be generally assessed at the micro level, while buildings are assessed at meso level (Figure 2) [11].

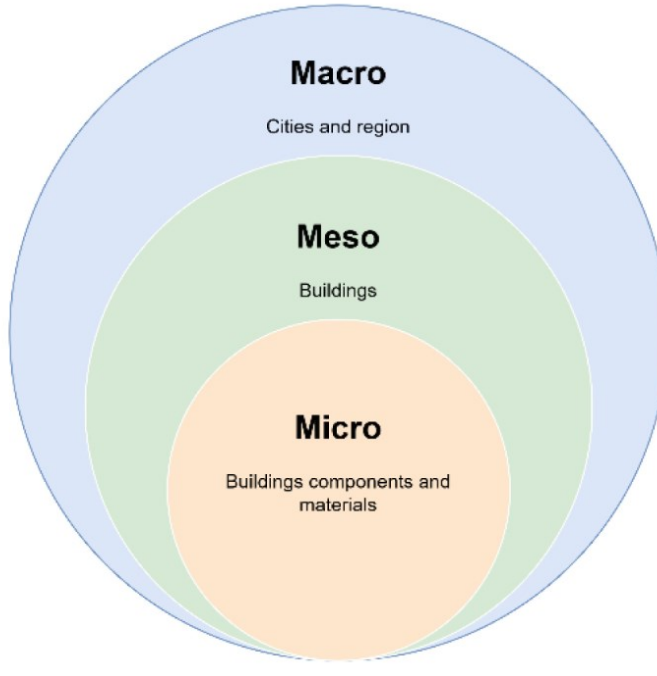


Figure 2. Circularity indicator levels [11]

2.1. Circularity of building materials and construction products

With the construction sector under growing pressure to reduce its environmental impact, benchmarking the circularity of building materials and construction products is increasingly recognized as a key step toward sustainable building design and renovation.

Among existing approaches, *Material Circularity Indicator* (MCI), developed by the Ellen MacArthur Foundation [12] is considered as the most used circularity indicator in the built environment [13]. MCI has proven to be a useful tool for the quantitative assessment and comparison of the circularity of building materials and construction products. It is based on three main parameters: (1) the amount of virgin material, (2) the product utility (lifespan) and (3) the amount of unrecoverable waste generated (Figure 3) [14].

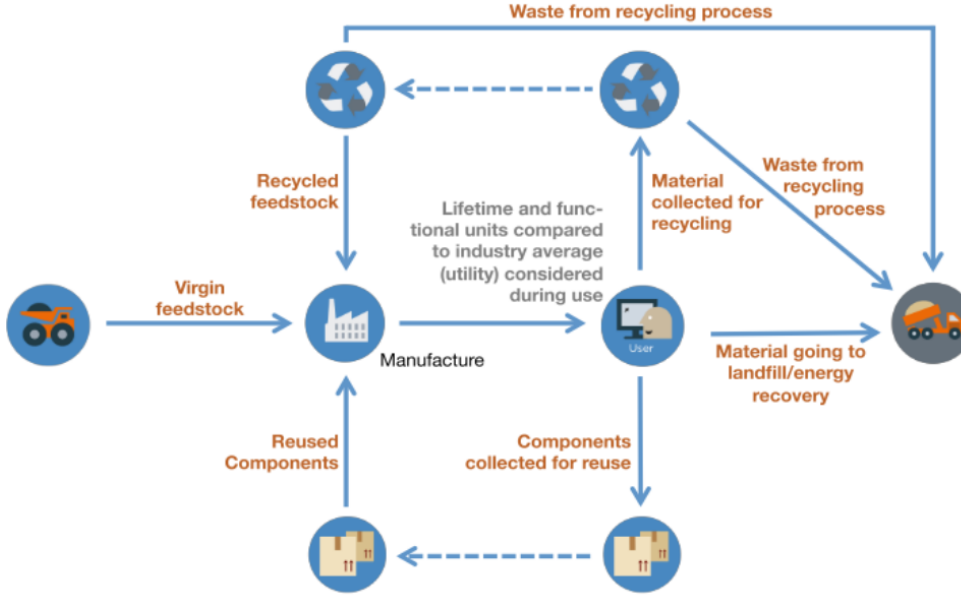


Figure 3. Material circularity indicator – representation of material flows [12]

MCI has been utilized in several studies to benchmark material circularity in practice [14-16]. MCI framework was applied for benchmarking of carbon footprint and circularity in production process of two widely used thermal insulation materials – stone wool and extruded polystyrene, within the Greek market context. The paper [15] highlights the importance of integrating carbon footprint and circularity assessments when choosing building materials to enhance overall sustainability outcomes.

Similarly, adapted MCI methodology was used to benchmark the circular performance of 89 construction products, including insulation materials, flooring systems, and wall components, using Life Cycle Assessment (LCA) data available in German ÖKOBAUDAT database. The study [16] emphasizes the significance of harmonized environmental datasets and transparent indicator frameworks in supporting comparative benchmarking across a wide range of construction materials, as well as the necessity of integrating digital data infrastructures to enable consistent and scalable circularity evaluations at the product level.

The application of MCI has also been expanded within the *DRIVE 0* project (2019-2023) [14], where it was employed to benchmark the circularity of various construction products and materials used for deep renovations in seven demonstration buildings across Europe (Netherlands, Estonia, Italy, Ireland, Spain, Greece and Slovenia). The benchmarking process facilitated the identification of solutions with high potential for reuse, recycling, and design for disassembly (DfD), thus supporting circular renovation strategies.

2.2. Circularity of buildings and built environment

While material- and product-level assessments provide valuable insights into circularity potential, achieving a truly circular built environment requires evaluation at the scale of entire buildings and urban systems. Several frameworks have been developed to guide this broader assessment.

LCA methodologies play a critical role in evaluating the environmental impacts associated with building materials, products, and buildings. Integrating LCA data into circularity assessments allows for a more comprehensive understanding of material flows and environmental performance. LCA provides the analytical foundation for many benchmarking systems, including *Level(s)*, a European framework providing a common language for assessing sustainability across a building's lifecycle [17]. Level(s) addresses six sustainability performance macro-objectives, including objective 2: Ensuring resource efficient and circular material lifecycles, featuring four indicator areas, designed to assess the most important opportunities for improving resource efficiency and circularity: *(1) Bill of quantities, materials and lifespans*; *(2) Construction and demolition waste*; *(3) Design for adaptability and renovation* and *(4) Design for deconstruction*.

To facilitate the integration of circularity strategies into building design, construction, and operation phases, decision making process should be supported by *Building Information Modeling* (BIM) tools [18, 19]. Measurement of building circularity often focuses on key principles such as adaptive reuse [20], DfD and adaptability. DfD in construction represents a sustainable design strategy that focuses on creating buildings and structures that can be easily dismantled at the end of their lifecycle [21]. DfD enables recovery, reuse, and recycling of materials and components, reducing waste and environmental impact. By incorporating reversible connections, modular designs, and traceable materials, DfD supports circular economy principles, extends the lifecycle of building materials, and promotes flexibility for future modifications or relocations. Furthermore, maintainability is also recognized as an important aspect of overall building circularity [22].

Emerging digital technologies are pivotal in accelerating the transition toward circular building design. The development of tools such as *Material Passports* and BIM 3D design tools, which contain an extensive database of components and materials, can boost the high-quality reuse of components and materials, and even replace current municipal archives in the long run [23]. Notably, the *Buildings as Material Banks* (BAMB) project (2015-2019) played a key role in advancing concepts such as material passports and reversible building design, further reinforcing the foundation for digital circularity strategies in the built environment [24].

Another emerging tool supporting circular renovation strategies is the *Building Renovation Passport* (BRP) which, alongside *Digital Building Logbooks* (DBL), is helping

to drive the circular economy in building retrofits. The European Commission promotes the development and adoption of BRPs and DBLs as part of its *Renovation Wave* strategy to accelerate the transition towards a decarbonized and circular building stock [25, 26]. In line with these European efforts, an original DBL version, specifically designed to address the unique needs of Serbia, was proposed as an innovative decision-support tool for building renovation management. This version of DBL functions as a comprehensive repository, incorporating crucial information on a building's performance, such as energy efficiency, durability, and fire safety, and offers insights into potential improvement measures [27].

Adopting digital tools to create digital twins, ie. identical digital models of physical buildings, will make data available to aid decision making for the future disassembly and reuse of components in a building when it reaches its end-of-life phase. Additionally, the combination of new digital technologies with advanced analytics can provide critical guidance on materials and parts with high replacement rates and therefore make buildings last longer.

In the European context, innovative applications of digital technologies for circularity assessment are already emerging. For instance, a recent case study, developed in Belgrade, Serbia, proposed and proved the applicability of a *CityGML-based methodology* for mapping the existing residential building stock and calculating *Material Intensity Coefficients* (MICs) at an urban scale [28]. This methodology aligns with the objectives of the EU Taxonomy for Sustainable Activities, supports DBLs, and is compatible with the Level(s) framework, offering additional applications in decarbonization strategies and disaster recovery planning.

The building stock could also be considered for urban mine - evaluate the materials stocks of the built environment in an urban mining (UM) perspective, to assess their different potentialities in terms of materials and recovery availability, for reuse or recycling scopes [29].

3. IDENTIFIED GAPS AND CHALLENGES

Despite growing efforts to develop circularity assessment frameworks for building materials, products, and entire buildings, several methodological and practical gaps remain:

1. *Lack of standardized and harmonized methodologies*

Although multiple frameworks and indicators exist, there is no universal standard for benchmarking circularity in buildings. Differences in scopes, metrics, and levels of assessment lead to inconsistent results, making cross-study or cross-project comparisons difficult.

2. *Limited coverage of circularity dimensions*

Most existing circularity assessment tools primarily focus on material flows and end-of-life strategies, neglecting broader dimensions. For example, while MCI effectively captures material inputs and outputs, it does not address economic or social aspects.

3. *Limited access to reliable and harmonized data*

The availability of robust, harmonized material databases is critical for accurate circularity benchmarking. However, many regions lack comprehensive and standardized

datasets. Although initiatives like ÖKOBAUDAT in Germany or DRIVE 0 at the European level provide valuable data, similar infrastructures are absent or fragmented in many countries, limiting the scalability and comparability of benchmarking efforts.

4. Insufficient integration of digital tools

Despite the recognized potential of digital tools such as BIM models, material passports, DBLs, BRPs and digital twins, their practical implementation in circularity benchmarking remains limited. Fragmented digital infrastructures and lack of interoperability between systems hinder the systematic tracking of material flows, disassembly potential, and future reuse opportunities.

5. Fragmented consideration of the building lifecycle

Circularity assessments often focus predominantly on the end-of-life phase, while overlooking critical opportunities during the operation, maintenance and renovation phases.

Identifying these gaps highlights the urgent need for further methodological development, harmonization of indicators, enhanced data infrastructures, and deeper integration of circular economy principles throughout the entire building lifecycle.

4. CONCLUSION

This paper reviewed current approaches to benchmarking circularity in the built environment, focusing on building materials, construction products, and entire buildings. The analysis highlighted the growing relevance of circularity assessment in the transition towards sustainable construction practices, while also revealing critical gaps related to methodological fragmentation, limited dimensional coverage, lack of reliable data, and insufficient integration of digital tools.

Although tools such as the MCI and Level(s) offer valuable foundations, there remains a strong need for standardization, comprehensive lifecycle coverage, and harmonization across methodologies. Digital technologies, including material passports, BRPs, DBLs and digital twins offer significant opportunities for enhancing circularity benchmarking but require broader adoption and interoperability.

Future research and practice should prioritize the development of comprehensive, interoperable, and user-friendly frameworks that support reliable circularity benchmarking and facilitate informed decision-making for a circular and sustainable built environment.

Acknowledgement

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

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CHANGES IN SOIL STRUCTURE DURING CHEMICAL STABILIZATION BY ADDING DIFFERENT REAGENTS

Summary: *Soil stabilization represents a very important technique in geotechnical engineering aimed at enhancing the mechanical and physical properties of soil. Among the various approaches, chemical stabilization stands out as an effective method for achieving permanent improvement of soil properties, utilizing different additives that react with soil particles and alter its (micro)structure. This paper presents an analysis of the impact of various chemical stabilizers (traditional and alternative) on soil structure modification, with a particular focus on their efficiency, mechanisms of action and long-term effects.*
Keywords: *Soil stabilization, pH value, FTIR spectroscopy, XRF analysis, SEM analysis*

PROMENE STRUKTURE TLA PRI HEMIJSKOJ STABILIZACIJI DODAVANJEM RAZLIČITIH REAGENASA

Summaryt: *Stabilizacija tla predstavlja izuzetno važnu tehniku u geotehničkom inženjerstvu kojom se poboljšavaju mehaničke i fizičke karakteristike tla. Među različitim metodama, hemijska stabilizacija se ističe kao efikasan način za trajno poboljšanje karakteristika tla, pri čemu se mogu koristiti različiti aditivi koji reaguju sa česticama tla i menjaju njegovu (mikro)strukturu. U radu je data analiza uticaja različitih hemijskih stabilizatora (tradicionalnih i alternativnih) na promene strukture tla, sa fokusom na njihovu efikasnost, mehanizme delovanja i dugoročne efekte.*
Ključne reči: *Hemijska stabilizacija tla, pH vrednost, FTIC spektroskopija, XRF analiza, SEM analiza*

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1. INTRODUCTION

The addition of an optimal quantity of stabilizing agents enhances soil strength, stability and load-bearing capacity, simultaneously reducing its plasticity, permeability and susceptibility to swelling. Traditional chemical stabilizers used across various soil types are well established in practise and widely applied. This group of additives includes lime and cement (commercial calcium based products), as well as fly ash, a byproduct of thermal power plants. The stabilization mechanisms of traditional additives include ion exchange, flocculation, pozzolanic reactions and carbonate cementation [3].

The most recognized theories explaining soil stabilization processes are Le Chatelier's crystallization theory and Taylor's gel theory, which are often applied in combination to more accurately describe the various stages of soil hardening [1]. Numerous studies indicate the high effectiveness of traditional stabilizers in improving expansive soils when used individually, without mixing with each other. However, their application may cause certain problems. One such issue is the reaction between lime and sulfates present in the soil, which can lead to excessive swelling and subsequent damage of the stabilized soil. Additionally, organic matter within the soil can interact with calcium ions, thereby affecting the stabilization potential. Furthermore, the production of cement carries significant environmental impacts, whereas carbonate reactions can lessen the long-term stability of treated soils. Due to these limitations, researchers are increasingly exploring alternative, sustainable and effective materials for stabilizing soils with insufficiently good natural properties.

This study presents an analysis of changes in the structure and fundamental chemical properties of clayey soil considering a variety of reagents – traditional (lime, cement and fly ash) and alternative (potassium hydroxide (KOH), magnesium carbonate ($MgCO_3$) and sodium silicate (Na_2SiO_3, commonly known as water glass)). Each of the selected additives has shown effectiveness in improving various soil characteristics, however, changes in the soil's internal structure or specific chemical properties have not been thoroughly examined so far [4,11]. The findings presented in this paper highlight structural transformations, assessed through pH value variation, FTIR spectroscopy, XRF analysis and SEM analysis.

2. MATERIALS USED IN THE RESEARCH

2.1. Clayey soil

The clayey soil used in this study was sourced from the Crvena Reka site, located in the southeastern region of the Republic of Serbia. It was sampled during landslide stabilization activities carried out as part of the construction of the E-80 highway.

The mineralogical composition of the natural clay soil was determined using X-ray diffraction (XRD) analysis, which is most commonly employed for the qualitative identification of mineral content. The spectrum shows that calcite is the most abundant mineral in the soil (39%), followed by quartz (25%) and clay minerals present at more than 10%: illite (11%), chlorite (10%) and montmorillonite (15%).

The physical and mechanical properties of natural clayey soil were determined by laboratory tests in the Laboratory for Geotechnics, Faculty of Civil Engineering and Geotechnics of Niš, and are shown in Table 1.

Soil property	Symbol (unit)	Value
Particle Density	Gs (–)	2.705
Grain Size Distribution	Gravel (%)	1.2
	Sand (%)	4.9
	Silt (%)	40.6
	Clay (%)	53.3
Coefficient of Uniformity	Cu (–)	8.0
Coefficient of Curvature	Cc (–)	2.0
Maximum Dry Density	MDD (g/cm^3)	1.903
Optimum Moisture Content	OMC (%)	18.5
Uniaxial Compressive Strength	UCS (kPa)	205
Liquid Limit	LL (%)	49
Plastic Limit	PL (%)	23
Plasticity Index	PI (%)	26
pH Value	pH (–)	9.5
Modulus of Compressibility	Mv (MPa)	12.945
California Bearing Ratio	CBR (%)	2.71
Swelling	s (%)	2.91
Cohesion	c (kPa)	16.9
Internal friction angle	φ (°)	16.5

Table 1. Properties of natural clayey soil

2.2. Stabilizing reagents

The stabilizing reagents analyzed in this study can be categorized into two main groups: traditional stabilizers (lime, cement and fly ash) and alternative stabilizers (potassium hydroxide (KOH), magnesium carbonate ($MgCO_3$) and sodium silicate (Na_2SiO_3)). Traditional stabilizers, which have been used in civil engineering and geotechnical applications for an extended period, are known to significantly enhance the mechanical properties of soil. In practice, these stabilizers have been successfully applied in over 80% of cases [6].

To determine the optimal lime content, samples of clayey soil were prepared with 3%, 5% and 7% additions of commercially available hydrated lime, designated as CL90S, with the following chemical composition: CaO (71.01%), MgO (3.05%) and SiO_2 (2.14%).

With regard to cement, HRB E2 was employed, with a chemical composition of CaO (51.73%), SiO_2 (15.20%), Al_2O_3 (5.13%), Fe_2O_3 (2.39%), SO_3 (1.87%) and MgO (1.02%). This type of cement binder is primarily used for the stabilization of coarse grained soils. Therefore, a higher percentage of cement was applied in the experiments compared to lime, specifically in quantities of 10%, 15% and 20%.

The fly ash used in this research was sourced from the Kostolac thermal power plant and had the following chemical composition: SiO_2 (56.38%), Al_2O_3 (17.57%), Fe_2O_3 (10.39%), CaO (7.46%) and MgO (2.13%). The effectiveness of soil stabilization with fly ash greatly depends on the soil's properties, the time elapsed since treatment, and

the moisture content at the time of application [10]. To determine the optimal fly ash content, clayey soil samples were prepared with 10%, 15% and 20% additions of fly ash.

Research in the field of chemical soil stabilization has demonstrated that traditional additives can be effectively replaced with stabilizing reagents based on alternative elements. This shift is primarily due to their high water solubility, which allows for more efficient mixing with the soil, and their ability to supply a sufficient amount of cations necessary for ion exchange processes. Upon contact with water, these stabilizers initiate both immediate and long-term chemical reactions with soil particles, leading to an improvement in the soil structure.

Magnesium carbonate has emerged as a sustainable option for enhancing the physical and mechanical properties of soils, particularly those with a high content of clay minerals. This additive has been shown to effectively increase cohesion, reduce plasticity and improve the bearing capacity of high-plasticity soils [9].

Sodium silicate, also known as water glass, has been utilized under specific conditions for soil stabilization and is considered an environmentally friendly and non-toxic material. Its benefits are especially notable in reducing soil permeability and increasing cohesion in fine-grained soils. For mixtures involving multiple additives, sodium silicate should be combined with the soil prior to the addition of other stabilizers [5]. In this study, the binding agent used was liquid sodium silicate SP 3.0–3.3, manufactured by Galenika MagmaSil, with a density of 1.38 g/cm³ and pH value ranging from 10 to 11.

Potassium hydroxide (KOH) is particularly effective in enhancing the load-bearing capacity and compactness of clay soils. The addition of KOH initiates chemical reactions that increase soil strength and reduce its sensitivity to moisture fluctuations. The K^+ ion enables additional compact and close polycondensation reactions, significantly increasing the long-term strength of alkali-activated soils. Previous studies have shown that compounds containing potassium ions tend to exhibit lower degrees of crystallinity and higher specific surface areas [7].

In this study, natural clayey soil was treated with 3%, 5% and 7% additions of each of the considered alternative reagents.

The effects of adding various inorganic reagents (e.g., $Ca(OH)_2$, KOH, KCl, etc.) can play a significant role in soil stabilization. When the soil comes into contact with a salt solution, cations (Na^+, K^+, Mg^{2+}, Ca^{2+}) present in the clay tend to migrate into the solution, where multivalent cations (e.g., Ca^{2+}, Al^{3+}) are capable of replacing relatively weaker cations (e.g., Na^+ or K^+) through a cation exchange process, as illustrated in Figure 1. Furthermore, with an increase in salt content, the ion concentration in soil pores rises, facilitating rapid flocculation of soil particles, thereby increasing the effective particle size. Simultaneously, the elevated cation concentration reduces interparticle repulsion, which leads to a decrease in the thickness of the diffuse double layer of the soil. This accelerates the entire process of cation exchange and ultimately promotes the stabilization of the soil matrix.

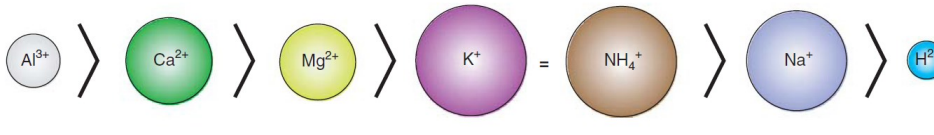


Figure 1. Tendency of cation exchange depending on valence [8]

3. RESEARCH METHODS AND RESULTS

3.1. pH value

High pH values in soil facilitate pozzolanic reactions between the reagents and the soil, leading to the formation of new compounds that contribute to soil binding and stabilization. For instance, when lime is added to clayey soil, calcium silicate hydrates (CSH) and calcium aluminate hydrates (CAH) are formed gel-like compounds with strong binding properties. The results of this research indicate that all reagents used in this study contributed to an increase or modification of the pH of the treated clayey soil (Table 2). A notable rise in pH value was recorded during the initial period (24 hours) following the mixing of the soil with the corresponding reagent, after which the values either remained stable or showed a slight decrease. Similar results were reported by leading researchers in their respective studies. The findings clearly demonstrate that, in addition to traditional additives, alternative stabilizers also possess the potential to induce long-term changes in the chemical properties of soil. The optimal reagent content refers to the proportion of reagent in the soil mixture that yields the greatest improvement and stabilization effect. The most favorable results are achieved at a soil pH of 12.4. For pH values exceeding 12.4, the crystalline structure of clay becomes unstable, reducing the efficiency of stabilization [2,3]. Based on the change in pH values, as well as representative geotechnical properties of the soil [4,11], next investigations were conducted with the optimal content of each of the examined reagents, specifically: 5% lime, 15% cement, 20% fly ash, 5% $MgCO_3$, 5% Na_2SiO_3 and 7% KOH.

Soil			**pH value**		
			After 1 day	**After 3 days**	**After 28 days**
Natural soil			9.5	-	-
Soil treated by the chemical stabilization technique	Lime ($Ca(OH)_2$)	3%	12.1	12.0	12.0
		5%	12.5	12.5	12.4
		7%	12.8	12.8	12.7
	Cement	10%	11.6	11.6	11.6
		15%	12.0	12.0	12.0
		20%	12.4	12.4	12.4
	Fly Ash	15%	11.2	11.2	11.0
		20%	11.6	11.3	11.3
		25%	12.0	11.8	11.6
	Magnesium Carbonate ($MgCO_3$)	3%	10.9	10.7	10.7
		5%	12.1	12.1	12.0
		7%	12.7	12.7	12.6
	Sodium Silicate (Na_2SiO_3)	3%	11.7	11.7	11.7
		5%	12.0	12.0	12.0
		7%	12.1	12.1	12.0
	Potassium Hydroxid (KOH)	3%	12.0	11.8	11.8
		5%	12.2	12.2	12.1
		7%	12.5	12.4	12.3

Table 2. Measured pH values of clayey soil after 1, 3 and 28 days

3.2. FTIR spectroscopy

FTIR (Fourier transform infrared spectroscopy) spectra were recorded in the Laboratory for General and Inorganic Chemistry, Faculty of Natural Sciences and Mathematics of Niš. FTIR analysis of the samples was performed using a Bruker Tensor 27 spectroscope, in the region of 4000-400 cm^{-1}, at a resolution of 2 cm^{-1}. For recording FTIR spectra, the so-called "potassium bromide" procedure for the preparation of solid samples was used. The tested samples were finely chopped and evenly dispersed in KBr (1.5 mg/150 mg KBr). The mixture was then subjected to vacuuming and pressing under a pressure of 200 MPa, whereby a suitable lozenge was obtained. Spectroscopically pure KBr, manufactured by Merck, was pre-melted at 700ºC, pulverized and stored in an electric desiccator heated at 30ºC to protect against moisture. A pellet of pure KBr was used as a reference. Based on the data provided by infrared spectra, it is possible to determine the fundamental characteristics of molecules (such as the presence of functional groups), perform qualitative analysis of organic compounds, as well as qualitative analysis of multicomponent mixtures. Additionally, these data can facilitate isotopic analysis of light elements, determination of the structure of crystalline substances, and other related applications. FTIR spectra were recorded in the wavelength range between 460 cm^{-1} and 4000 cm^{-1} and are presented in Figure 2.

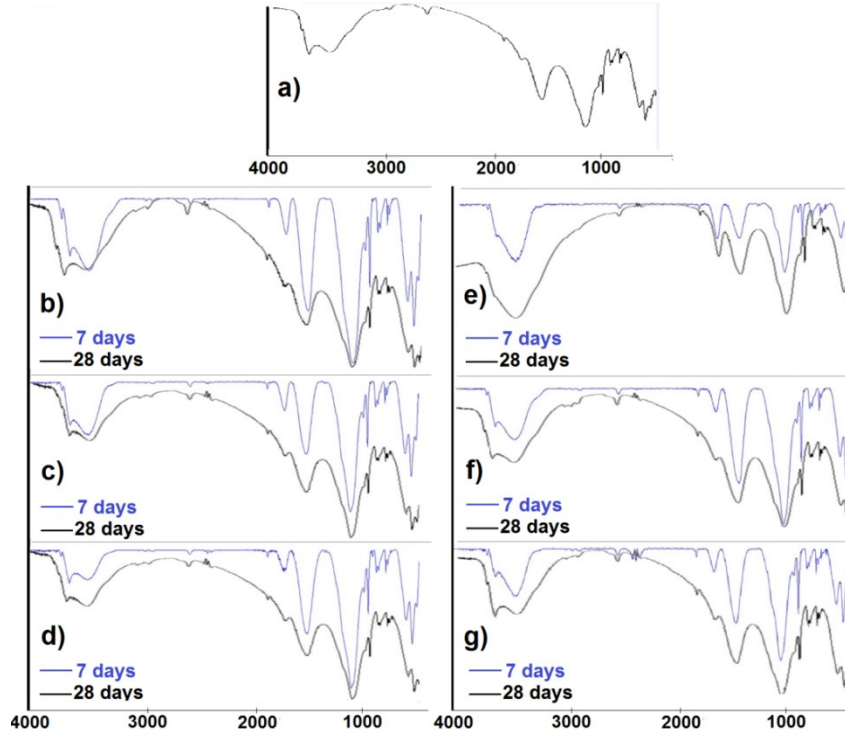


Figure 2. FTIR spectra: a) natural clayey soil; soil stabilized with b) lime, c) cement, d) fly ash, e) magnesium carbonate, f) sodium silicate, g) potassium hydroxid, after 7 and 28 days

As observed in Figure 2a, i.e., the spectrum of the natural clayey soil, the bands in the wavelength region from 3000 to 3500 cm^{-1} correspond to the stretching vibrations of OH groups. The band at 1636 cm^{-1} is attributed to the deformation vibrations of the carbonyl group, which likely originate from calcite. Weak bands appearing in the region between 690 and 780 cm^{-1} correspond to bending vibrations of the OH group.

The bands in the range of 950–1200 cm^{-1} are associated with asymmetric stretching vibrations of Si–O bonds (attributable to quartz bands), as well as to the structural units of aluminosilicate tetrahedral frameworks, Si(Al)–O, typical of clay minerals. In the spectra of samples stabilized with various chemical reagents (Figures 2b–2g), no significant spectral changes were observed. The bands characteristic of the natural clayey soil are still evident in these spectra. However, variations in the intensity of the aforementioned bands are noticeable. This suggests that certain changes in the soil structure have occurred, indicating a degree of chemical stabilization.

3.3. XRF analysis

Samples were recorded with a handheld XRF spectrometer (X-ray Fluorescence), brand LANScientific, model RTU-XRF7460, in the Laboratory for General Inorganic Chemistry of the Faculty of Science of Niš. The device is used to determine the concentration of chemical elements in the soil on samples in a solid aggregate state, in the form of dust, powder, pieces of soil or even thin layers of soil with a detection range of Mg-U. This method of testing is non-destructive and adaptable to the sampled material, as there are almost no requirements regarding the shape or size of the sample.

The sampled material was in a partially loose state. The material was dried and then crushed to achieve homogeneity of the test sample. Each of the samples was recorded three times for a duration of 30 seconds. The mean value of all recordings of each individual sample was adopted as the final value. The results of the XRF analysis conducted after 28 days are presented in Table 3. It is evident to what extent each of the used reagents influences the change in the concentration of individual elements in the soil. The addition of lime and cement increases the content of calcium (Ca), while the addition of $MgCO_3$ significantly increases the magnesium (Mg) concentration, but also reduces the silicon content. The addition of KOH leads to a multiple increase in the potassium (K) content. No significant changes in element concentrations were observed with the addition of fly ash and water glass in comparison to the natural soil. These results may help to better understand the changes in the physical and mechanical characteristics of soil during chemical stabilization.

Elements		Si (%)	Ca (%)	Mg (%)	Al (%)	K (%)
Natural soil		34.789	5.409	10.744	8.325	0.732
Soil treated by the chemical stabilization technique	Lime ($Ca(OH)_2$)	33.074	8.473	12.805	8.145	0.768
	Cement	33.421	9.449	10.126	7.736	0.711
	Fly Ash	30.155	5.875	11.085	8.752	0.991
	MagnesiumCarbonate ($MgCO_3$)	27.347	5.353	14.164	9.321	0.699
	Sodium Silicate (Na_2SiO_3)	34.445	5.901	10.164	7.311	0.867
	Potassium Hydroxid (KOH)	31.781	5.250	11.609	12.131	4.658

Table 3. Change in element concentration (±5%) in clayey soil 28 days after soil treatment

3.4. SEM analysis

The SEM analysis (scanning electronic microscopy) was conducted to observe the microscopic morphological changes occurring in the soil upon the addition of different reagents after treatment. Preparation of samples for SEM analysis, as well as the analysis itself were conducted in the Laboratory for Electronic Microscopy of the University of Niš. In order to make the sample conductive for the electron beam, a thin layer of gold was deposited on one side of the sample, as a preparation. For the purpose of SEM analysis, the samples were placed into the scanning electronic microscope JEOL JSM-5300, which was operated at a voltage of 30 kV, and the penetration depth of the electronic beam was 10 μm. The characteristics of the sample surfaces were observed at various magnifications (×100, ×1000, ×2000) and then photographed. SEM analysis was conducted, and it did not require further sample preparation.

The results of the SEM analysis can be seen in Figure 3. The addition of reagents has a similar effect on the nature of morphological changes. By comparing the images, it can be observed that the flat and smooth particles of the natural soil break down into coarse and fragmented parts with the addition of reagents. Such a change may contribute to an increase in cohesion, thereby enhancing the soil's strength and reducing its plasticity.

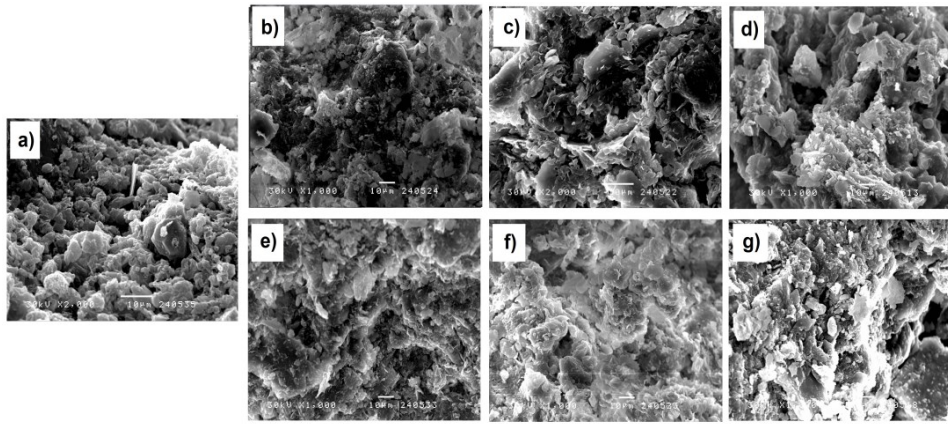


Figure 3. SEM photographs (×2000) of natural clayey soil and soil stabilized with different reagents after 28 days: a) natural clayey soil; soil stabilized with b) lime, c) cement, d) fly ash, e) magnesium carbonate, g) sodium silicate, h) potassium hydroxid

SEM figures indicate a characteristic compact structure of the sample with very few pores and the presence of crystalline particles. It is also observed that the surface of the soil is uneven, complex and irregular in shape. The predominant number of particles is irregular in shape. The finest particles tend to agglomerate with larger particles.

4. CONCLUSIONS

The analysis conducted in this study indicates that various reagents, both traditional (lime, cement and fly ash) and alternative (magnesium carbonate, potassium hydroxide and water glass), affect the soil structure through different mechanisms, ranging from flocculation to pozzolanic reactions and the formation of secondary minerals. The changes in the soil structure resulting from the application of these reagents directly influence the physical–mechanical properties, such as increased compressive strength, reduced plasticity and improved frost resistance and permeability. A special emphasis is placed on durability of these changes, which is crucial for the design and durability of infrastructure facilities. The use of alternative reagents, particularly industrial by-products, demonstrates significant potential, not only from a technical, but also from an ecological and economic perspectives, contributing to the sustainable development of the construction industry.

Future research could focus on more detailed investigations into the interactions between specific soil types and various reagents, with a particular emphasis on microstructural changes and their correlation with engineering characteristics. This would pave the way for the development of personalized stabilization approaches tailored to the characteristics of specific soils, which will contribute to enhanced efficiency and longevity of engineering structures.

Acknowledgment

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

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Prethodno saopštenje

OPTIMALNI SADRŽAJ SITNIH ČESTICA I MINERALA GLINE U AGREGATU DACITA, POREKLOM IZ KAMENOLOMA "ĆERAMIDE", ZA PROIZVODNJU BETONA

Rezime: *Eruptivne stene se, kao visokokvalitetni agregati, mogu koristiti za proizvodnju betona. Da bi se ovaj potencijal i ostvario, agregati moraju da prođu, između ostalih, i određene testove koji se tiču količine i kvaliteta sitnih čestica. Agregat dacita proizveden je na separaciji bez dodatnog pranja i otprašivanja. U sitnom agregatu utvrđen je povećan sadržaj sitnih čestica i štetnih minerala glina. U cilju utvrđivanja njihovog optimalnog sadržaja, laboratorijski su projektovane tri mešavine betona sa različitim sadržajem sitnih čestica. Na betonskim kockama ispitana je čvrstoća pri pritisku, savijanju i upijanje vode nakon 7 i 28 dana. Rezultati ispitivanja pokazali su koja mešavina pokazuje najbolja svojstva i data su preliminarna rešenja za uklanjanje viška sitnih čestica iz agregata.*
Ključne reči*: silikatni agregat, ekvivalent peska, metilen-plavo, minerali glina, beton*

OPTIMAL CONTENT OF FINES AND CLAY MINERALS IN DACITE AGGREGATE, ORIGINATING FROM "ĆERAMIDE" QUARRY, FOR CONCRETE PRODUCTION

Summary: *Eruptive rocks, as high-quality aggregates, can potentially be used for concrete production. To realise this potential, aggregates must pass, among others, specific tests concerning the quantity and quality of fine particles. The dacite aggregate was produced on a separation plant without additional washing and dusting. An increased content of fines and harmful clay minerals was found in the fine aggregate. To determine their optimal content, three concrete mixtures with different content of fines were designed in the laboratory. Concrete blocks were tested for compressive strength, flexural strength and water absorption after 7 and 28 days. The test results showed which mixture showed the best properties, and preliminary solutions were given to remove excess fine particles from the aggregate.*
Keywords: *silicate aggregate, sand equivalent, methylene blue, clay minerals, concrete*

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1. UVOD

U našoj zemlji, u građevinskoj industriji, za proizvodnju betona i izradu asfaltnih mešavina uglavnom se koriste magmatske vulkanske (eruptivne) stene silikatnog sastava i sedimentne i/ili metamorfne stene karbonatnog sastava [4]. U drugim zemljama, agregati magmatskih plutonskih (dubinskih) stena glavni su izvor drobljenih agregata za proizvodnju betona. Bez obzira na poreklo i sastav, agregati proizvedeni od ovih stena moraju da zadovolje niz specifikacija koje se odnose na čvrstoću i trajnost. Brojni su standardni testovi koji se u današnje vreme koriste kako bi se osiguralo da agregati ispunjavaju odgovarajuće specifikacije. Petrografska analiza, na primer, predstavlja jedan od njih i smatra se važnim testom za predviđanje ukupnih performansi agregata za beton, naročito kada su u pitanju mehanička i hemijska svojstva betona [6].

Ocena kvaliteta sitnih čestica u agregatu za beton vrši se ispitivanjem na metilen-plavo. Ovaj test nije bio deo standardnih SRPS testova, već je sa usvajanjem evropskih standarda počeo kod nas da se sve češće koristi. Kameni agregati se smatraju prihvatljivim ako su vrednosti adsorpcije metilen-plavog ispod granice propisanih vrednosti. Kod nas je ova granica data u dokumentu Smernice i pravila za primenu nacionalnih tehničkih zahteva za proizvodnju betona koji se ugrađuje u betonske, armiranobetonske i prethodno napregnute konstrukcije SRPS U.M1.206:2023 i iznosi $MB_{1,5}$ [18]. Osim za metilen-plavo, isti dokument propisuje i granice za sadržaj sitnih čestica (f) i ekvivalenta peska (SE). Ova tri testa smatraju se međusobno povezanim i krucijalnim kada je u pitanju ocena kvaliteta sitnih čestica u agregatu. Povećan sadržaj sitnih čestica često prate niže vrednosti ekvivalenta peska i povećane vrednosti za metilen-plavo. Odnosno, u frakciji agregata izgrađenoj od prašinastih i glinovitih čestica, često je smanjen sadržaj peska a povećan sadržaj gline. U određenim slučajevima ovi testovi nisu u saglasnosti.

Istraživanja su pokazala da povećanje MB vrednosti u drobljenom pesku utiče na pucanje svežeg betona i povećano sušenje očvrslog betona. Kada je ova vrednost veća ili jednaka 1,45 g/kg, otpornost betona na pojavu pukotina usled skupljanja je niža. Može se zaključiti da na metilen-plavo ne utiče toliko sadržaj sitnih čestica, već sadržaj minerala glina [12]. Fina granularna struktura i površinska aktivnost minerala glina povećava količinu vode koja je potrebna da bi se postigla obradivost betona. Čak i kada je sadržaj sitnih čestica u agregatu mali, vrednost MB se povećava ako su u agregatu prisutni minerali glina. Sadržaj sitnih čestica (manjih od 0,063 mm) se ne reflektuje na svojstva betona. Odnosno, da bi se predvidelo ponašanje betona, potreban je metilen-plavo test, koji daje indikaciju kvaliteta sitnih čestica. Proizvođači agregata bi stoga trebalo da naprave određena poboljšanja u procesu proizvodnje betona uzimajući u obzir rezultate metilen-plavo testa [9].

Sa druge strane, postoje studije koje pokazuju da u agregatima andezita može da bude i povećan sadržaj sitnih čestica (14%) u sitnom agregatu 0/4 mm, a da je MB 0,75%, odnosno da je sadržaj štetnih minerala glina (montmorionita) ispod granice od 1% [2]. Isto je zapaženo i kod mermera koji se, kao mekanije stene u odnosu na magmatske, lakše drobe i često imaju povećan sadržaj sitnih čestica, ali je kvalitet tih čestica zadovoljavajući, odnosno vrednosti MB su niske. U isto vreme rezultati ekvivalenta peska pokazuju niže vrednosti (SE) pa se čini da testovi nisu u saglasnosti [19]. U ovom slučaju preporuke su da se pri oceni kvaliteta agregata prednost daje vrednosti metilen-plavog [13].

Test metilen-plavog ne funkcioniše tačno i zadovoljavajuće u svim litotipovima magmatskih stena. Sadržaj smektita u intermedijarnim stenama je snažno povezan sa vrednostima testa metilen-plavog zbog svojstva povećanog upijanja vode i bubrenja minerala iz grupe smektita. Tip i procenat sadržaja hlorita ne utiče na vrednosti metilen-plavog u bazičnim stenama. Međutim, sadržaj serpentina u ultrabazičnim stenama je odlučujući faktor za tačnost testa MB (70% kritičan procenat) [8].

Cilj ove studije bio je da objasni kako sadržaj štetnih minerala glina i hlorita u sitnom agregatu dacita utiču na svojstva betona. Upoređena su svojstva betona pripremljena sa sitnim agregatom iz proizvodnje, opranim na situ 0,063 mm i kombinacijom ova dva agregata pri čemu je učešće krupnog agregata bilo nepromenjeno. U cilju poboljšanja svojstava betona, dodatno je pripremljen beton tako što je korigovan granulometrijski sastav opranog agregata sa prirodnim peskom. U radu su analizirane fizičko-mehaničke karakteristike ovako izrađenih betona. Istraživanja

treba da pomognu proizvođaču u izboru metode otprašivanja i/ili pranja sitnih čestica na drobilično-separacionom postrojenju i/ili transporta do neke druge separacije gde će se agregati oprati i pripremiti za dalju upotrebu za proizvodnju betona.

2. PRIPREMA MATERIJALA I METODE

Iz kamenoloma "Ćeramide" dopremljeno je oko 500 kg agregata u frakcijama 0/4 mm, 4/8 mm i 8/16 mm koje su se koristile za izradu tri mešavine betona pripremljene po istoj recepturi. Frakcije su uzorkovane i dopremljene od strane proizvođača. Po dopremanju u laboratoriju sve frakcije su se sušile razasute na vazduhu u laboratorijskim uslovima. Frakcije 4/8 mm i 8/16 mm nisu prane, a frakcija 0/4 mm je korišćena u tri varijante i to kao:

- 100% frakcija iz proizvodnje na drobilično-separacionom postrojenju;
- 50% frakcija iz proizvodnje + 50% vodom oprana frakcija na situ 0,063 mm;
- 100% vodom oprana frakcija na situ veličine otvora 0,063 mm.

Ispitivanje karakteristika agregata izvršeno je pomoću standardizovanih metoda. Prvo je određen sadržaj vlage (SRPS B.B8.035:1984, povučen), oblik zrna krupnog agregata (SRPS B.B8.048:1987, povučen) i upijanje vode (SRPS ISO 6783:1999, povučen). Kvalitet sitnih čestica ispitan je preko ekvivalenta peska (SRPS EN 933-8:2016) i metilen-plavog (SRPS EN 933-9:2023). Ekvivalent peska rađen je na nekorigovanim frakcijama 0/2 mm ili 0/4 mm, a metilen-plavo na frakciji 0/2 mm i fileru 0/0,125 mm. Ispitivanja agregata izvršena su u Laboratoriji za kamen i kamene agregate Instututa za puteve ad, Beograd.

Metodom rendgenske difrakcije na polikristalnom materijalu utvrđeno je prisustvo nekoliko kristalnih faza [10]. Pored potvrđenih faza u uzorku je konstatovano prisutno i nekoliko različitih minerala iz grupe slojevitih silikata za čiju identifikaciju su urađene dodatne metode pripreme i analize na orijentisani preparatima [11]. Ispitivanja su izvršena na Rudarsko-geološkom fakultetu Univerziteta u Beogradu.

Za izradu probnih betonskih mešavina upotrebljen je cement proizvođača LAFARGE BFC SRBIJA DOO BEOČIN: Portland kompozitni cement oznake PC 20M (S-L) 42,5R, ili CEM II/A-M (S-L) 42,5R, po Pravilniku o kvalitetu cementa [16].

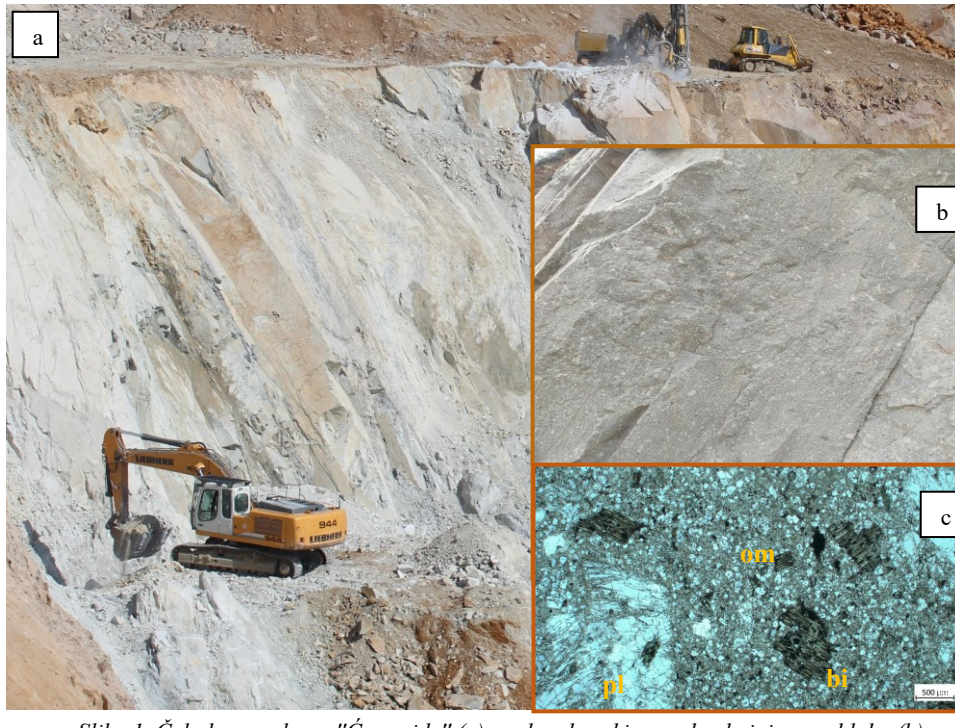
Granulometrijski sastav frakcija agregata određen je metodom prosejavanja na suvo uz prethodno potapanje u vodu, ispiranje na situ veličine otvora 0,063 mm i sušenje agregata po metodi iz standarda SRPS EN 933-1:2013. Granulometrijski sastavi pripremljenih mešavina agregata za probe betona određeni su na osnovu učešća pojedinih frakcija. Ova ispitivanja kao i ispitivanja betonskih mešavina izvršena su u Laboratoriji za beton i veziva Instututa za puteve ad, Beograd. Sastav betonskih mešavina je projektovan tako da sadržaj cementa bude 380 kg/m^3, a dodatak vode do postizanja konzistencije svežeg betona, mereno metodom sleganja, koja odgovara klasi konzistencije S3 (sleganje od 100 do 150 mm) i ispitan u skladu sa SRPS EN 12350-2:2019.

Nakon završenih ispitivanja i analize dobijenih rezultata, pripremili smo i četvrtu probu betona samo sa poboljšanim granulometrijskim sastavom oprane frakcije 0/4 mm na situ 0,063 mm, dodavanjem prirodnog peska 0/4 mm sa separacije "Duvanište" u količini od 25% u mešavini, odnosno 50% od ukupnog učešća frakcije 0/4 mm. Sadržaj ostalih frakcija, receptura betona i sadržaj cementa ostao je isti.

3. POVRŠINSKI KOP "ĆERAMIDE"

Petrografskim ispitivanjem utvrđeno je da stenska masa genetski pripada magmatskim vulkanskim (eruptivnim) stenama, dacitima porfirske strukture i masivne teksture koji se, zbog svog povoljnog mineralnog sastava i sklopa, mogu koristiti za proizvodnju agregata za spravljanje betona i izradu asfaltnih mešavina. Bližom petrološkom odredbom, na osnovu mineralnog i hemijskog sastava, ove stene su određene kao kvarclatiti. Fenokristali su predstavljeni kvarcom, plagioklasom, sanidinom, biotitom i amfibolom. Krupni fenokristali sanidina dimenzija do 10 cm retko su zahvaćeni

mikroskopskim preparatom pri čemu se stena često određuje kao dacit. Osnovna masa je holokristalasta do hipokristalasta. Sekundarni minerali su sericit i hlorit, [5], [7], [3]. Minerali glina se ne mogu identifikovati mikroskopskim ispitivanjem. Njihova identifikacija utvrđena je rendgenskim ispitivanjem i prikazana u odeljku 4.2. Na površinskom kopu kamenoloma "Ćeramide" (slika 1) je otvoreno je nekoliko etaža.



Slika 1. Čelo kamenoloma "Ćeramide" (a), makroskopski uzorak odminiranog bloka (b), mikrofotografija dacita porfirske strukture sa fenokristalima plagioklasa - pl, biotita – bi i osnovne mase - om (c), (Izvor: N.Hrnjez pod a i b, april 2025)

Tehnološki proces započinje otkopavanjem i uklanjanjem otkrivke (jalovine), bušenjem i miniranjem, utovarom odminiranog dacita i transportom do drobiličnog postrojenja koji se nalazi neposredno do kamenoloma. Komadi kamena dimenzija 0-1500 mm se ubacuju u čeljusnu IBAG drobilicu na primarno drobljenje do 400 mm. Ovako izdrobljen materijal ide na sekundarno drobljenje konusnom drobilicom KUBRIA do 120 mm. Tercijalna drobilica OMNICOM ulazni materijal od 0-40 mm predrobljava na 0-22 mm. Drobilično-separaciono postrojenje ima sledeći proizvodni program: frakcije 0/2 mm, 2/4 mm, 0/4 mm, 4/8 mm, 8/11 mm, 8/16 mm, 0/4 mm II kl; 31,5/63 mm; 0/31,5 mm; 0/63 mm; 31,5/125 mm. Nusproizvodi su tamponi druge klase 0/31,5 mm II kl, 0/63 mm II i alterisani dacit. Po potrebi i zahtevu kupaca moguće je praviti i druge nestandardne frakcije. Kvalitet agregata se menja zavisno od vrste kamena i vremenskih uslova, od čega zavisi i upotrebljivost materijala. Godišnje se na separacionom postrojenju proizvede oko 300.000 t. Proizvodnja varira od godine do godine zavisno od potražnje kupaca i infrastrukturnih projekata. Problemi koji se u novije vreme javljaju vezani su za deponovanje velikih količina frakcija 0/2 mm, 0/4 mm, 4/8 mm i tampona druge klase. Neophodno je naći primenu ovih frakcija kako bi se smanjile deponovane količine i rasteretilo opterećenje na životnu sredinu.

Trenutno separacija poseduje važeće Sertifikate o stalnosti performansi za spravljanje betona i izradu asfaltnih mešavina (gornjeg nosećeg i habajućeg sloja kolovozne konstrukcije na putevima svih grupa saobraćajnog opterećenja) za frakcije 2/4 mm, 4/8 mm, 8/11 mm i 8/16 mm.

4. REZULTATI I DISKUSIJA

4.1. Agregati za izradu proba betona

Nakon sušenja frakcija agregata na vazduhu, određen im je sadržaj vlage. Dobijeni rezultati prikazani su u tabeli 1. Zapaženo je da je frakcija 0/4 mm iz proizvodnje nakon prosušivanja zadržala dvostruko veći sadržaj vode u odnosu na opranu i na vazduhu sušenu frakciju.

Karakteristika agregata	Metoda isptivanja	Rezultati ispitivanja
Određivanje vlažnosti agregata (%)		
- frakcija 0/4 mm iz proizvodnje	SRPS B.B8.035:1984, povučen	1,68
- frakcija 0/4 mm vodom oprana na situ 0,063mm i na vazduhu prosušena		0,86
- frakcija 4/8 mm iz proizvodnje		1,25
- frakcija 8/16 mm iz proizvodnje		1,05

Tabela 1. Sadržaj vlage u frakcijama agregata nakon sušenja na vazduhu

Granulometrijski sastavi frakcija sitnog agregata (iz proizvodnje i vodom oprane na situ 0,063 mm) i krupnog agregata prikazani su u tabeli 2. Sadržaj sitnih čestica u frakciji 0/4 mm iz proizvodnje malo prelazi maks. dozvoljenih 10% (f_{10}) prema SRPS U.M1.206:2023 Smernicama i pravilima za primenu nacionalnih tehničkih zahteva za proizvodnju betona koji se ugrađuje u betonske, armiranobetonske i prethodno napregnute konstrukcije [18]. Oprana frakcija 0/4 mm ima mali sadržaj sitnih čestica (1,2%) koje su prošle kroz sito 0,063 mm, što je zanemarljiva količina obzirom da je oko 100 kg agregata oprano na najmanjem situ. Frakcija 4/8 mm takođe pokazuje povećan sadržaj sitnih čestica (maks. 1%) prema Pravilniku o tehničkim zahtevima za frakcionisani agregat za beton i asfalt [17]. Frakcija je uzorkovana u zimskom periodu sa deponije na separaciji gde je segregacija materijala moguća.

Otvor sita (mm)	Prolaz na sitima za frakcije agregata (%), SRPS EN 933-1:2013			
	0/4 mm, iz proizvodnje	0/4 mm, vodom oprana	4/8 mm, iz proizvodnje	8/16 mm, iz proizvodnje
31,5	100,0	100,0	100,0	100,00
16,0	100,0	100,0	100,0	99,7
8,0	100,0	100,0	93,5	8,2
4,0	93,9	94,6	13,8	1,6
2,0	65,7	64,4	5,4	1,5
1,0	39,6	34,5	4,5	-
0,5	28,4	21,2	4,2	-
0,25	20,6	11,7	-	-
0,125	15,3	5,7	-	-
0,063	10,8	1,2	2,5	0,9

Tabela 2. Granulometrijski sastav frakcija agregata

Tehničke karakteristike agregata prikazane su u tabeli 3. Oblik zrna agregata je kubičast. Sadržaj zrna nepovoljnog oblika je mali (6%). Prema Pravilniku za agregate oblik zrna se određuje preko zapreminskog koeficijenta. Smernice za proizvodnju betona propisuju evropski standard za oblik zrna SI_{20}.

Upijanje vode je rađeno na sitnom i krupnom agregatu. Tehnički uslovi po Pravilniku ograničavaju kamen za beton na 1%, ali uslova za upijanje vode agregata nema. Analizirajući rezultate upijanja vode sitnog agregata (tabela 3) može se zaključiti da oprana frakcija ima najveće vrednosti upijanja vode (1,78%).

Karakteristika agregata	Metoda isptivanja		Rezultati ispitivanja agregata			Pravilnik za agregat [17]	SRPS U.M1.206:2023 [18]
			proizvodnja	50%:50%	opran na situ 0,063 mm		
Oblik zrna 3:1 (%) - frakcija 4/8 mm - frakcija 8/16 mm	SRPS B.B8.048:1987, povučen (Prilog III-R)		6,0 6,3	-	-	-	SRPS EN 933-4:2010 SI_{20}
Upijanje vode (%) - frakcija 0/4 mm - frakcija 4/8 mm - frakcija 8/16 mm	SRPS ISO 6783:1999, povučen		1,60 2,29 2,15	1,56 - -	1,78 - -	-	-
Ekvivalent peska (%) nekorigovan	SRPS EN 933-8:2016	SE (0/2)	**44**	**61**	**91**	-	-
		SE_4	55	67	86	-	eruptivna – krečnjak SE_{60}
Metilen-plavo MB (g/kg)	SRPS EN 933-9:2023	0/2 mm	**7,6**	**3,0**	**1,0**	-	eruptivna - krečnjak $MB_{1,5}$
		0/0,125 mm	15,0	8,3	1,7	-	-

Tabela 3.Tehničke karakteristike agregata

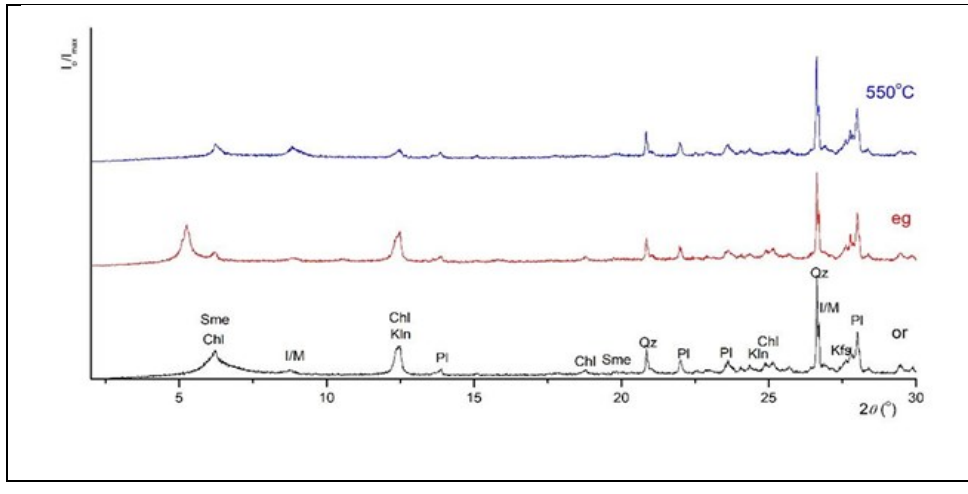
Obzirom da su čestice gline i prašine oprane, ova voda je vezana isključivo za vodu koja ulazi u pukotine zrna agregata. Upijanje vode frakcije 0/4 mm iz proizvodnje je manja brojčano (1,60%). Vrednost nije reprezentativna za poređenje, zato što se potapanjem uzorka u vodu gubi deo kamene prašine i gline iz agregata, odnosno njihovo upijanje ovom metodom nije moguće izmeriti.

Kvalitet sitnih čestica izražen preko ekvivalenta peska u frakciji iz proizvodnje iznosi SE 44 u nekorigovanoj frakciji 0/2 mm, pri čemu je sadržaj metilena-plavog MB 7,6 g/kg, što je dosta povećana vrednost u odnosu na nacionalnu granicu (f_{16}, SE_{60} i $MB_{1,5}$ za krečnjake). Granica za prirodni agregat je takođe propisana (f_{10}, SE_{70} i $MB_{1,5}$). Međutim, za drobljene agregate silikatnog sastava nije propisana granica za kvalitet sitnih čestica, osim f_{10}. Ovo je razumljivo imajući u vidu da se kod nas uglavnom koriste rečni agregati (šljunak i pesak) za proizvodnju betona. Obzirom da u zemljama u okruženju nije dozvoljena upotreba šljunka i peska, i da se evropska regulativa sve više inkorporira u našu regulativu, neophodno je u narednim revizijama standarda SRPS U.M1.206:2023 ove granice propisati. Prema preporučenim specifikacijama za granitne materijale kao sitne agregate u betonu, koje daje "Building and Construction Authority (BSA)", sadržaj sitnih čestica je kategorisan na f_3, f_{10} i f_{16}, pri čemu je MB 0.8 g/kg [1]. U nacionalnim dodacima Slovenaca i Francuza ova granica je MB 1 g/kg [13], [15], [20].

Frakcija 0/4 mm mešavine zrna agregata 50% iz proizvodnje, a 50% oprane na situ 0,063 mm ima povećanu vrednost MB koja iznosi 3 g/kg. Ovo ukazuje da će proizvođač teško delimičnim pranjem ili otprašivanjem agregata postići zadovoljavajući kvalitet ulazne sirovine za proizvodnju agregata i pored zadovoljavajuće vrednosti SE 61. Ovaj kvalitet se postiže potpunim pranjem agregata (SE 91 i MB_1) kao što se pere prirodni agregat na separacijama šljunkova.

4.2. Rendgenska ispitivanja agregata

U ispitivanom uzorku agregata 0/4 mm, rengenskom analizom utvrđeno je prisustvo nekoliko kristalnih faza - mineral kvarc, mineral iz grupe plagioklasa, mineral iz grupe kalijumskih feldspata (ortoklas) i mineral kalcit. Pored potvrđenih faza u uzorku je prisutno i nekoliko različitih minerala iz grupe slojevitih silikata za čiju identifikaciju su urađene dodatne metode pripreme i analize (slika 2) [10].



Slika 2. Difraktogrami orijentisanih preparata uzorka agregata 0/4 mm sa kamenoloma "Ćeramide". Legenda: Sme – mineral iz grupe smektita, Chl – mineral iz grupe hlorita, I/M – mineral iz grupe ilit/liskun, Kln – mineral iz grupe kaolinita, Qz – kvarc, Pl – mineral iz grupe plagioklasa, Kfs – mineral iz grupe kalijumskih feldspata.

Analizom orijentisanih i različito tretiranih preparata koji su snimljeni u karakterističnom opsegu 2–30° 2θ (slika 2) potvrđeno je prisustvo minerala glina iz grupe smektita, minerala iz grupe hlorita, minerala iz grupe ilit/liskuna i minerala iz grupe kaolinita [11].

Prisustvo minerala glina (smektitska i kaolinitska grupa) i minerala iz grupe ilita/liskuna vezano je za procese hidrotermalne alteracije feldspata, dok su minerali iz grupe hlorita rezultat hidrotermalne alteracije biotita i amfibola. Ovi sekundarni minerali prisutni su kako u fenokristalima, tako i u osnovnoj masi stene. U krupnom agregatu dacita sekundarni minerali su čvrsto vezani u strukturi stene i ne bi trebalo da značajno utiču na karakteristike betona. Problemi nastaju u sitnom agregatu, kada dolazi do drobljenja osnovne mase i oslobađanja sekundarnih minerala koji su štetni za kvalitet betona.

4.3. Mešavine agregata za izradu proba betona

Sastav mešavine frakcija agregata za izradu proba betona je bio sa sledećim učešćem pojedinih frakcija, prikazanim u tabeli 4.

Mešavina	Učešće pojedinih frakcija u mešavini za probu betona			
	0/4 mm, iz proizvodnje	0/4 mm, vodom oprana	4/8 mm, iz proizvodnje	8/16 mm, iz proizvodnje
I	50%	0%	16%	34%
II	25%	25%	16%	34%
III	0%	50%	16%	34%
IV	0%	25% + 25% (prirodni pesak)	16%	34%

Tabela 4. Sastav mešavina agregata za izradu proba betona

Granulometrijski sastav pripremljenih mešavina agregata za probe betona je prikazan u tabeli 5.

Otvor sita (mm)	Prolaz na sitima (%), SRPS EN 933-1:2013		
	Mešavina I	Mešavina II	Mešavina III
31,5	100,0	100,0	100,0
16,0	99,9	99,9	99,9
8,0	67,8	67,8	67,8
4,0	49,7	49,9	50,1
2,0	34,2	33,9	33,6
1,0	20,5	19,2	18,0
0,5	14,9	13,1	11,3
0,25	10,3	8,1	5,8
0,125	7,6	5,2	2,8
0,063	6,1	3,7	1,3

Tabela 5. Granulometrijski sastav mešavina agregata

4.4. Karakteristike upotrebljenog cementa

Ispitane su osnovne karakteristike upotrebljenog uzorka cementa, a rezultati ispitivanja su prikazani u tabeli 6.

Karakteristika cementa	Metoda ispitivanja [16]	Rezultat ispitivanja
Gubitak žarenjem (%)	SRPS EN 196-2:2015, т.4.4.1	7,14
Nerastvorljivi ostatak u HCl/Na2CO3 (%)	SRPS EN 196-2:2015, т.4.4.3	0,29
Sadržaj sulfata, SO3 (%)	SRPS EN 196-2:2015, т.4.4.2	3,61
Sadržaj hlorida, Cl^- (%)	SRPS EN 196-2:2015, т.4.4.16	0,041
Voda za standardnu konzistenciju (%)	SRPS EN 196-3:2017	28,8
Vreme početka vezivanja (minuta)	SRPS EN 196-3:2017	165
Vreme kraja vezivanja (minuta)	SRPS EN 196-3:2017	220
Finoća mliva, prosejavanjem 90μm (%)	SRPS EN 196-6:2019	0,45
Čvrstoća pri savijanju (MPa), - na 2 dana - na 7 dana - na 28 dana	SRPS EN 196-1:2017, т.9.1	 5,2 7,2 8,4
Čvrstoća pri pritisku (MPa), - na 2 dana - na 7 dana - na 28 dana	SRPS EN 196-1:2017, т.9.2	 24,6 39,0 51,4

Tabela 6. Karakteristike upotrebljenog cementa CEM II/A-M (S-L) 42,5R

4.5. Rezultati ispitivanja betona

Rezultati ispitivanja cementbetonskih mešavina prikazani su u tabeli 7.

Karakteristika betona	Metoda ispitivanja	Mešavina **I**	Mešavina **II**	Mešavina **III**	Mešavina **IV**
Cement (kg/m^3)		380	384	378	384
Vodocementni faktor, W/C		**0,776**	**0,713**	**0,710**	**0,629**
Konzistencija, sleganje (mm)	SRPS EN 12350-2:2019	**140**	**130**	**100**	**155**
Zapreminska masa svežeg betona (kg/m^3)	SRPS EN 12350-6:2019	**2186**	**2217**	**2262**	**2272**
Zapreminska masa betona na 7 dana (kg/m^3)	SRPS EN 12350-7:2019; /AC:2021	2176	2206	2250	2271
Čvrstoća pri pritisku na 7 dana (MPa)	SRPS EN 12390-3:2019	**15,2**	**19,2**	**21,2**	**26,4**
Čvrstoća pri savijanju na 7 dana (MPa)	SRPS EN 12390-5:2019	**2,9**	**3,0**	**3,5**	**-**
Ukupno upijanje vode na 28 dana (%)	SRPS EN 1338:2012, Прилог E	12,9	12,0	10,3	**-**
Zapreminska masa betona na 28 dana (kg/m^3)	SRPS EN 12350-7:2019; /AC:2021	2186	2213	2254	**-**
Čvrstoća pri pritisku na 28 dana (MPa)	SRPS EN 12390-3:2019	**23,1**	**29,1**	**31,5**	**-**
Čvrstoća pri savijanju na 28 dana (MPa)	SRPS EN 12390-5:2019	**3,9**	**4,1**	**4,2**	**-**
Ukupno upijanje vode na 28 dana (%)	SRPS EN 1338:2012, Прилог E	12,7	11,2	9,7	**-**

Tabela 7. Rezultati ispitivanja betona

Dobijeni rezultati ispitivanja cementbetonskih mešavina su pokazali da je W/C faktor izuzetno visok ukoliko se koristi frakcija 0/4 mm iz proizvodnje. Zamenom dela (50%) frakcije 0/4 mm iz proizvodnje sa frakcijom 0/4 mm koja je oprana na situ otvora 0,063 mm postiže se slična konzistencija svežeg betona pri nešto manjem W/C faktoru (za 8%). Kada je upotrebljena samo oprana frakcija 0/4 mm nije došlo do daljeg pada W/C faktora ali je došlo do izvesnog pada ugradljivosti betona na osnovu izmerene konzistencije metodom sleganja.

Zapreminska masa svežeg i očvrslog betona ima najniže vrednosti kod mešavine I gde je upotrebljena samo frakcija 0/4 mm iz proizvodnje, a raste do mešavine II i dalje do mešavine III (gde je upotrebljena samo oprana frakcija 0/4 mm).

Dobijene vrednosti za čvrstoću pri pritisku betona na 7 i 28 dana kod mešavine I su značajno niske, obzirom da je mešavina sa 380 kg cementa po m^3 betona i klasom konzistencije S3, što je posledica visokog sadržaja po beton štetnih minerala glina. Zamenom dela (50%) frakcije 0/4 mm iz proizvodnje sa opranom frakcijom 0/4 mm, kod mešavine II dobija se značajno povećanje vrednosti od 26% za čvrstoću pri pritisku betona na 7 i 28 dana. Kada je u mešavini III upotrebljena oprana frakcija 0/4 mm, dobijeno je povećanje čvrstoće pri pritisku u odnosu na mešavinu I od oko 38% a u odnosu na mešavinu II samo za 8-10%, uz navedeni pad ugradljivosti betona.

Čvrstoća pri savijanju betona raste od mešavine I do mešavine III, a vrednosti za ukupno upijanje vode betona opadaju. Ukupno upijanje vode betona na 28 dana je nešto manje od upijanja vode na 7 dana, kod sve tri mešavine.

Za mešavinu IV, gde je oprana frakcija 0/4 mm delom (50%) zamenjena sa separisanom prirodnom rečnom frakcijom 0/4 mm postignuta je značajno bolja ugradljivost betona (sleganje 155 mm) pri manjem W/C faktoru, a na 7 dana dobijena je čvrstoće pri pritisku betona od 26,4 MPa, što je poboljšanje u odnosu na mešavinu III za 25%, a u odnosu na mešavinu I za 74%.

5. MOGUĆA REŠENJA

Analizom dobijenih rezultata karakteristika betona (tabela 7) zaključeno je da mešavina betona III, koja je uključila frakciju 0/4 mm opranu na situ 0,063 mm, postiže najveće vrednosti čvrstoće pri pritisku na 7 dana (21,2 MPa) i na 28 dana (31,5 MPa). Međutim, obzirom da je granulometrijski sastav oprane frakcije bio krupnozrn, odnosno da su nedostajale sitne čestice dobrog kvaliteta, granulometrijski sastav je korigovan dodavanjem 0/4 mm frakcije prirodnog agregata količini od 25% u mešavini, odnosno 50% od ukupnog učešća frakcije 0/4 mm. Čvrstoća pri pritisku tako projektovanog betona nakon 7 dana nege u vodi iznosila je 26,4 MPa.

Rešenja koje proizilaze iz ovih ispitivanja ukazuju da proizvođač mora da ukloni sa drobljenog agregata dacita sadržaj sitnih čestica koje u sebi sadrže štetne minerale glina i hlorita (smektit, ilit/liskun, kaolinit, hlorit). Uklanjanje može da učini na jedan od sledećih načina:

- Nastaviti proizvodnju frakcija bez pranja na drobilično-separacionom postrojenju. Neoprane frakcije transportovati do separacionog postrojenja u blizini fabrike betona gde bi se frakcije drobljenog agregata oprale. Pri pravljenju betona dodavati prirodni pesak u cilju korekcije granulometrijskog sastava ulaznih komponenti betona.
- Otprašivati sitne čestice na samom separacionom postrojenju ciklonom ili hidrociklonom odmah nakon drobljenja [14]. Pronaći rešenja za dalju upotrebu ovako nastalog nusproizvoda koji se sastoji od smeše kaolinita, smektita i hlorita kao što je npr. građevinska keramika (cigle, blokovi), geobarijere (visok sadržaj smektita), aditivi (potencijal za nano primene uz obradu) itd.

Ekonomska opravdanost ponuđenih rešenja ukazuju na sledeće:

- Frakcija 0/4 mm u stanju gde ne zadovoljava standarde kvaliteta, cena se okvirno kreće oko 2-3 €/t, sto je ispod proizvodne cene koja se kreće oko 6 €/t, jer se ne može koristiti u betonima i asfaltima, vec isključivo za nasipanje ili zapunjavanje kanala cevovoda ili podzemnih rezervoara.
- U slučaju delimičnog pranja ove frakcije i njenog korišćenja za proizvidnju betonskog crepa, koji Lafarge planira da proizvodi u Obrenovcu, cena bi mogla da bude oko 10 €, s tim da je potrebno probama potvrditi ove pretpostavke.
- Bilo bi korisno da se kroz betonske probe razmotre procenti mešanja eruptivne (delimično oprane ili oprane) frakcije sa rečnom frakcijom, ali tako da beton, odnosno finalni proizvod ne poskupljuje, tj. da količina cementa koja se koristi u mešavini ostane u najmanju ruku ista kao kod betona iste klase čvrstoće koji se proizvodi isključivo od rečne frakcije.
- S tim u vezi, ukoliko bi se potvrdilo da je moguće mešanje od 25-30% eruptivne frakcije u recepturama za beton, takođe bi cena ovakve frakcije mogla da bude od 8-10 € (u zavisnosti od više faktora od kojih su najznačajniji udaljenost betonskih baza tj. pripadajući trošak transporta, kao i uslovi, osnosno trošak mešanja frakcija na samim bazama).
- Iz ovih proba koje su rađene, jasno je da se oprana frakcija mnogo bolje ponaša u odnosu na frakciju 0/4 mm u izvornom stanju, međutim, 380 kg cementa za postizanje betona klase čvrstoće C25/30 je verovatno veći utrošak cementa u odnosu na beton iste klase čvrstoće spravljen od rečnog materijala, te je potrebno razmotriti ekonomsku isplativost ovog rešenja i eventualno upotrebu hemijskih dodataka betonu, tipa superplastifikatora.

6. ZAKLJUČCI

Zaključci izvedeni iz gore pomenutih istraživanja su sledeći:

- sa povećanjem sadržaja sitnih čestica u frakciji 0/4 mm drobljenog agregata dacita, povećava se sadržaj štetnih minerala glina i hlorita;

- minerali iz grupe smektita i hlorita u agregatu dacita (eruptivnog porekla) drastično utiču na povećanje MB, odnosno smanjenje SE i SE_4;

- delimičnim pranjem frakcije 0/4 mm, kada je sadržaj sitnih čestica f_5 u agregatu, ne postiže se $MB_{1.5}$;

- potpunim pranjem frakcije 0/4 mm, kada je sadržaj sitnih čestica f_1 u agregatu, postiže se $MB_{1.5}$, ali su zbog manjka sitnih čestica čvrstoće betona < 35 MPa;

- dodavanjem peska opranoj frakciji 0/4 mm, odnosno korigovanjem granulometrijskog sastava frakcije 0/4 mm postiže se čvrstoća betona nakon 7 dana nege u vodi (26,4 MPa);

- proizvođač ne može samo otprašivanjem na separacionom postrojenju postići zadovoljavajući kvalitet agregata, već mora kombinovati otprašivanje i pranje ili samo pranje agregata;

- analiza ekonomske opravdanosti ponuđenih rešenja ukazala su da je neophodno nastaviti istraživanja u pravcima delimičnog pranja ove frakcije i njenog korišćenja za proizvodnju betonskog crepa, kao i razmatranja mešanja eruptivne sa rečnom frakcijom, ali tako da beton, odnosno finalni proizvod ne poskupljuje;

- oprana frakcija se mnogo bolje ponaša u odnosu na frakciju 0/4 mm u izvornom stanju, međutim, 380 kg cementa za postizanje betona klase čvrstoće C25/30 je verovatno veći utrošak cementa u odnosu na beton iste klase čvrstoće spravljen od rečnog materijala, te je potrebno razmotriti ekonomsku isplativost ovog rešenja i eventualno upotrebu hemijskih dodataka betonu, tipa superplastifikatora;

- u narednoj reviziji standarda SRPS U.M1.206:2023 izvršiti dopunu graničnih vrednosti MB i SE za drobljeni agregat eruptivnog porekla.

Zaključci u prezentovanom radu izvedeni su iz jednog uzorka agregata. Potrebno je uraditi više analiza agregata proizvedenih sa različitih mesta na kamenolomu. Paralelno sa istraživanjima agregata potrebno je izvršiti optimizaciju načina uklanjanja sitnih čestica iz agregata na drobilično-separacionom postrojenju, pronaći rešenja za dalju upotrebu ovako nastalog nusproizvoda koji se sastoji od smeše kaolinita, smektita i hlorita, kako bi se uskladila istraživanja i realizovala njihova primenljivost.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

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PERFORMANCE INDEX FOR REPAIR CEMENT MORTARS MODIFIED WITH SLAG FROM THE ASPECT OF SELECTED PROPERTIES

Summary: *Worldwide, the concrete structures under harsh environment cannot reach the expected service life. In order to restore the damaged, different types of repair mortars can be used. The aim of this paper is to evaluate the physicomechanical properties of repair cement-based mortars that contain 0, 10, 20 and 30% slag as SCM, prepared with a water-to-binder ratio of 0.5, by using the concept of performance index. Applied slag represents a by-product from the one company located in Serbia. The compressive strength, modulus of elasticity, and capillary water absorption of examined mortars were determined at the age of 28 days.*
Keywords: *EN 1504-3; Compressive strength; Elastic modulus; Capillary water absorption*

INDEKS PERFORMANSE ZA REPARATURNE CEMENTNE MALTERE MODIFIKOVANE ZGUROM SA ASPEKTA ODABRANIH SVOJSTAVA

Rezime: *Širom sveta, betonske konstrukcije izložene agresivnim uslovima okoline ne mogu da dostignu očekivani upotrebni vek. Za potrebe sanacije oštećenja mogu se koristiti različite vrste reparaturnih maltera. Cilj ovog rada je procena fizičko-mehaničkih svojstava reparaturnih maltera na bazi cementa koji sadrže 0, 10, 20 i 30% zgure kao SCM, spravljenih sa vodovezivnim faktorom od 0,5, primenom koncepta indeksa performansi. Primenjena zgura predstavlja nusproizvod jedne kompanije koja se nalazi u Srbiji. Čvrstoća pri pritisku, modul elastičnosti i kapilarno upijanje vode ispitivanih maltera su određeni za starost od 28 dana.*
Ključne reči: *EN 1504-3; Čvrstoća pri pritisku; Modul elastičnosti; Kapilarno upijanje vode.*

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1. INTRODUCTION

Considering that nowadays a large number of concrete structures are approaching the end of their expected service life, there is an increasing need for their rehabilitation and repair [8]. Moreover, recent research [15] in the field of construction sector emphasizes the importance of sustainability assessment and the consideration of impacts in the maintenance phase throughout the entire infrastructure and construction cycle. In order to restore the performance of the deteriorated structures, different types of repair mortars could be used, such as cement-based mortars.

Partial replacement of ordinary Portland cement (OPC) with supplementary cementitious materials (SCM), such as ground-granulated blast furnace slag (SL), in addition to environmental benefits, could have physicomechanical [10][13][14][16] and durability [7][12][16] advantages for certain levels of cement replacement in cement-based composites. SL represents a by-product of iron and steel production, and it is commonly used for cement production or as a partial replacement of OPC in cement-based composites, i.e. as type II addition in accordance with standard SRPS EN 206 [18].

In order to determine the optimal level of the cement replacement with SCM in cement-based composites, in terms of the certain physicomechanical properties and durability, the concept of performance index was introduced by [11]. According to this concept, numerical performance index values can be assigned to the physical, mechanical and durability characteristics of concretes/mortars. These values can serve as a means to make decisions about the optimal level of cement replacement with SCM.

The aim of the research presented in this paper was to evaluate the selected physicomechanical properties of repair cement-based mortars that contain 0, 10, 20 and 30% SL as SCM, prepared with a water-to-binder ratio of 0.5, in accordance with classes defined in BS EN 1504-3 [4], but also by applying the concept of performance index. Applied SL is a by-product from the company Hesteel Serbia, located in Smederevo, Serbia. Following the standard BS EN 1504-3 [4], the compressive strength of mortars, their modulus of elasticity in compression and capillary absorption were determined at the age of 28 days in accordance with standards BS EN 12190 [2], BS EN 13412 [3], and EN 13057 [9], respectively.

2. MATERIALS AND METHODS

2.1. Component materials and mortar mixtures

In order to evaluate the effect of SL as SCM on compressive strength, modulus of elasticity in compression, and capillary water absorption of the repair cement-based mortars, the following component materials were used for the repair mortars preparation:

- Ordinary Portland cement CEM I 42.5R (Lafarge-BFC Serbia);
- SL from the company Hesteel Serbia, located in Smederevo, Serbia (sieved through a sieve opening 125 μm and additionally ground for 3h);
- CEN standard sand, in accordance with SRPS EN 196-1 [17];
- Deionized water.

True densities of OPC (3.126 g/cm^3) and SL (2.766 g/cm^3) were determined in accordance with BS EN 1097-7 [1], while the specific surface of OPC (4189 cm^2/g) and SL (5855 cm^2/g) were determined in accordance with the procedure specified in standard

BS EN 196-6 [6]. The SL activity indexes, determined in accordance with the standard BS EN 15167-1 [5], were 74% after 7 which is higher than the required 45%, and 87% after 28 days which is higher than the required 70%.

One reference and three different cement-based repair mortar mixtures containing 10, 20 and 30% of SL as SCM by mass, were made to determine their compressive strength, modulus of elasticity in compression, and capillary water absorption. Mortar mixtures were prepared with a water-to-binder ratio of 0.5 according to BS EN 12190 [2]. Composition of mortar mixtures is shown in Table 1.

Component material	PC	SL10	SL20	SL30
CEM I 42.5R, (g)	450	405	360	315
SL, (g)	-	45	90	135
CEN standard sand, (g)	1350	1350	1350	1350
Deionized water, (g)	225	225	225	225

Table 1. Composition of reference and cement-based mortar mixtures that contain SL as SCM

2.2. Methods

2.2.1. Compressive strength

The 28-day compressive strength of the mortars was determined according to the standard BS EN 12190 [2] using prism-shaped specimens with dimension of 40 mm ×40 mm ×160 mm after recommended curing resume (covered in film for 24 h, demoulded after 24 h and cured under water at (21 ± 2)°C for 27 days). The mean strength values were obtained by testing the six prism halves per mortar mixture. The load at failure was determined using a hydraulic press with a capacity of 150 kN.

2.2.2. Modulus of elasticity in comprassion

Modulus of elasticity in compression of mortars was determined according to Method 2 given in the standard BS EN 13412 [3] using prism-shaped specimens with dimension of 40 mm ×40 mm ×160 mm, at the age of 28 days. After recommended curing resume (covered in film for 24h, demoulded after 24h and cured 27 days under water at (21 ± 2)°C); the test specimens were taken from water and conditioned for 24h under the standard laboratory climate, of (21 ± 2)ºC and (60 ± 10)% RH prior to testing). The elastic modulus was established by measuring the change in the strain in the specimen when loaded to produce a stress of between 1.0 N/mm^2 and one-third of the compressive strength of the specimen. Compression testing machine with a capacity of 150 kN was used for pre-loading as well as for the test loading.

2.2.3. Capillary water absorption

Capillary water absorption of mortars was determined according to the standard EN 13057 [9] on discs with a diameter of 100 mm and height of 25 mm, at the age of 28

days, after recommended curing resume and conditioning. After preparing mortar mixtures and embedding in moulds, they were covered in film for 24h, demoulded after 24h and cured under water at (21 ± 2)°C for 27 days. At the end of required curing period, the specimens were dried in the oven for a minimum of 7 days at (40 ± 2)°C, and conditioned for 24h under the standard laboratory climate, at (21 ± 2)°C and (60 ± 10)% RH.

During the examination, the specimens were placed in separate shallow flat-bottomed trays on knife-edge supports to achieve a depth of water immersion of (2.0 ± 1.0) mm, as it is shown in Figure 1. The mass of the specimens was measured immediately before their exposure to the deionized water, as well as after 12', 30', 1h, 2h, 4h and 24h from the beginning of the test. Between measurements, the trays were covered with a lid, and after each specimen weight measurement, the water level was checked and corrected if it was necessary.



Figure 1. Examination of capillary water absorption

2.2.4. Performance index

Instead of using qualitative terms to describe the performances of SL as partial replacement of OPC in repair cement-based mortars, the quantitative values can be assigned instead. Considering the performance of reference mortar as 1.0 for any physicomechanical property, the performance of each cement-based mortar may be expressed as either a value less than 1.0, which means its performance is relatively inferior to reference one, or as a value greater than 1.0, which means that its performance is better than the reference. The performance index values for each mortar type were then summed to determine the optimal level of OPC replacement with SL.

3. RESULTS AND DISCUSSION

3.1. Compressive strength

The 28-day compressive strengths of mortars containing SL as SCM, prepared with a water-to-binder ratio of 0.5, are shown in Figure 2.

Based on the obtained results at the age of 28 days, the mortars made with SL as SCM fulfilled the requirement for class R4 of structural repair mortars according to the standard BS EN 1504-3 [4] in terms of compressive strength [2] i.e. their mean compressive strength values were higher than 45 MPa.

The range of compressive strength was from 50.16-52.92 MPa. It can be seen that mortar containing 20% SL as SCM had the highest compressive strength, i.e. 5.5% higher in comparison to the reference one. Mortars with 10% and 30% had strengths at the same level as the reference cement mortar. Considering that the differences in compressive strength of cement-slag mortars were less than 10% compared to reference cement mortar, it can be concluded that the level of substitution of OPC with SL up to 30% did not have a significant impact on the 28-day compressive strength of blended cement mortars made with the water-to-binder ratio of 0.5.

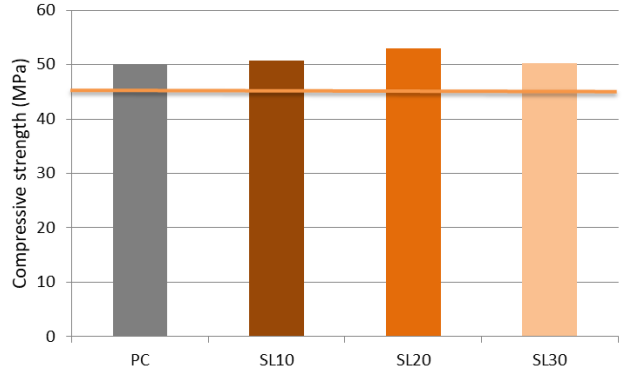


Figure 2. The 28-day compressive strength of tested mortars

3.2. Modulus of elasticity in compression

The elastic modulus results of mortars containing SL as SCM, at the ages of 28 days, and prepared with a water-to-binder ratio of 0.5, are shown in Figure 3. Based on the obtained results, the mortars made with SL as SCM fulfilled the requirement for class R4 of structural repair mortars according to the standard BS EN 1504-3 [4] in terms of modulus of elasticity in compression [3] i.e. their mean values were higher than 20 GPa.

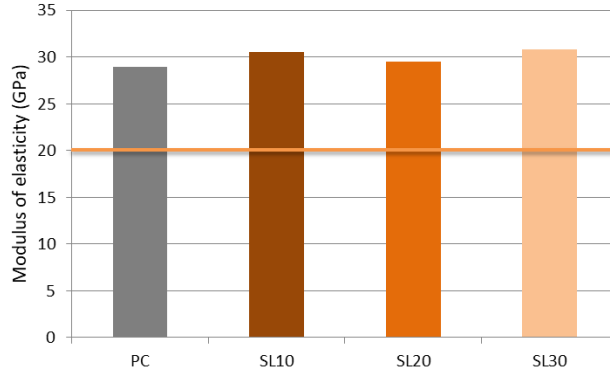


Figure 3. The 28-day modulus of elasticity in compression of tested mortars

The range of the modulus of elasticity in compression of blended cement mortars after 28 days of curing were: 28.92-30.84 GPa. After 28 days, the modulus of elasticity of SL10, SL20 and SL30 were 5.6%, 2.1% and 6.6% higher in comparison with reference one. Based on the presented results, it can be concluded that the level of substitution of OPC with SL up to 30% had a certain impact on the 28-day elastic modulus of cement-based mortars containing SL as SCM, made with the water-to-binder ratio of 0.5, although the difference was less than 10%.

3.3. Capillary water absorption

The kinetics of capillary water absorption of the 28-day age mortars that contain SL as SCM, prepared with a water-to-binder ratio of 0.5, are shown in Figure 4. Based on the shape of the presented diagrams, it can be concluded that the kinetic of the capillary absorption was influenced by the level of OPC substitution with SL.

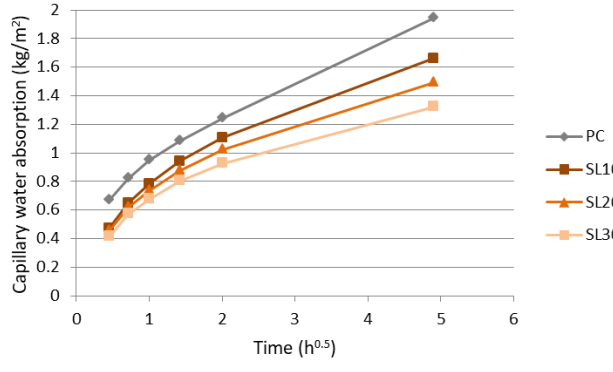


Figure 4. Capillary water absorption of 28-day age mortars

The mean value of capillary absorption (after 24h of exposure) of reference mortar PC was 1.944 kg/m^2, for mortars containing up to 30% SL it was in the range of 1.324 - 1.662 kg/m^2. Reference mortar PC had the highest capillary absorption of water, while the capillary absorption of water decreased with an increase in the proportion of SL. In comparison to the capillary water absorption of the reference PC mortar, measured after 24 hours from the beginning of the test, the relative values of the capillary water absorptions of the 28-day old mortars SL10, SL20 and SL30 were lower by 14.5%, 23.2% and 31.9%, respectively. It is also important to note that the first measurement of capillary absorption was after 12 minutes, and that already at that moment the reference mortar had significantly higher capillary absorption (by about 50%) compared to the mortars containing SL as SCM.

Based on the 28-day sorption coefficient of tested mortars, shown in Table 2, all mortars containing SL as SCM met the criteria for class R4 structural repair mortar, i.e. their sorption coefficients at the age of 28 days were ≤ 0.5 kg/(m^{2}h$^{0.5}$).

Mortar mixture	Sorption coefficient, ($kg/(m^2h^{0.5})$)	Resulting line
PC	0.2728	Linear
SL10	0.3393	Non-linear
SL20	0.3046	Non-linear
SL30	0.2702	Non-linear

Table 2. Sorption coefficient of tested mortars

Although all tested mortar types belong to the same class of structural repair mortar from the aspect of capillary absorption, the sorption coefficient of reference mortar can not be directly compared to ones of mortars containing SL. In the case of reference mortar, the sorption coefficient was determined as the gradient of the resulting line (linear trendline) based on results from 12 minutes to the 24h, while for mortars containing up to 30% SL, resulting lines were non-linear, and therefore, the sorption coefficients were determined as the slope from the intercept to the last reading (after 24h) in accordance with the second special case defined in EN 13057 [9]. As it can be seen from Table 2, increasing the level of OPC replacement with SL from 10 to 30% resulted in a decrease in the value of the sorption coefficient.

3.4. Performance index

Based on the obtained results presented in parts 3.1.-3.3. of this paper, all examined mortars met the criteria for class R4 structural repair mortar in accordance with the standard BS EN 1504-3 [4] in terms of the compressive strength, modulus of elasticity in compression and capillary absorption, determined for the age of 28 days. Since all tested mortars belong to the same class in terms of tested properties, it can be recommended to replace 30% of OPC with SL. On the other hand, the aim of this research was to determine more precisely the impact of different levels of OPC replacement with SL, for which the concept of performance index was applied. Performance index values per each examined mortar type are presented in Table 3.

	PC	SL10	SL20	SL30
(I) Compressive strength	1	1.011	1.055	1.001
(II) Elastic modulus	1	1.056	1.021	1.066
(III) Sorption coefficient	1	0.756	0.884	1.010
(IV) Capillary water absorption*	1	1.145	1.232	1.319
(A) Sum of performance indexes (I, II and III)	3	2.824	2.960	3.077
(B) Sum of performance indexes (I, II and IV)	3	3.212	3.309	3.386

* Water uptake per unit area after 24 hours of exposure

Table 3. Performance index values and sums

Performance index values of mortars contained SL as SCM were sligthly higher than 1.0 in the cases of compressive strength (sign as I) and elastic modulus (sign as II), although the difference was less than 10% compared to the reference value (1.000). Considering that based on the value of the sorption coefficient (sign as III), the classes of repair mortars were determined from the aspect of capillary absorption of water, the performance indexes for the given property were calculated and included in Table 3. Considering that the capillary absorption trendline of the reference mortar is linear in contrast to mortars containing SL (non-linear), the obtained sorption coefficients as well as the corresponding performance indexes for the observed property do not reflect the realistic situation (i.e. in fact, reference mortar had the highest values of capillary water absorption, while with an increase in the level of OPC replacement with SL the values decreased, Figure 4), therefore the authors suggest that the performance index be calculated based on the water uptake per unit area after 24 hours of exposure, (IV) in Table 3. According to the performance index values in term of the water uptake per unit area after 24 hours, with increasing OPC replacement with SL, there was also the increase in the performance index values, i.e. with increasing the level of SL up to 30% as SCM there was the decrease in capillary absorption.

In order to determine the total performance of examined mortar types, the sums (A) and (B) were calculated and presented in Table 3. The first sum (A) of performance indexes includes values of sorption coefficient, while the second (B) contains values of the water uptake per unit area after 24 hours of exposure (i.e. the capillary absorption) instead. In both cases, with the addition of 10-30% SL as SCM, the sum values increase. Furthermore, in the case of sum (A), the mortars with 10 and 20% had slightly lower sums (2.824 and 2.960, respectively), while the mortar with 30% of SL had a sum (3.077) slightly higher than the reference mortar. In the case of sum (B), with with increasing levels of OPC substitution with SL, sum of performance indexes increased from 3.212-3.386, compared to the reference sum (3.000).

The differences in sums (A) for cement-based mortars were less than 10% compared to the sum corresponding to reference cement mortar, while in the case of sums (B), mortars containing 10, 20 and 30% SL as SCM had for 7.1%, 10.3% and 12.9% higher sums of performance indexes in comparison to the sum of reference one, respectively. Based on the all presented sums, it can be concluded that best total performance had the mortar containing 30% of SL as SCM.

4. CONCLUSION

Based on the obtained experimental results, in terms of the 28-day compressive strength, modulus of elasticity in compression and capillary water absorption of the repair cement-based mortars that contain 0, 10, 20 and 30% SL as SCM, prepared with water-to-binder ratio of 0.5, the following can be concluded:

- Mortars containing up to 30% SL as SCM fulfilled the requirement for class R4 of structural repair mortars according to the standard BS EN 1504-3 [4] in terms of compressive strength, modulus of elasticity in compression, and capillary absorption;
- Level of substitution of OPC with SL up to 30% did not have a significant impact on the compressive strength and elastic modulus of cement-based mortars compared to reference mortar;

- Reference mortar had the highest capillary absorption, while with an increase in the proportion of SL as SCM up to 30% the capillary water absorption decreased;
- Sorption coefficient of reference mortar was determined as the gradient of the resulting line (linear trendline) based on results from 12 minutes to the 24h, while for mortars containing up to 30% SL, resulting lines were non-linear, and the sorption coefficients were determined as the slope from the intercept to the last reading (after 24h);
- Increasing the level of OPC replacement with SL from 10 to 30% resulted in a decrease in the value of the sorption coefficient;
- The total sums of performance indexes values of compressive strength, modulus of elasticity in compression, and the sorption coefficient in the first case and the water uptake per unit area after 24 hours of exposure in the second, showed that the best total performance had the mortar containing 30% of SL as SCM.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

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GREEN ROOFS – LEGISLATION IN THE REPUBLIC OF SERBIA AND EUROPE

Summary: *The adoption of green roofs in the Republic of Serbia remains limited due to multiple barriers, including the absence of supportive legislation, lack of financial incentives, low public awareness, high initial costs, and ongoing maintenance requirements. In contrast, green roofs are increasingly integrated into urban planning globally, particularly in major cities where building regulations often mandate their inclusion. In Europe, green roofs have become a standard, with Germany leading the way through pioneering laws that encourage their installation on both new and existing structures. This paper examines international legislative frameworks to assess the potential for developing technical regulations to advance green roof implementation in Serbia.*
Keywords: *green roofs, urban planning, legislation*

ZELENI KROVOVI – REGULATIVA U REPUBLICI SRBIJI I EVROPI

Rezime: *Implementacija zelenih krovova je slabo zastupljena u Republici Srbiji, usled više faktora: nedostatak regulative, nepostojanje finansijske podrške države, neinformisanost ljudi o benefitima implementacije zelenih krovova, visoki, početni, investicioni troškovi, potreba redovnog održavanja. Zeleni krovovi se primenjuju u velikim, svetskim gradovima usled sistemskih zahteva koji se odnose na projektovanje i izvođenje novih objekata. U Evropi, zeleni krovovi postaju standard u urbanom planiranju. Nemačka je jedan od pionira i lidera u implementaciji ove tehnologije, gde su u poslednjim decenijama usvojeni zakoni koji podstiču implementaciju zelenih krovova na novim i postojećim zgradama. U cilju sagledavanja mogućnosti razvoja domaćih, tehničkih propisa, u radu je dat pregled regulative u drugim zemljama u kojima je primena zelenih krovova u većoj upotrebi, nego u Republici Srbiji.*
Ključne reči: *zeleni krovovi, urbano planiranje, regulative*

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1. INTRODUCTION

Green roofs, also known as ecological roofs or living roofs, represent an innovative solution in urban design that allows for the integration of natural elements into urban infrastructure. This type of roof includes a system of vegetation growing on the building's surface, often supported by specialized materials and layers that provide optimal conditions for plant growth. In addition to their aesthetic contribution, green roofs offer numerous ecological, energy, and functional benefits, making them a key element in the sustainable development of cities. They represent a fusion of nature and technology, improving the quality of life for building occupants and the broader community, while making cities healthier and more pleasant places to live.

The application of green roofs is possible on various types of buildings, from residential to commercial, and in different climatic conditions. Thanks to their ability to reduce the effect of urban heat islands, improve air quality, and reduce noise [10], they have become an important part of environmentally responsible and energy-efficient architecture.

In the Republic of Serbia, green roofs are a relatively new area that is still not covered by specific regulations. Their implementation is mainly based on local initiatives in urban areas, with limited institutional support.

On a global scale, while there are no universal laws specifically addressing green roofs, many countries have developed detailed guidelines for their design, implementation, and maintenance [15]. These guidelines vary depending on the climatic, economic, and social specifics of each country.

Green roofs bring significant quantitative benefits, including energy cost reduction, improved biodiversity, and stormwater management. Numerous authors [1, 19, 28, 15, 10] emphasize the importance of local regulations and financial incentives for their wider adoption, with particular attention to the impact of European Union (EU) policies.

In Serbia, scientific literature on regulations for green roofs is still in its early stages, as this field is in development. However, there are works and initiatives addressing green infrastructure and urban planning, which can serve as a foundation for further development [5, 29, 20].

This paper provides an informative overview of some regulations and incentives implemented in Serbia and other countries to promote the implementation of green roofs, with an emphasis on their significance for the sustainable development of urban environments.

2. LEGISLATION IN THE REPUBLIC OF SERBIA

In the Republic of Serbia, there is no specific law or regulation that exclusively addresses green roofs. However, this area is indirectly covered by several existing legal and strategic documents that promote sustainable development, energy efficiency, and environmental protection.

The *Energy Efficiency Law* [31] is aligned with EU directives and pertains to reducing energy consumption, protecting the environment, and decreasing CO_2 emissions, while encouraging investments related to energy retrofitting and renewable energy sources. Law articles [31] that address the energy efficiency of buildings include the following points:

- Minimum requirements for energy efficiency of new and renovated buildings
- Mandatory energy retrofitting of government buildings over 250 m²
- Mandatory preparation of energy efficiency reports
- Energy services for efficiency improvements (such as energy audits, design, renovation, energy retrofitting of buildings, and energy management)
- Establishment of a budget fund for financing energy efficiency projects in existing buildings
- Introduction of energy management systems, especially in public and industrial buildings (which includes monitoring and analyzing energy consumption in buildings, conducting energy audits, and planning measures to improve efficiency)
- Financing energy efficiency measures using funds from the state budget (energy retrofitting of buildings, equipment replacement, and the introduction of renewable energy sources in buildings).

The *Planning and Construction Law* [32] introduces the concept of green building, stating that green building represents a way of planning, designing, performing construction works, using, and maintaining buildings in a way that reduces greenhouse gas emissions, uses ecologically certified building materials and equipment, efficiently utilizes all resources, reduces production and increases the use of construction and demolition waste in a circular model, uses renewable energy sources, thus improving the life quality of users, protecting the environment, and enhancing sustainability. Encouraging green building is allocated from the national building fund on the proposal of the Ministry of Construction. The construction rules specify that during construction, a mandatory percentage of green areas must be maintained on the plot. Although the term "green infrastructure" is not explicitly mentioned in the documents, the regulations point to the framework that should be followed when implementing green roofs.

The *Environmental Protection Law* [33] is the fundamental legal act regulating environmental protection and improvement in the Republic of Serbia. Its goal is to preserve environmental quality, promote sustainable development, and align with international standards, particularly those of the European Union. Although green roofs are not explicitly mentioned in the text of the law, their application can be linked to several key principles and provisions of this regulation. Sustainable development is listed as a fundamental principle, balancing economic, social, and ecological needs. Considering that green roofs reduce energy consumption for heating and cooling, improve air quality, and mitigate the effect of heat islands in urban environments, these benefits directly support the goal of sustainable development, making green roofs a contribution to implementing this principle. The law also stipulates measures to reduce air pollution, including monitoring emissions. Green roofs contribute to the reduction of emissions as the vegetation on the roofs absorbs harmful particles (PM10, PM2.5) and gases such as nitrogen dioxide (NO_2). Green roofs retain rainwater and slow down the runoff of atmospheric waters, reducing the burden on sewage systems and contributing to water management and urban flood prevention. Biodiversity conservation is also recognized as another benefit of green roofs, as they support the protection and enhancement of biodiversity. Green roofs provide habitats for birds, insects, and plants, and can be used to plant native species, which aligns with the goal of biodiversity conservation. Legal entities and citizens are encouraged to take ecological measures, thus emphasizing the importance of implementing green roofs.

In addition to the laws mentioned above, Serbia has strategies and programs for environmental development and preservation, creating opportunities for the broader

implementation of green roofs. The *Sustainable Development Strategy of the Republic of Serbia* and the *Green Agenda for the Western Balkans* [13] from 2020 highlight the need for green building and the reduction of greenhouse gas emissions, which indirectly supports the implementation of green roofs. The *National Environmental Protection Program* [16] also promotes green infrastructure, but without detailed regulations concerning green roofs. At the local level, cities such as Belgrade and Novi Sad occasionally initiate green roof-related projects and pilot programs, often with the support of non-governmental organizations like the *National Association of Green Roofs* [17].

3. LEGISLATION IN EUROPE

In Europe, particularly in countries such as Germany, the Netherlands, the United Kingdom, and Switzerland, green roofs are becoming a standard in urban planning. While the EU does not have a unified, binding regulation specifically dedicated to green roofs, they are integrated into a broader framework of sustainable urban development policies, energy efficiency, and climate change adaptation. Key documents and directives include:

- *Directive on the Energy Performance of Buildings* [6] – promotes green roofs as part of solutions to reduce energy consumption in buildings, particularly by improving thermal insulation and mitigating the heat island effect.
- *EU Green Infrastructure Strategy* [30] – encourages the implementation of green roofs as part of urban green infrastructure to enhance biodiversity and manage stormwater.
- *EU Climate Change Adaptation Strategy* [21] – emphasizes the role of green roofs in mitigating flooding and heatwaves in urban environments.
- *Urban Innovative Actions (UIA)* [27] – funds projects such as RESILIO in Amsterdam, which explores green and blue roofs for stormwater management and enhancing climate resilience.

Germany is one of the pioneers and leaders in the implementation of this technology [34], where laws have been adopted in recent decades to promote the construction of green roofs on both new and existing buildings. The *German Federal Building Code (BauGB)* [9] and regulations at the state level, such as in North Rhine-Westphalia and Baden-Württemberg, promote the implementation of green roofs through subsidies, tax incentives, and integration into urban planning. For example, cities like Berlin actively promote green roofs through its urban sustainability strategy, particularly through the *Berlin Climate Protection Plan* and the *Urban Climate Plan (StEP Klima)* [25]. The city offers subsidies that cover up to 60% of the costs of installing green roofs, according to the guidelines of the Senate Department for Urban Development of Berlin [25].

Switzerland has a developed and advanced policy related to the implementation of green roofs, particularly in cities like Basel, Zurich, and Geneva. The *Green Building Blog* mentions the Swiss *Minergie standard*, which promotes sustainable building practices, including green roofs [24]. The application of the standard is not mandatory. In 2002, Basel, Switzerland, amended its Building and Construction Law to mandate green roofs on all new and renovated buildings with flat roofs, promoting biodiversity and energy efficiency [2]. For flat roofs exceeding 1,000 m², consultation with a green roof expert is required during design and construction [2].By 2010, Basel had significantly expanded its green roof coverage, with approximately 23% of its 2.4 million m² of flat roof area (about 290,000 m²) greened by 2006, and an additional 80,000 m² added annually thereafter,

according to estimates from green roof research and policy analyses [3]. The claim of over 1.5 million m² by 2010, attributed to the Basel-Stadt Department of Environment, could not be verified with available sources and may reflect later estimates or a specific report not accessed here [3]. According to Zurich's urban planning policies, building regulations introduced in 1991 require green roofs on new municipal buildings with flat roofs, while private developers receive financial subsidies and incentives for implementing green roofs [4]. These measures are part of a broader strategy to enhance biodiversity and sustainability in urban areas [26]. The *Swiss Energy Strategy 2050* [22] promotes green roofs as part of a broader approach to energy-efficient urban planning, supporting sustainable building practices that reduce energy consumption and contribute to climate neutrality. This strategy, in combination with local regulations and incentives in cities like Zurich, encourages the installation of green roofs to improve building energy efficiency and manage the urban environment. These regulations mandate that green roofs are required for certain types of buildings, particularly flat-roofed ones in specific cantons.

The Netherlands, on the other hand, is known for its long-standing tradition of using green roofs, especially due to high population density and the need to create green spaces in urban areas [18]. According to *EFB*, cities like Rotterdam and Amsterdam have been promoting green roofs since the early 2000s, partly due to dense urbanization and the need for sustainable water management and green spaces [7]. Rotterdam is known for its strategy of promoting green roofs, including subsidies introduced as early as 2008 for the installation of green roofs. This initiative is a response to high population density and climate challenges such as stormwater management. In the Netherlands, authorities promote the implementation of green roofs with subsidies and tax incentives, while the city of Amsterdam also has specific goals to increase the number of green roofs as part of its sustainable development plans [32].

In the United Kingdom, green roofs are used on both commercial and residential buildings [14]. According to the *London Environment Strategy* [11] published in 2018, London has set ambitious targets for 50% of its area to be green by 2050, with green roofs being a key part of this strategy. The strategy integrates approaches to address key ecological challenges, including air quality, climate change, waste management, the need for increased green space, and biodiversity. The main objectives include:

- Reducing air pollution – implementing the Ultra Low Emission Zone (ULEZ) and stricter vehicle emission standards.
- Achieving carbon neutrality – becoming a zero-carbon city by 2050 through a transition to a low-carbon, circular economy and energy efficiency.
- Green infrastructure – expanding green spaces and improving resilience to climate change, such as flooding and heatwaves.
- Waste management – maximizing recycling and reducing waste.

In Europe, there are several countries that have introduced mandatory regulations and incentives for green roofs, while others still rely on voluntary initiatives and subsidies. In France, a national legal framework has been established requiring all new commercial buildings to have either green roofs or solar panels [23].

3.1. Germany – A Good Practice Example

In Germany, since 1982, there has been a document known as the*FLL Richtlinien für die Planung, Ausführung und Pflege von Dachbegrünungen* [8] (translated as

"Guidelines for the Planning, Construction, and Maintenance of Green Roofs"), which represents one of the most influential and recognized standards for green roofs (hereafter referred to as the FLL document), not only in Germany but also globally. This document was developed by the *Forschungsgesellschaft Landschaftsentwicklung Landschaftsbau*, a non-profit organization founded in 1975, which focuses on research and development in landscape architecture and green technologies.

In Germany, the *FLL* document is used in conjunction with national building standards (*DIN* standards). It was first published in 1982 under the title *Principles for Roof Greening*, and since 1990, it has been revised several times to adapt to the latest technical and ecological advancements. The most recent version, published in 2018, offers significant updates compared to previous editions and serves as a comprehensive guide for designers, architects, engineers, and contractors. The guidelines are often used as the basis for the development of national regulations in other countries.

Within the *FLL* document, several areas related to green roofs are covered, such as:

- Classification of Green Roofs
- Structural Requirements (roof load capacity, root penetration protection, waterproofing, etc.)
- Layers of Green Roofs
- Stormwater Management
- Biodiversity
- Use with Solar Systems
- Maintenance.

The *FLL* document includes tables with reference values, such as values for roof load, annual water retention depending on the layer thickness. In addition to the guidelines, the document specifies methods for testing materials, such as resistance of layers to root penetration, drainage capacity, thus ensuring a standardized approach. The *FLL* document is adapted to various climatic conditions, which makes it applicable beyond the borders of Germany. It has become a model for countries that are in the process of developing their own regulations, such as the United States and Australia, where American *ASTM* standards are partially inspired by the *FLL* document.

4. CONCLUSION

Green roofs represent an innovative solution for improving urban environments, reducing ecological footprints, and adapting to climate change. In the Republic of Serbia, the regulatory framework regarding green roofs is still in the developmental phase, with limited legislation and incentives compared to countries like Germany and Switzerland, where standards, subsidies, and mandatory measures have already reached an impressive level. Globally, there is a growing trend toward creating favorable conditions for the implementation of such solutions through urban planning and environmental policies, while Serbia has the opportunity to improve its regulations by integrating international practices. The lack of clear regulations and financial incentives limits broader adoption. The main challenges in Serbia include high initial costs, complicated material imports, and insufficient awareness of the benefits of green roofs. Aligning domestic legislation first with European, and then global standards, alongside clear financial incentives and education, is crucial for expanding the application of green roofs and realizing their full potential in sustainable development.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

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Stručni rad

STEEL-TIMBER COMPOSITE STRUCTURES

Summary: *This paper analyzes composite steel-timber structures. Composite beams are designed to maximize the use of all materials the composite beams are made from. In addition to monolithic timber, cross-laminated timber is also used in structures, which requires special analysis. The paper presents an analysis of a steel-cross-laminated timber composite beam based on existing standards, with recommendations for solving problems due to the lack of specific standards for this type of structure. This paper aims to contribute to a better understanding of this type of structure and to encourage the development of standards, which would enable their wider application in the future.*
Keywords: *composite structures, steel, cross-laminated timber, standard*

SPREGNUTE KONSTRUKCIJE TIPA ČELIK-DRVO

Rezime: *Predmet ovog rada je analiza spregnutih konstrukcija od čelika i drveta. Spregnuti nosači se projektuju tako da se maksimalno iskoriste svi materijali od kojih su nosači napravljeni. Pored monolitnog drveta, u konstrukcijama se primenjuje i unakrsno lamelirano drvo, što zahteva posebnu analizu. U radu je data analiza spregnutog nosača od čelika i unakrsno lameliranog drveta na osnovu postojećih standarda, sa preporukama za rešavanje problema zbog nepostojanja konkretnih standarda za ovu vrstu konstrukcija. Ovaj rad ima za cilj da doprinese boljem razumevanju ove vrste konstrukcija i da podstakne razvoj standarda, što bi omogućilo njihovu širu primenu u budućnosti.*
Ključne reči: *spregnute konstrukcije, čelik, unakrsno lamelirano drvo, standard*

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1. INTRODUCTION

In contemporary structural engineering, the exigency for efficacious, sustainable, and economical solutions is pervasive. Composite structures, which synergistically amalgamate disparate materials to capitalize on their complementary attributes, constitute a salient advancement in this context. Among these, steel-timber composite systems are gaining considerable prominence, predicated on their potential to achieve elevated load-bearing capacities, mass reduction, and the utilization of a renewable resource – timber.

Conventionally conceived, steel-timber composite beams primarily encompassed the fusion of ferrous elements with monolithic timber beams or slabs [1, 2]. However, the progressive development in cross-laminated timber (CLT) production technology has engendered novel perspectives for the implementation of timber in more intricate load-bearing assemblies, including composite structures with steel. CLT is characterized by high strength, stability, and dimensional precision, rendering it a compelling material for diverse construction applications [3].

The integration of steel and CLT in composite beams presents the opportunity for optimizing the utilization of both constituent materials. Steel affords superior tensile strength and ductility, while CLT contributes substantial resistance to shear and compression, in conjunction with favorable stiffness characteristics and a reduction in the self-weight of the structure when compared to purely steel alternatives. Such composite systems can be particularly advantageous in scenarios necessitating large spans, high load-bearing capacities, or accelerated construction schedules.

Notwithstanding the burgeoning interest and application of steel-CLT composite structures, there is a current paucity of unified and comprehensive standards that would govern their analysis and design. Existing norms for steel-monolithic timber composite structures, as well as those that individually address CLT structures, can serve as a foundational basis for analysis; however, the specific interaction between steel and CLT necessitates a discrete methodological approach.

This paper analyzes the structural behavior of a composite beam formed from steel and cross-laminated timber, predicated on pertinent existing standards. The primary objective is to delineate the salient aspects of the design of such structures, underscore the challenges arising from the absence of specific norms, and propose guidelines for mitigating these issues. The ultimate outcome of this paper is intended to be a contribution to an enhanced understanding of this structural typology and the initiation of the development of standards that would enable their more widespread and reliable implementation in future construction endeavors.

2. THEORY OF COMPOSITE STRUCTURES

Composite girders are designed and analyzed to ensure maximal utilization of all constituent materials. For a girder to function compositely, appropriate shear connectors (e.g., dowels, studs) must be implemented to transfer shear forces at the interface between the two materials. Consequently, the characteristics of the shear connectors employed for composite action are paramount to the overall structural behavior of the girder. These connectors prevent independent deformation of the elements, thereby resulting in a load-bearing capacity and stiffness of the composite girder that exceed the arithmetic sum of those properties of its individual components.

In the event that the connection between the elements being coupled is not adequately achieved, shear force transfer at the interface between the composite elements is impeded. This implies that the girder elements behave independently under loading and deformation, and slippage occurs at the contact surface. If a sufficient number of shear connectors are incorporated into such a system to prevent slippage, the composite section exhibits monolithic behavior. In this scenario, shear forces exist within the slip planes, slippage is practically negligible, and this type of connection is termed complete or rigid composite action. Conversely, when the implemented shear connectors lack sufficient load-bearing capacity and stiffness to facilitate complete interaction, the composite action is partial [4].

The subsequent illustration (Figure 1) depicts the static effects in a simply supported beam subjected to a concentrated load at mid-span. The beam comprises a steel profile and a slab, which can be constructed from cross-laminated timber (CLT) panels or reinforced concrete.

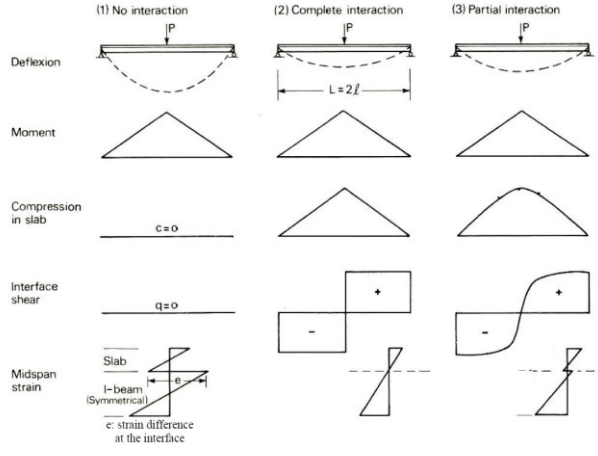


Figure 1. Effects of composite action: no composite action, rigid composite action, and partial composite action [5]

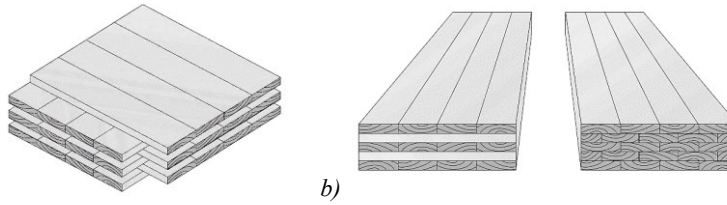
When girders are constructed using a combination of solid timber and steel, which is a frequent occurrence in scaffold structures, the dimensioning is performed based on the condition that the deformations of the steel and timber are mutually identical [6]. Assuming that the steel is of class S235 and the timber is of class C22 or C24 [7], it can be inferred that the ratio of normal stresses in the steel and timber, under equal strains, is approximately equal to the ratio of their respective moduli of elasticity. Consequently, it can be concluded that steel and timber are suitable for the fabrication of composite girders, as the exhaustion of their load-bearing capacity occurs simultaneously. This ensures maximal utilization of both materials.

Composite structures made of steel and cross-laminated timber can be distinguished as a specific class of composite structures within the broader category of steel-timber composite structures. The specificities of such composite structures lie in the mechanical characteristics of cross-laminated timber, which differ from those of solid timber and therefore necessitate a distinct analysis.

3. CROSS-LAMINATED TIMBER

Cross-laminated timber (CLT) is an engineered wood panel fabricated from a minimum of three orthogonally bonded lamellar layers, with configurations extending up to thirteen layers, though typically not exceeding seven. This orthogonal arrangement of lamellae in successive layers (Figure 2-a) is predominantly at a right angle, although alternative angular orientations between adjacent layers are also employed in certain panel types. The primary method of lamination involves adhesive bonding.

The resultant cross-section exhibits symmetry, with the outermost lamellar layers possessing identical grain orientations. While an odd number of layers is the prevalent configuration, even-numbered layer arrangements are also utilized. Figure 2-b further delineates the distinguishing characteristics between cross-laminated timber (CLT) and glued laminated timber (GLT), highlighting their structural and manufacturing differences.



a) *b)*

Figure 2. a) Configuration of CLT panels, b) CLT and GLT panel [3]

The thickness of lamellae used in CLT panel production typically ranges from 6 to 45 mm, and, the width of individual lamellae spans between 80 and 240 mm [8]. Maximum production dimensions for CLT panels are up to 500 mm in thickness, 3 m in width, and 16-20 m in length [9].

CLT panels are advantageous for planar structural applications such as intermediate floor constructions or bridge deck slabs. A significant benefit of utilizing CLT panels over concrete is the reduced structural mass, leading to considerably faster construction times [3].

CLT employed in structural engineering is predominantly manufactured from softwood (coniferous species). Spruce is the most common species used in Europe, but other suitable species include fir, pine, and larch. In North America, Douglas fir is frequently utilized, while Australian pine and radiata pine are common in Australia. Regardless of the timber species, the lamellae must possess a defined strength class according to the relevant standard [7].

Adhesives are employed for bonding the lamellae. Several adhesive types are suitable for CLT production, including phenolic, emulsion, and polyurethane-based adhesives. Polyurethane adhesives are commonly used in the production of load-bearing structures, particularly for GLT in Europe, whereas phenolic adhesives are more prevalent in North America [3].

Unlike other construction materials such as concrete or steel, whose standards have been significantly refined in recent decades, design standards for timber structures are still in a relatively earlier stage of development. This slower progress is primarily attributed to the inherent complexities in fully comprehending the material mechanics of wood-based products [10].

Initial standardization efforts for CLT in Europe commenced in 2008, culminating in the proposal of the first draft standard [8]. A group of Canadian researchers initiated extensive testing, which resulted in the publication of a comprehensive CLT Handbook [3]. This handbook was also released in the USA [11].

4. CURRENT APPROACH TO THE STATIC ANALYSIS OF STEEL-CLT COMPOSITE STRUCTURES

Given the absence of specific standards for steel-CLT composite structures, the current methodology for analyzing static effects involves treating the structure as an assembly of discrete components. The total static effects (bending moments, shear forces, and axial forces) acting on the composite structure are distributed to the individual elements – primarily the steel girders and CLT panels – based on assumptions regarding their relative stiffness and the connection mechanism. Following this distribution, each individual element is analyzed and dimensioned separately (Figure 3).

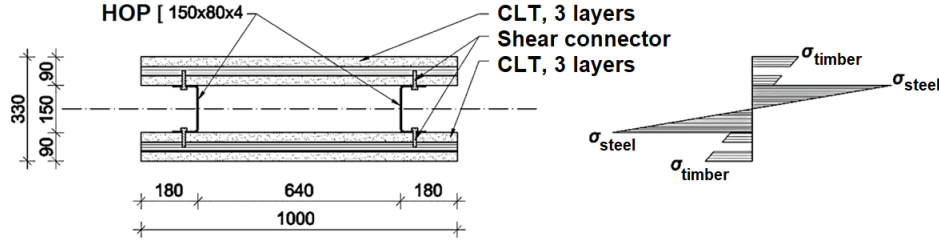


Figure 3. Observed steel-CLT composite structure. Normal stress distribution

The assessment of the load-bearing capacity and stability of steel girders is conducted in accordance with Eurocode 3 [12]. This standard provides comprehensive guidelines for the design of cross-sections subjected to bending, shear, axial forces, and their combinations.

The analysis of the load-bearing capacity of CLT panels is performed in accordance with Eurocode 5 [13]. The load-bearing capacity of CLT panels is determined by the capacity of the weakest element within the panel, according to the criteria specified in Eurocode 5 for the combined effects of tension and bending or compression and bending. Failure of CLT panels typically occurs due to the combined action of normal tensile forces and bending moments in the outermost lamellae.

The initial step involves determining the idealized moment of inertia with respect to steel:

$$I_i = I_{steel} + I_{timber} \cdot E_{steel}/E_{timber} \quad (1)$$

$$E_0/E_{90} \approx 0 \quad (2)$$

where: I_{steel} - moment of inertia of steel profile,
I_{timber} – moment of inertia of CLT panels,
E_{steel} – steel elasticity modulus,
$E_{timber} = E_0$ – timber elasticity modulus parallel to the grain,
E_{90} – timber elasticity modulus perpendicular to the grain.

Subsequently, the normal force and bending moment in the outermost lamellae of the CLT panel are obtained using the following relationships:

$$N_1 = M_{Ed}/I_i \cdot e \cdot A_1 \tag{3}$$

$$M_1 = M_{Ed}/I_i \cdot I_1 \cdot E_{steel}/E_{timber} \tag{4}$$

where: M_{Ed} – design bending moment,
e – distance between neutral axis and outermost lamellae centroid,
A_1 – outermost lamellae area,
I_1 – outermost lamellae moment of inertia.

These calculated values are then utilized to verify the load-bearing capacity of the CLT panel in accordance with Eurocode 5. The bending moment in the steel girder is determined using the relationship:

$$M_{steel} = M_{Ed}/I_i \cdot I_{steel} \tag{5}$$

The resulting bending moment value in the steel girder is then used to verify its load-bearing capacity in accordance with Eurocode 3.

The calculation of the load-bearing capacity of shear connectors, which transfer shear forces at the steel-CLT interface, depends on the type of connector used.

Headed studs embedded in mortar. The shear capacity is often calculated according to Eurocode 4 [14]. This application draws an analogy with steel-concrete composite structures, where the mortar surrounding the stud is considered to perform a similar function to concrete.

Screw-type dowels directly connecting steel and CLT. The shear capacity is calculated based on the rules for mechanical joints in timber structures, as defined in Eurocode 5.

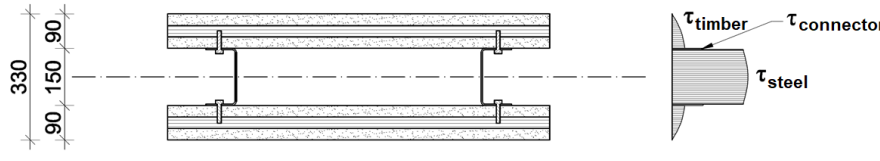


Figure 4. Observed steel-CLT composite structure. Shear stress distribution

The shear stress in the slipping plane is calculated according to the Zhuravsky formula, and the shear force in a single shear connector is:

$$F_{Ed} = \tau \cdot b_0 \cdot e \leq F_{Rd} \tag{6}$$

where: τ – shear stress in the slipping plane,
b_0 – flange width of the steel profile,
e – spacing between the shear connectors,
F_{Rd} - design shear resistance of a single shear connector.

A more precise analysis of the load-bearing capacity of the shear connector can be conducted by applying the FEM or through experimental testing [1, 15].

5. DISCUSSION

The current methodology, which relies on the separate analysis of components and the application of standards for individual materials, has certain limitations. The complex interactive behavior of the integrated steel-CLT system is not fully accounted for, and the application of standards for other types of composite or exclusively timber structures can lead to overly conservative or potentially unsafe solutions in specific situations. Therefore, there is a clear need for the development of specific standards and guidelines that would encompass a unified analysis and design approach for steel-CLT composite structures, considering the specificities of the interaction between these two materials and the behavior of the connection elements within such systems.

Replacing the central part of the CLT girder with steel profiles offers several significant advantages. The most notable benefit is that the manufacturing complexity of two thinner CLT panels is considerably less than that of producing a single thicker panel. Creating a void within the composite girder achieves a substantial saving in timber and also allows for the potential routing of installations.

In most cases, the critical component in the load-bearing capacity calculation of the observed composite girder is the outermost tension lamella. Under tension, timber exhibits brittle behavior, which means that girders made solely from CLT panels are not well-suited for structures requiring a certain level of ductility (e.g., bridges). From a ductility standpoint, a composite structure is more favorable for two primary reasons. One is the inherent ductility of steel profiles, and the other is the presence of shear connectors, which can be designed to allow slippage at the steel-timber interface in the event of structural overload.

6. CONCLUSSION

Steel-timber composite structures represent an efficient structural system that synergistically combines the advantages of steel (high tensile strength and ductility) and timber (good compressive and shear strength, relatively low weight, and renewability). The principle of composite action is achieved by connecting the steel and timber components, enabling them to jointly bear loads and utilize the load-bearing capacity of both materials more effectively compared to their separate use. This approach results in higher load-bearing capacity, stiffness, and optimized structural mass, which is particularly significant for elements with larger spans and loads.

The application of composite structures made of steel and wood-based materials offers the advantage of lower mass compared to other types of composite floor structures (steel-concrete and timber-concrete). A significant reason for employing this type of composite structure is its considerably lower environmental impact compared to concrete, primarily because concrete production releases substantial amounts of carbon dioxide into the atmosphere, whereas wood absorbs and incorporates carbon into its structure during growth.

Despite these advantages, such structures also have drawbacks. The main disadvantage is the complex fabrication process of the composite girder, which involves multiple stages: manufacturing the CLT panels, producing the steel profiles, and installing the shear connectors. The stiffness ratio of the composite floor structure in the direction of support and perpendicular to it is less favorable than that of solid CLT

panels. This is because CLT panels possess significant stiffness in both orthogonal directions, while the proposed composite structure has considerably greater stiffness along the axis of the steel profiles.

Based on the presented advantages and disadvantages of a steel-CLT composite girder compared to a CLT girder, it can be concluded that the composite girder offers more benefits than drawbacks. Material savings and more ductile behavior make the composite girder more suitable for application in complex structures such as bridges, where the use of solid timber girders is often not feasible due to their very low ductility. Furthermore, a composite girder can be fabricated with a significantly greater height than a solid CLT girder, meaning it can be used for bridging larger spans than is typical for timber bridges.

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

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Jelisaveta Nikolić[3]

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Stručni rad

SMERNICE ZA TRETIRANJE OTPADA OD RUŠENJA I GRAĐENJA U CILJU DOBIJANJA RECIKLIRANOG AGREGATA, KVALITET DOBIJENOG AGREGATA NA BAZI IZVRŠENIH LABORATORIJSKIH ISPITIVANJA

Rezime: *Reciklirani agregat, koji obuhvata otpad od rušenja, renoviranja i samog procesa izgradnje, i u Srbiji polako počinje da dobija na značaju i da pronalazi svoje mesto na tržištu. O tome svedoče sve češći zahtevi za ispitivanja i dokazivanje upotrebljivosti ovog materijala, kao i uspostavljanje određenih zakonskih okvira, koji prepoznaju značaj ove vrste agregata. U radu su dati osnovni preduslovi proizvodnje kvalitetnog recikliranog agregata i iskustva Instituta IMS stečena kroz vršenje laboratorijskih ispitivanja.*
Ključne reči: *reciklirani agregat, laboratorijska ispitivanja, beton, kamen*

GUIDELINES FOR THE TREATMENT OF DEMOLITION AND CONSTRUCTION WASTE TO OBTAIN RECYCLED AGGREGATE, THE QUALITY OF THE AGGREGATE OBTAINED BASED ON THE PERFORMED LABORATORY TESTS

Summary: *Recycled aggregate, which includes waste from demolition, renovation and the construction process itself, is slowly beginning to gain importance and find its place on the market in Serbia as well. This is evidenced by the increasingly frequent requests for testing and proving the usability of this material, as well as the establishment of certain legal frameworks, which recognize the importance of this type of aggregate. The present paper provides the basic prerequisites for the production of high-quality recycled aggregate and the experience of the IMS Institute gained through conducting of laboratory tests.*
Keywords: *recycled aggregate, laboratory testing, concrete, stone.*

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1. UVOD

Reciklirani agregat (RA) se, u skladu sa definicijama postavljenim odgovarajućim standardima [13-15] dobija obradom neorganskog materijala prethodno korišćenog u građevinarstvu. Obuhvata otpad od rušenja, renoviranja i samog procesa izgradnje. Obuhvata beton, opeku, kamen, malter, gips, drvo, staklo, plastiku, gumu, metal, bitumenske proizvode, zemljište i sl.

U svetskim okvirima, najveću količinu otpada od rušenja generiše Kina (~2360 miliona tona), zatim Sjedinjene Američke Države (~600 miliona tona) i Indija (~530 miliona tona u 2016). Evropska zajednica takođe generiše značajnu količinu ove vrste otpada, gde su Francuska i Nemačka u 2018. godini bile zastupljene sa oko 240, odnosno 225 miliona tona [9]. U Evropi je građevinski otpad i otpad od rušenja tokom 2018. godine činio čak 36% od oko $2{,}5 \cdot 10^9$ t ukupno proizvedenog otpada [6].

Sa ciljem da se smanje negativni ekološki i ekonomski uticaji ovako velike količine otpada u većini razvijenih zemalja, ulažu se veliki napori za pronalaženje naprednih načina za prevođenje "otpada" u materijal koji ima vrednost; istražuju se tehnološki procesi koji postupak recikliranja čine ekonomski opravdanim i stimuliše se njihova primena [2]. Zamena prirodnog agregata recikliranim, stekla je značaj širom sveta kao način za smanjenje potrošnje prirodnog agregata i zadovoljavanje građevinskih zahteva na održiv način.

U Srbiji je situacija znatno drugačija. Proizvodnja RA je više nego skromna, nema potrebne tehnološke infrastrukture, nema odgovarajućeg podsticaja od strane države i, ne manje važno, nema poverenja u kvalitet ovog materijala [2]. Takođe, upotreba alternativnih materijala, zbog značajnih troškova vezanih za njihov transport i tretman, uglavnom nije ekonomski isplativa [3].

Mogućnost upotrebe RA u Srbiji je istraživana uglavnom sa aspekta akademskih istraživanja, publikovanih radova i studija. Ignjatović i Marinković [4] i Jevtić i dr. [5] su se bavili svojstvima betona na bazi RA. Radević (2017) i Radević i dr. [7,8] istraživali su mogućnosti proizvodnje asfalta zadovoljavajućeg kvaliteta delimičnom zamenom prirodnog krečnjačkog agregata recikliranim. Đokić i dr. [3] bavili su se istraživanjima mogućnosti primene mešavina prirodnih i RA u kolovoznim konstrukcijama. Abramović i dr. [1] istraživali su kvalitet i ekološke i ekonomske aspekte korišćenja RA sa područja Beograda i utvrdili da se RA može koristiti kao zamena za prirodne agregate i istovremeno dokazali da se njihovom upotrebom umesto proizvodnje agregata iz prirodnih resursa postiže ušteda od 23% energije potrebne za proizvodnju recikliranog agregata i ušteda od skoro 0,5 evra/t agregata.

Uprkos tome, primena RA u Srbiji u građevinskoj praksi još uvek nije našla svoje mesto. Situaciju je u određenoj, zasad skromnoj meri, počeo da menja nedavno doneti Pravilnik o vrstama otpada za koje se može podneti zahtev, dozvoljenim postupcima i tehnologijama tretmana za vrste otpada i drugim posebnim elementima za određivanje prestanka statusa otpada [12], koji daje odredbe za različite vrste otpada. Između ostalog, pravilnik daje tehničke zahteve i kriterijume i za reciklirani agregat.

2. PROIZVODNJA RECIKLIRANOG AGREGATA

Svojstva recikliranih agregata određena su ulaznim materijalom, vrstom mašina korišćenih za preradu i samim postupkom prerade [10]. Transformacija građevinskog

otpada u RA željenog kvaliteta, zahteva određene pripremne radnje, koje obuhvataju više različitih postupaka:

Odvojeno skladištenje otpadnog materijala na izvoru omogućava delotvornije recikliranje i bolji kvalitet agregata. Ovo podrazumeva razvrstavanje otpada, odvajanje gipsa, drveta, papira, čelika i različitih drugih nečistoća, ostataka i sl. Stepen odvajanja zavisi od mogućnosti dostupnih na lokaciji (npr. prostora i radne snage), ali i od troškova i prihoda ove faze pripremnih radova [11]. Takođe, istovremeno je potrebno vršiti dokumentovanu kontrolu ulaznog materijala, koja treba da identifikuje: prirodu ovog materijala i izvor i mesto porekla.

Sledi usitnjavanje, odnosno propuštanje otpada kroz stabilne ili mobilne mašine za drobljenje, uz, po potrebi, korišćenje magnetnih separatora radi uklanjanja preostalih komada gvožđa. Vrsta drobilice koja se koristi, npr. čeljusna ili rotirajuća, brzina rotacije i sl., određuju intenzitet usitnjavanja ulaznog materijala i kvalitet gotovog proizvoda [10].

Potrebno je napomenuti da se mešanjem različitih ulaznih materijala, može uticati na kvalitet RA i u određenoj meri ga prilagoditi postojećim potrebama [10].

Svođenje RA na odgovarajuću granulaciju vrši se kroz sistem za prosejavanje, ili ponekad regulisanjem postupka drobljenja putem podešavanja samog uređaja u pogledu izlazne veličine zrna. Dobar izbor postrojenja za drobljenje i separisanje u velikoj meri doprinosi proizvodnji kvalitetnog RA odgovarajuće granulacije.

Poslednju fazu predstavlja laboratorijsko ispitivanje fizičkih, mehaničkih i hemijskih svojstava agregata u skladu sa relevantnim standardima, tehničkim propisima, tehničkim specifikacijama ili na drugi način definisanim utvrđenim zahtevima.

3. KVALITET RECIKLIRANOG AGREGATA

Kvalitet RA predstavlja jedan od osnovnih preduslova za njegovu buduću namenu. Sa zakonske strane definisan je istim onim standardima kojima je definisan kvalitet prirodnog agregata, te podleže primeni u skladu sa:

- SRPS EN 12620:2010 Agregati za beton
- SRPS EN 13043:2007 Agregati za bitumenske mešavine i površinsku obradu kolovoza, aerodroma i drugih saobraćajnih površina
- SRPS EN 13242:2010 Agregati za nevezane i hidraulički vezane materijale za upotrebu u građevinskim radovima i izgradnji puteva

Neophodno je napomenuti da svi ovi EN standardi napominju da su nove metode ispitivanja za RA u odmakloj fazi pripreme i da je rad na standardizaciji nedovoljno poznatih materijala iz sekundarnih nalazišta, odnosno RA, otpočeo tek nedavno, te da je potrebno više vremena da se jasno odrede njihovo poreklo i karakteristike. Navodi se i da se mogu utvrditi dodatne karakteristike i zahtevi za pojedinačne slučajeve, zavisno od iskustva sa upotrebom proizvoda i specifičnim ugovornim dokumentima [13-15].

Generalno, odavno je poznato da je primena RA ograničena zbog slabijeg kvaliteta u odnosu na prirodni: većeg upijanja vode, manje zapreminske i specifične mase, veće količine sitnih čestica, većeg sadržaja organskih i eventualno drugih štetnih materija, manje otpornosti prema habanju, drobljenju i prema dejstvu mraza, lošijeg oblika zrna i dr. [5,9]. Osim toga, nedovoljno je proučeno ponašanje određenih sastojaka RA nakon ugradnje u betonske i druge konstrukcije (npr. staklo, keramičke pločice, gips, plastika itd.).

4. ISKUSTVA LABORATORIJSKIH ISPITIVANJA

Prilikom razmatranja kvaliteta RA i sagledavanja mogućnosti njegove primene, neophodno je imati na umu da se zbog vrste izvornog materijala i načina na koji se proizvodi, mehanička svojstva i druge karakteristike RA mogu značajno razlikovati od polaznog materijala.

Sistematska ispitivanja recikliranog agregata u Srbiji do sada nisu vršena. Ispitivanja izvršena u akreditovanim laboratorijama Instituta IMS vršena su po principu studija slučaja, na uzorcima RA dobijenih iz građevinskog otpada i otpada od rušenja odloženog na deponije i kasnije prerađenog drobljenjem na različitim industrijskim drobilicama od strane različitih operatera. Sva izvršena ispitivanja, i to u Laboratoriji za kamen i agregat, Laboratoriji za puteve i geotehniku i Laboratoriji za veziva, hemiju i maltere, mogu se sagledati sa dva osnovna aspekta. Prvi je ispitivanje i istraživanje u cilju dokazivanja funkcionalne mogućnosti korišćenja građevinskog otpada i otpada od rušenja nakon transformacije u RA, na prvom mestu kao zamene prirodnog agregata u izgradnji puteva za izradu nasipa i nevezanih nosećih slojeva i, u manjoj meri, za izradu betona.

Drugi aspekt ispitivanja predstavlja dokazivanje neštetnosti za okolinu i zdravlje ljudi, odnosno pribavljanje dokaza da upotreba ove vrste agregata neće uzrokovati veći rizik u smislu opasnog svojstva od rizika koji postoji kod primene agregata proizvedenog iz konvencionalnih mineralnih nalazišta [12].

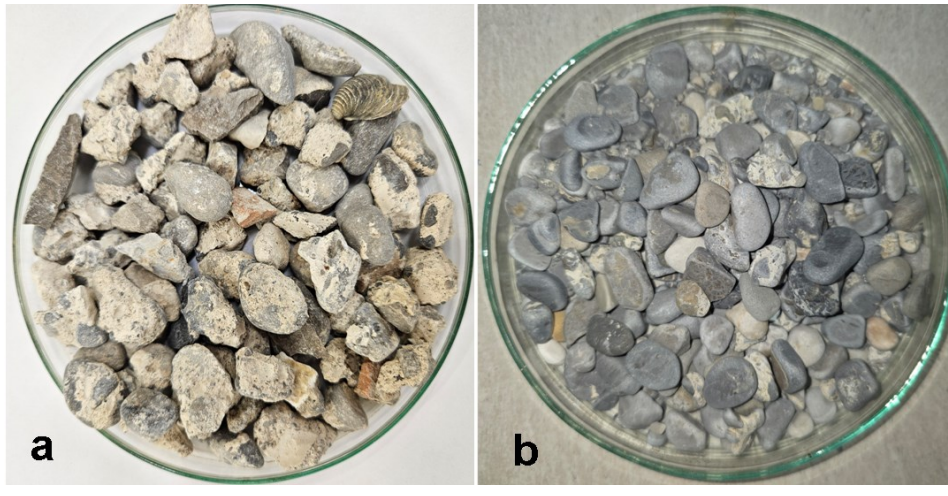
Izvršeni obim ispitivanja i broj uzoraka su zasad relativno skromni, ali uprkos tome, daju određene smernice za definisanje mogućih načina korišćenja RA i postavljaju temelje sagledavanju kvaliteta ovog u Srbiji novog proizvoda.

Obavljena ispitivanja RA dobijenih drobljenjem mešanog građevinskog otpada pokazala su nezadovoljavajuće rezultate. Najpre je, u cilju identifikacije sastojaka mešanog agregata, izvršeno ispitivanje u skladu sa SRPS EN 933-11:2009 Ispitivanje geometrijskih svojstava agregata, Klasifikacija sastojaka krupnog recikliranog agregata [17], koja su pokazala da se u ovakvim agregatima kao komponente pojavljuju beton (grupa Rc), prirodni agregat i kamen (grupa Ru), proizvodi od gline za zidanje (grupa Rb - blokovi za zidanje, opeka, crep, keramičke pločice), staklo (grupa Rg - ambalažno, ogledalsko), bitumenski materijali (grupa Ra) i druge komponente iz grupe X, kao što su: komadi plastike, gipsano-kartonski paneli, metal (ekseri, šrafovi, žica). Uzorci koji su sadržali fragmente gipsa očekivano su pokazali značajno veće sadržaje sulfata rastvorljivih u vodi, koji se smatraju potencijalno reaktivnim, jer mogu da dovedu do ekspanzivnog razaranja betona i kolovoznih konstrukcija. Osim toga, kod ovakvih uzoraka mogu se pojaviti i pojavljuju se i plutajući sastojci organskog hemijskog sastava, kao što su stiropor, poliuretanska pena, hartija, drvo itd., koji mogu ugroziti kvalitet RA i čije ponašanje nakon ugradnje u konstrukcije nije dovoljno proučeno. Ispitivanje otpornosti na mraz mešanog agregata metodom SRPS EN 1367-2:2010 Ispitivanja toplotnih i vremenskih uticaja na svojstva agregata, Ispitivanje magnezijum-sulfatom pokazalo je veoma veliki gubitak mase, čak i preko 40%. Svim izvršenim ispitivanjima dodatno je potvrđen značaj razdvajanja različitih vrsta i komponenti otpada.

Rezultati ispitivanja RA dobijenih drobljenjem betona različitih klasa čvrstoće i drobljenjem elemenata od gline, oba iz izvora sirovine nepoznatog porekla i kvaliteta sagledavani su sa stanovišta zahteva JP Putevi Srbije, Tehničke specifikacije, t. 2 Posebni tehnički uslovi, 2018, kao agregat za nevezane noseće slojeve (NNS) i u skladu sa zahtevima standarda SRPS EN 12620:2010, kao agregat za beton.

Ispitivanje otpornosti na drobljenje vršeno je u skladu sa SRPS EN 1097-2:2020 (LA). Metoda Los Anđeles je dobro i široko poznata metoda, veoma dugo u upotrebi u svetu i kod nas. Dobijene vrednosti LA su iznosile od oko 25 do oko 35 % za agregat dobijen odradom betona i između 30 i 40 % za agregat dobijen preradom glinenih elemenata. Obzirom na Tehničkim specifikacijama [16] postavljene kriterijume za vrednost LA, za srednja ili teška saobraćajna opterećenja najviše 30 % i za mala saobraćajna opterećenja najviše 35 % dobijene vrednosti uglavnom daju mogućnost za korišćenje RA za NNS u putogradnji.

Ispitivanje otpornosti na drobljenje u skladu sa SRPS EN 1097-1:2013 (Mikro-Deval) pokazalo je širok opseg dobijenih vrednosti, od ispod 20 % za agregat dobijen odradom betona, do blizu 50 % za agregat dobijen preradom glinenih elemenata.



Slika 1. Agregat dobijen preradom betona: a. pre i b. nakon ispitivanja metodom Mikro-Deval.



Slika 2. Agregat dobijen preradom glinenih proizvoda: a. pre i b. nakon ispitivanja metodom Mikro-Deval.

Ispitivanje upijanja vode vršeno je metodom SRPS EN 1097-6:2016. Ispitivanje mehaničkih i fizičkih svojstava agregata, Određivanje stvarne zapreminske mase i upijanja vode. Za agregat dobijen odradom betona dobijeni su rezultati između 3 i 5 %, dok su kod agregata dobijenog preradom glinenih elemenata dobijene vrednosti između 13 i 14 %. Ovi rezultati nisu ograničavajući sa aspekta upotrebe za NNS, ali su vrlo visoki sa aspekta upotrebe kao agregat za beton.

Ispitivanje agregata na otpornost prema mrazu može se vršiti na više načina, pri čemu je za RA sa fragmentima cementnog veziva ispitivanje magnezijum-sulfatom, metodom SRPS EN 1367-2:2010 nepodesno [13]. Ispitivanje RA dobijenih preradom glinenih elemenata ispitanih ovom metodom pokazalo je maseni gubitak od oko 20%, što se može smatrati vrednošću koja otvara put daljim istraživanjima mogućnosti upotrebe ove vrste agregata. Ispitivanje betonskog RA potrebno je vršiti metodom SRPS EN 1367-1:2010 Ispitivanja toplotnih i vremenskih uticaja na svojstva agregata, Određivanje otpornosti prema zamrzavanju i odmrzavanju.

Jedno od bitnih svojstava koje određuje upotrebljivost RA je i oblik zrna koji se ispituje u skladu sa SRPS EN 933-4:2010. Indeks oblika značajno utiče na rezultate mehaničkih svojstava agregata, kako na vrednost otpornosti prema drobljenju i prema habanju, tako i na visinu Kalifornijskog indeksa nosivosti (CBR), pri čemu se ovim svojstvom u određenoj meri može upravljati odgovarajućim tehnološkim postupcima i prilagođenim režimom drobljenja izvornog materijala. Izvršena ispitivanja pokazala su da je RA pripremljen preradom glinenih proizvoda, a gde je izvor materijala sadržao značajniji udeo keramičkih pločica i blokovskih elemenata, dobijeni agregat pokazao značajnije učešće pločastih zrna i visoke vrednosti indeksa oblika, a samim tim i lošija mehanička svojstva.

Ispitivanja geomehaničkih svojstava bitnih za upotrebljivost agregata za nasipe i nevezane noseće slojeve vršena je pre svega Proktorovim opitom u skladu sa SRPS EN 13286-2:2012/AC:2013 i određivanjem Kalifornijskog indeksa nosivosti (CBR) u skladu sa SRPS EN 13286-47:2022. Napominjemo da zahtev Tehničkih specifikacija [16] za drobljene i mešane agregate koji sadrže više od 50 % drobljenih zrna iznosi najmanje 80%. Ispitivani agregati su pokazali širok opseg rezultata, tako da su agregati dobijeni preradom betona pokazali rezultate počev od 60 do čak 200 %, dok se vrednost CBR-a na agregatima dobijenim preradom glinenih proizvoda kretala od 30 do oko 100%. Ispitivanje na mešanim agregatima pokazalo je vrednost od oko 20 %. Napominjemo da je odgovarajući granulometrijski sastav, odnosno odgovarajuća kompozicija veličine zrna jedan od bitnih preduslova za dobijanje zadovoljavajućih rezultata. Ovo najčešće znači da sadržaj frakcije 0-4 mm u ukupnoj masi uzorka treba da bude dovoljno veliki da omogući dobru zbijenost prilikom ugradnje, kao i da sadržaj zrna manjih od 0,063 mm ne bude veći od 5 %. Obično je poželjno da sadržaj frakcije 0-4 mm iznosi minimum 30 do 40 % ukupne mase koja se drobi.

Hemijska ispitivanja izvršena u cilju dokazivanja neštetnosti za okolinu i zdravlje ljudi su pokazala da većina ispitanih uzoraka zadovoljava Pravilnikom [12] postavljene kriterijume, ali su se pojavljivali i uzorci sa nešto povećanim sadržajem olova i hroma kod betonskih agregata i značajnom količinom sulfata kod mešanog agregata. Nijedan od uzoraka nije pokazao nedozvoljeni sadržaj organskih materija.

5. ZAKLJUČAK

U Laboratorijama Instituta IMS u Beogradu izvršeno je laboratorijsko ispitivanje više različitih uzoraka recikliranih agregata dobijenih preradom otpada od rušenja i građenja odloženog na deponije, proizvedenih od strane različitih operatera i bez kontrole ulaznog, „izvornog“ materijala.

Tokom ispitivanja utvrđeno je da su homogeni uzorci poput pretežno betonskih ili keramičkih materijala imali mahom povoljna svojstva za dalju primenu u građevinarstvu. Ispitivanje njihovih fizičko-mehaničkih svojstava i zahtevana hemijska ispitivanja sprovedena u cilju utvrđivanja neštetnosti po ljude i prirodnu sredinu su većinski pokazala vrednosti koje kvalifikuju RA kao materijal za ugradnju u nasipe i nevezane noseće slojeve u putogradnji, ali i kao sirovinu za upotrebu u betonima kao zamena dela prirodnog agregata.

Uzorci heterogenog RA imaju veoma složen sastav, koji obuhvata ponekad i preko deset različitih sastojaka. Pored inertnih sastojaka, kao što su betoni i betonski proizvodi, kamen i elementi od gline, mogu se pojaviti i drugi, „zagađujući“ sastojci, kao što su gipsani paneli i sl., koji svojim prisustvom kontaminiraju i onemogućavaju upotrebu ovakvog RA kao građevinskog materijala.

Očekivano, bolja fizička i mehanička svojstva dobijena su na RA dobijenom preradom betona, dok su agregati proizvedeni od glinenih elemenata pokazali značajno variranje rezultata. Isto tako, razlike u rezultatima ispitivanja RA dobijenog preradom betona ukazuju da su neki od uzoraka pripremljeni od relativno visokokvalitetnog betona, dok su drugi dobijeni preradom betona nižeg kvaliteta.

Dobijeni rezultati ukazuju da je od velikog značaja za kvalitet i mogućnost upotrebe RA u građevinarstvu, pre svega neophodno u svakom konkretnom slučaju razraditi model razvrstavanja i razdvajanja deponovanog građevinskog otpada već tokom njegovog nastanka i odgovarajući način tretiranja i prerade izvorne sirovine u cilju postizanja odgovarajućeg granulometrijskog sastava i oblika zrna agregata. Sve navedeno ukazuje na potrebe praćenja izvorne sirovine, na značaj laboratorijskih ispitivanja i praćenja krajnjeg proizvoda.

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[17] SRPS EN 933-11:2009/AC:2012 Ispitivanje geometrijskih svojstava agregata - Deo 11: Klasifikacija sastojaka krupnog recikliranog agregata

SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

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POSSIBILITY OF APPLYING A BIOCOMPOSITE BASED ON WOOD SAWDUST AS A THERMAL INSULATION MATERIAL

Summary: *A strong relationship exists between renewable energy sources (wind, solar, hydro, geothermal, and biomass) and weather conditions that affect their capacity. Furthermore, climate change alters energy supply and demand patterns, affecting buildings insulation needs directly. All this represents an opportunity for the development of new biomass-based materials to reduce energy consumption and protect the environment. The goal of this research is the application of wood sawdust as an aggregate to obtain a new thermal insulation material and to improve the energy efficiency of buildings.*
Keywords: *Biocomposite; Thermal conduction; Thermal insulator; Energy efficiency.*

MOGUĆNOST PRIMENE BIOKOMPOZITA NA BAZI DRVNE PILJEVINE KAO TERMOIZOLACIONOG MATERIJALA

Rezime: *Postoji snažna povezanost između izvora obnovljive energije (vetar, solarna energija, hidroenergija, geotermalna energija i biomasa) i vremenskih uslova koji utiču na njihov kapacitet. Štaviše, klimatske promene menjaju obrasce snabdevanja energijom i potrošnje, što direktno utiče na potrebe za izolacijom zgrada. Sve ovo predstavlja priliku za razvoj novih materijala na bazi biomase koji mogu smanjiti potrošnju energije i zaštititi životnu sredinu. Cilj ovog istraživanja je primena drvne piljevine kao agregata za dobijanje novog materijala za termoizolaciju i poboljšanje energetske efikasnosti zgrada.*
Ključne reči: *Biokompozit; Toplotna provodljivost; Toplotni izolator; Energetska efikasnost..*

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1. INTRODUCTION

The use of renewable raw materials is one of the key aspects of environmentally friendly construction [13], and one example is that renewable raw materials can be used in the production of thermal insulation materials (TI). In addition to technical and technological requirements, such TI materials also meet economic requirements. The use of TI materials in the outer envelope of the building has a very significant role in increasing the energy efficiency (EE) of the building. The most frequently used materials for this purpose are synthetic, which have a certain number of disadvantages. For example, TI materials based on polyurethane and polystyrene can't be decomposed after use. Also, different synthetic binders and gases that create greater porosity of the material are used, while consuming a large amount of heat and electricity in the production process.

2. THERMAL INSULATION MATERIALS IN CONSTRUCTION

Thermal insulation materials in construction have the task of slowing down the transfer of thermal energy through the outer shell of the building and preventing the occurrence of moisture in the walls, i.e. on certain layers of the wall assembly during the winter period [9][22]. The coefficient of thermal conductivity (λ) defines how a material behaves in terms of thermal insulation capabilities. According to this coefficient, thermal insulation materials can be divided into two groups: first, where $\lambda < 0.06$ W/mK, and the second, where λ is between 0.06 - 0.3 W/mK [19]. Conventional synthetic materials with good thermal conductivity properties (0.025 to 0.048 W/mK) are commonly used as insulation. These materials are very long-lasting, and they are easily incorporated into building elements. In addition, they have disadvantages, such as non-sustainable raw material sources, excessive energy consumption, and the use of dangerous chemicals during production. Certain materials have reduced durability with an increase in temperature, and some are even toxic [22]. In the civil industry, the most frequently applied TI materials are (Figure 1):

1. Expanded and extruded polystyrene (EPS and XPS)
2. Polyurethane (PU)
3. Mineral wool (rock wool and glass wool).

All three types of materials have a high porosity of over 90%. EPS, XPS, and PU are characterised by the dominant closed porosity, water resistance and the impossibility of biological degradation. In contrast to them, stone and glass wool absorb water, but they are not combustible materials as previously mentioned. The common feature of all three types of TI materials is that when they are exposed to high temperatures, they release toxic gases and sooty smoke, which harms human health and the environment. They are mainly applied as the TI of the outer shell of the object [19][22].

On the other hand, by using biodegradable waste materials, which do not require technologies with high energy consumption, the principle of sustainable development is introduced. The use of biomass, primarily wood waste, meets the criteria of sustainable development, reduces pollution and the amount of material that is disposed of in landfills or used for burning.



Figure 1. Synthetic TI materials [9]

There are boards made of reed, cork, wool, baled straw, wood fibers (sawdust), as examples of less frequently used but also less environmentally harmful materials, shown in Figure 2. The coefficient of thermal conductivity of these materials is higher than conventional, and they are in the range from 0.037 to 0.145 W/mK. Due to the poor knowledge about this type of material as thermal insulation, they are rarely used in practice. However, scientific research is increasingly contributing to the application of these TI materials because they have numerous advantages, such as the renewability of the natural resources from which they originate, and at the end of their lifetime, they are biologically degradable. Also, they are not harmful to the environment and human health, and simpler and less energy-demanding production processes produce them. Research and development in science are increasingly producing materials whose components are natural materials, and they are called biocomposite materials. Biocomposites are a mixture of at least two materials that consist of components of natural origin - biodegradable fibers that have the role of reinforcement (aggregate) and binders [1][5][8]. The current research on these materials indicates that their application is possible and widespread. Studying the process of heat transfer through biocomposites is of great importance for determining the thermal properties of this type of material, the knowledge of which is important due to its application as an insulating material [2][14][15][17][20][22]. Curto et al. [7] studied the thermal and acoustic characteristics of materials obtained by mixing natural lime, water and hemp. Their research was conducted on precisely defined sizes and shapes of material samples. The obtained results were compared with the results of other authors, where it was concluded that the new material made of lime and hemp can replace the conventional materials that have been used for a long time. Ninikas et al. [16] studied a type of insulation made from hemp and pine tree bark. Four types of mixtures were made using fibers of the material that were joined with methyl cellulose glue. The difference was reflected in the ratio of tree bark and hemp fibers, which were mixed in the following ratio: 90:10, 80:20, 70:30 and 60:40. The results showed that with an increase in the hemp content, the mechanical properties are improved. However, at the same time, the thermal conductivity coefficient also increases, which represents the negative side of this mixture. Khoukhi [12] and

coworkers developed an environmentally friendly thermal insulation material based on short and long-grain rice. Tests have determined the optimal thermal conductivity of this bio-insulating material. Iqbal Cetiner and Andrew D. Shea [6] examined the possibility of using sawdust as a filler for prefabricated walls, which can be used to replace classic concrete walls with conventional TI materials. Christer Tashana Danne M. Gamiao et al. [21] based their research on the production of wooden panels from sawdust as thermal insulation of ceilings in buildings.



Figure 2. Biocomposites based on wood biomass [10]

3. ENERGY EFFICIENCY AND OVERALL HEAT TRANSFER COEFFICIENT

To meet the requirements of EE buildings, the Rulebook is applied, which prescribes the energy properties and the method of calculating the thermal properties of high-rise buildings, as well as the energy requirements for new and existing buildings. The energy efficiency of buildings has been achieved if the following properties are met [18]:

1) Provided minimum conditions,

2) Energy consumption for heating, cooling, preparation of hot sanitary water, ventilation and lighting of the building does not exceed the allowed maximum values per m^2, which are precisely defined according to the Rulebook.

Fulfilling the requirements for EE facilities also includes checking the diffusion of water vapour, which is calculated for external building structures and structures bordering unheated rooms (except rooms bordering the field). All construction structures of the building must be designed and built in such a way that water vapour does not condense on their surfaces. In case of condensation, the drying period is calculated.

In this paper, on the example of 2 walls assembly, the overall heat transfer coefficient is calculated using the URSA software package. As an insulation material for both wall assemblies, a panel made of biocomposite P is used. Biocomposite (P) panels are made from natural aggregate - sawdust procured as waste from the wood industry. Granules of expanded polystyrene (EPS) and lime-gypsum paste in a 4:1 ratio were added to the mixture as a binding agent. The use of EPS aims to enhance the thermal characteristics of the biocomposite P. The volume fractions of the materials in biocomposite P are presented in Table 1, while the component materials are depicted in Figure 3.

	sawdust	granules of expanded polystyrene (EPS)	lime-gypsum paste
Biocomposite P	53,5%	13,5%	33%

Table 1. Volume shares of component materials



a) sawdust b) polystyrene granules c) lime and gypsum

Figure 3. Component materials of biocomposite P

The average distribution of sawdust particles in the aggregate used for the production of the new biocomposite is shown in Table 2.

Biocomposite/ size (mm)	**< 0,4**	**0,4-0,5**	**0,5-1**	**1-2**	**2-4**	**4-8**	**> 8**
Sawdust (%)	22,5	6,8	48,3	8,1	12,2	2,1	0

Table 2. Average distribution of particles in the aggregate

Table 2 shows that the dominant particles are 0.5-1 mm (~48 %), followed by particles smaller than 0.4 mm (22.5 %). The size of the EPS granules is 0.3 cm.

Through preliminary testing on samples with the same composition, the coefficient of thermal conductivity for biocomposite P is calculated and used in this paper [4]. To check EE for two wall assemblies, the overall heat transfer coefficient was determined using the following equation [18]:

$$U = \frac{1}{R_{si} + \sum_m \frac{d_m}{\lambda_m} + R_{se}}$$

where R_{si} and R_{se} are the individual convective heat transfer coefficients for the inner and outer wall sides, λ_m is the coefficient of thermal conductivity, and d_m is the thickness of the m-th layer of the wall.

4. RESULTS AND DISCUSSION

The overall heat transfer coefficient ″U″ was determined for two wall assemblies, the one with a ceramic block and the second with a concrete sheet as a main structure layer, where the thermal insulation material in both cases is the plate made of biocomposite P. Also, the water vapour diffusion resistance factor is calculated for both cases.

4.1. Wall assembly 1 - TI biocomposite P/ceramic block

The characteristics of the wall assembly with TI panel made of biocomposite P, with a thickness of 50 mm and ceramic block as the basic material, are given in Table 3. The coefficient of thermal conductivity and water vapour diffusion resistance factors for all other layers except for biocomposite P are taken from the Rulebook. For biocomposite P, a thermal conductivity coefficient is 0,141 W/mK [4] and the water vapour resistance factor is determined using the dry cup test presented in the doctoral dissertation [3].

	Layers	d (m)	λ (W/mK)	μ	R (W/m^{2}K)
	i				0,125
1	Extension plaster	0,02	0,85	15	0,024
2a	Ceramic block	0,19	0,61	6	0,311
2b	Cement–lime plaster	0,19	0,87	15	0,218
3	TI biocomposite P	0,05	0,141	5,1	0,355
4	External finishing plaster	0,03	0,87	15	0,034
	e				0,04

Table 3: Wall assembly with TI biocomposite P and ceramic block as base material

Based on values shown in Table 3, the value of the overall heat transfer coefficient ″U″ for assembly 1 is

U= 1,138 W/m^{2}K

If EPS of the same thickness is used instead of biocomposite as a thermal insulation layer, this coefficient would be 0.572 W/m^{2}K, and if rock wool of the same thickness is used, this coefficient would be 0.490 W/m^{2}K. Based on the previous, it can be concluded that the overall heat transfer coefficient of a wall made of ceramic block and biocomposite P as a thermal insulation material is about two times higher than assemblies with EPS and rock wool. More precisely, it would mean that such an assembly is twice as bad in terms of thermal insulation compared to walls with EPS and

rock wool. To achieve the specified values of the overall heat transfer coefficient with biocomposite P as TI, the thicknesses must be 0.17 m compared to EPS and 0.21 m compared to rock wool.

4.1.1. The water vapour diffusion resistance factor calculation

The values of the water vapour diffusion resistance factor are shown in the following Table 4:

	Leyers	d (m)	λ (W/m K)	μ	R (W/m^2K)	r (m)	t (°C)	p'(kPa)	p(kPa)
	i				0,125		20,00 15,12	2,337 1,716	1,285 1,285
1	Extension plaster	0,02	0,85	15	0,024	0,30	15,12 14,20	1,716 1,618	1,285 1,127
2	**Ceramic block**	0,19	0,61	6	0,311	1,14	14,20 2,05	1,618 0,710	1,127 0,525
3	**TI biocomposite P**	0,05	0,141	5,1	0,355	0,26	2,05 -11,79	0,710 **0,221**	0,525 **0,390**
4	External finishing plaster	0,03	0,87	15	0,034	0,45	-11,79 -13,14	**0,221** 0,196	**0,390** 0,152
	e				0,04		-13,14 -14,70	0,196 0,169	0,152 0,152

Table 4. The water vapour diffusion resistance factor for assembly 1 with the ceramic block

The conditions for the calculation of water vapour diffusion resistance factor are as follows: t_i=-14,7°C, φ_i=55%, φ_e=90%, where t_i represents a projected air temperature in winter conditions for the city of Kraljevo, φ_i represents the inner, and φ_e the outer relative humidity of air.

Based on values shown in Table 4, it can be seen that condensation occurs between the 3rd and 4th layer (p>p′), and it will dry out in 23 days, which is allowed values provided by the Rulebook.

4.2. Wall assembly 2 - TI biocomposite P/ concrete panel

The characteristics of the wall assembly 2 with TI biocomposite P, thickness 50 mm and concrete panel as the basic material are given in Table 5. The coefficient of thermal conductivity and water vapour diffusion resistance factors for all other layers except for biocomposite P are taken in the same way as for wall assembly 2.

	Leyers	d (m)	λ (W/mK)	μ	R (W/m²K)
	i				0,125
1	Extension plaster	0,02	0,85	15	0,024
2	**Concrete panel**	0,1	2,04	60	0,049
3	**TI biocomposite P**	0,05	0,141	5,1	0,355
4	External finishing plaster	0,03	0,87	15	0,034
	e				0,04

Table 5. Wall assembly 2 with TI biocomposite P and concrete panel as the basic material

Based on values shown in Table 5, the value of the overall heat transfer coefficient ″U″ for assembly 2 is:

U= 1,596 W/m²K

If EPS of the same thickness is used instead of biocomposite as a thermal insulation layer, this coefficient would be 0.670 W/m²K, and if rock wool of the same thickness is used, this coefficient would be 0.559 W/m²K. Based on the previous, it can be concluded that the overall heat transfer coefficient of a wall made of a concrete panel and bicomposite P as a thermal insulation material is about two times higher than assemblies with EPS and rock wool. To achieve the specified values of the overall heat transfer coefficient with biocomposite P as TI, the thicknesses must be 0.17 m in the case of EPS and 0.21 m compared to rock wool.

4.2.1. The water vapour diffusion resistance factor calculation

The values of the water vapour diffusion resistance factor are shown in the following Table 6:

	Leyers	d (m)	λ (W/mK)	μ	R (W/m²K)	r (m)	t(°C)	p'(kPa)	p(kPa)
	i				0,125		20,00 13,08	2,337 1,506	1,285 1,285
1	Extension plaster	0,02	0,85	15	0,024	0,30	13,08 11,78	1,506 1,384	1,285 1,237
2	**Concrete panel**	0,1	2,04	60	0,049	6,00	11,78 9,06	1,384 1,156	1,237 0,266
3	**TI biocomposite P**	0,05	0,141	5,1	0,355	0,26	9,06 -10,58	1,156 0,245	0,266 0,225
4	External finishing plaster	0,03	0,87	15	0,034	0,45	-10,58 -12,49	0,245 0,207	0,225 0,152
	e				0,04		-12,49 -14,70	0,207 0,169	0,152 0,152

Table 6. The water vapour diffusion resistance factor for assembly 2 with a concrete panel

It can be seen from the above table that there is no condensation in this assembly.

5. CONCLUSION

In the final analysis of the obtained values presented in the tables, it can be noted that in assembly 1, which uses ceramic block as the basic material, condensation occurs between the 3rd and 4th layers. In the case of assembly 2, which uses a concrete panel as the basic material, no condensation is observed. However, despite the absence of condensation in assembly 2 with the concrete panel, it was found to perform worse in terms of the overall heat transfer coefficient U, calculated by EE. When comparing walls with biocomposite P as an insulating layer, it can be concluded that the tested wall assemblies are twice as ineffective as walls built with EPS and rock wool. To achieve the same energy efficiency, the thickness of the biocomposite P as the insulation layer must be greater. Nevertheless, this method of insulating buildings using a biocomposite is far superior in terms of protecting the living environment, reducing energy consumption during production processes, minimizing CO_2 emissions, and positively impacting human health. These are important factors in the selection of TI materials.

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
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Stručni rad

PROCENA STANJA, ODRŽAVANJE, SANACIJA I REKONSTRUKCIJA OBJEKATA – ISKUSTVA IZ PRAKSE

Rezime: *Ovaj rad analizira procenu stanja, održavanje i sanaciju/rekonstrukciju građevinskih objekata uzimajući u obzir nedovoljno jasnu zakonsku regulativu, primenu strogih Evrokod normi, različite vrste konstrukcija koje su predmet procene kao i period izgradnje objekata.*
U Srbiji, zakonodavni okvir za održavanje i sanaciju/rekonstrukciju objekata često je nedovoljno precizan, što dovodi do problema u implementaciji standarda i propisa.
Različite vrste konstrukcija i periodi izgradnje utiču na metode procene stanja i sanacije/rekonstrukcije. Stariji objekti često zahtevaju posebnu pažnju zbog zastarelih materijala i tehnologija gradnje i teško ih je proglasiti podobnim za sanaciju/rekonstrukciju što u praksi predstavlja dodatni izazov investitorima.
Rad pruža osvrt na postojeću srpsku zakonsku regulativu koja se tiču održavanja i sanacije/rekonstrukcije objekata i analizira izazove u primeni Evrokodova u ovoj oblasti.
Ključne reči: *procenu stanja, održavanje, sanacija, rekonstrukcija, Eurocode*

CONDITION ASSESSMENT, MAINTENANCE, REHABILITATION AND RECONSTRUCTION OF FACILITIES - EXPERIENCE FROM PRACTICE

Summary: *This paper analyzes the assessment of the condition, maintenance, and rehabilitation/reconstruction of building structures, considering the insufficiently clear legal regulations, the application of strict Eurocode standards, different types of structures subject to assessment, as well as the construction periods of the buildings.*
In Serbia, the legislative framework for the maintenance and rehabilitation/reconstruction of buildings is often imprecise, leading to problems in the implementation of standards and regulations. Different types of structures and construction periods affect the methods of condition assessment and rehabilitation/reconstruction. Older buildings often require special attention due to outdated materials and construction technologies, making it difficult to declare them suitable for rehabilitation/reconstruction, which in practice leads to problems for investors.
The paper provides an overview of the existing Serbian legal regulations related to the maintenance and rehabilitation/reconstruction of buildings, analyzes the challenges in applying Eurocodes in this field, and discusses the issues related to adapting existing buildings to the required regulations.
Keywords: *condition assessment, maintenance, renovation, reconstruction, Eurocode*

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1. UVOD

Rekonstrukcija građevinskih objekata predstavlja značajan deo savremene inženjerske prakse, naročito u urbanim sredinama, gde prostorni i infrastrukturni resursi zahtevaju očuvanje i prilagođavanje postojećih objekata savremenim potrebama. Za razliku od novogradnje, proces rekonstrukcije podrazumeva delovanje u okviru postojećih konstruktivnih sistema, često nepoznatog ili oštećenog stanja, uz primenu savremenih tehničkih standarda i zakonske regulative koja se u međuvremenu promenila. Zbog toga, projektovanje rekonstrukcije predstavlja izazov koji zahteva visok stepen stručnosti, analitičkog razmišljanja i fleksibilnosti u prilagođavanju različitim uslovima.

Cilj ovog rada je da se, kroz prikaz konkretnih primera iz prakse, prikažu iskustva, problemi i inženjerski pristupi u procesu rekonstrukcije objekata. Analizirani su projekti rekonstrukcije dela obaloutvrde na kanalu DTD, kao i industrijskog objekta u oblasti prehrambene proizvodnje.

2. REKONSTRUKCIJA DELA DESNE OBALOUTVRDE KANALA DTD

Desna obala kanala DTD u Novom Sadu je zaštićena kosom obaloutvrdom. Od platoa naniže izvedena je kosina pod nagibm 1:1.25. Kosine obaloutvrde su obložene kamenim blokovima. U dnu kosine nalazi se potporna noseća greda koja je oslonjena na kameni nabačaj, na sredini kosine se nalazi betonska inspekciona staza dok je na vrhu postavljena završna betonska greda kao kruna nasipa. Nakon detaljnog geomehaničkog istraživanja i geodetskog snimanja na obalskom delu, kao i u koritu vodotoka (hidrografski snimak) utvrđene su morfološke karakteristike postojeće obaloutvrde.



Slika 1. Deo postojeće kose obaloutvrde na desnoj obali kanala

Rekonstrukcija je započeta nakon obilaska terena, vizuelnog pregleda, uvidom u arhivsku dokumentaciju i prikupljanjem svih neophodnih podataka za početak izrade statičkog proračuna, odnosno proračuna stabilnosti kosine.

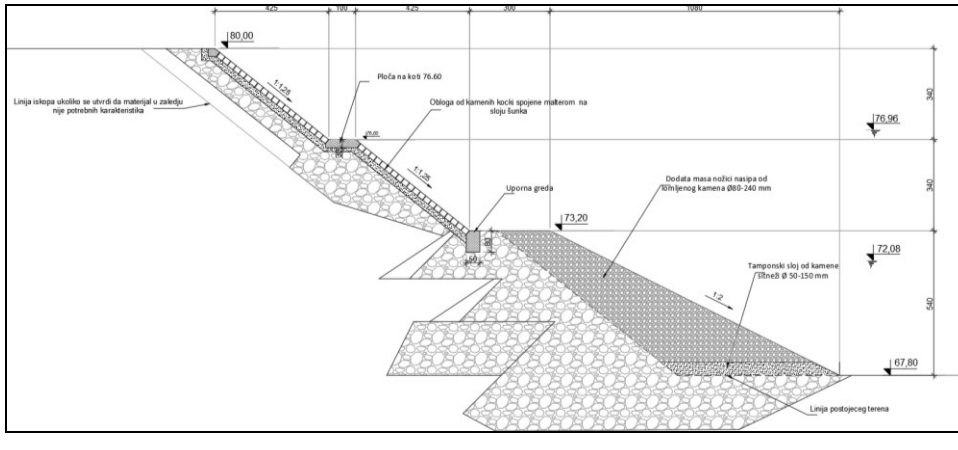
2.1. Proračun stabilnosti kosine

Gledajući Pravilnik za građevinske konstrukcije, u članu 24. nalaze se dva kriterijuma koja su nas navela da ovu konstrukciju proračunamo po novim propisima (Eurocode) i projekat rekonstrukcije uradimo tako da rekonstruisana konstrukcija zadovolji sve uslove propisane Eurocode proračunom. Tu su kriterijumi: „3) rekonstrukcija objekta čija je građevinska konstrukcija oštećena tako da postoji opasnost po život i zdravlje ljudi, životnu sredinu, druge objekte i dobra, ili stabilnost tla okolnog zemljišta, i 4) „rekonstrukcija objekta koja prema projektnom zadatku ima za cilj produžavanje eksploatacionog veka objekta“.

Problematika analize stabilnosti zemljanih masa svodi se na određivanje odnosa između raspoložive smičuće čvrstoće i prosečnog smičućeg napona, koja je potrebna da potencijalno klizno telo drži u ravnoteži. Geotehnički model terena i parametri tla usvojeni su na osnovu rezultata istražnih radova koji su prikazani u geomehaničkom elaboratu koji je bio jedan od ulaznih parametara za izradu ovog proračuna.

2.1.1. Proračun sa projektnim ubrzanjem 0.1g – definisan Pravilnikom za aseizmičko projektovanje na lokaciji Novi Sad prema Evrokodu 8

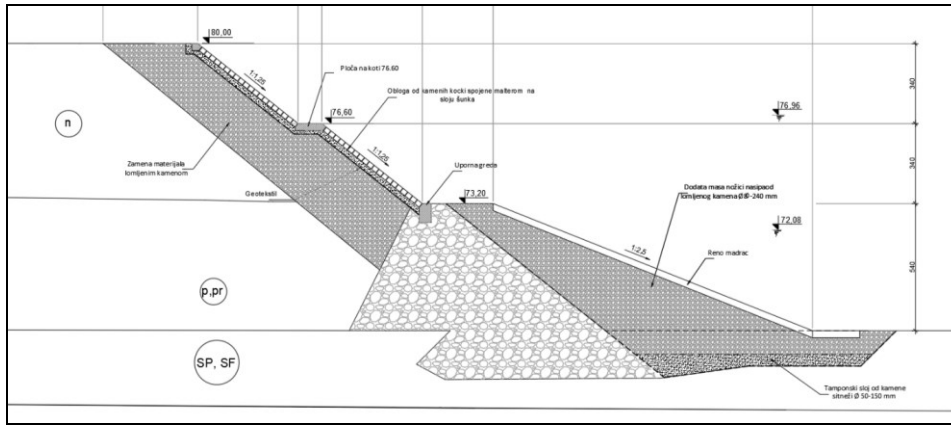
Nakon urađene analize opterećenja, kombinacija opterećenja i proračuna globalne stabilnosti prema projektantskom pristupu br.3.Evrokoda 7, usvojen je poprečni profil prikazan na slici 2. Proračun stabilnosti je urađen i za nožicu obaloutvrde čija je svrha da prihvati opterećenje od obloge i da ga prenese na dno rečnog korita. Telo nožice je trapezastog poprečnog preseka, a dimenzionisanje se sastoji na proveri njene stabilnosti na klizanje.



Slika 2. Izgled poprečnog profila

2.1.2. Proračun sa projektnim ubrzanjem 0.16g – definisan geomehaničkim elaboratom za predmetnu lokaciju

Geomehanički elaborat koji smo koristili je, u delu geofizičkih ispitivanja, definisao maksimalno projektno ubrzanje tla kao 0,16g. Eurokodom definisano ubrzanje je uvećano zbog uticaja fruškogorskih zemljotresa koji se razmatraju lokalno. Ovaj podatak nas je obavezao da uradimo proračun sa uvećanim koeficijentom i rezultati su dali znatno više kamene mase koja je trebala da se ugradi. Prikaz količine kamena se može videti na slici 3.



Slika 3. Izgled poprečnog profila

2.2. Rezultati projekta rekonstrukcije obaloutvrde

Rekonstrukciji obaloutvrde smo pristupili poštujući pravilnik o građevinskim konstrukcijama. Vodeći se navedenim kriterijumima objekat smo projektovali poštojući Eurocode norme. Pored toga, rezultati geofizičkih ispitivanja dodatno su uvećali količinu materijala za oko 60%. Kao krajnji rezultat dobili smo konstrukciju koja je znatno skuplja od onoga što je u startu očekivano i projekat izgradnje je obustavljen.

3. REKONSTRUKCIJA INDUSTRIJSKOG OBJEKTA U PREHRAMBENOJ INDUSTRIJI

Predmetni objekat je izgrađen 70-ih godina prošlog veka. Po karakteru opterećenja i broju tehničkih etaža, objekat se može podeliti u dva dela, viši deo (gabarita 14.5x26x18m) i niži deo (gabarita 14.5x46x10m). Na taj način je i formirana statička dispozicija objekta i dilatacija između istih. Objekat je fundiran na plitkim temeljima. Glavna konstrukcija je čelična formirana od stubova povezanih međusobno gredama, međuspratnim tavanicama, kao i krovnom konstrukcijom koja se sastoji od glavnih nosača i rožnjača, a za prijem horizontalnih sila na objektu su predviđeni horizontalni i vertikalni spregovi.

Objekat je ranije imao još jedan deo, ali je pre početka izrade projekta rekonstrukcjie taj deo srušen (slika 4).



Slika 4. Izgled dela srušenog objekta u procesu rušenja

Rušenje je obavljeno bez projekta, vučenjem i sečenjem čeličnih elemenata na način da su stubovi susednog, nižeg dela pretrpeli plastične deformacije što je dalje imalo značajan uticaj na mogućnost rekonstrukcije preostalog dela objekta.

3.1. Ocena primerenosti objekta za rekonstrukciju

Pre izrade projekta rekonstrukcije, urađena su ispitivanja, merenja i pregledi da bi se došlo do relevantnog zaključka o stanju postojeće konstrukcije i primerenosti objekta za rekonstrukciju.

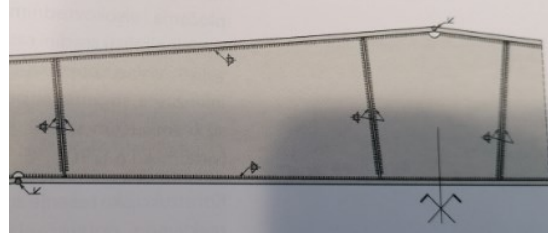
3.1.1. Vizuelni pregled

Vizuelnim pregledom primećeno je da zavareni spojevi nisu izvedeni u skladu sa standardom. Zavareni spojevi imaju nepravilnu površinu, mestimično nadvišenje a

osnovni materijal je mestimično iskrivljen. Montažni nastavci glavnih nosača nisu formirani na način koji propisuje Eurocode.



Slika 5. Neadekvatno izvođenje glavnih nosča u slemenu (montažni nastavak)



Slika 6. Izgled ispravnog konstrukcijskog oblikovanja zavarenog krovnog nosača u slemenu

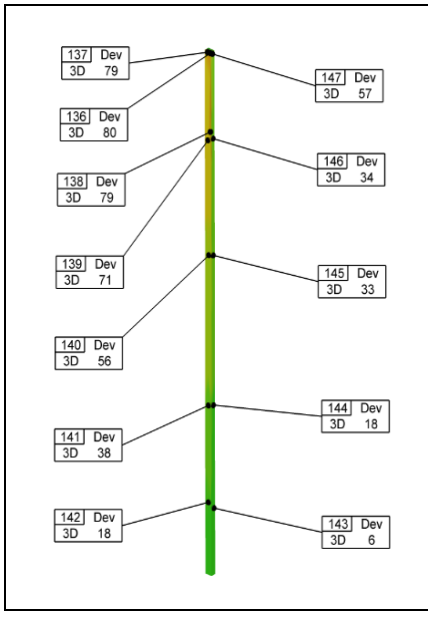
Glavni nosač je promenjivog I poprečnog preseka zavaren iz delova, raspona 14.5m. Kod krovnih nosača velike visine preseka (visina u vencu 227mm, u slemenu 532mm), koji su opterećeni momentima savijanja znatnog intenziteta, montažni nastavci treba da budu pozicionirani van sredine preseka, kako i nalaže SRPS EN 1993-1-8 za proračun i izradu čeličnih veza.

Vizuelnim pregledom je, takođe, uočeno da su stubovi deformisani što je kasnije i dokazano geodetskom kontrolom geometrije glavne konstrukcije.

3.1.2. Geodetsko osmatranje

Da bi se stekao uvid u veličinu geometrijskih deformacija osnovne konstrukcije, urađeno je 3d skeniranje konstrukcije oblakom tačaka. U okviru izveštaja urađena je detaljna 3D analiza geometrije elemenata.

Detaljnom analizom došlo se do podataka da su neki stubovi pretrpeli deformacije koje dostižu i 92mm u vrhu stuba, koja su veća od dopuštenih. Na prvi pogled, pre detaljne analize, stekao se utisak da su se stubovi deformisali usled preopterećenja i posledica rušenja, i da bi ovaj problem mogao da se reši ispravljanjem stubova i adekvatnih ojačanja kosnicima, no međutim, sagledavši rezultate snimanja, došlo se do zaključka da su se stubovi izvili i da je čelik došao u stanje plastifikacije, što direktno utiče na stabilnost cele konstrukcije i sam eksploatacioni vek objekta. Na slici 7 su jasno prikazane deformacije jednog stuba po visini i intenziteti pomeranja.



id	Meas X (mm)	Meas Y (mm)	Meas Z (mm)	Ref X (mm)	Ref Y (mm)	Ref Z (mm)	Dev X (mm)	Dev Y (mm)	Dev 3D (mm)
136	52560	16079	16978	52504	16137	16978	56	-57	80
137	52462	16027	16968	52427	16098	16968	36	-71	79
138	52558	16080	14518	52504	16137	14518	55	-57	79
139	52473	16041	14224	52440	16104	14224	32	-64	71
140	52487	16065	10638	52462	16115	10638	25	-50	56
141	52452	16068	5924	52435	16102	5924	17	-34	38
142	52425	16077	2882	52417	16093	2882	8	-16	18
143	52466	16224	2692	52461	16221	2692	5	3	6
144	52495	16193	5932	52479	16185	5932	16	8	18
145	52520	16178	10625	52490	16163	10625	29	15	33
146	52524	16172	14275	52493	16157	14275	30	15	34
147	52558	16156	16932	52504	16137	16932	54	19	57

Slika 7. Deformacije stubova B6-max.pomeranje 80mm

3.1.3. Kontrola kvaliteta materijala

Kontrola materijala vršena je uzimanjem uzoraka na licu mesta. Uzimani su uzorci u vidu kernova iz temelja radi utvrđivanja mehaničkih karakteristika betona i ispitani su, nedestruktivnim metodama, zavareni spojevi na montažnom nastavku krovnog nosača i zavarenim spojevima na stubovima.

3.2. REZULTATI PROJEKTA REKONSTRUKCIJE

Nakon detaljno sprovedenih istražnih radnji, vizuelnog pregleda, geodetskih snimaka i ispitivanja materijala, došli smo do zaključka da čelična konstrukcija nije primerena za rekonstrukciju. Niži deo nije bio u upotrebi te je bilo moguće uraditi rušenje čelične konstrukcije i izraditi novu na starim temeljima koji su, za razliku od čeličnog dela, pokazali zadovoljavajuća svojstva. Viši deo objekta je deo koji je u aktivnoj funkciji, gde se odvija proizvodnja koju nije moguće lako zaustavitim, te je iz tog razloga izostavljen iz projekta rekonstrukcije.

4. SLOŽENOST REKONSTRUKCIJE OBJEKATA: IZMEĐU PROPISA, PRAKSE I REALNOSTI

Rekonstrukcija objekata predstavlja složen i višeslojan proces koji zahteva pažljivo planiranje, detaljno istraživanje i multidisciplinarni pristup. Za razliku od nove gradnje, rekonstrukcija podrazumeva rad sa postojećim, često nepoznatim ili nepouzdanim elementima konstrukcije i materijalima.

4.1. Specifičnosti i izazovi u projektovanju rekonstrukcije objekata

Pri izradi projekta rekonstrukcije objekta moramo voditi računa o mnogo faktora, kao što su: Zatečeno stanje konstrukcije, mogućnost primene aktuelne zakonske regulative, poznavanje propisa iz perioda kada je objekat građen, poznavanje materijala od kojih je objekat građen, dostupnost arhivskih projekata, kulturno istorijski značaj obejekta, obezbeđivanje radnog procesa u objektu tokom rekonstrukcije, uticaj rekonstrukcije na polisu osiguranja, neplanirani troškovi.

4.1.1. Zatečeno stanje konstrukcije

Ovo je faza projektovanja koja je najizazovnija, najduže traje i finansijski nezanemariva. Da bi se utvrdila zatečena svojstva konstrukcije obavezno se sprovodi vizuelni pregled, uvid u dokumentaciju objekta, uzimanje uzoraka, ispitivanje uzoraka i delova građevinske konstrukcije, proračuni ili slično. Zatečena tehnička svojstva za postojeće stanje građevinske konstrukcije je obavezno utvrditi pre početka izrade projekta rekonstrukcije. Ocenom primerenosti objekta za rekonstrukciju započinje projekat i utvrđuje se potrebni obim radova.

4.1.2. Mogućnost primene aktuelne zakonske regulative

Veliki je izazov zadovoljiti čitav niz uslova iz zakonske regulative pri projektovanju rekonstrukcije postojećih objekata izgrađenih u vreme važenja ranijih regulativa. Ovde se prvenstveno misli na uslove konstrukcije i proračuna koje nosi Eurocode, protivpožarne zaštite, upravljanje otpadom (ukoliko postoji azbest ili slični štetni materijali), energetska efikasnost, konzervatorske smernice.

4.1.3. Poznavanje propisa iz period kada je objekat građen

Ukoliko rekonstrukcija nema bitan uticaj na tehnička svojstva postojeće građevinske konstrukcije dovoljno je da zadovolji propise perioda u kojem je građena. Izazov je za projektanta da poznaje propise raznih perioda i da sprovede efikasno i brzo proračun iz ranijih perioda.

4.1.4. Poznavanje materijala od kojih je objekat građen

Poznavanje materijala od kjih je objekat građen je veoma značajno. U praksi nailazimo na materijale čija svojstva nisu dovoljno poznata ili su potencijano štetna za okolinu. Ispitivanje materijala se sprovodi analizom dokumentacije, ispitivanjima na licu mesta, laboratorijskim analizama itd. Bez ovog saznanja nije moguće pravilno projektovati rekonstrukciju ni dokazati da će konstrukcija nakon rekonstrukcije ispuniti tehničke zahteve – čak i ako se ne radi o suštinskoj izmeni konstrukcije.

4.1.5. Dostupnost arhivskih projekata

Često je dostupnost arhivskog projekta upitna. Čak i kada postoje, arhivski projekti su ili oštećeni ili ne sadrže sve delove koje su originalno imali a koji su neophodni za celokupno sagledavanje. Ukoliko nedostaje arhivski projekat, projektantu

sledi mukotrpan posao izrade projekta postojećeg stanja pre izrade projekta rekonstrukcije.

4.1.6. Kulturno istorijski značaj obejkta

Ukoliko objekat koji je predmet rekonstrukcije ima kulturno istorijski značaj, to dodatno komplikiuje izradu projekta rekonstrukcije. Sam obim radova, materijali koji se koriste i metode za izvođenje dodatno konstrolišu nadležne institucije što utiče na rok i koštanje projekta

4.1.7. Obezbeđivanje radnog procesa u objektu tokom rekonstrukcije

Još jedan izazov pri projektu rekonstrukcije je zahtev za obezbeđivanje kontinuiranog procesa rada u objektu koji se rekonstruiše. Ovo je često zahtev u industrijskim postrojenjima i nosi veliki izazov kako za samu tehnologiju izvođenja radova tako i za obezbeđivanje bezbednosti radnika koja je najbitnija.

4.1.8. Uticaj rekonstrukcije na polisu osiguranja

Uticaj rekonstrukcije objekta na polisu osiguranja može biti značajan, jer osiguranje uvek zavisi od stanja, vrednosti i rizika povezanih sa objektom. Promena vrednosti objekta, povećani ili smanjeni rizici (broj korisnika, protivpožarna zaštita i sl.) i slično utiče na promenu vrednosti polise osiguranja.

4.1.9. Neplanirani troškovi

Neplanirani troškovi su gotovo neizbežan deo rekonstrukcije, i često glavni uzrok prekoračenja budžeta i rokova. Skrivena oštećenja (vlaga u zidovima, korozija armature, trule grede, slaba temeljenja), Asbest, olovo, azbestne cevi – zahtevaju specijalno uklanjanje, neslaganje sa dokumentacijom – zidovi na pogrešnom mestu, drugačiji materijali od očekivanih i slično može značajno povećati troškove tokom projekta.

5. ZAKLJUČAK

Uspešna rekonstrukcija građevinskih objekata zahteva visok stepen stručnosti, međusobnu koordinaciju svih učesnika i fleksibilnost u pristupu, kako bi se obezbedila tehnička ispravnost, bezbednost i ekonomska isplativost projekta. Pored tehničkih izazova kao što su analiza zatečenog stanja, izbor adekvatnih materijala i ocena nosivosti konstrukcije, rekonstrukcija nosi i pravne i proceduralne izazove koje projektant mora da razume i primeni.

Zakonska regulativa u oblasti projektovanja i rekonstrukcije u Srbiji, pre svega kroz Zakon o planiranju i izgradnji i Pravilnik o građevinskim konstrukcijama, postavlja opšti okvir i obaveze za sve faze projektovanja i izvođenja. Pravilnik jasno propisuje da se za rekonstrukcije koje imaju bitan uticaj na tehnička svojstva objekta, posebno kada je cilj produženje veka trajanja, moraju primeniti savremeni propisi i proračuni u skladu sa važećim standardima (uključujući Eurokodove).

Međutim, praksa pokazuje da se značajan broj postojećih objekata teško može uskladiti sa novim normama bez opsežnih intervencija, što dodatno opterećuje projektante, kako tehnički, tako i u odnosu na investitore koji često vrše pritisak da se projekat pojednostavi ili ubrza – neretko na uštrb bezbednosti i zakonitosti. Uočava se i nedovoljna razrađenost pravilnika u delu koji se odnosi na rekonstrukciju, što projektantima ostavlja previše prostora za slobodnu interpretaciju, ali istovremeno povećava rizik od odgovornosti.

Zbog toga bi značajan korak unapred bio dopuna pravilnika konkretnijim smernicama koje bi uključivale:

- razvrstavanje vrsta rekonstrukcija prema nivou intervencije i vrsti objekta,
- jasnije kriterijume za ocenu podobnosti konstrukcije za rekonstrukciju,
- integraciju praktičnih priručnika sa realnim primerima,
- uspostavljanje stručne platforme ili portala za razmenu iskustava i dobre prakse među projektantima i inženjerima.

U suštini, iako je pravni okvir formalno zadovoljavajući, prostor za unapređenje postoji, naročito u pravcu praktične primene, transparentnosti i podrške struci. Samo sinergijom između jasnih propisa, stručne prakse i institucionalne podrške može se postići održiva i bezbedna rekonstrukcija objekata, u interesu svih – struke, investitora i krajnjih korisnika.

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[3] ZAKON O PLANIRANJU I IZGRADNJI, ('Sl. glasnik RS', br. 72/2009, 81/2009 - ispr., 64/2010 - odluka US, 24/2011, 121/2012, 42/2013 - odluka US, 50/2013 - odluka US, 98/2013 - odluka US, 132/2014, 145/2014, 83/2018, 31/2019, 37/2019 - dr. zakon, 9/2020, 52/2021 i 62/2023)

DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

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Pregledni rad

MOGUĆNOST INDUSTRIJSKE PRIMENE NUSPRODUKATA NASTALIH U TEHNOLOŠKOM PROCESU PROIZVODNJE ELEKTRIČNE ENERGIJE TERMOELEKTRANAMA NA UGALJ

Rezime: *Proizvodnja električne energije u termolektaranama ima za posledicu stvaranje različitih materija poput pepela, šljake, SOx, NOx, CO_2, otpadne vode i sintetičkog gipsa. Pored značajnog potencijala za ekonomsku valorizaciju ovih materijala, veoma je važno i smanjenje njihovog negativnog uticaja na životnu sredinu. Ekonomska isplativost može biti od značaja za poljoprivredu, građevinsku industriju, reciklažu i sanaciju okoliša. Korišćenje ovog materijala pomaže u smanjenju otpada i doprinosi održivom razvoju. Ipak, obavezno je izvršiti analize i pratiti propise kako bi se osigurala bezbednost primene nusprodukata sagorevanja uglja u termoelektranama. Ukratko, pepeo, šljaka i gips imaju širok spektar primena, ali je važno pravilno ih koristiti i u skladu sa propisima i standardima životne sredine.*
Ključne reči: *pepeo, šljaka, gips*

POSSIBILITY OF INDUSTRIAL APPLICATION OF BY-PRODUCTS CREATED IN THE TECHNOLOGICAL PROCESS OF ELECTRICITY PRODUCTION BY COAL-FIRED THERMAL POWER PLANTS

Summary*: The production of electricity in thermal power plants results in the creation of various substances such as ash, slag, SOx, NOx, CO_2, waste water and synthetic gypsum. In addition to the significant potential for economic valorization of these materials, it is also very important to reduce their negative impact on the environment. Economic profitability can be important for agriculture, construction industry, recycling and environmental rehabilitation. Using this material helps reduce waste and contributes to sustainable development. However, it is mandatory to carry out analyzes and follow the regulations in order to ensure the safety of the use of coal combustion by-products in thermal power plants. In short, ash, slag and gypsum have a wide range of applications, but it is important to use them correctly and in accordance with environmental regulations and standards*
Keywords: *ash, slag, gypsum*

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1. UVOD

Ugalj predstavlja jedan od najekonomičnijih izvora energije, čijim se sagorevanjem proizvodi oko 40 % električne energije u svetu a kao nusproizvodi pored ostalog nastaju pepeo i šljaka. Skladištenje ovih materijala predstavlja značajan ekološki i ekonomski problem.

Korišćenje otpada od pepela i šljake, kao sirovine, različitim industrijskim sektorima pruža mogućnost ne samo da se smanji uticaj na životnu sredinu, već i da se obezbede značajne ekonomske koristi.

Kao potpisnica Ugovora o osnivanju Energetske zajednice, R. Srbija je bila u obavezi da se uskladi sa zahtevima iz Direktive o velikim ložištima 2001/80/EC počevši od 1. januara 2018. Kao meru za smanjenje emisija, ovaj plan predviđa izgradnju postrojenja za odsumporavanje. Na osnovu prethodno prihvaćenih analiza kao referentni postupak odsumporavanja definisan je vlažni krečnjak/gips.

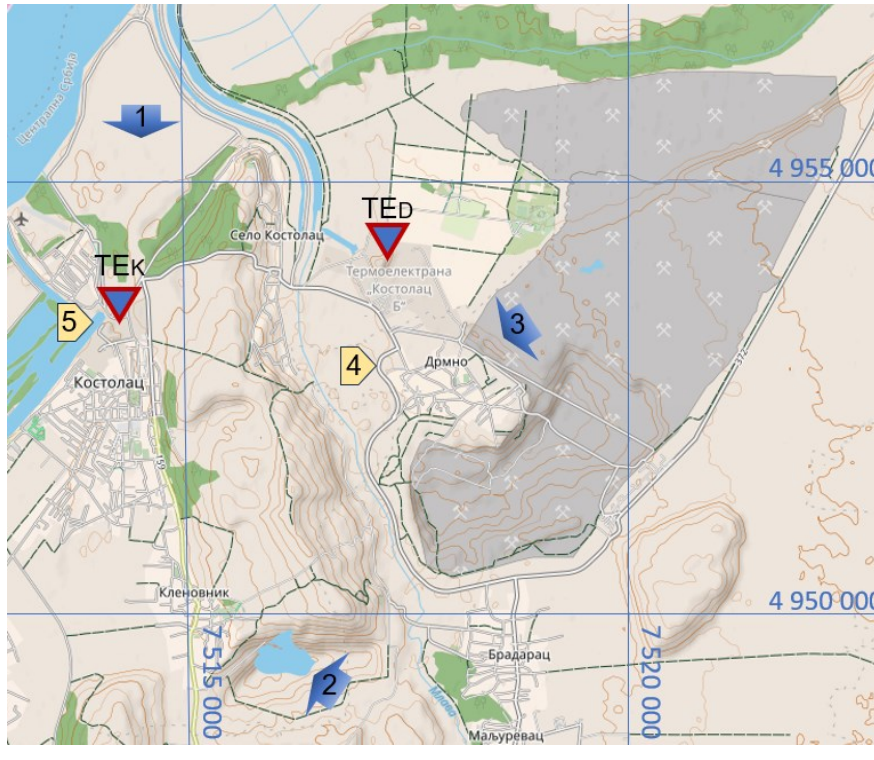
Briga za životnu sredinu i ekološki uslovi pružaju sve više mogućnosti za korišćenje antropogenih materijala kao što su nusproizvodi sagorevanja. Oni značajno utiču na smanjenje emisije CO_2 kao i na smanjenje eksploatacije prirodnih resursa. Nusproizvodi sagorevanja su uz odgovarajuću kontrolu i tretman bezbedni za ljude i životnu sredinu a njihova upotreba u industriji odgovara modelu cirkularne ekonomije.



Slika 1. Principijelna šema nastajanja nusprodukata sagorevanjem uglja u termoelektranama uz proces odsumporavanja dimnih gasova

2. DEPONIJSKI PROSTOR ZA SMEŠTAJ PEPELA, ŠLJAKE I ODG GIPSA

Deponovanje bilo kog materijala, pa i pepela i šljake, vrši se kada je dalja upotreba tog materijala nemoguća ili kada nije dovoljno detaljno sagledana mogućnost njegove dalje primene, kao i kada se želi smanjenje uticaja tog otpada na životnu sredinu i zdravlje ljudi [2]. U cilju očuvanja životne sredine, kao i u skladu sa normativima EU za potrebe odsumporavanja dimnih gasova iz termoelektrana „Kostolac B1“ i „Kostolac B2“, tokom prethodnih godina izgrađen je sistem za preuzimanje i odlaganje gipsa, koji nastaje kao nusproizvod u procesu vezivanja sumpora iz dimnih gasova sa krečnjakom [1]. Postrojenje za odsumporavanje dimnih gasova omogućava smanjenje emisije sumpornih oksida. Gips koji nastaje kao nusproizvod radom ODG postrojenja upisan je u Registar nusproizvoda i u Evropski registar hemikalija (REACH), što predstavlja preduslov za prodaju zainteresovanim kupcima. Smeštajni prostori nusproizvoda termoelektrana u Kostolcu imaju povoljnu poziciju za preuzimanje.



Slika 2. Lokacije termoelektrana i deponija na prostoru Kostolca

1-SKO; 2-PK Ćirikovac; 3-PK Drmno; 4-Državni put II reda; 5-Pristanište (Luka)

Na srednjem kostolačkom ostrvu (SKO) odloženi su pepeo i šljaka, kao i na deponiji unutrašnjeg odlagališta PK Ćirikovac, dok su na unutrašnjem odlagališu PK Drmno selektivno odloženi u prvoj kaseti gips, a u drugoj pepeo, šljaka i gips objedinjeno [3]. Trenutno se samo iz prve kasete za potrebe tržišta izuzima gips iz procesa odsumporavanja, dok su preostali odloženi nusprodukti neiskorišćeni.

U tom smislu, kako bi se sa jedne strane smanjili troškovi smeštaja ovih materijala, a sa druge iskoristio potencijal raspoloživih nusprodukata, potrebno je pokrenuti aktivnosti u cilju obezbeđivanja uslova za upotrebu pepela, šljake i ODG gipsa. U realizaciji takvog poduhvata treba da učestvuju resorno ministarstvo, opštinski organi i privredna društva. Proceduralni elementi za što jednostavniju primenu nusprokata rada termoelektrana su u nadležnosti zakonodavstva pa je iz tog razloga važna uloga resornog ministarstva. Opštinski organi su neminovno subjekt koji treba da animira lokalnu privredu i obezbedi logističku podršku. I na kraju privredna društva su akteri koji treba da budu konstrukt sveobuhvatnog ciklusa. Umesto fokusa na smeštaj nusprodukata, orijentacija svih relevantnih institucija treba da bude usmerena ka tržišnoj ekonomiji sa jedne i zaštiti životne sredine sa druge strane.

3. PEPEO, ŠLJAKA I (ODG) GIPS KAO NUSPRODUKTI SAGOREVANJA UGLJA U TERMOELEKTRANAMA

Pepeo je prateći proizvod sagorevanja uglja u termoelektranama. Po krupnoći se klasifikuje na krupniji, koji pada direktno na dno kotla i lakši tzv. leteći, finozrnasti koji se stvara pri sagorevanju na visokoj temperaturi. Pepeo sadrži minerale kao što su silikati, oksidi gvožđa, aluminijuma, kalcijuma i magnezijuma, a ponekad i tragove teških metala. Deo letećeg pepela, koristi se kao dodatak u proizvodnji betona, jer poboljšava njegova svojstva, poput čvrstine i otpornosti na vlagu. Ovaj pepeo deluje kao mineralni aditiv koji smanjuje količinu korišćenih prirodnih sirovina i smanjuje emisiju CO_2. Neki oblici pepela se koriste u industriji stakla i keramike, proizvodnji cementa, zamenjujući deo klinkera, što može doprineti smanjenju emisije ugljen-dioksida u cementnoj industriji.

Šljaka je otpadni materijal koji nastaje u termoelektranama kao posledica sagorevanja uglja, ali u manjoj meri od pepela. Šljaka je obično krupnija i teža i često sadrži visok nivo kalcijuma, silicijuma i aluminijuma. Šljaka koja se generiše iz termoelektrana, može se koristiti u proizvodnji asfaltnih mešavina, gde ima ulogu agregata. Ovo može smanjiti potrebu za prirodnim agregatima i negativan uticaj na životnu sredinu. Koristi se i kao materijal za izgradnju saobraćajnica, puteva ili železničkih pruga, pružajući stabilnu podlogu za saobraćajnu infrastrukturu.

Proizvodi od pepela i šljake iz procesa sagorevanja u termoelektranama, mogu imati različite primene u industriji i građevinskoj proizvodnji. Ovi materijali se obično nazivaju „industrijski nusproizvodi" i mogu biti od koristi u različitim sektorima, ali i predstavljaju ekološke izazove. Neki oblici pepela i šljake mogu se koristiti kao filtracioni materijali u procesima pročišćavanja vode, jer imaju sposobnost apsorpcije nečistoća. U nekim slučajevima, pepeo može biti korišćen kao dodatak zemljištu, posebno u poljoprivredi, kako bi se poboljšala njegova struktura i povećala pH vrednost zemljišta. Deo pepela može se koristiti u proizvodnji ugljeničnih materijala za različite industrijske procese, uključujući elektroindustriju.

ODG (odsumporavanje dimnih gasova) gips je otpad, koji nastaje kao rezultat uklanjanja sumpor-dioksida iz dimnih gasova u termoelektranama. Iako sadrži tragove teških metala i drugih nečistoća, ekološki je prihvatljiv oblik gipsa jer sadrži niz drugih korisnih materija. Koristi se u proizvodnji gips-kartonskih ploča koje se široko primenjuju u građevinskoj industriji, naročito za unutrašnju završnu obradu prostora. Može se koristiti kao dodatak u proizvodnji maltera ili cementa, poboljšavajući njegovu plastičnost, stabilnost i vreme očvršćavanja.

Takođe se može koristiti u poljoprivredi za poboljšanje kvaliteta zemljišta, naročito u regionima gde je zemljište previše kiselo, jer gips pomaže u stabilizaciji pH vrednosti tla. Primera radi, visokotemperaturno izlaganje betona dovodi do ozbiljnih oštećenja betonskih konstrukcija, što za posledicu ima značajno propadanje mehaničkih svojstava i ljuštenje betona [4]. Pokazalo se da ugradnja gipsa pozitivno utiče na mehanička svojstva kada se koristi u kontrolisanim količinama. Ovo uzrokuje smanjenje poroznosti i poboljšava povećanje snage. Ugradnja letećeg pepela i šljake dodatno poboljšava termičku izdržljivost [5].

Korišćenje pepela, šljake i ODG gipsa smanjuje potrebu za vađenjem prirodnih resursa kao što su pesak, šljunak, cement, gips itd. Upotrebom otpadnih materijala smanjuje se potražnja za proizvodnjom novih, što direktno utiče na smanjenje emisija ugljen-dioksida i drugih gasova sa efektom staklene bašte. Upotrebom ovih materijala smanjuje se količina i troškovi za zbrinjavanje otpada koji bi inače bio odložen na deponijama.

Pre nego što se ovi materijali upotrebe, važno je izvršiti temeljnu analizu na prisustvo teških metala i drugih štetnih supstanci. Nepravilna upotreba može izazvati zagađenje.

Da bi se ovi materijali koristili u industriji, mora se osigurati njihov kvalitet i bezbednost. Potrebno je razviti i primeniti odgovarajuće zakone, standarde i regulative koji omogućavaju bezbednu i efikasnu upotrebu ovih materijala.

Industrijska primena pepela, šljake i ODG gipsa iz termoelektrana pruža značajne ekološke i ekonomske koristi, ali zahteva pažljivo upravljanje i nadzor kako bi se osigurala sigurnost i efikasnost upotrebe ovih materijala.

4. PRIMERI PRIMENE PEPELA, ŠLJAKE I ODG GIPSA

U mnogim zemljama leteći pepeo tretiraju kao proizvod koji ima svoju upotrebnu vrednost, tržište i cenu. Ovakav pristup je doveo do toga da u nekim zemljama praktično više ne postoji nijedna deponija pepela. Mogućnost primene tzv. letećeg pepela u građevinskoj industriji otkrivena je još 1950-ih godina u SAD. Upotrebom letećeg pepela smanjuje se potrošnja cementa, za čiju se proizvodnju koristi dosta energije. Rezime opisanog je da se korišćenjem pepela kao sirovine u građevinarstvu smanjuje potrošnja i resursa i energije. Da bi leteći pepeo bio kvalitetan građevinski materijal i da bi mogao da se nađe na tržištu mora biti u skladu sa međunarodnim standardima. Zato je obaveza elektrana da vode računa da proizvode pepeo koji se uklapa u standarde. Transport letećeg pepela je moguć ali, troškovi prevoza su visoki. U neke od najvećih građevina u evropskim gradovima ugrađene su stotine tona letećeg pepela. Tako je u kule blizance u Frankfurtu Kastor i Poluks ugrađeno 180 kg cementa i 120 kg letećeg pepela po m^3. Takođe je i najviša zgrada u Madridu Pikaso kula, visoka 171 m, građena sa betonom „pojačanim“ letećim pepelom. Beton kojim je izliven most, koji povezuje Kopenhagen sa kontinentom, inače u pitanju je viseći most sa rasponom od 1.624 m, sastoji se od smeše cementa, letećeg pepela i mikrosilike. Leteći pepeo se ugrađuje i u brane („Puylaurent“ u Francuskoj, „Planatovryssi“ u Grčkoj, „Siergrist“ u Pensilvaniji), rashladne tornjeve elektrane („Niederaussem“ u Nemačkoj), puteve širom Evrope (RN-47 u Lens La-Basse, Francuska, Staffordshire u V.Britaniji, US-278 u SAD-u) i dr.



Slika 3. Proizvodi od letećeg pepela: opeka / asfaltna mešavina / cement / keramika [6]



Slika 4. Proizvodi od ODG gipsa: gipsane i/ili kartonske ploče / klinker [7]

5. ZAKLJUČAK

Industrijska primena pepela, šljake i ODG gipsa iz termoelektrana ima značajan potencijal za ekonomsku valorizaciju ovih otpadnih materijala ali i za smanjenje negativnog uticaja na životnu sredinu.

Iako postoji niz koristi u upotrebi pepela, šljake i ODG gipsa, važno je napomenuti da sadrže teške metale i druge štetne materije, zbog čega je neophodna pažljiva regulacija i kontrola pri njihovoj upotrebi. Nepoštovanje ekoloških standarda može dovesti do kontaminacije zemljišta, vode i vazduha.

U tom smislu, potrebno je da se izvrše ispitivanja sastava i karakteristika pepela, šljake i ODG gipsa sa potencijalnim upotrebnim mogućnostima, kako bi se obezbedio pouzdan plasman i na taj način smanjili troškovi skladištenja.

U mnogim zemljama postoje propisi koji regulišu upotrebu ovih nusproizvoda kako bi se smanjio njihov negativan uticaj na životnu sredinu, dok istovremeno omogućavaju ekonomsku isplativost njihovog korišćenja u industriji.

Kako bi se postigla optimalna primena, potrebno je razvijati odgovarajuće tehnologije tretmana ovih otpada, kao i sprovoditi stalna istraživanja u cilju pronalaženja novih i održivih načina njihove upotrebe.

Umesto skladištenja nusprodukata potrebno je pojednostaviti proceduru za njihovu upotrebu. Za to je potrebno da resorno ministarstvo uz pomoć države podstakne sa jedne strane proizvođače a sa druge potencijalne kupce nusprodukata, kroz različite zakonske olakšice i eventualne subvencije, da se po uzoru na razvijene evropske zemlje, ovi materijali iskoriste u proizvodnji i na taj način u izvesnoj meri uštedi na rezervama sirovinske osnove.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
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EVALUATION OF COFFEE BIOWASTE AS A CARRIER FOR BACTERIA IMMOBILIZATION FOR SELF-HEALING EFFECT OF CEMENT

Summary: *The construction industry faces significant challenges related to the durability and sustainability of traditional concrete structures. This study explores the innovative use of spent coffee grounds (i.e. coffee biowaste) as a multifunctional carrier for bacteria engineered to induce the autonomous self-healing effect of the cement-based matrix. Coffee biowaste, a readily available and sustainable byproduct, was evaluated for its potential to serve as both a nutrient source and protective matrix for bacteria capable of precipitating calcium carbonate ($CaCO_3$) to seal cracks. The bacterium, Bacillus licheniformis, isolated from a Danube-related source, was immobilized within the selected carrier, and their viability, release kinetics, and self-healing efficiency were assessed under various environmental conditions.*
Keywords: *Microbial technologies; Bacillus licheniformis; Spent coffee grounds.*

EVALUACIJA BIOOTPADA KAFE KAO NOSAČA ZA IMOBILIZACIJU BAKTERIJA ZA SAMOZACELJUJUĆI EFEKAT CEMENTA

Rezime: *Građevinska industrija se suočava sa značajnim izazovima vezanim za trajnost i održivost tradicionalnih betonskih konstrukcija. Ova studija istražuje inovativnu upotrebu taloga od kafe (tj. biootpada od kafe) kao multifunkcionalnog nosača za bakterije koje indukuju samoobnavljajući efekat cementne matrice. Biootpad dobijen nakon pravljenja napitka kafe, lako dostupan i održiv nusproizvod, evaluiran je u pogledu potencijala da služi i kao izvor hranljivih materija i kao zaštitna matrica za bakterije sposobne da precipitiraju kalcijum karbonat ($CaCO_3$) radi postizanja efekta popunjavanja prslina/pukotina. Bakterija Bacillus licheniformis, izolovana iz izvora povezanog sa rekom Dunav, imobilisana je unutar odabranog nosača, a njena viabilnost, kinetika oslobađanja i efikasnost samozaceljivanja su procenjene pod različitim uslovima okoline.*
Keywords: *Mikrobiološke tehnologije; Bacillus licheniformis; Talog od kafe.*

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1. INTRODUCTION

Cement and cement-based materials remain the most widely used construction materials globally due to their availability, affordability, and mechanical properties [6]. However, one of their enduring limitations is the formation of microcracks over time, caused by shrinkage, mechanical stress, or environmental cycles. These microcracks often go unnoticed in their early stages but can evolve into structural vulnerabilities, allowing moisture and aggressive agents to penetrate the matrix and lead to corrosion of the reinforcement and long-term degradation. Traditional repair methods are costly, time-consuming, and often environmentally unsustainable [1]. As such, interest in self-healing materials—especially those enabled by biological processes—has grown significantly. One of the most promising innovations in this field is microbially induced calcium carbonate precipitation (MICP), a naturally occurring process that can be harnessed to seal cracks in these types of materials [13]. Among the microbial candidates, *Bacillus* species have attracted particular attention due to their ability to survive extreme environmental conditions typical for the cement matrix. Their spore-forming capability allows them to remain dormant during mixing and curing, then reactivate upon exposure to moisture. Moreover, some *Bacillus* strains possess urease or other metabolic pathways that convert organic substrates into carbonate ions, which then react with calcium to form solid $CaCO_3$—effectively plugging microcracks and improving durability [11]. Despite these advantages, the success of MICP in real-world applications largely depends on how well bacteria are immobilized and protected within the matrix. Direct incorporation often exposes microbial cells to mechanical shear, high pH, and desiccation, severely limiting their viability and healing effectiveness [9] while only a few studies implemented the direct incorporation of bacteria in cement effectively. For this reason, the development of suitable carrier materials is a crucial step in advancing bio-based technologies in the cement industry. Materials such as expanded clay, perlite, silica gel, and synthetic polymers have been explored. Many of these provide structural protection and delayed activation, but they are frequently associated with high costs, synthetic origins, limited nutrient content, or negative environmental footprints [2]. In recent years, attention has shifted toward more sustainable, organic-based immobilization systems, particularly those derived from biowaste. These materials offer dual benefits: they serve as effective bacterial carriers and reduce environmental burden through waste valorization [2, 4]. Among them, coffee biowaste, after coffee drink preparation, stands out as a particularly promising option, but it has not been used as a carrier of *Bacillus* bacteria for the MICP process yet. Produced globally in vast quantities as a byproduct of coffee brewing, coffee biowaste is nutrient-rich, biodegradable, and possesses a porous, fibrous texture ideal for microbial attachment and moisture retention [4]. Their carbon-rich composition enables alteration of protection of bacterial cell but may also contribute modestly as a nutrient source upon metabolic activation [3].

This study evaluates spent coffee grounds (SCG) as a carrier for *Bacillus licheniformis* in a self-healing system, which application is planned to be in cement-based materials. After formation of SCG-bacteria system, the research was structured to assess: (i) cell release rate and (ii) efficiency of calcium carbonate production under various environmental conditions, as well as (iii) overall MICP performance, summarized through a self-healing index (SHI). Additionally, long-term bacterial viability during storage was evaluated. This work aims to contribute to the design of low-cost, bio-based self-healing systems that align with circular economy principles.

2. MATERIALS AND METHODS

2.1. Spent coffee grounds biowaste

Spent coffee grounds (SCG) were sourced from a local café within 24 hours of brewing to preserve their chemical characteristics and microbiological stability (Figure 1a). The collected SCG were oven-dried at 60 °C for 24 hours to reduce moisture content below 5%, which is crucial to prevent microbial overgrowth and degradation. Dried samples were then sieved through a <1 mm mesh to ensure particle uniformity and maximize surface area for bacterial immobilization (Figure 1b).

2.2. Bacterial strain and cultivation conditions

A native strain of *Bacillus licheniformis*, previously isolated from a Danube sediment sample [10] was utilized due to its demonstrated ureolytic activity and high tolerance to alkaline environments. The strain was maintained on tryptic soy agar (TSA), and colonies were subsequently transferred to tryptic soy broth (TSB). Cultivation was performed at 30 °C with constant agitation at 150 rpm for 72 hours. Bacterial concentrations were standardized to approximately 8 log CFU/mL using spectrophotometric calibration at OD_{600} (Figure 1c).

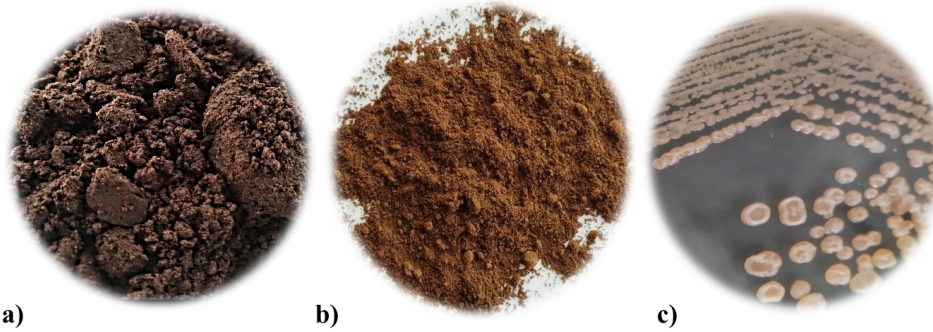


Figure 1. Spent coffee grounds: a) wet, freshly obtained after use; b) dried and prepared for use; c) B. licheniformis colonies at TSA medium

2.3. Formation of SCG-bacteria system

To create the microbial carrier system, the dried SCG were first sterilized via autoclaving at 121 °C for 15 minutes and allowed to cool under aseptic conditions. For bacterial immobilization, 10 g of sterile SCG were submerged in 50 mL of bacterial suspension and incubated at 30 °C for 24 hours with periodic shaking to promote uniform bacterial attachment. Post-incubation, the SCG loaded with bacteria were filtered and air-dried at ambient temperature. The viability of immobilized cells was assessed by resuspending 1g of the SCG-bacteria system in 10 mL of sterile saline (0.9% NaCl), followed by serial dilution and plating on nutrient agar. Colony-forming units (log CFU/g) were enumerated after a 24-hour incubation at 30 °C.

2.4. Assessment of self-healing potential of SCG-bacteria system

The self-healing potential of the SCG–bacteria system was evaluated by examining cell release rate and $CaCO_3$ production as a measure of healing efficacy, as well as self-healing index (SHI) as a result of the previously mentioned parameters. In brief, the rate of bacterial release from SCG was tested under varying pH conditions (7, 9, and 11) and temperatures (15, 28, and 37 °C) using the same protocol as described in subsection 2.3. The cell release rate was calculated as the ratio of released cells to initial bacterial concentration at day zero. Self-healing potential, calculated as SHI was indirectly assessed by quantifying calcium carbonate yield in a calcium-rich medium under the same pH and temperature conditions as for the previous step. The SHI was correlated with the extent of $CaCO_3$ formation, serving as a relative proxy for crack-healing potential. Additionally, bacterial viability in the SCG-bacterial system was measured immediately after system preparation and after storage at standard room temperature after 7, 14, 28, 60, and 90 days of storage. All experiments were conducted in triplicate.

3. RESULTS AND DISCUSSION

The present study investigated the self-healing potential of a sustainable microbial system based on SCG as a bio-carrier and nutrient source for *B. licheniformis* bacterium with strong MICP potential. The system was evaluated for its ability to retain bacterial viability, promote controlled release under environmental stimuli, and induce $CaCO_3$ precipitation. As shown in Figure 2, the SCG-bacteria system undergoes a notable transformation during the precipitation process in calcium-rich medium. Initially (Figure 2a), the SCG-bacteria system appears as a porous material with a relatively uniform and granular structure. After exposure to precipitation conditions, visible white inclusions develop on the SCG surface (Figure 2b), consistent with the formation of $CaCO_3$ crystals. This morphological change supports the occurrence of the MICP process, a critical mechanism for the autonomous sealing in self-healing cement-based materials [5]. These visual findings provide qualitative evidence that the SCG-bacteria system supports bacterial immobilization but also fosters biochemical activity essential for biomineralization. The rough, porous nature of SCG likely enhances bacterial adherence



Figure 2. The SCG—bacteria system: a) before use; b) after induction of $CaCO_3$ formation

and nutrient exchange, while its organic content may help maintain local microenvironments favorable for microbial survival and metabolism [3], especially under alkaline and moist conditions typical of cement microcracks [5].

After preparation, the efficiency of bacterial activation from the SCG carrier is a key determinant of self-healing performance. Figure 3 presents the release rate of *B. licheniformis* cells into a liquid medium under different combinations of pH and temperature. A clear trend is observed: release rates increased with both temperature and moderate alkalinity, indicating that these factors synergistically enhance bacterial mobilization. The highest release rate (84.9%) was observed at pH 9 and 37 °C, highlighting this combination as the most favorable for cell desorption. This likely reflects both optimal membrane fluidity and enzymatic activity of *B. licheniformis* under such conditions. A similar, albeit slightly reduced, release was also recorded at pH 7 and 37 °C (70.1%), suggesting that temperature plays a primary role while pH modulates the overall response. Conversely, the lowest release rate (22.6%) was observed at pH 11 and 15 °C, where both extreme alkalinity and reduced molecular diffusion likely suppressed microbial activation. Interestingly, even at pH 11, elevated temperatures (28–37 °C) improved release by over 30%, indicating partial resilience of the bacteria when thermally stimulated. These results reinforce the environment-sensitive release behavior of the SCG-bacteria system and confirm that moderately alkaline and warm conditions are optimal for effective bacterial activation - a crucial prerequisite for initiating the self-healing cascade.

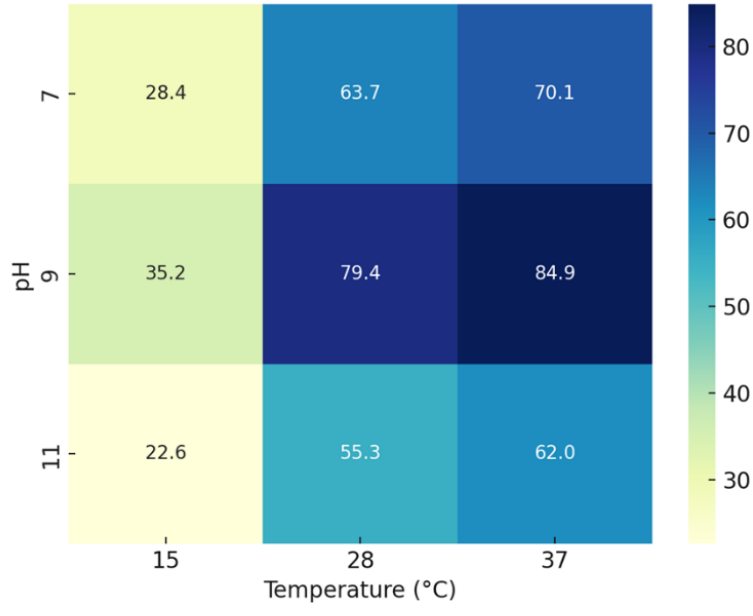


Figure 3. Bacterial cell release rate (%) in liquid medium at different pH and temperature combinations

$CaCO_3$ formation is the key biochemical output of microbial self-healing serving to fill and seal microcracks in the cement matrix. As shown in Figure 4, $CaCO_3$ yield was significantly influenced by both pH and temperature, with the highest precipitation (151.2 mg/g) observed at pH 9 and 37 °C. This aligns with the peak bacterial release (Figure 4), indicating that the extent of biomineralization is closely tied to the availability of metabolically active cells. At moderate conditions (pH 7–9 and 28–37 °C), $CaCO_3$ yields exceeded 100 mg/g, which is within the range known to be effective for visible crack healing in cement-based materials. In contrast, yields dropped sharply under low temperature or high pH, suggesting that either stress impairs the microbial urease activity required for $CaCO_3$ precipitation. Interestingly, *B. licheniformis* remained viable in all tested conditions (Figure 4), but its mineralization output was environment-dependent, confirming that metabolic performance is a more sensitive marker of healing potential than survival alone. This reinforces the need to tailor environmental exposure conditions—particularly pH and temperature—to match the microbial operational window for optimal self-healing behavior.

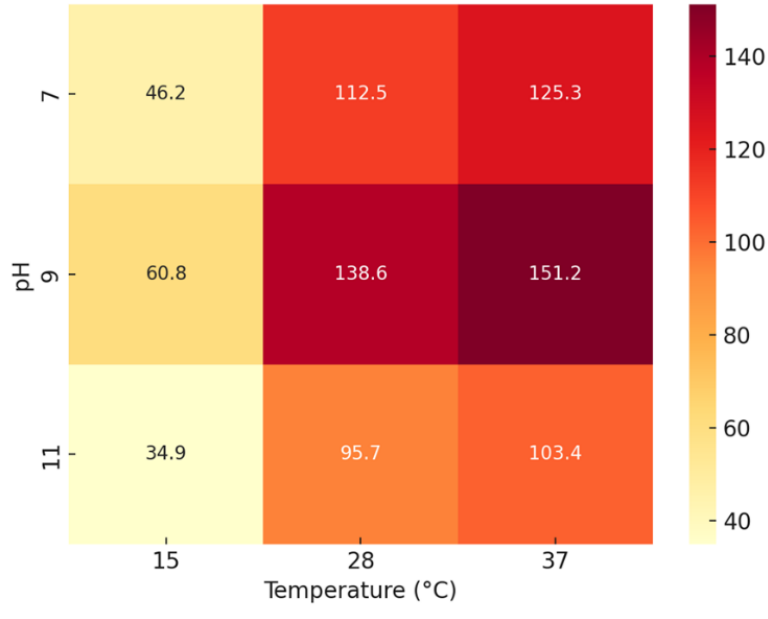


Figure 4. $CaCO_3$ yield (mg/g) in liquid medium at different pH and temperature combinations

To provide an integrated measure of performance, a self-healing index (SHI) was developed (Figure 5a), combining bacterial release efficiency and $CaCO_3$ yield under each pH-temperature condition.

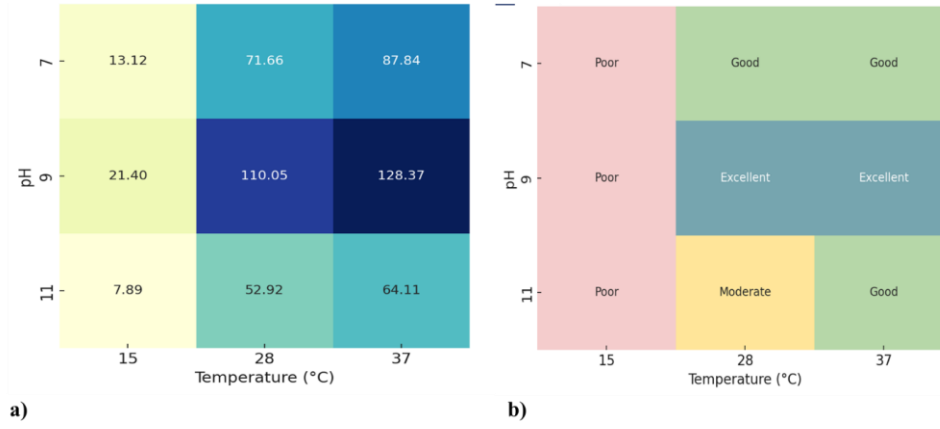


Figure 5. Self-healing assessment: a) self-healing index (SHI) (based on the correlation between results presented in Figures 4 and 5); b) relative interpretation of SHI based on scale: poor (<25): insufficient healing — not suitable for engineering use; moderate (25–60): limited healing potential; good (60–100): effective in protective or semi-structural applications; excellent (>100): strong healing — suitable for structural deployment;

The SHI values, shown in Figure 5a, illustrate a distinct pattern: self-healing potential peaks at pH 9 and 37 °C (SHI = 128.37), while it is markedly reduced under suboptimal environmental conditions. Furthermore, the SHI serves as a meaningful predictor of real-world healing efficiency by capturing both microbial responsiveness and biomineralization output. It links the gap between biological viability and engineering function, making it a valuable metric for material designers and field engineers alike. To aid interpretation, Figure 5b classifies each SHI value into four practical categories: poor, moderate, good, and excellent. According to this scale given in Figure 5, excellent healing performance was confirmed under pH 9 in range of 28-37 °C, ideal for real-world application in temperate environments. Good performance was observed at pH 7 and 37 °C, and pH 11 and 37 °C, suggesting that slightly more extreme conditions can still support self-repair. Poor and moderate effects (e.g., pH 11 at 15 °C) highlight configurations unlikely to produce meaningful crack sealing. This classification system provides actionable insights into when and system based on SCG and *B. licheniformis* may be deployed effectively.

As Omoregie et al. [8] emphasize, long-term storage stability is critical for the practical deployment of microbial self-healing systems in construction materials. As shown in Figure 6, the viability of *B. licheniformis* immobilized within the SCG matrix remained consistently high over a 90-day period when stored in dried form at standard room conditions.

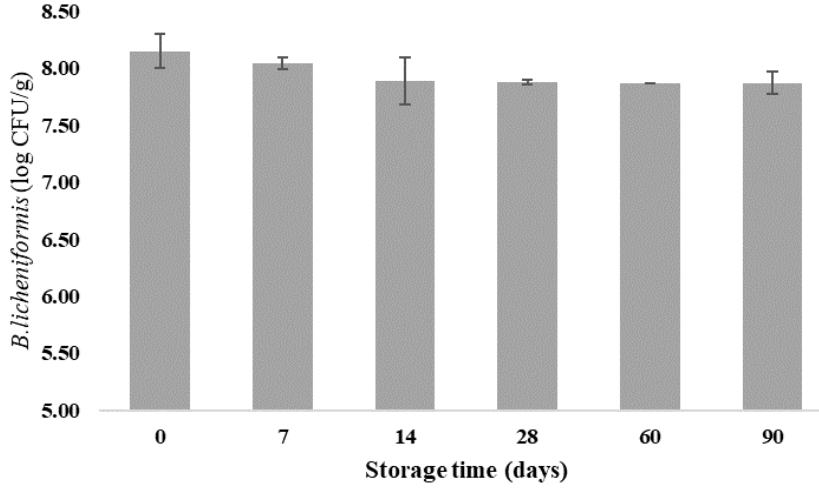


Figure 6. SCG-bacteria system stability during storage at standard room temperature

The initial bacterial concentration was approximately 8.15 log CFU/g, and minimal declines were observed across time points, with the final count at day 90 still maintaining 7.87 log CFU/g. These results highlight the protective nature of the SCG matrix, which effectively preserves bacterial viability by providing both physical shelter and a stabilizing microenvironment. The slight decrease in CFU values over time may reflect natural desiccation stress, but the magnitude of reduction (approx. 0.2 log units) is within acceptable ranges reported for other organic carriers like expanded perlite or diatomaceous earth [7,12]. The ability of *B. licheniformis* to remain viable for extended periods without refrigeration makes this system particularly attractive for real-world applications, where storage and transport under non-controlled conditions are often necessary. This resilience also ensures that bacteria can remain dormant in the cement matrix and activate upon moisture/water due to crack formation.

4. CONCLUSIONS

This study demonstrated the feasibility and effectiveness of utilizing spent coffee grounds (SCG) as a sustainable carrier matrix for the immobilization of *B. licheniformis* in self-healing systems. Environmental factors, particularly pH and temperature, strongly influenced the performance of the microbial system. Both cell release rate and $CaCO_3$ precipitation peaked at pH 9 and 37 °C, with a maximum $CaCO_3$ yield of 151.2 mg/g and cell release up to 84.9%. The self-healing index (SHI), developed as a composite metric of healing efficiency, further confirmed that this condition represents the optimal operational window for self-repair. Additionally, the SCG-based system supported long-term bacterial viability, maintaining over 7.9 log CFU/g even after 90 days of storage under standard room conditions, making it suitable for practical field applications. These results suggest that SCG is not only a viable waste-derived carrier but also an enabler of environmentally responsive $CaCO_3$ precipitation which is preconditions for healing effect in cement-based materials.

Acknowledgments

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

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Miloš Marković[3]

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Stručni rad

ESTIMATING THE CONCRETE COMPRESSIVE STRENGTH USING ULTRASOUND TOMOGRAPHY

Summary: *Evaluating the compressive strength of concrete without altering the structure is often achieved using non-destructive testing (NDT) methods. These techniques are popular in practice due to their low cost and the fact that they leave the structural elements intact. Among them, the rebound hammer and ultrasonic pulse velocity tests are the most frequently applied. However, a major drawback is their limited precision when compared to destructive methods. To overcome this issue, various analytical models have been introduced to better link NDT measurements with actual compressive strength values. In this investigation, a combined approach using both destructive and non-destructive techniques was adopted to assess in-situ concrete quality. Due to structural limitations, core extraction was only feasible from fourteen columns. A polynomial regression model was applied to establish a relationship between the compressive strength and the velocity of shear waves obtained from ultrasonic tomography.*
Keywords: *Structural engineering, ultrasonic pulse velocity, compressive strength, concrete assessment*

PROCENA ČVRSTOĆE BETONA PRI PRITISKU PRIMENOM ULTRAZVUČNE TOMOGRAFIJE

Rezime: *Procena pritisne čvrstoće betona bez narušavanja konstrukcije često se sprovodi primenom nedestruktivnih ispitivanja (NDT) metoda. Ove tehnike su u praksi popularne zbog niskih troškova i činjenice da ne oštećuju konstruktivne elemente. Među njima se najčešće koriste ispitivanje veličine odskoka čekića i ispitivanje brzinom ultrazvučnog impulsa. Ipak, glavni nedostatak ovih metoda je ograničena preciznost u poređenju sa destruktivnim metodama. Kako bi se prevazišao ovaj problem, uvedeni su različiti analitički modeli koji bolje povezuju NDT merenja sa stvarnim vrednostima pritisne čvrstoće. U ovom istraživanju primenjen je kombinovani pristup koji uključuje i destruktivne i nedestruktivne metode u cilju ocene kvaliteta betona na licu mesta. Zbog konstruktivnih ograničenja, uzorkovanje jezgara bilo je moguće samo sa četrnaest stubova. Za uspostavljanje veze između pritisne čvrstoće i brzine prostiranja transverzalni talasa dobijene ultrazvučnom tomografijom, primenjen je polinomni regresioni model.*
Ključne reči: *građevinsko inženjerstvo – konstrukcije, brzina ultrazvučnog impulsa, pritisna čvrstoća, ocena stanja betona.*

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1. INTRODUCTION

Non-destructive testing (NDT) methods play a vital role in evaluating the physical and mechanical characteristics of concrete. Accurate analysis of these characteristics contributes significantly to improving structural performance and reducing construction costs [1,2]. For reinforced concrete structures, compressive strength is often the primary parameter of interest, largely because it's relatively simple to test and serves as a basis for estimating other properties such as the elastic modulus and durability in harsh environments [1,3].

NDT techniques are widely used not only for evaluating newly constructed elements but also for diagnosing aging or damaged structures, both prior to and following rehabilitation interventions. Their main objectives include assessing structural soundness, identifying defects, performing quality assurance, and estimating relevant material properties.

Ultrasonic waves, which are commonly employed in NDT, propagate through materials based on how particles vibrate. The two most commonly utilized wave types in solid materials are longitudinal waves (P-waves) and shear waves (S-waves), with longitudinal waves traveling at higher speeds [4,5].

While destructive testing remains the benchmark for determining concrete compressive strength, especially due to its high reliability, it is not always feasible. Typically, specimens are cast and tested in laboratories. However, in cases where laboratory-prepared samples are unavailable or unsuitable, core samples are drilled directly from structural components, then prepared and tested under controlled conditions [6,7].

Ultrasonic pulse velocity (UPV) testing involves sending a pulse through a concrete element and measuring the time it takes to pass through. From this, wave velocity is calculated and can be related to the concrete's compressive strength. Although NDT methods cannot replace destructive techniques, they are useful for identifying regions with potentially poor mechanical performance.

For more accurate compressive strength predictions, it is essential to develop a reliable estimation model. Besides conventional regression-based approaches, recent studies have explored advanced operational research techniques to evaluate concrete properties.

Both pressure and shear wave velocities are utilized to determine mechanical parameters such as strength and elastic modulus. These velocities also help detect internal flaws, such as voids or cracks, by analyzing reflections or variations in wave speed. UPV measurements are thus a practical means of evaluating concrete density, uniformity, and structural integrity [8,9,10].

2. EXPERIMENTS

The experimental study was carried out on newly constructed columns in public buildings in Belgrade, following a request from the Supervising Engineer to perform quality testing on the concrete. To determine and estimate the compressive strength of the reinforced concrete (RC) columns, both destructive and non-destructive testing methods, including ultrasound tomography, were employed.

A total of fourteen core samples were extracted and tested using a concrete compression testing machine with a 2000 kN capacity. The loading rate during the tests was approximately 0.2 MPa/s, and the results are summarized in Table 1.

Sample code / Šifra uzorka	Mark sample/ Oznaka uzorka	Construction element/ Konstruktivni element	Mix design/ Receptura	Odnos h/d	Prečnik jezgra d (mm)	Dužina jezgra na prijemu H (mm)	Prosečna dužina pripremljenog jezgra h (mm)	Čvrstoća pri pritisku jezgra (N/mm²)	Redukovana čvrstoća (N/mm²)
BET-20304/22 (1)	K-1 S-18	PTB 2.2 columns/ stubovi	P - 3218	1.1	50	120	56	48.5	57.0
BET-20304/22 (2)	K-2 S-17		P - 3218	1.1	50	130	56	46.7	54.9
BET-20304/22 (3)	K-3 S-10		P - 3218	1.1	50	110	56	43.9	51.6
BET-20304/22 (4)	K-4 S-24		P - 3218	1.1	50	140	56	42.3	49.8
BET-20304/22 (5)	K-5 S-38		P - 3218	1.1	50	125	56	50.3	59.2
BET-20304/22 (6)	K-6 S-39		P - 3405	1.1	50	140	56	49.7	58.5
BET-20304/22 (7)	K-7 S-43		P - 3218	1.1	50	125	56	49.4	58.1
BET-20304/22 (8)	K-8 S-12		P - 3218	1.1	50	140	56	46.2	54.3

Table 1. Compressive strength on cores (Institute IMS Report BET 20304/22)

P-Wave Velocity Measurement

The P-wave ultrasonic measurement is based on the time it takes for a stress wave to arrive at the receiver. In this study, the direct transmission technique was used, which involves transmitting P-wave energy through a section between the transmitter and the receiver (Figure 1).



Figure 1. P-Wave velocity measurement equipment

In the through-transmission method, ultrasound waves are transmitted through one surface of the material by the transmitter, and the receiver detects the waves on the opposite side. The output from the receiver is displayed on an oscilloscope as a time-domain waveform. When the wave speed in the material is known, the travel time of the

wave can be used to calculate the depth of the tested element. For concrete testing, transducers that emit short-duration, low-frequency (50–100 kHz) focused waves are commonly used [3,4].

S-Wave Velocity Measurement

S-wave measurements were carried out using a Proceq Pundit PD8000, which was equipped with 24 sensors (Figure 2). The system employed dry point contact (DPC) technology, eliminating the need for a couplant, and A-scans were recorded using ultrasonic multi-channel pulse echo technology with shear waves. Real-time B-scans were later obtained (Figure 3). Since shear waves cannot propagate through liquids or gases, the presence of water in the concrete did not affect the measurements.



Figure 2. S-Wave velocity measurement equipment

Each measurement was taken three times at each point, and the average value was computed. Shear waves cause particle motion that occurs perpendicular to the direction of wave propagation, with a velocity roughly 60% of that of P-waves.

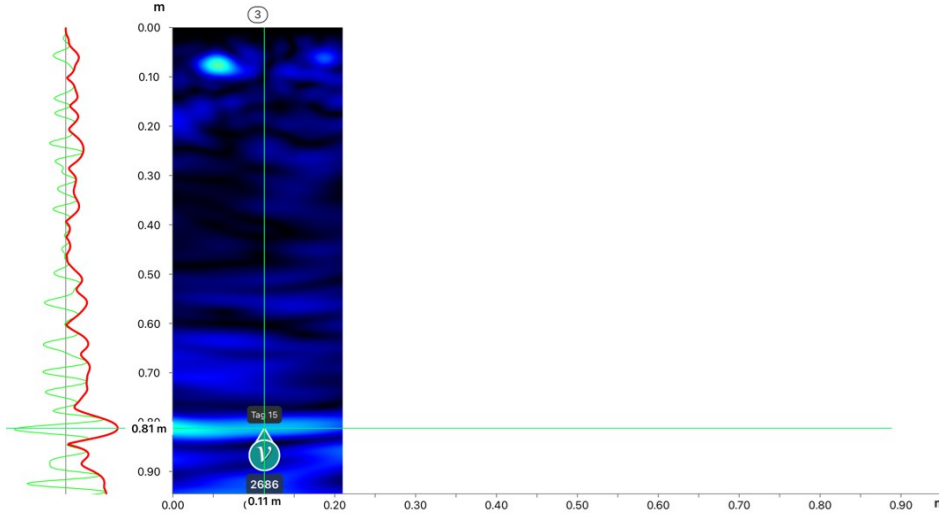


Figure 3. Example of ultrasound tomogram

Sample code	Shear velocity [m/s]	Longitudinal velocity [m/s]	Compressive strength [MPa]	Estimated compressive strength [MPa]
S 12 2B	2701	4745	54.3	**54.6**
S 14 2B	2674	4698		**52.8**
S 19 2B	2696	4736		**54.3**
S 21 2B	2719	4777		**55.6**
S 26 2B	2686	4719		**53.7**
S 28 2B	2731	4729		**56.2**
S 37 2B	2833	4997		**58.7**
S 38 2B	2830	4985	59.2	**58.7**
S 39 2B	2820	4954	58.5	**58.7**
S 40 2B	2644	4645		**50.4**
S 43 2B	2810	4770	58.1	**58.6**
S 49 2B	2641	4640		**50.2**
S 55 2B	2688	4722		**53.8**
S 57 2B	2698	4740		**54.4**
S 59 2B	2723	4799		**55.8**
S 10 2B	2656	4666	51.6	**51.4**
S 11 2B	2641	4640		**50.2**
S 17 2B	2702	4747	54.9	**54.7**
S 18 2B	2809	4935	57.0	**58.5**
S 24 2B	2648	4652	49.8	**50.8**
S 25 2B	2672	4907		**58.3**

Table 2. Estimated compressive strength of each column based on correlation between ultrasound shear velocity and compressive strength of cores.

This study, which relies on a relatively small set of comparative data (Table 2), centers on the polynomial regression model. The model was used to describe the relationship between ultrasonic shear wave velocity and compressive strength. The parameters of the model were determined using the least squares method. Once the model was fitted, its accuracy was assessed through standard regression evaluation metrics (Figure 4).

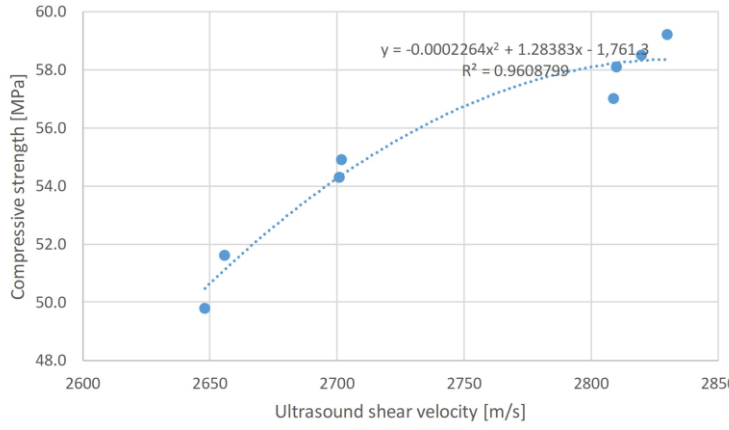


Figure 4. Polynomial regression curve

Table 3 displays an additional dataset to complement the previously analyzed results, with the aim of investigating how the coefficient of determination (R^2) is affected by a narrower range of input values. The intention was to demonstrate how limiting the dataset to a smaller span of shear wave velocities and corresponding compressive strengths influences the accuracy and predictive strength of the polynomial regression model. This comparison helps to assess the model's stability and its sensitivity to data variability, thereby offering deeper insight into its performance under different data conditions.

Sample code	**Shear velocity** [m/s]	**Compressive strength** [MPa]
1-11	2194	29.2
2-5	2213	32.9
3-1	2134	25.5
4-1	2192	32.6
4-6	2178	32.5
4-11	2186	29.2

Table 3. Comparison between shear wave velocity and compressive strength on cores

Table 3 presents the results of shear wave velocity measurements and corresponding compressive strength values obtained from core samples extracted from reinforced concrete columns. The recorded velocities ranged from 2134 to 2213 m/s, while the compressive strengths varied between 25.5 MPa and 32.9 MPa. These results were used to develop and validate a polynomial regression model (Figure 5), aiming to establish a correlation between shear wave velocity and compressive strength. The consistency of the data supports the potential of ultrasound tomography as a non-destructive method for identifying variations in concrete quality within structural elements.

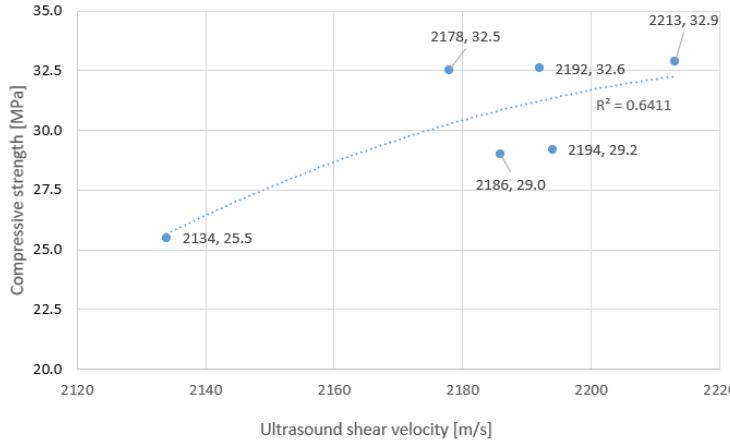


Figure 5. Polynomial regression curve

3. CONCLUSION

The samples for experimental analysis were obtained by extracting cores from the columns for testing. A combination of destructive and non-destructive methods was employed to assess the concrete quality in the structure. Core sampling was restricted to fourteen columns, which had not been fully coated with the final stone layer. A polynomial regression model was utilized to establish a relationship between the shear wave ultrasound velocity (measured via ultrasound tomography) and the concrete's compressive strength. Compressive strength tests were carried out to evaluate the quality of the concrete in the existing reinforced concrete (RC) elements.

From the experimental results, the following conclusions were made:

- The correlation between ultrasound tomography and compressive strength indicates that ultrasound tomography can quickly and effectively pinpoint areas of weaker concrete within RC elements,
- The quality of concrete placement and curing has a significant influence on the achieved strength, and ultrasound tomography is capable of detecting these variations in RC elements,
- Although the correlation may slightly vary with a larger sample size, the relationship between these factors remains clearly evident,
- The R-squared factor is lower than one due to the narrow range of test results, as well as the inherent variability and limitations of both destructive and non-destructive testing methods.

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

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ANALIZA MODELA NAPON–DILATACIJA U TEHNOLOGIJI 3D ŠTAMPE BETONOM

Rezime: *U radu je analizirana primena matematičkih modela Popovics, Evrokod 2 i Stojkovic za opisivanje krive napon–dilatacija 3D štampanog betona (3DPC) na osnovu dostupnih eksperimentalnih podataka. Na osnovu RMSE kriterijuma, utvrđeno je da model Stojkovic najbolje aproksimira eksperimentalne rezultate. Predložene su i funkcije zavisnosti parametara od čvrstoće betona, čime se doprinosi preciznijoj numeričkoj analizi konstrukcija izrađenih 3D štampom.*
Ključne reči: *3D štampani beton, tehnologija 3D štampe, veza napon-dilatacija.*

STRESS-STAIN MODEL FOR 3D PRINTED CONCRETE

Summary: *This paper analyses the application of the Popovics, Eurocode 2 and Stojkovic models for describing the stress–strain curve of 3D printed concrete (3DPC) based on available experimental data. Based on RMSE criteria, it was determined that the Stojkovic model best approximates the experimental results. Functions describing the dependence of model parameters on concrete compressive strength were also proposed, contributing to more accurate numerical analysis of 3D printed structures.*
Keywords: *3D printed concrete, 3D printing technology, stress-strain curve.*

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1. UVOD

Sve je češća upotreba inovativnih materijala u oblasti građevinarstva. Mnogi istraživači se trude da ispitaju različite mogućnosti primene 3D štampanih betona (3DPC). Pri tome, javlja se potreba za adekvatnim opisom određenih korelacija koje su dobijene eksperimentalnim istraživanjem, kako bi se ti materijali koristili pri različitim numeričkim analizama. U radu su analizirane matematičke formulacije krive napon-dilatacija za 3DPC na osnovu podataka eksperimentalnih istraživanja na osnovu dostupne literature.

Za primenu 3DPC potrebno je izraditi smešu specijalne recepture, najčešće ojačanu vlaknima, koja se pomoću mlaznice 3D štampača nanosi u slojevima, horizontalno, sloj po sloj. Proces štampe obuhvata tri osnovne faze: digitalnu pripremu modela i pravca nanošenja slojeva, pripremu betona i samu izradu elementa [1-2].

Tehnologija 3D štampe omogućava izgradnju objekata sa višim stepenom automatizacije, uz istovremeno smanjenje vremena izgradnje, angažovane radne snage i ukupnih troškova [3-4]. Takođe, doprinosi i smanjenju negativnog uticaja na životnu sredinu, jer omogućava upotrebu ekološki prihvatljivijih materijala i smanjenje količine građevinskog otpada [1]. Tehnologija omogućava izgradnju objekata sa složenom geometrijskom formom, koje je teško realizovati konvencionalnim metodama. Međutim, metoda je ograničena dimenzijama objekata, a dodatni izazovi odnose se na tehničke karakteristike smeše, koje moraju obezbediti visoku obradivost i dovoljnu ranu čvrstoću kako bi slojevi mogli biti uspešno naneseni.

Pored analize mehaničkih svojstava materijala, razumevanje veze napon–dilatacija ima značajnu primenu u upravljanju procesom građenja u 3D štampi. Modeli koji opisuju ovu vezu mogu poslužiti kao osnova za razvoj sistema nadzora i kontrole štampe kod fazne gradnje objekata ovom tehnologijom. Time se omogućava optimizacija parametara štampe i unapređenje kvaliteta konačnog proizvoda.

Ovaj rad se bavi analizom modela veze napon-dilatacija kod 3DPC sa ciljem utvrđivanja jednačine modela koji najbolje opisuje tu vezu na osnovu dostupnih eksperimentalnih podataka. Eksperimentalni podaci koji su korišćeni za potrebe ovog istraživanja predstavljeni su u radu [5].

2. 3D ŠTAMPANI BETON

Beton koji se koristi za izradu konstruktivnih elemenata metodom 3D štampe može se podeliti na beton na bazi portland cementa i geopolimerni beton. Obe vrste betona mogu biti sa ili bez ojačanja vlaknima, odnosno armature. Ta vlakna, koja su relativno malih dimenzija mogu biti čelična, na bazi stakla, kompozitnih materijala, itd.

Glavna prednost 3DPC i tehnologije 3D štampe je izrada elemenata bez upotrebe oplate, povećana arhitektonska sloboda, kao i smanjenje troškova i vremena izgradnje. Kod konvencionalne gradnje betonskih konstrukcija, više od 50% ukupnih troškova otpada na oplatu i radnu snagu [6]. Glavni nedostatak konstrukcija od 3DPC jeste ograničenje njihovih dimenzija i izrada samo vertikalnih elemenata.

3. MODELI VEZE NAPON-DILATACIJA

3.1. Model prema Popovics-u

Popovics [7] je razvio model koji omogućava aproksimaciju bilo koje krive zavisnosti napona i deformacije za beton. Navedeni model opisuje ovu zavisnost pomoću sledeće matematičke relacije:

$$\frac{\sigma_0}{\sigma_{0,c}} = \frac{\varepsilon_c}{\varepsilon_{0,c}} \cdot \frac{n}{n-1+\left(\frac{\varepsilon_c}{\varepsilon_{0,c}}\right)^n} \tag{1}$$

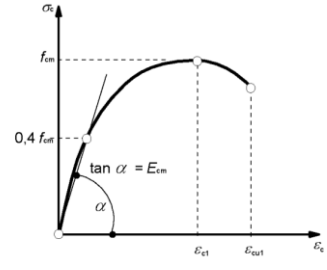
gde je n parametar kojim se model prilagođava eksperimentalnim podacima. Popovics je predložio da n zavisi isključivo od čvrstoće betona na pritisak i da se može izraziti kao $0.4\times10^{-3}\sigma_{0,c}$. Dalja istraživanja, pokazala su da ova zavisnost nije potpuna, budući da u značajnoj meri zavisi od odnosa količina agregata i cementa, kao i veličine agregata [8].

3.2. Model prema Evrokodu 2

U okviru standarda Evrokod 2 [9], kriva napona i dilatacije za beton (slika 1) definiše se na osnovu čvrstoće na pritisak i odgovarajuće dilatacije prema izrazu:

$$\sigma_c = \frac{k\eta-\eta^2}{1+(k-2)\eta} f_c ; \eta = \frac{\varepsilon_c}{\varepsilon_{c1}} ; k = 1.05E_c \frac{\varepsilon_{c1}}{f_c} \tag{2}$$

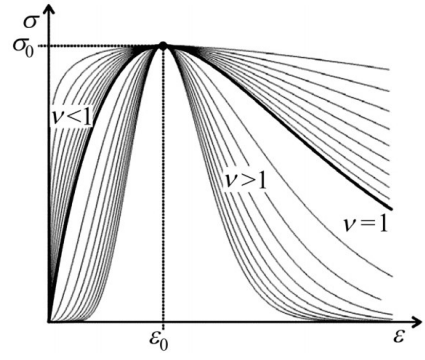
gde σ_c predstavlja napon pritiska u betonu, ε_c je dilatacija betona pri pritisku, $\varepsilon_{c1} = 0.00205$ je dilatacija betona pri maksimalnom naponu pritiska i $\varepsilon_{cu1} = 0.0035$ granična dilatacija betona pri pritisku.



Slika 1. Šematski prikaz dijagrama napon–dilatacija za analizu konstrukcija, prema [9]

3.3. Model prema Stojkovicu

U radu [10] autori su predložili jednačinu koja dobro opisuje oblik dijagrama napona i dilatacije betona. Određena neslaganja sa eksperimentalnim podacima su uglavnom rezultat parametra ν. Na slici 2 prikazano je kako ovaj parametar utiče na oblik krive napona i deformacije.



Slika 2. Uticaj parametra v na oblik dijagrama napona i dilatacije, prema [10]

Za određivanje parametra ν, odnosno prilagođavanje modela eksperimentalnim krivama, Stojković i dr. su u radu [11] koristili metodu najmanjih kvadrata [12]. Kako bi se postigao puni potencijal prilagođavanja krive, urađeno je prilagođavanje krive isključivo za uzlazni deo grane eksperimentalne krive. Predložen je promenljiv parametar ν, sa vrednostima α na početku i β na kraju. Vrednosti variraju u zavisnosti od odnosa $\varepsilon_c / \varepsilon_{0,c}$, što je predstavljeno izrazom:

$$\frac{\sigma_0}{\sigma_{0,c}} = \left(\frac{\varepsilon_c}{\varepsilon_{0,c}} \cdot \mathrm{e}^{\left(1-\frac{\varepsilon_c}{\varepsilon_{0,c}}\right)} \right)^{\left(\alpha+\frac{\varepsilon_c}{\varepsilon_{0,c}}\cdot(\beta-\alpha)\right)} \tag{3}$$

gde je σ_0 napon, ε_c dilatacija, $\sigma_{0,c}$ maksimalni napon, $\varepsilon_{0,c}$ dilatacija koja odgovara maksimalnom naponu, dok se parametri α i β mogu odrediti prilagođavanjem krive eksperimentalnim podacima. Iz jednačine (3) proizilazi da za vrednost $\varepsilon_c = 0$ parametar ν ima vrednost α, dok za $\varepsilon_c = \varepsilon_{0,c}$ vrednost parametra ν iznosi β. Parametri α i β mogu se odrediti korišćenjem metode najmanjih kvadrata, koja se pokazala kao veoma efikasna u tu svrhu.

4. POREĐENJE MODELA I EKSPERIMENTALNIH PODATAKA

Parametri α i β koji su dobijeni prilagođavanjem uzlaznoj grani krive daju znatno bolje slaganje sa eksperimentalnim podacima u poređenju sa parametrima dobijenih iz prilagođavanja duž cele krive. Iz tog razloga je u ovom radu razmatrana samo prva opcija. Vrednosti parametara prikazani su u Tabeli 1.

Prilagođavanjem uzlazne grane pomoću jednačine (3) dobijeni su parametri α i β. Generalized Reduced Gradient (GRG) Nonlinear u Excel Solveru je metoda optimizacije koja rešava nelinearne probleme, koristeći sofisticiranu matematičku proceduru.

Broj dana	Mešavina	α	β	n
3	a	0,983538296	1,207905679	1,857180511
3	b	1,141430522	1,881718909	1,8423181
3	c	1,051955	1,018548	1,805985324
3	d	0,980088	1,44938	1,818966
3	e	0,754484	0,835821	1,587169
3	f	1,026765	0,384136	1,657506
7	a	0,902934	1,512721	1,901155
7	b	0,973646	1,572265	1,891724
7	c	1,100482	1,672495	1,81975
7	d	1,079785	1,022584	1,862611
7	e	0,844681	0,90928	1,656309
7	f	1,139721	0,610266	1,717056
28	a	1,102352	1,414652	1,962039
28	b	1,070678	1,40842	1,986604
28	c	1,000264	0,880285	1,91319
28	d	0,809227	1,1411588	1,947273
28	e	1,038896	0,965853	1,742356
28	f	1,059282	1,338181745	1,770776

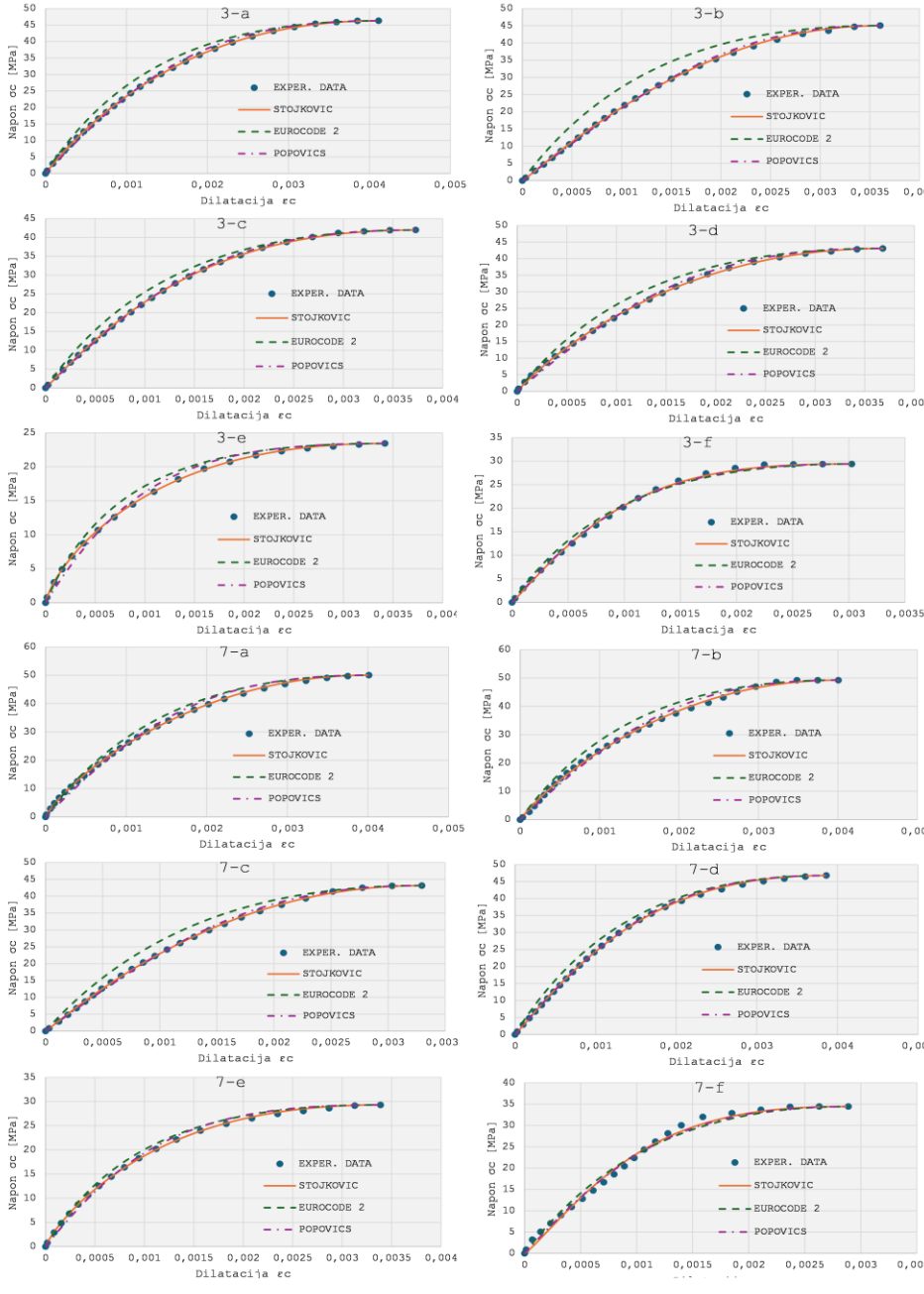
Tabela 1. Vrednosti parametara uzlazne grane krive napona i dilatacije

Određena je funkcija zavisnosti parametara u odnosu na čvrstoću pri pritisku betona:

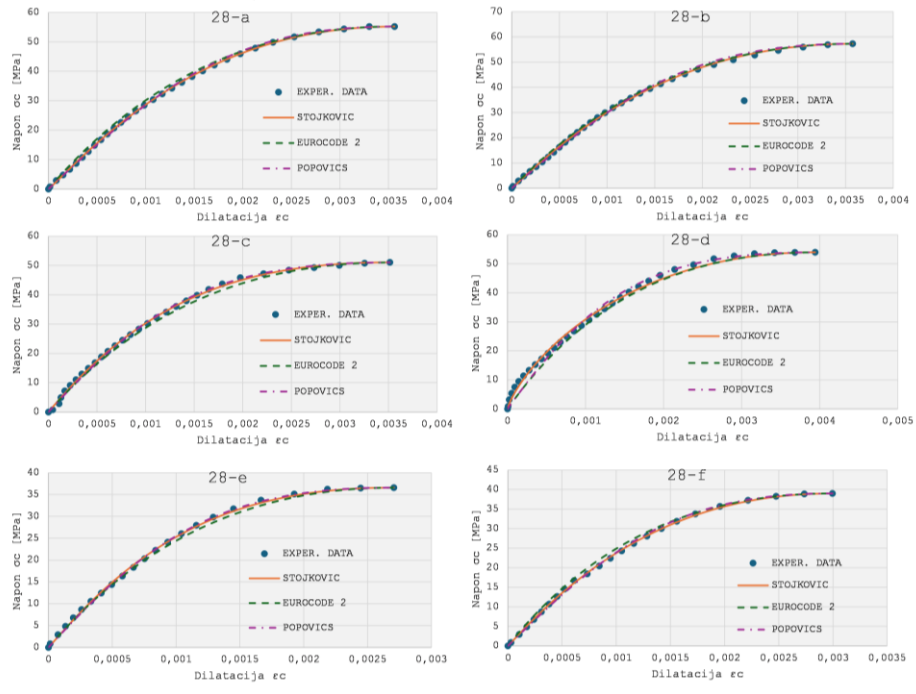
$$\begin{aligned} \alpha &= 0,003 \cdot \sigma_0 + 0,8721 \\ \beta &= 0,0234 \cdot \sigma_0 + 0,1697 \\ \mathrm{n} &= 0,0118 \cdot \sigma_0 + 1,3106 \end{aligned} \tag{4}$$

Slike 3 i 4 prikazuju poređenje između eksperimentalnih dijagrama napona i dilatacije i krivih predstavljenih pomoću jednačina (1), (2) i (3) za uzlazni deo krive i parametre prilagođene eksperimentalnim podacima.

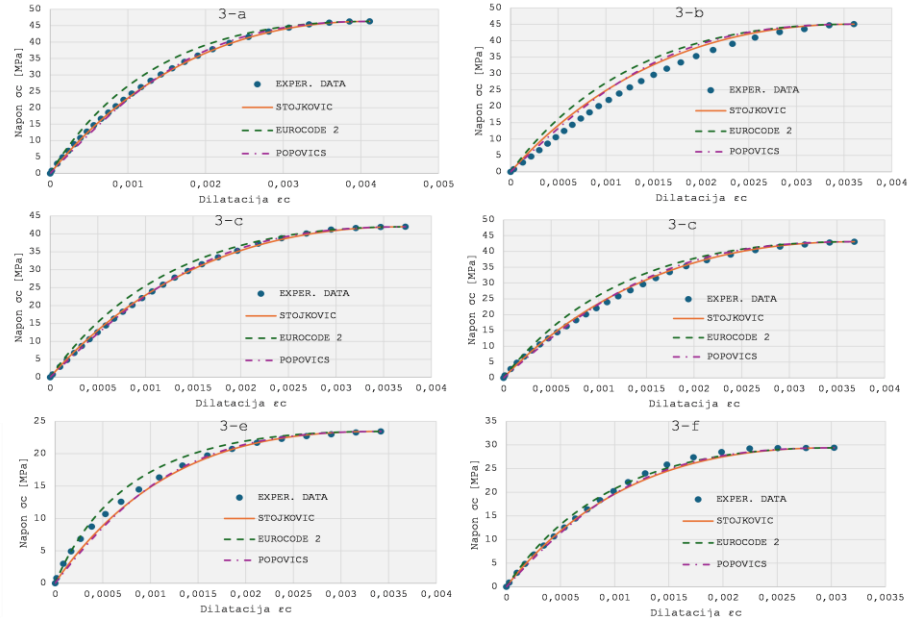
Na slikama 5 i 6 prikazani su dijagrami napon-dilatacija na osnovu eksperimentalnih podataka i modela krivih sa parametrima izračunatim prema izrazu (4).



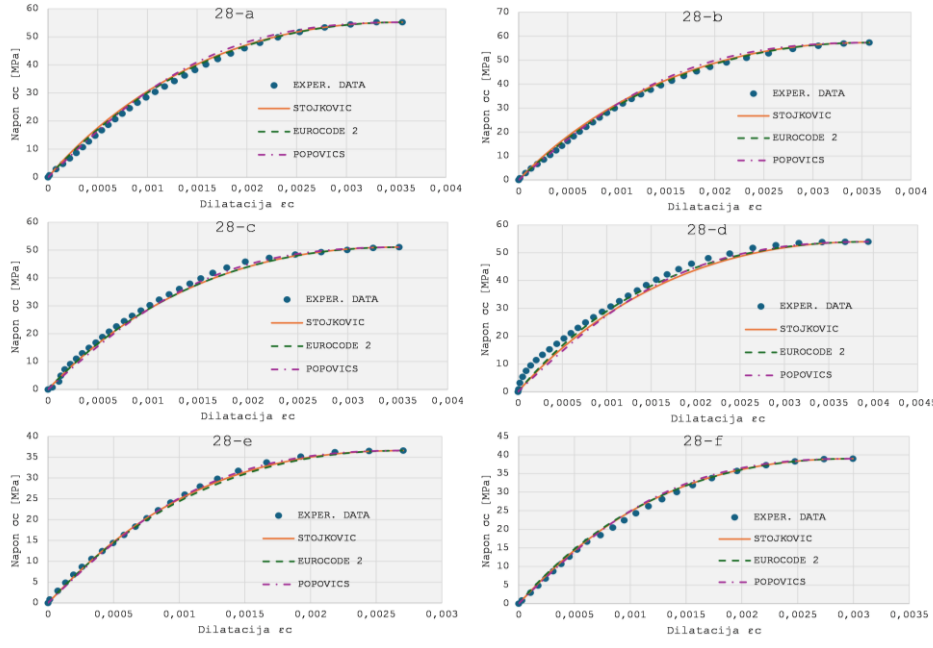
Slika 3. Poređenje kriva napon-dilatacija dobijene na osnovu eksperimenta i modela za prilagođene parametre



Slika 4. Poređenje kriva napon-dilatacija dobijene na osnovu eksperimenta i modela za prilagođene parametre



Slika 5. Poređenje kriva napon-dilatacija dobijene na osnovu eksperimenta i modela za usvojene parametre prema izrazu (4)



Slika 6. Poređenje kriva napon-dilatacija dobijene na osnovu eksperimenta i modela za usvojene parametre prema izrazu (4)

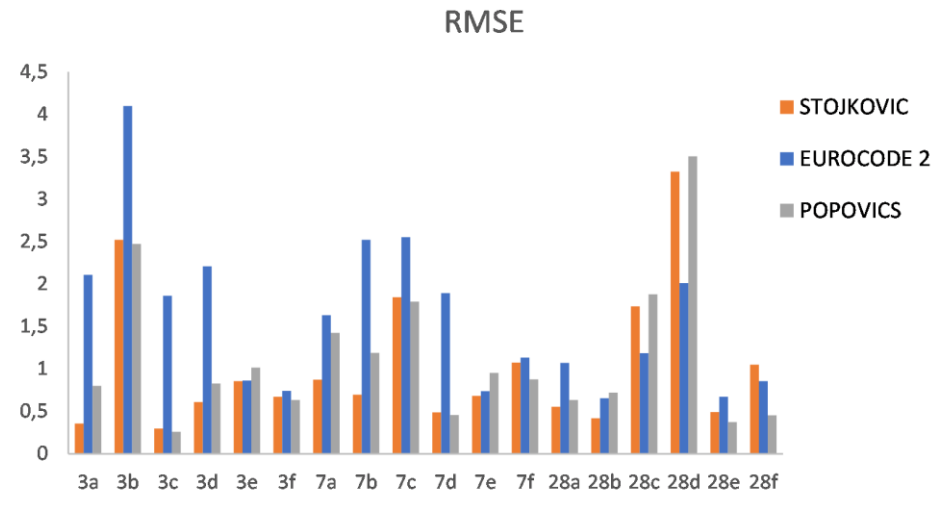
Na osnovu Root Mean Square Error (RMSE) [13], kao i sa slika 3 i 4 može se zaključiti da model koji je predstavljen jednačinom (3) pokazuje najbolje prilagođavanje podacima, budući da ostvaruje najnižu vrednost RMSE u poređenju sa ostalim razmatranim modelima. RMSE se računa na osnovu izraza:

$$\mathrm{RMSE} = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(\hat{y}_i - y_i)^2} \tag{4}$$

gde je $\hat{y}_i$ predviđena vrednost, y_i stvarna vrednost i n broj primena.

5. ZAKLJUČAK

Na osnovu prilagođavanja parametara α i β za uzlazni deo krive model opisan u radu [11] najbliže opisuje veze napon-dilatacija na osnovu eksperimentalnih podataka iz rada [5]. Ipak, te vrednosti parametara su računate na osnovu najmanjeg RMSE. Kako bi se odredila matematička formula tih parametara određena je njihova zavisnost u odnosu na čvrstoću pri pritisku betona, na osnovu izraza (4). Prema tom izrazu su određeni parametri α i β. Potom je za svaki model i uzorak izračunat RMSE i to je prikazano na slici 7.



Slika 7. Veličine RMSE svih uzoraka i modela za parametre prema izrazu (4)

Sa slike 5 se uočava da najmanju srednju vrednost RMSE ima Model Stojkovic ($\Sigma \text{RMSE} = 1{,}029728646$), potom Model Popovics ($\Sigma \text{RMSE} = 1{,}125733834$) i Model Evrokod 2 ($\Sigma \text{RMSE} = 1{,}599848538$).

Prema sprovedenoj analizi proizilazi zaključak da svi modeli predstavljeni u ovom radu mogu biti primenjeni za numeričku analizu konstrukcija napravljenih tehnologijom 3D štampe. Takođe, u radu su predložene funkcije parametara u zavisnosti od čvrstoće betona pri pritisku. Sa tako usvojenim parametrima model koji najbolje prikazuje vezu napon-dilatacije jeste Model Stojkovic.

Osim što omogućava bolju karakterizaciju materijala, model napon–dilatacija ima i značajnu ulogu u upravljanju procesom 3D štampe betona.

Za dalja istraživanja jeste preporuka da se obuhvati što veći broj eksperimentalnih podataka i na taj način poboljšaju jednačine parametara.

Zahvalnica

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

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Stručni rad

POBOLJŠANJE IZOLACIJE OD VAZDUŠNOG ZVUKA MASIVNOG ZIDA OD GITER BLOKA DODAVANJEM LAKIH SISTEMA SA GIPS-KARTONSKIM PLOČAMA

Rezime: *U Laboratoriji za akustiku i vibracije Instituta IMS u laboratorijskim uslovima sprovedena su ispitivanja na masivnom zidu od giter bloka i istog zida dodavanjem lakih obloga od mineralne vune i gips-kartonskih ploča sa jedne i sa obe strane masivnog zida. Ispitivanja su sprovedena u serijama prilikom instalacije svake od obloga kako bi se pratile merodavne vrednosti izolacionih moći u različitim fazama instalacije, a sve u cilju da se vidi koliko je poboljšanje zvučne izolacije dodavanjem lakih obloga na masivni zid.*
Ključne reči: *Izolacija od vazdušnog zvuka, masivni zid, lake obloge, giter blok, mineralna vuna, gips-kartonske ploče.*

AIRBORNE SOUND INSULATION OF A SOLID CEMENT BLOCK WALL IMPROVEMENT BY ADDING LIGHTWEIGHT SYSTEMS WITH GYPSUM BOARDS

Rezime: *In the Laboratory for Acoustics and Vibrations of the IMS Institute, tests were conducted under laboratory conditions on a masive cobble block wall and the same wall by adding lightweight mineral wool and gypsum board linings on one and both sides of the masive wall. The tests were conducted in series during the installation of each lining in order to monitor the relevant singl-numbervalues of the sound reduction index at different stages of installation, all with the aim of seeing how much the sound insulation is improved by adding lightweight linings to a masive wall.*
Keywords: *Airborne sound insulation, masive wall, lightweight linings, cement block, mineral wool, gypsum board.*

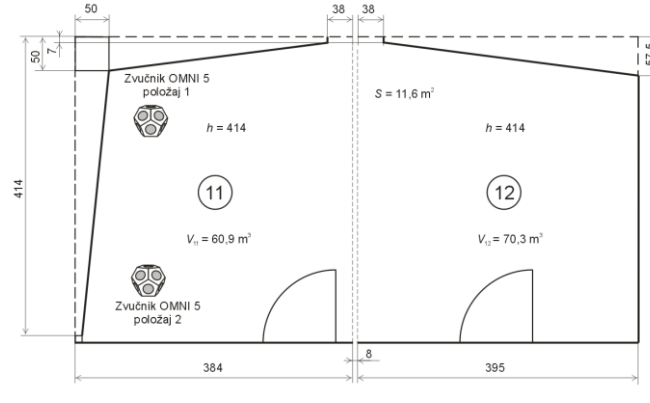
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1. UVOD

Ispitivanja zvučne izolacije (izolacije od vazdušnog zvuka), čiji će rezultati biti prezentovani u radu, sprovedena su u laboratorijskim uslovima, u specijalizovanim ispitnim prostorima (komorama) Laboratorije za akustiku i vibracije Instituta IMS u Beogradu, izrađenim za datu namenu, bez bočnog provođenja zvuka (sa fugama koje razdvajaju prostor između njih), a koje u svemu zadovoljavaju zahteve za ispitnim prostorima u kojima se vrše laboratorijska ispitivanja zvučne izolacije u građevinskoj akustici [1]. Izgled ovih komora u horizontalnom preseku sa označenim mestom između njih u koji se ugrađuje ispitni uzorak prikazan je na Slici 1. Ispitni uzorak je ugrađen u ispitni otvor tako da se uzorak nalazi na strani predajne prostorije (komore u kojoj se nalazi zvučnik) sa vidljivom fugom iz prijemne prostorije (komore) u skladu sa pravilima primene [2].

Ispitivanje nivoa zvučnog pritiska za potrebe ispitivanja zvučne izolacione moći (izolacije od vazdušnog zvuka) je izvršeno za dva položaja zvučnika u predajnoj prostoriji u po šest mernih tačaka (pozicija mikrofona) u predajnoj i prijemnoj prostoriji, a izmerene vrednosti nivoa zvučnog pritiska su logaritamski i prostorno usrednjene kako bi se dobile vrednosti nivoa zvučnog pritiska u predajnoj i prijemnoj prostoriji potrebnih za proračun zvučne izolacione moći u skladu sa postupkom definisanim u [3]. Vreme reverberacije u prijemnoj prostoriji kao parametar koji je takođe potrebno izmeriti za potrebe ispitivanja zvučne izolacione moći je izvršeno za dva položaja zvučnika u tri merne tačke (pozicije mikrofona) sa snimanjem po dva zapisa krive opadanja nivoa zvuka za svaku mernu tačku te je na osnovu dvanaest krivih opadanja određena srednja vrednost vremena reverberacije potrebnog za proračun zvučne izolacione moći u skladu sa preciznim postupkom definisanim u [4]. Ispitni uzorak je bio površine 11,6 m^2, a zapremine ispitnih komora 60,86 m^3 (predajna komora, komora sa oznakom 11) i 70,33 m^3 (prijemna komora, komora sa oznakom 12). Ispitivanje je izvršeno u standardnom frekvencijskom opsegu od 100 Hz do 5000 Hz u skladu sa [5], a merodavna vrednost izolacione moći (jednobrojna vrednost) određena je u frekvencijskom opsegu od 100 Hz do 3150 Hz u skladu sa postupkom datim u [6].

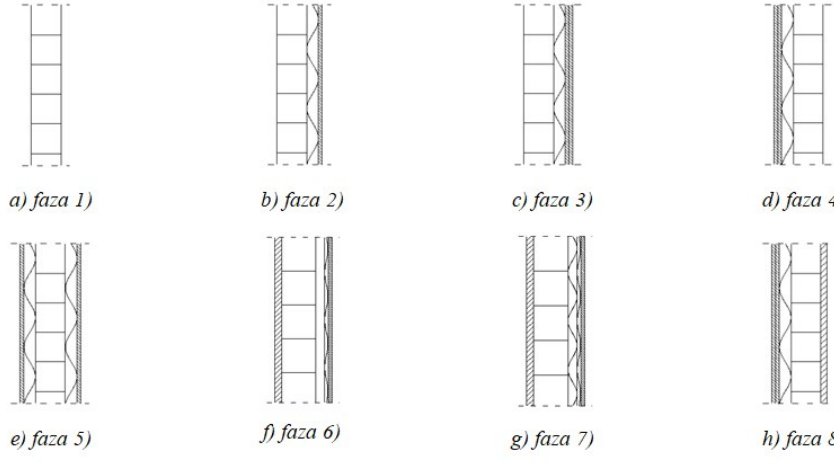
U radu su prikazani rezultati izolacione moći po frekvencijama u standardnom frekvencijskom opsegu, $R(f)$, kao i merodavna (jednobrojna) vrednost izolacione moći, R_w za nekoliko pregradnih konstrukcija sa osnovnim zidom od giter bloka.



Slika 1. Ispitne prostorije u horizontalnom preseku

2. PREDMET ISPITIVANJA

Uzorak zida ugrađen je u ispitni otvor tako da su na osnovni zid od giter bloka debljine 10 cm (GB10) dodavani slojevi i sa jedne i sa druge strane zida. Na Slici 2. su date skice slojeva (sistema) dodatih na osnovni zid prikazanih po fazama. U daljem tekstu su korišćene skraćenice za pojedine slojeve sistema: osnovni zid od giter bloka (GB), gips-kartonska ploča (GKP), gips-kartonska ploča sa mineralnom vunom (GKPMV), mineralna vuna (MV), vazduh (VA) i malter (M).



Slika 2. Faze instalacije zida

Giter blok korišćen u fazama 1, 2, 3, 4, 5 i 8 je klasični glineni blok sa vertikalnim šupljinama proizvođača IGM Mladost bruto zapreminske mase 818,4 kg/m^3 sa procentom šupljina 58%, dok je giter blok korišćen u fazama 6 i 7 iste debljine ali drugog proizvođača i sa različitom bruto zapreminskom masom i procentom šupljina. Gips-kartonske ploče korišćene u fazama 2, 3, 4, 5 i 8 su standardne ploče površinske mase 8,5 kg/m^2, dok su u fazama 6 i 7 korišćene gips-kartonske ploče sa mineralnom vunom kao kompaktan gotov proizvod.

Ispitivanja zida su izvršena u osam faza sa sledećim opisima konstrukcije zida:

F1. GB10: zid od giter-bloka debljine d = 100 mm,

F2. GB10+MV4+GKP1,25: GB10 sa dodatom oblogom koja se sastoji od mineralne vune debljine d = 40 mm i jednog sloja gips-kartonskih ploča debljine d = 12,5 mm sa strane osnovnog zida prema prijemnoj prostoriji ukupne debljine 162,5 mm,

F3. GB10+MV4+2GKP1,25: GB10 sa dodatom oblogom koja se sastoji od kamene vune debljine d = 40 mm i dva sloja gips-kartonskih ploča debljine d = 2x12,5 mm sa strane osnovnog zida prema prijemnoj prostoriji ukupne debljine 175 mm,

F4. 2GKP1,25+MV4+GB10: GB10 sa dodatom oblogom koja se sastoji od kamene vune debljine d = 40 mm i dva sloja gips-kartonskih ploča debljine d = 2x12,5 mm sa strane osnovnog zida prema predajnoj prostoriji ukupne debljine 175 mm,

F5. GKP1,25+MV4+GB10+MV4+GKP1,25: GB10 sa dodatim oblogama koje se sastoje od kamene vune debljine d = 40 mm i jednog sloja gips-kartonskih ploča debljine d = 12,5 mm sa obe strane osnovnog zida ukupne debljine 225 mm,

F6. M2+GB10+VA2,7+GKPMV2,25: GB10 sa dodatim oblogama koje se sastoje od vazduha $d = 27$ mm i jednog sloja gips-kartonskih ploča sa mineralnom vunom debljine $d = 22,5$ mm ukupne debljine 169,5 mm,
F7. M2+GB10+MV2,7+GKPMV2,25: GB10 sa dodatim oblogama koje se sastoje od mineralne vune $d = 27$ mm i jednog sloja gips-kartonskih ploča sa mineralnom vunom debljine $d = 22,5$ mm ukupne debljine 169,5 mm i
F8. 2GKP1,25+MV4+GB10+MV4+M2: GB10 sa dodatim oblogama koje se sastoje od kamene vune debljine $d = 40$ mm i dva sloja gips-kartonskih ploča debljine $d = 2$x12,5 mm sa strane osnovnog zida prema predajnoj prostoriji i od sloja maltera $d = 20$ mm sa strane osnovnog zida prema prijemnoj prostoriji ukupne debljine 195 mm.

3. REZULTATI ISPITIVANJA

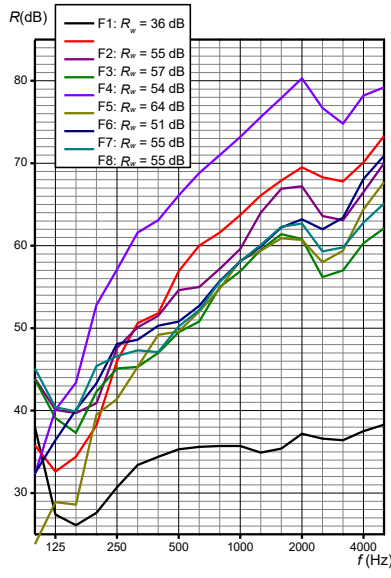
Rezultati ispitivanja prikazani u radu su dati prema ispitnim fazama i označeni osnovnom konstrukcijom (GB10) i dodatim sistemom od gips-kartonskih ploča i mineralne vune (GKP, MV), gips-kartonskim sistemom sa mineralnom vunom (GKPMV) i dodatom slojem vazduha odnosno mineralne vune (VA, MV) kao i slojem maltera (M) kako je opisano u tekstu.

Izmerene vrednosti zvučne izolacione moći za osnovni zid GB10 i dodate sisteme obloge su prikazani uporedno u Tabeli 1 i na dijagramu na Slici 3.

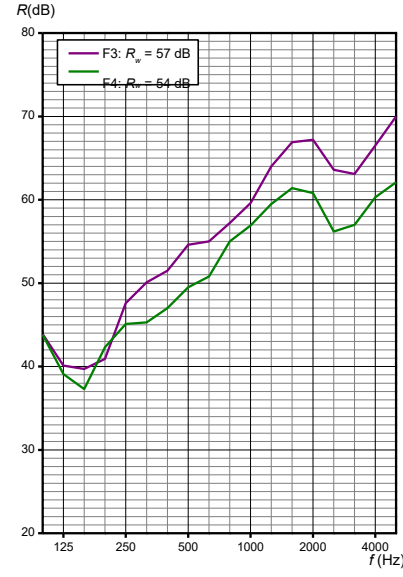
***f* (Hz)**	***R* (dB)**							
	F1	**F2**	**F3**	**F4**	**F5**	**F6**	**F7**	**F8**
100	38.0	35.8	43.9	43.8	32.3	23.8	32.4	45.1
125	27.4	32.6	40.1	39.1	40.1	28.9	36.4	40.4
160	26.1	34.4	39.7	37.3	43.4	28.6	40.1	39.9
200	27.6	38.2	40.9	42.3	52.8	39.5	43.3	45.4
250	30.7	46.0	47.6	45.1	57.1	41.4	48.1	46.6
315	33.4	50.6	50.1	45.3	61.6	45.3	48.6	47.3
400	34.4	51.8	51.5	47.0	63.1	49.2	50.3	47.1
500	35.3	56.9	54.6	49.5	66.1	49.6	50.8	50.2
630	35.6	60.0	55.0	50.8	68.8	52.0	52.7	52.2
800	35.7	61.6	57.2	55.0	71.0	54.8	55.7	55.6
1000	35.7	63.7	59.6	56.9	73.2	58.2	58.1	58.1
1250	34.9	66.1	64	59.5	75.6	59.4	59.9	60.1
1600	35.4	67.9	66.9	61.4	77.9	60.9	62.2	62.3
2000	37.2	69.5	67.2	60.8	80.3	60.7	63.2	62.7
2500	36.6	68.3	63.6	56.2	76.7	58.0	62.0	59.3
3150	36.4	67.8	63.1	57.0	74.8	59.4	63.4	59.8
4000	37.5	70.1	66.5	60.3	78.2	64.4	68.1	62.8
5000	38.3	73.3	70	62.1	79.2	67.7	70.9	65.1
Rw	**36**	**55**	**57**	**54**	**64**	**51**	**55**	**55**

Tabela 1. Izmerene vrednosti zvučne izolacione moći u zavisnosti od vrste obloge dodate na osnovni zid

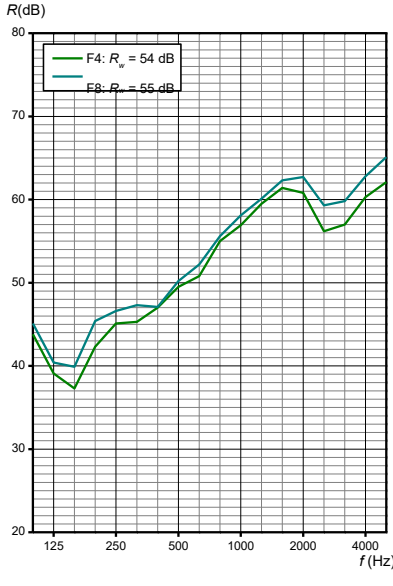
Na dijagramima na Slici 4, Slici 5 i Slici 6 dati su uporedni rezultati po karakterističnim fazama koje su interesantne za analizu.



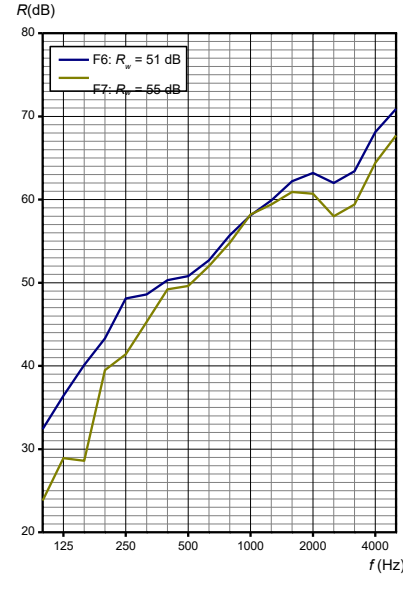
Slika 3. Izmerene vrednosti zvučne izolacione moći u zavisnosti od vrste obloge dodate na osnovni zid



Slika 4. Karakteristični dijagram: izmerene vrednosti zvučne izolacione moći u zavisnosti od mesta pobude



Slika 5. Karakteristični dijagram: izmerene vrednosti zvučne izolacione moći u zavisnosti od tipa završnog sloja



Slika 6. Karakteristični dijagram: izmerene vrednosti zvučne izolacione moći u zavisnosti od tipa međuprostora između osnovnog zida i gips-kartonskog sistema sa mineralnom vunom

4. ANALIZA REZULTATA

S obzirom na to da je ispitivanje zvučne izolacije sprovedeno po fazama parcijalni rezultati ispitivanja pokazuju očekivani trend porasta (poboljšanja) merodavne (jednobrojne) vrednosti izolacione moći dodavanjem slojeva na osnovnu konstrukciju zida.

Merenjem izolacione moći zida u fazi 1) dobijena je očekivana vrednost merodavne izolacione moći $R_w = 36$ dB.

Dodavanjem slojeva u fazama 2) i 3) dobijeno je poboljšanje merodavne izolacione moći od 19 dB kada se doda sistem sa jednom gips-kartonskom pločom ($R_w = 55$ dB) odnosno 21 dB kada se doda sistem sa dve gips-kartonske ploče ($R_w = 57$ dB).

Ipak, u fazama 3) i 4) za nominalno isti zid dobijene su vrednosti izolacione moći koje se razlikuju za 3 dB ($R_w = 57$ dB i $R_w = 54$ dB, respektivno) a s obzirom na to da se desila pojava da se dodavanjem obloge od gips-kartonskih ploča i mineralne vune u sistemu sa strane prijemne prostorije dobija veće poboljšanje zvučne izolacije nego dodavanje iste obloge sa strane predajne prostorije (sa strane koja je izložena zvuku).

Kao posebno interesantno za datu konstrukciju zida od giter bloka je dodavanje gotovog sistema od gips-kartonskih ploča i mineralne vune u fazama 6) i 7) koje ne daje nikakav doprinos zvučnoj izolaciji u fazi 6) u kombinaciji sa vazdušnim slojem ($R_w = 51$ dB) u odnosu na sistem sa odvojenim slojevima mineralne vune i gips-kartonskih ploča ($R_w = 51$ dB), čak ni dodavanjem dodatnog sloja mineralne vune u gotov sistema od gips-kartonskih ploča i mineralne vune u fazi 7) ($R_w = 55$ dB).

U radu je dat rezultat i jednog kompletnog zida sa oblogana od mineralne vune i gips-kartonskih ploča sa obe strane osnovnog zida čijim dodavanjem se postiže zvučna izolacija čak za 9 dB viša ($R_w = 64$ dB) u odnosu na istu oblogu samo sa jedne strane zida ($R_w = 55$ dB), a kao ilustracija je prikazan i rezultat sa dodavanjem maltera sa jedne strane osnovne konstrukcije od giter bloka u fazi 8) koji donosi neznatno poboljšanje izolacije od 1 dB ($R_w = 55$ dB) u odnosu na konstrukciju bez maltera u fazi 4) ($R_w = 54$ dB).

5. ZAKLJUČCI

Kako je svako dobijeno poboljšanje zvučne izolacione moći značajno s obzirom na to da nije dobijena visoka vrednost izolacione moći u fazi 1) ono pokazuje da se dodavanjem lake konstrukcije (mineralna vuna i gips-kartonske ploče) na homogenu konstrukciju (zid od bloka) postiže da se vrednosti zvučne izolacije značajno povećaju što za posledicu ima da se, kada je u pitanju izolacija od vazdušnog zvuka, ne moraju izrađivati suviše debeli masivni zidovi koji zauzimaju znatne zapremine prostora već se isti efekat dobija izradom tanjeg masivnog zida i dodavanjem slojevite lake konstrukcije.

Interesantna je pojava da se dodavanjem obloge od gips-kartonskih ploča i mineralne vune u sistemu sa strane prijemne prostorije dobija veće poboljšanje zvučne izolacije od čak 3 dB nego dodavanje iste obloge sa strane predajne prostorije (sa strane koja je izložena zvuku).

Najbolje poboljšanje na svim frekvencijama svakako daje konstrukcija sa oblogama sa obe strane osnovnog zida od giter bloka, ali na niskim i srednjim frekvencijama dobro poboljšanje daje i konstrukcija sa sistemom od sloja mineralne vune

i gips-kartonske ploče sa jedne i malterom sa druge strane osnovnog zida od giter bloka, dok dobro poboljšanje na visokim frekvencijama daje bilo koja konstrukcija sa sistemom od mineralne vune i gips-kartonskih ploča.

Svakako je najinteresantniji rezultat u primeni gotovih proizvoda koji se u poslednje vreme plasiraju na tržište baš za primenu u zvučnoj zaštiti, od kojih su posebno interesantni proizvodi kod kojih se obloge sklapaju na licu mesta a sadrže i komponenete gips-kartonske ploče i mineralne vune u jednom elementu. Kod ovih proizvoda, a kako su rezultati pokazali, ipak je nedostatak što ne mogu ostvariti poboljšanja koja su reda veličine poboljšanja koja se dobijaju kada se koriste razdvojene komponente mineralne vune i gips-kartonskih ploča u sistemu, već se gotovom sistemu mora dodati još jedan sloj mineralne vune kako bi poboljšanje izolacione moći moglo biti ostvareno, što poskupljuje intervenciju i dovodi u pitanje smisao gotovih sistema naspram dosadašnje uobičajene prakse kada su u pitanju poboljšanja izolacije od vazdušnog zvuka.

Zahvalnica

Rad je nastao u vreme trajanja ugovora između Instituta IMS i Ministarstva nauke, tehnološkog razvoja i inovacija (evidenioni broj ugovora: 30-1331 od 8. 2. 2024).

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[8] A. Milenković, D. Boljević, "Doprinos slojeva u složenim konstrukcijama na zvučnu izolaciju", Zbornik radova LXVII konferencija ETRAN, Istočno Sarajevo, BiH, ISBN 978-86-7466-969-3, 2023, AK2.5.

[9] A. Milenković, D. Boljević, " Doprinos sistema sa gips-kartonskim pločama na zvučnu izolaciju osnovnog zida od giter bloka ", Zbornik radova LXVIII konferencija ETRAN, Niš, Srbija, 978-86-6200-002-6, 2024, AK1.3.

DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

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Stručni rad

DIJAGNOSTIKA STANJA, PREDLOG MERA ČIŠĆENJA I ZAŠTITE OBJEKATA KULTURNOG NASLEĐA: STUDIJA SLUČAJA SRPSKE PRAVOSLAVNE CRKVE U SREMSKOJ KAMENICI

Rezime: *Savremeni principi odgovorne i održive zaštite objekata kulturnog nasleđa zasnivaju se na interdisciplinarnom pristupu, saradnji naučnika i stručnjaka, i upotrebi metoda koje osiguravaju očuvanje autentičnosti objekata. Intervencije na ovim objektima, uključujući čišćenje, sanaciju i konzervaciju, moraju biti zasnovane na temeljnim istraživanjima materijala, mehanizama degradacije i specifičnih uslova sredine. U okviru rada predstavljena je metodologija dijagnostike stanja, čišćenja, desalinacije, konsolidacije i zaštite fasade srpske pravoslavne crkve posvećene rođenju Presvete Bogorodice u Sremskoj Kamenici. Na osnovu kombinovanih in-situ i laboratorijskih analiza definisane su i preporučene metode čišćenja, konsolidacije i dugoročne zaštite, dok je realizovan naučno-stručni nadzor osigurao doslednu primenu predložene metodologije. Ovaj pristup rezultovao je ne samo kvalitetno izvedenim radovima, već i potpunom dokumentacijom procesa, čime je obezbeđena transparentnost i prenosivost znanja za buduće slične konzervatorske intervencije.*
Ključne reči: *nedestruktivne metode ispitivanja, kulturno nasleđe, konzervacija, čišćenje, sanacija, novi funkcionalni materijali*

DIAGNOSIS, CONSERVATION PLANNING, CLEANING AND PROTECTION OF MONUMENTS: CASE STUDY OF THE SERBIAN ORTHODOX CHURCH IN SREMSKA KAMENICA

Summary: *Modern principles of responsible and sustainable heritage protection are based on an interdisciplinary approach, collaboration between scientists and experts, and the application of methods that ensure the preservation of the authenticity of heritage structures. Interventions on such structures—including cleaning, repair, and protection—must be grounded in thorough*

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research of the materials, degradation mechanisms, and specific environmental conditions. This paper presents the methodology for condition assessment, cleaning, consolidation, desalination and protection of the façade of the Serbian Orthodox Church dedicated to the Nativity of the Blessed Virgin Mary in Sremska Kamenica. Based on combined in situ and laboratory analyses, recommended methods for cleaning, consolidation, and long-term protection were defined, while the scientific and professional supervision ensured the consistent implementation of the proposed methodology. This approach resulted not only in high-quality conservation work, but also in comprehensive documentation of the entire process, thus ensuring transparency and the transferability of knowledge for future similar interventions.
Keywords: *non-destructive testing, cultural heritage, cleaning, protection, conservation, new functional material*

1. UVOD

Objekti kulturnog nasleđa, su naše nasledstvo iz prošlosti, ono sa čim živimo danas i što prenosimo budućim generacijama, oni su naše uporište, tačke oslonca i identitet. Upravo zbog njihove neprocenjive vrednosti i jedinstvenog karaktera, sve intervencije na objektima kulturnog nasleđa, uključujući njihovo čišćenje, sanaciju i zaštitu, moraju biti prethodno zasnovane na temeljnom razumevanju materijala, uzroka degradacije i specifičnih uslova sredine. Veliki broj naučnika bavio se postavljanjem metodologije ispitivanja, kao i nedestruktivnim i destruktivnim ispitivanjem istorijskih materijala, kao konstitutivnih elemenata objekata kulturnog nasleđa. Karakterizacija istorijskih materijala predstavlja detaljno istraženu oblast, koja datira još od 60-ih godina prošlog veka. Danas je dostupan značajan broj publikacija i rezultata istraživanja vezanih za ispitivanje istorijskih materijala. Primena nedestruktivnih metoda ispitivanja u dijagnostici stanja, predlogu mera čišćenja i zaštite, kao i kontroli izvođenja radova, predstavlja ključan korak ka odgovornoj i stručno utemeljenoj konzervaciji. Opšteprihvaćeni princip odgovorne i održive zaštite objekata kulturnog nasleđa podrazumeva upotrebu naučnih metoda i rad u multidisciplinarnim timovima. Sistematski i sveobuhvatni pristup zaštiti spomenika kulture se sastoji od tri osnovna koraka: anamneze, dijagnoze i predloga sanacije [4].

Čišćenje i zaštita objekata kulturnog nasleđa može promeniti njihovu boju i površinske karakteristike, utičući na kvalitet i izgled, a u nekim slučajevima može izazvati i nepovratna oštećenja ako pristupi, materijali i metode nisu adekvatno izabrani. Stoga se ovim tretmanima mora pristupiti s krajnjim oprezom, s ciljem da se vizuelno poboljša izgled površine, ali da se pri tome ne ugrozi njen krhki karakter (tj. „da lek ne bude gori od bolesti“). Svi potencijalni materijali i metode za čišćenje i zaštitu fasada moraju se prethodno ispitati na adekvatnim modelima u laboratorijskim uslovima, i na probnim površinama manjih dimenzija na samom objektu. Takođe, pored rezultata o izvornim materijalima sa samog objekta, izbor metoda i materijala za konzervatorski tretman treba biti u skladu sa zaštitom životne sredine ali i bezbednošću izvođača radova. Za sve navedeno potrebna je saradnja interdisciplinarnih timova stručnjaka i naučnika, sposobnih da razumeju strukturu i svojstva konstitutivnih elemenata objekata, otkriju mehanizme degradacije, „zaleče“ i produže životni vek materijala uz zadržavanje funkcionalnih i estetskih svojstava objekta.

Studija slučaja predstavljena u ovom radu imala je za cilj da se na osnovu rezultata nedestruktivnih ispitivanja in-situ i u laboratorijskim uslovima utvrdi stanje fasade srpske pravoslavne crkve u Sremskoj Kamenici posvećene rođenju Presvete Bogorodice, kako bi se adekvatno postavila metodologija za postupke čišćenja, konsolidacije i zaštite

pomenutog objekta. Ova istraživanja sprovedena su u periodu oktobar 2022 – mart 2023., nakon čega je u periodu januar 2024 – april 2025. kontinualno sprovođen i stručan nadzor izvođenja konzervatorskih radova. Ova faza je naročito bila značajna u pogledu praćenja doslednosti sprovođenja predložene metodologije čišćenja i zaštite objekta, ali i u slučaju da je bilo neophodno upotrebiti drugačije materijale ili sredstva od predloženih zbog objektivnih razloga. Takođe, stručni nadzor i prateća dokumentacija sa rezultatima naučnih metoda značajno je doprinela formiranju detaljne dokumentacije i transparentnosti samog procesa izvođenja radova. Zbog svega navedenog, može se konstatovati da detaljna dijagnostika stanja i odabir adekvatnih sredstava-tehnika za postupak konzervacije uz stručni nadzor, ne samo da garantuje kvalitet i dugotrajnost konzervatorskih intervencija, već i predstavlja osnovu za odgovoran i održiv pristup očuvanju kulturnog nasleđa.

2. MATERIJAL I METODE ISPITIVANJA

Sama istraživanja sprovedena su na Srpskoj Pravoslavnoj crkvi u Sremskoj Kamenici i to u četiri faze, tako da rezultati ispitivanja prve faze, odnosno sprovedena dijagnostika stanja, predstavljaju ulazne parametre za drugu i treću fazu, odnosno odabir kompatibilnih materijala za čišćenje, konsolidaciju i zaštitu, dok je četvrtu fazu predstavljao stručni nadzor izvođenja konzervatorskih radova, ali i ispitivanja stenskog materijala za nedostajuće elemente, materijala za fugovanje i zamenskih hemijskih sredstava u odnosu na ona koja su inicijalno preporučena. Ovo je naročito važno uzimajući u obzir činjenicu da se u tenderskoj dokumentaciji ne može navesti tačan naziv nekog hemijskog sredstva, već se taj zahtev prikazuje opisno, uz dodatak „ili slično“.

Rezultati su prikazani kroz četiri detaljna elaborata sa rezultatima ispitivanja i preporukama po zonama objekta.U okviru prvog elaborata pod nazivom **Istraživački rad mobilnom laboratorijom - „in situ“,** ispitani su: struktura i sastav originalnog materijala fasade i dat je predlog materijala za zamenu (stenski materijal i malteri), a takođe i mapiranje oštećenja i analiza mogućih uzroka degradacije materijala. U okviru drugog elaborata pod nazivom **Komparativna analiza sa predlogom čišćenja, desalinacije i konsolidacije** prikazani su rezultati proba u laboratoriji i na samom terenu. U okviru trećeg elaborata pod nazivom **Predlog mera za dugotrajnu zaštitu fasada** prikazani su rezultati proba u laboratoriji i na samom terenu, a u okviru poslednjeg elaboratapod nazivom **Naučno-stručni nadzor u izvođenju konzervatorskih radova,** dat je prikaz praćenja izvođenja konzervatorskih radova

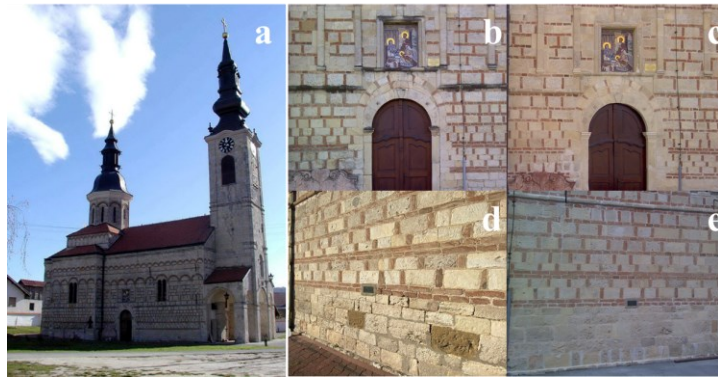
Prema saznanju tima Laboratorije za ispitivanje materijala u kulturnom nasleđu i u skladu sa dostupnim publikacijama, prikazana istraživanja predstavljaju jedinstven primer dobre prakse sprovedenih sveobuhvatnih ispitivanja naučnim metodama na objektu srpske pravoslavne crkve u Republici Srbiji pre bilo koje konzervatorske intervencije. U okviru istraživačkih radova uzorkovano je više od 30 uzoraka i snimljeno preko **70 zona in-situ ispitivanja (dijagnostika stanja) i 12 zona (nakon čišćenja, konsolidacije, desalinacije i zaštite**), nakon čega su sledili konzervatorski radovi[1].

[1] Radove je izvodila firma Ornament investinženjering, koja je u timu podizvođača za stručni deo konzervatorskih radova angažovala Natašu Petrović (master konzervator sa položenom licencom iz oblasti zaštite nepokretnih kulturnih dobara), preduzetnička radnja Nataša Petrović PR Motivi. Pored nadzora Laboratorije tokom izvođenja radova postojao je i konzervatorski nadzor ispred

2.1. Materijal

Kao što je i navedeno, sva sprovedena ispitivanja urađena su na fasadi SPC u Sremskoj Kamenici koja je posvećena Rođenju Presvete Bogorodice, Slika 1. Crkva je podignuta u periodu od 1737. do 1758. godine i pod zaštitom je države kao spomenik kulture u kategoriji nepokretnih kulturnih dobara od velikog značaja [1].

Crkva je posvećena Rođenju Presvete Bogorodice, predstavlja jednobrodnu građevinu, izgrađenu u duhu srpske tradicionalne arhitekture sa elementima baroka. Fasade crkve zidana je kamenom i opekom, a u 18. veku dozidan je kameni trospratni zvonik sa tremom i baroknom kapom.



Slika 1. SPC u Sremskoj Kamenici (a) [1]; detalj severne fasade pre (b) i nakon (c) konzervatorskih radova; detalj južne fasade pre (d) i nakon (e) konzervatorskih radova

2.2. Metode ispitivanja

Za gorepomenut eksperimentalni rad u svim fazama upotrebljene su sledeće metode ispitivanja i odgovarajući instrumenti:

- **Mikroskopska ispitivanja**: Stereo mikroskop, OMANO OMXTL/V7 Articulated Boom Microscope sa sopstvenim izvorom vidljive svetlosti i USB kamerom.;
- **Hemijsko- mineraloška ispitivanja:** Infracrvena spektrometrija sa Furijeovom transformacijom (FTIR), Alpha Bruker Optics (BRUKER OPTICS, Nemačka) i Rendgenska fluorescentna spektrometrija (XRF), ARTAX 200 μ-XRF spectrometer (BRUKER Nano, Nemačka);
- **Kolorimetrija sa spektrofotometrijom,** prenosni spektrofotometar CM-700D, Konica Minolta (Japan);
- **Analiza sadržaja rastvorljivih soli**: kvalitativno i semikvantitativno test trakama;
- **Mikrobiološka analiza**: obuhvatila je utvrđivanje prisustva i procenu broja: aerobnih mezofilnih bakterija, aerobnih oligotrofnih bakterija, amonifikatora, ureolitskih mikroorganizama, plesni i kvasaca, algalnih vrsta, lišajeva;
- **Fizičko-mehanička ispitivanja:** Živina porozimetrija i Ispitivanje sile otpora bušenja (Drilling resistance measurements, eng. *DRMS*) upotrebom uređaja DRMS SINT Technology (Italija). DRMS je urađen sa ciljem da se proceni efikasnost konsolidacije, od površine ka dubini materijala, u zoni u kojoj su primenjeni konsolidanti.;

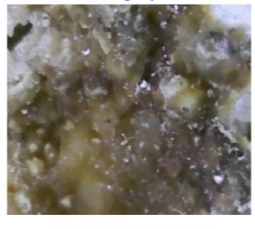
Gradskog zavoda za zaštitu spomenika kulture Grada Novog Sada, Miodrag Mišljenović, dipl. inž. građevinarstva

- **Termovizija**, termovizijska kamera T440bx (FLIR, SAD).

3. REZULTATI I DISKUSIJA

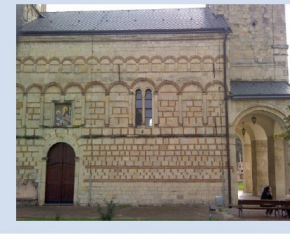
3.1. Dijagnostika stanja i predlog novih materijala za zamenu

Primenom in-situ metoda ispitivanja ceo objekat je mapiran i utvrđena su sledeća oštećenja: atmosferska zaprljanja, sulfatacija sa i bez formiranja crnih kora, postojanje akrilnih premaza, prisustvo rastvorljivih soli sulfata, hlorida i nitrata, neadekvatni materijali u prethodnim intervencijama (stenski materijali i malteri), oksalatne kore, mikrobiološka korozija, frakture, ispiranje materijala, erozija stenskog materijala i mreža pukotina, Slika 2. Na osnovu identifikovanih oštećenja birana su i sredstva i neophodni tretmani, poglavlja 3.2 i 3.3.

Zona	Fotografija/ Stereo-optička mikroskopija	Hemijski sastav / vrste hemijskih veza	Kolorimetrijska analiza	Rastvorljive soli (mg/l)
II.2.1	 Fotografija  Optička mikroskopija x10	Element / Conc./%: Al 5,910; Si 13,143; S 7,594; K 1,028; Ca 23,453; Ti 0,114; Fe 0,976; Sr 0,107	L 52.75 a 5.06 b20.59	Cl^-:0 NO_3: 250 SO_4: 0

Komentar: U ovoj zoni snimanja konstatuje se značajna sulfatacija uz pojavu crnih kora: Izrazito viok sadrž sumpora uz odsustvo rastvornih sulfata. U pitanju je nerastvorna površinska promena. Pored visokog sadržaja sumpora-S, identifikuje se i povećan sadržaj aluminijuma-Al i silicijuma-Si, koji su inkorporani u identifikovanu crnu koru.

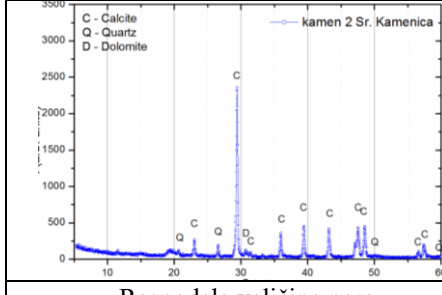
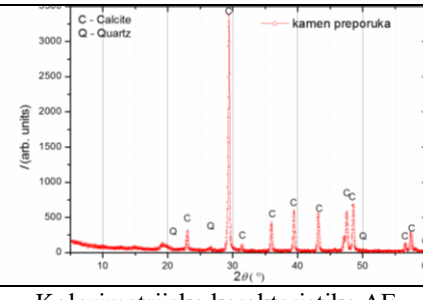
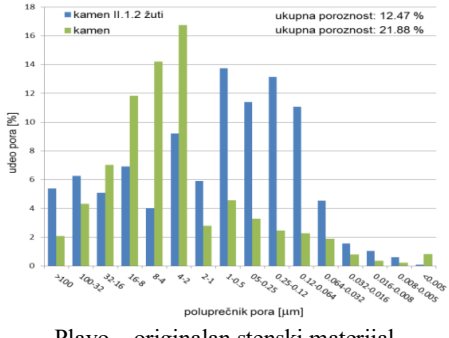




Komentar: Severna strana objekta odlikuje se većim zadržavanjem vlage u konstitutivnim elementima u odnosu na južni i istočnu stranu (naročito izraženo u temeljnoj zoni). Sa obzirom na prethodno prikazana mikrobiološka ispitivanja, identifikovana raspodela vlage je posledica značajne mikroobiološke korozije. Iako ova strana objekta ne zahteva poseban mikroobiološki tretman prisustne grupe mikroorganizama zadržavaju vlagu koja im je neophodna za njihov rast i razvoj. Zbog identifikovane raspodele vlage ovu stranu objekta bi trebalo čistiti parom (što slabiji pritisak uz poštovanje procedura datih u poglavlju koje sledi).

Slika 2. Primeri rezultata u jednoj od ukupno 70 ispitanih zona

U pogledu materijala za zamenu, na osnovu sprovedenih ispitivanja originalnih materijala urađen je odabir stenskog materijala i projektovani su malteri koji bi bili kompatibilni sa postojećim. Za sve laboratorijski projektovane sisteme upotrebljene su komercijalno dostupne komponente kako bi se preporučene recepture kasnije mogle bez poteškoća reprodukovati u realnim uslovima. Projektovano je ukupno 6 sistema, koji su sazrevali (početna karbonizacija) i bili ostareni (simulacija ekstremnih realnih uslova) u laboratorijskoj komori za starenje Binder Climate Chamber KBWF 240. Od projektovanih 6 sistema, 4 su se nakon starenja izdvojila kao potencijalno adekvatni (nepostojanje mikroprslina i adekvatne mehaničke karakteristike u početnom stepenu karbonizacije). Starenje projektovanih sistema sprovedeno je u periodu decembar 2022. – mart 2023. godine.(tabele 1 i 2). Prilikom projektovanja novih maltera, stepen kompatibilnosti procenjivan je u skladu sa indikatorima kompatibilnosti koji su uzimali u obzir ocenu sledećih kriterijuma: estetska kompatibilnost, slična priroda agregata i veziva, mehaničke karakteristike, ukupna poroznost i raspodela veličine pora, formiranje kontaktne zone između novog i starog maltera i udeo agregata i veziva (teži se da faktor rizika bude 0, da bi se neki sistem smatrao kompatibilnim).

<table>
<tr><th>Original stenski materijal</th><th>Predloženi stenski materijal</th></tr>
<tr><td colspan="2">Mineraloška analiza</td></tr>
<tr><td> 
</td><td> 
</td></tr>
<tr><td>Raspodela veličine pora</td><td>Kolorimetrijske karakteristike ΔE</td></tr>
<tr><td> 

Plavo – originalan stenski materijal
Zeleno- predloženi stenski materijal</td><td>
<table>
<tr><td>ΔL</td><td>-1,83</td></tr>
<tr><td>Δa</td><td>-0,91</td></tr>
<tr><td>Δb</td><td>0,19</td></tr>
<tr><td>ΔE</td><td>2,5</td></tr>
</table>
</td></tr>
</table>

Tabela 1. Komparativna analiza originalnog i predloženog zamenskog stenskog materijala

Na osnovu tabele 1 konstatuje se da je predloženi stenski materijal kompatibilan sa postojećim. U pitanju je krečnjak veoma sličnog mineraloškog sastava kao i originalni stenski materijali (XRD analiza). Takođe, razlika u boji (ukupna promena boje ΔE) je u

granicama vizuelno nedetektabilne razlike. Osim navedenog, u pogledu ukupne poroznosti i raspodele veličine pora, predloženi stenski materijal zadovoljava sve kriterijume kompatibilnosti. Naime, predloženi materijal poseduje veću ukupnu poroznost od originalnih, i veći udeo pora u intervalu krupnih pora poluprečnika 2-32 µm, kao i prisustvo sitnih pora. Ovakva raspodela pora omogućuje nesmetano kretanje vlage i rastvorljivih soli identifikovanih u i na fasadi, što će doprineti očuvanju originalnog stenskog materijala.

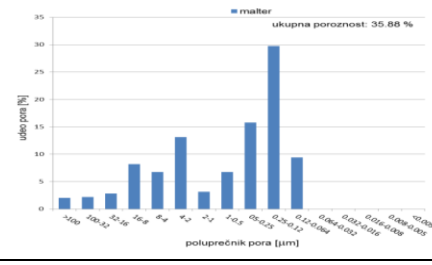
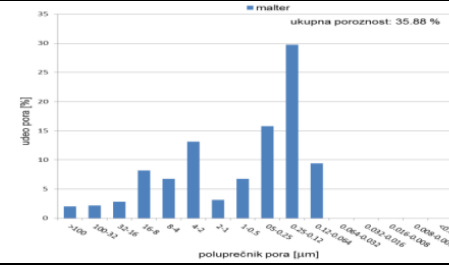
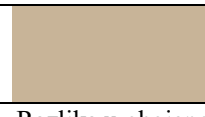
Originalni malter	Novo Projektovani malter
	
Ukupna poroznost 35,55 %	Ukupna poroznost 38,82 %
	
Razlika u obojenosti; ΔL -4,3 Δa -1,89 Δb 1,2 **ΔE = 4,84 <5**	
formiranje kontaktne zone, rizik 0 – kontaknta zona formirana u ranoj fazi karbonizacije	
Estetska kompatibilnost, rizik 0; Teksturalne karakteristike, rizik 0	
Hemijsko – mineraloška kompatibilnost, rizik 0 (isti odnos i priroda agregata I veziva)	

Tabela 2. Komparativna analiza originalnog i predloženog novo-projektovanog maltera

Na osnovu komparativne analize originalnog maltera za fugovanje i projektovanog maltera, konstatuje se da je faktor rizika nekompatibilnosti 0 odnosno ne postoji rizik nekompatibilnosti dva ispitana materijala – nakon veštačkog starenja novo – projektovanog maltera. Po pitanju estetske kompatibilnosti i vizuelnog izgleda projektovanog sistema, važno je istaći da će u realnim uslovima eksploatacije doći do povećanja L koordinate, odnosno povećanja svetline. U pogledu formiranja kontaktne zone između postojećeg maltera i novo - projektovanog maltera faktor rizika je 0, odnosno kontakt je dobro formiran i nisu identifikovane mikropukotine (mikroskopska ispitivanja). U pogledu pozornosti, ukupna poroznost je veća kod novo–projektovanog maltera u odnosu na originalni malter, što je neophodno za kretanje vlage i identifikovanih rastvorljivih soli.

3.2. Predlog materijala-tehnika za čišćenje, desalinaciju i konsolidaciju

U okviru proba čišćenja i konsolidacije ispitane su komercijalno dostupne tehnike i sredstva za čišćenje i konsolidaciju stenskih materijala. Poseban aspekt čišćenja predstavljao je razvoj desalinacionog sistema na bazi bakterijske kulture denitrifikatora, kako bi došlo do potpune eliminacije nitrata i njihovog prevođenja u azot [4].

Prvo je u laboratorijskim uslovima urađeno ispitivanje sredstava za čišćenje, desalinaciju i konsolidaciju. Nakon izbora sredstava koja su zadovoljila postavljene kriterijume u laboratorijskim uslovima, sredstva/tehnike su ispitivani na terenu u realnim uslovima.

Ispitivanja u laboratorijskim uslovima urađena su na krečnjačkom stenskom materijalu koji je u prvoj fazi istraživanja odabran kao kompatibilan zamenski materijal. Prvi eliminacioni kriterijum je da primenjeni tretman tehnika čišćenja ne izaziva promenu boje na čistom kamenu i ne menja njegove mehaničke karakteristike. Terenske probe su takođe podrazumevale in-situ ispitivanja nakon tretmana (slika 3). Rezultat ovih ispitivanja je spisak preporuka za čišćenje, desalinaciju i konsolidaciju, po zonama fasade.



levo-arte mundit; desno-clean galena

Sprovedeni tretman čišćenja

- Nakon pranja parom, posle 48 h, primenjen je sapun koji je odmah nakon primene uklonjen takođe parom;
- Nakon pranja parom i sapunom jedan deo zone 10 tretiran artemunditom: na jedan deo zone urađena je dvostepena aplikacija sredstva Arte mundit (2X48h), dok je drugi deo tretiran sredstvom clean galena (jedno-stepeno 48h)
- Ispitivanja su urađena nedelju dana nakon uklanjanja primenjenih sredstava

Rezultati



nakon aplikacije arte munditom



nakon aplikacije clean gaalenom

Mikroskopski snimci, uvećanje X40

Element	Arte Mundit	Clean galena
Al	0,87	0,27
Si	1,138	0,038
P	0,044	0,734
S	0,06	0,106
K	0,37	0,639
Ca	45,074	49,321
Fe	0,302	0,41
Sr	0,202	0,100

Komentar: Postupak čišćenja je uspešno sproveden uz izraženo uklanjanje crnih kora uz napomenu da sredstvo Clean galena nije pogodno za stenski material kod kojeg je odmakla faza čestičnog odvajanja jer dodatno narušava stanje površine (mikroskopski snimak). U pogledu raspodele elemenata i kolorimetrijskih karakteristika, rezultati odgovaraju čistom originalnom stenskom materijalu.

Slika 3. Primer proba čišćenja na terenu – Uklanjanje crnih kora, atmosferskog zaprljanja i teško rastvornih oksalata (uz prisustvo organske materije)

3.3. Predlog materijala za dugotrajnu zaštitu

Sredstva za zaštitu fasade odabrana su po istoj metodologiji koja je prikazana u poglavlju 3.2. Za primenu i ispitivanje na terenu odabrana su sredstva za koja je na osnovu rezultata dobijenih u laboratoriji utvrđeno da odgovaraju kriterijumu kompatibilnosti sa supstratom. Ispitivanja efikasnosti odabranih sredstava na terenu su urađena na prethodno očišćenim zonama. Zahtev koji su odabrana sredstva trebalo da zadovolje je funkcija hidrofobizacije i dugotrajne zaštite sa efektom samočišćenja fasade (slika 4). Ispitivanje efekta samočišćenja urađeno je za **fotokatalitički materijal NANOFAS** koji se na osnovu relevantnih naučnih izvora, izdvojio kao potencijalno kompatibilan sa supstratom [2]. Aplikacija „samočistećih" materijala u zaštiti kulturnog nasleđa pokazala se kao adekvatan pristup dugotrajnoj zaštiti fasada objekata u poslednjim decenijama. Materijali sa efektima samočišćenja se zasnivaju na fotokatalitički aktivnim komponentama koje se usled delovanja sunčevog zračenja aktiviraju i razgrađuju atmosferske polutante do neorganskih (neštetnih) jedinjenja. Istovremeno sa razgradnjom polutanata, usled dejstva sunčevog zračenja, površina na koju je materijal nanet postaje hidrofilna, što omogućava da se razgrađene nečistoće spiraju usled dejstva atmosferskih padavina. Funkcionalne karakteristike predloženih zaštitnih sredstava proverene su merenjem ugla kvašenja.



Slika 4. Primer provere funkcionalnih karakteristika sredstva za hidrofobizaciju i samočišćenje

4. ZAKLJUČAK

Na osnovu prikazane metodologije i integrisanog pristupa koji je obuhvatio laboratorijska i in-situ ispitivanja, uspešno su razvijena i primenjena sredstva i tehnike čišćenja, konsolidacije, desalinacije i zaštite fasade srpske pravoslavne crkve posvećene Presvetoj Bogorodici u Sremskoj Kamenici. Detaljna dijagnostika (preko 30 uzoraka i 70 zona in-situ ispitivanja) omogućila je preciznu identifikaciju stepena, vrste i uzroka degradacije konstitutivnih elemenata objekta, što je predstavljalo osnovu za odabir optimalnih tehnika i sredstava za konzervatorske radove.

Na osnovu analize stanja, definisani su svi koraci za izvođenje konzervatorskih radova. Sve preporučene metode i sredstva testirani su u laboratorijskim uslovima, u kontrolisanim uslovima na samom objektu, a njihova efikasnost i kompatibilnost potvrđene nakon primene kroz sistematski nadzor tretiranih površina.

Prikazana istraživanja i celokupan pristup zaštiti kulturnog nasleđa od suštinskog su značaja, jer obezbeđuju naučno utemeljene, održive i reverzibilne metode koje

omogućavaju dugoročnu očuvanost autentičnosti materijala i arhitektonske vrednosti objekata, čime se aktivno doprinosi očuvanju kulturnog nasleđa.

Zahvalnica

Ministarstvu nauke, tehnološkog razvoja i inovacija Republike Srbije, projekat No. 451-03-136/2025-03/200134, 451-03-137/2025-03/200134. Autori se zahvaljuju SPC, eparhiji Sremskoj, kompaniji Ornament Investinženjering, Zavodu za zaštitu spomenika kulture grada Novog Sada i Nataši Petrović na izuzetnoj saradnji i poverenju pri dijagnostici stanja i izvođenju radova u toku in-situ analiza i nadzora konzervatorskih radova na SPC rođenja Presvete Bogorodice u Sremskim Karlovcima.

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA
UDRUŽENJE SAVREMENE INDUSTRIJE GLINENIH PROIZVODA SRBIJE

XXIX KONGRES DIMK I X KONGRES SIGP
SA MEĐUNARODNIM SIMPOZIJUMOM
O ISTRAŽIVANJIMA I PRIMENI SAVREMENIH DOSTIGNUĆA U
GRAĐEVINARSTVU U OBLASTI MATERIJALA I KONSTRUKCIJA
Sokobanja, 21 - 23. maj 2025.

Goran Mladenović[1]

https://doi.org/29DIMK.479M
Pregledni rad

PRIMENA POROZNIH ASFALTNIH HABAJUĆIH SLOJEVA

Rezime: *U radu su prikazane osnovne karakteristike poroznih asfaltnih mešavina. Posebno su obrađene prednosti njihove primene koje se ogledaju u povoljnim uticajima vezanim za bezbednost odvijanja saobraćaja, uslove vožnje i smanjenje štetnog uticaja na okolinu, prevashodno kroz poboljšanje kvaliteta vode i smanjenje saobraćajne buke. Objašnjeni su koncepti projektovanja i ispitivanja ovih mešavina i prikazano je projektovanje asfaltne mešavine od poroznog asfalta PA 11 za potrebe izrade habajućeg sloja kolovozne konstrukcije u infleksionim zonama autoputeva. Za mešavinu je korišćen kameni agregat eruptivnog porekla Mrčići, Divčibare, kameno brašno Bekament, polimer modifikovani bitumen PmB 45/80-65 i vlakna Viatop. Za poboljšanje veze bitumena i agregata korišćeni su hidratisani kreč i hemijski aditiv Iterlene. Ispitivanjem mešavina dobijeni su zadovoljavajući rezultati i u pogledu dreniranja veziva i u pogledu gubitka materijala.*
Ključne reči: *Porozni asfalt, habajući sloj, kolovozne konstrukcije, infleksione zone, autoputevi, gubitak materijala, Cantabro*

USE OF POROUS ASPHALT WEARING COURSES

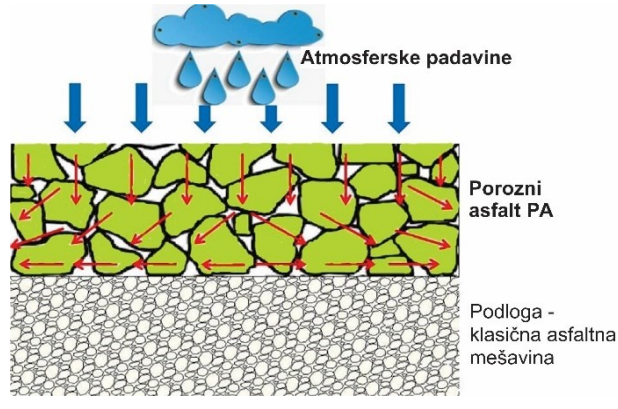
Summary: *The paper presents the basic characteristics of porous asphalt mixtures. The advantages of their application are particularly discussed, which are reflected in the favorable impacts related to road safety, improvement of driving conditions and the reduction of harmful impacts on the environment, primarily through improving water quality and reducing traffic noise. The concepts of designing and testing these mixtures are explained and the design of an asphalt mixture made of porous asphalt PA 11 for wearing course of the pavement structure in the inflection zones of motorways is presented. The mixture uses eruptive stone aggregate Mrčići, Divčibare, stone filler Bekament, polymer modified bitumen PmB 45/80-65 and Viatop fibers. Hydrated lime and the chemical additive Iterlene were used to improve the bond between bitumen and aggregate. The testing of the mixtures yielded satisfactory results both in terms of binder drainage and material loss.*
Keywords: *Porous asphalt mixtures, wearing course, pavement, inflection zones, motorways, material loss, Cantabro*.

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1. UVOD

Primena poroznih asfaltnih mešavina je počela u razvijenim zemljama u sredinom XX veka, imajući u vidu niz prednosti koje one imaju u pogledu povećanja bezbednosti odvijanja saobraćaja, poboljšanja uslova vožnje, kao i povoljnog uticaja na okolinu u pogledu smanjenja buke generisane na kontaktu između pneumatika i kolovoza.

Porozne asfaltne mešavine predstavljaju grupu diskontinualnih mešavina koje se ugrađuju kao habajući slojevi kolovoznih konstrukcija i karakterišu se visokim sadržajem krupnozrnog agregata koji čini kameni skelet, i takođe visokim sadržajem povezanih šupljina ispunjenih vazduhom koji se generalno kreće od 15% do 30%. Visok sadržaj šupljina rezultuje visokom vodopropustljivošću ovih mešavina, jer one omogućavaju prodiranje vode sa površine kolovoza i dreniranje u poprečnom pravcu po sloju vodonepropustne asfaltne podloge, do ivica kolovoza gde se ova voda prihvata elementima za odvodnjavanje (slika 1). Pored toga ove mešavine imaju i srazmerno visok sadržaj bitumena u poređenju sa odgovarajućim klasičnim asfalt-betonskim mešavinama.



Slika 1. Koncept odvodnjavanja poroznih asfaltnih slojeva [1]

U pogledu terminologije, postoji čitav niz različitih naziva koji se koriste za ovaj tip asfaltnih mešavina u različitim delovima sveta, poput: vodopropusni sloj sa visokim trenjem (permeable friction course - PFC), asfaltna mešavina sa "otvorenim" granulometrijskim sastavom (open-graded asphalt - OGA), asfaltni sloj sa "otvorenim" granulometrijskim sastavom i visokim trenjem (open-graded friction course - OGFC), drenažni asfalt (DA) i drugi. Svi ovi nazivi ukazuju na određene pozitivne karakteristike i povoljne efekte ovih mešavina koje će u širem smislu u ovom radu biti razmatrane kao porozni asfalti, u skladu sa terminologijom koja je usvojena u Evropskim normama [2].

U okviru projekta RESAFE - Road and environmental safety related to water-pavement interactions, koji finansira Fond za nauku Republike Srbije, između ostalog vrši se istraživanje mogućnosti primene habajućeg sloja od poroznog asfalta u infleksionim zonama autoputeva u kojima je ugroženo površinsko odvodnjavanje zbog malih rezultujućih nagiba površine kolovoza. Cilj rada je da sumira iskustva u projektovanju mešavine od poroznog asfalta koja je razvijena u okviru ovog projekta.

2. EKSPLOATACIONE KARAKTERISTIKE POROZNIH ASFALTNIH SLOJEVA

Povoljni efekti koje imaju porozni asfaltni habajući slojevi se mogu grupisati u tri kategorije: efekti u pogledu bezbednosti odvijanja saobraćaja, uslova vožnje i u pogledu uticaja na okolinu.

U pogledu bezbednosti odvijanja saobraćaja, primena ovih slojeva dovodi do:

- smanjenja akvaplaning efekta, čak i u uslovima bliskim zasićenju [3]
- smanjenja količine vodenih čestica koje se raspršuju pneumaticima (engl. "splash and spray"), čime se poboljšava vidljivost 2.5 do 3 puta u odnosu na klasične asfalt-betonske mešavine [4]
- smanjenja odsjaja, čime se omogućava bolja vidljivost horizontalne signalizacije u noćnim uslovima [5] i
- povećanja trenja na kontaktu pneumatika i kolovoza, pogotovo u uslovima vlažnih kolovoza, odnosno pri padavinama [6].

Svi ovi efekti dovode do smanjenja broja i posledica saobraćajnih nesreća, pogotovu pri atmosferskim padavinama [7,8].

U pogledu uslova vožnje, porozni zastori:

- omogućavaju veće brzine, pa samim tim i veći kapacitet saobraćajnica, odnosno manja zagušenja koja su karakteristična pogotovo za gradske saobraćajnice u vlažnim uslovima [5,9] i
- imaju istu ili bolju ravnost i obezbeđuju bolji kvalitet vožnje (engl. ride quality) u poređenju sa drugim tanko-slojnim asfaltnim mešavinama za habajuće slojeve [10], što ima povoljne efekte u pogledu smanjenja potrošnje goriva.

Konačno, u pogledu uticaja na okolinu ovi zastori omogućavaju prečišćavanje vode koja otiče sa kolovoza i značajno smanjenje buke. Ispitivanje uzoraka vode je pokazalo značajno smanjenje koncentracije (i do 90%) suspendovanih čvrstih materija, azota, ukupnog fosfora, cinka, olova i bakra u poređenju sa oticajima sa klasičnih asfalt-betonskih zastora [11]. Nivo buke pri prolasku vozila se smanjuje za 3 dB do 6 dB u poređenju sa klasičnim asfaltnim mešavinama i od 5.5 dB do 10.5 dB u odnosu na betonske kolovoze [12].

Smanjeni uticaj na okolinu je primarni razlog zašto se porozni asfaltni zastori koriste u velikom broju Evropskih zemalja, dok su koristi vezane za povećanje bezbednosti osnovni razlog zašto se ovi slojevi primenjuju u Sjedinjenim Američkim Državama [13]. Ove karakteristike poroznih asfaltnih mešavina imaju sve veći značaj, imajući u vidu konstantno povećanje saobraćajnog opterećenja na putevima sa jedne strane, i uticaj klimatskih promena, pri čemu su posebno značajni sve češći ekstremni događaji sa velikim količinama padavina u kratkim vremenskim periodima, što ugrožava bezbednost odvijanja saobraćaja.

Pored ovih vrednosti, primena poroznih asfaltnih mešavina je vezana i sa određenim problemima. Pre svega, oni su teži za održavanje, jer nije moguće lako izvršiti popravku većih površina bez značajnog uticaja na njihovu vodopropustljivost [9]. Pored toga, ove kolovoze je značajno teže održavati u zimskim uslovima. Oni se zbog svoje strukture zamrzavaju brže u odnosu na klasične mešavine i duže ostaju zamrznuti. Pri tome nije dopuštena upotreba soli ili rizle zbog mogućeg zapušavanja šupljina i posledičnog smanjenja funkcionalnosti, a za zimsko održavanje se koristi veća količina hemikalija [14].

Porozni asfaltni slojevi imaju veću cenu o odnosu na klasične mešavine, a pri tome treba imati u vidu da je njihov životni vek kraći i da se kreće tipično od 6 do 12 godina.

Drugi izazovi vezani za primenu poroznih asfalta se odnose na njihovu trajnost i funkcionalnost (mogućnost da održe svoje povoljne karakteristike, pre svega u pogledu poroznosti, tokom celokupnog životnog veka). Osnovni vid propadanja ovih zastora je odnošenje materijala (engl. raveling) koje predstavlja gubitak agregata iz habajućeg sloja i nastaje usled abrazionog efekta saobraćajnog opterećenja. S druge strane funkcionalnost ovih zastora opada kroz vreme zbog zapunjenja šupljina raznim otpadnim materijalima, što dovodi do smanjenja vodopropustljivosti i umanjuje efikasnost smanjenja buke.

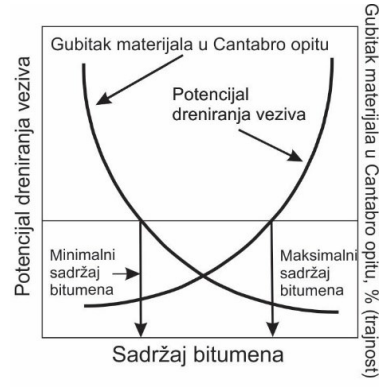
3. MATERIJALI I MEHANIČKO PONAŠANJE POROZNIH ASFALTA

Učešće krupnozrnog agregata u poroznim asfaltnim mešavinama je relativno veliko (preko 80%) što dovodi do velikog broja kontakata između zrna agregata. Zbog toga može doći do loma zrna prilikom ugradnje/valjanja ili tokom eksploatacije, što zahteva da se primenjuje agregat visokog kvaliteta koji ima dobru otpornost na drobljenje, što je tipično agregat eruptivnog porekla. Rezultujući kameni skelet daje tipično dobru otpornost na trajnu deformaciju ovih mešavina. Pored toga, treba obratiti pažnju na druge karakteristike agregata, poput habanja, oblika zrna, čvrstoće, čistoće i upijanja vode, jer sve one utiču na ponašanje poroznih mešavina u eksploataciji [13].

Zbog velikog sadržaja šupljina u poroznim asfaltnim mešavinama, veća površina bitumena je neposredno izložena vazduhu ili vodi, u odnosu na klasične asfaltne mešavine. Pored toga, voda može narušiti kontakt između agregata i veziva ili izazvati lom u okviru samog sloja veziva. Zbog toga se generalno primenjuju modifikovani bitumeni, koji imaju veću otpornost na starenje [15], a ostvaruju i bolju otpornost na narušavanje kontakta sa agregatom. Aditivi za poboljšanje veze između bitumena i agregata, poput hidratisanog kreča, ili hemijskih aditiva, se takođe koriste i oni generalno poboljšavaju otpornost ovih mešavina na dejstvo vode.

Granulometrijski sastav agregata treba da bude takav da se ostvare velike šupljine u mineralnom materijalu. Trenutno se metode projektovanja poroznih asfaltnih mešavina primarno zasnivaju na određivanju zapreminske strukture u smislu poroznosti [1]. Pored toga, određivanje sadržaja veziva i vlakana se bazira na Šelenbergovom opitu dreniranja veziva (koji je dat u standardu SRPS EN 12697-18), dok se Cantabro test (dat u SRPS EN 12697-17), koji je razvijen 90-tih godina na Univerzitetu u Kantabriji, koristi za utvrđivanje osetljivosti mešavine na habanje, i potencijalno kao indikator mogućnosti odnošenja materijala, što predstavlja osnovno oštećenje koje utiče na trajnost poroznih slojeva.

Koncept projektovanja poroznih asfaltnih mešavina i određivanja optimalnog sadržaja bitumena (slika 2) se svodi na utvrđivanje kompromisa između rezultata ova dva opita.



Slika 2. Koncept projektovanja poroznih asfaltnih mešavina [9]

4. PREGLED SPECIFIKACIJA ZA POROZNI ASFALT

Porozne asfaltne mešavine predstavljaju relativno novi tip mešavina koji nema široku primenu u Srbiji, imajući u vidu njihovu trajnost, koja je značajno kraća od klasičnih asfaltnih mešavina poput asfalt-betona i skeletnih mastiks asfalta, kao i visoke troškove održavanja, kako bi se tokom životnog veka održala njihova funkcionalnost. Iz tog razloga postojeće tehničke specifikacije nisu u potpunosti razvijene, tako da se u Srbiji porozni asfalti pominju samo u tehničkim uslovima javnog preduzeća (JP) Putevi Srbije, čija je najnovija verzija izdata 2018. godine [16]. Prema ovim specifikacijama, postoje mešavine sa maksimalnom veličinom zrna od 8 mm i 11 mm, i za svaku od njih data je granulacija za drenažni asfalt standardizovanog sastava i granulacija za drenažni asfalt otvorenog sastava „og".

Pored ovih tehničkih uslova, analizom su obuhvaćeni i zahtevi za porozni asfalt u Tehničkom propisu u Hrvatskoj [17], koji suštinski predstavlja nacionalni aneks za seriju standarda EN 13108 koji se odnose na različite tipove asfaltnih mešavina, uključujući i porozni asfalt. Takođe je analiziran i Slovenački standard SIST 1038-7 [18] koji predstavlja nacionalni aneks za porozne asfaltne mešavine, i koji daje uslove kvaliteta i za mešavinu PA16 pored mešavina PA8 i PA11.

U tabeli 1 dat je pregled specifikacija za granulometrijski sastav asfaltnih mešavina PA11.

Otvor sita (mm)	TU Putevi Srbije		Hrvatski Tehnički propis	SIST 1038-7
	PA 11	PA 11 og	PA 11	PA 11
	Prolaz kroz sito (% m/m)			
0.063	2 – 5	0 – 5	2 – 5	2 – 5
0.25		0 – 7		3 – 8
0.71	5 – 12	0 – 10		
1.0			6 – 13	

2.0	7 - 15	4 – 14	8 – 18	7 – 15
4.0	12 - 20	15 – 37		12 - 20
8.0	18 - 28	60 – 80	20 – 35	18 - 28
11.2	90 – 100	85 – 100	90 – 100	90 – 100
16.0	100	100	100	100

Tabela 1. Granulometrijski pojasevi za mešavine PA 11 prema specifikacijama u regionu

Zahtevi u pogledu zapreminske strukture mešavina na Maršalovim uzorcima i sadržaja bitumena prikazani su u tabeli 2. Tabela 2 takođe prikazuje i zahteve u pogledu gubitka čestica u Cantabro opitu prema SRPS EN 12697-17 i u pogledu dreniranja veziva prema SRPS EN 12697-18. Ovo su dve kritične osobine poroznih asfaltnih mešavina, jer se sa jedne strane mora odrediti dovoljan sadržaj vlakana da ne dođe do curenja veziva, jer je specifična površina mineralnog materijala mala s obzirom da dominira krupnozrni agregat, a sa druge strane, zbog male količine mastiksa, kritičan je gubitak zrna u asfaltnoj mešavini koja je izložena habanju.

Hrvatski Tehnički propis zahteva da se za teško saobraćajno opterećenje koristi polimer modifikovani bitumen.

Parametar	TU Putevi Srbije		Hrvatski Tehnički propis	SIST 1038-7
	PA 11	PA 11 og	PA 11	PA 11
Minimalni sadržaj bitumena (% m/m)	5.0			5.0
Sadržaj šupljina ispunjenih vazduhom (% v/v)	16 - 26		18 - 28	16 - 26
Gubitak materijala prema SRPS EN 12697-17 (% m/m)	30		30	30
Dreniranje veziva prema SRPS EN 12697-18 (% m/m)	0			0
Minimalni odnos čvrstoća na indirektno zatezanje prema SRPS EN 12697-12 (%)			80	

Tabela 2. Zahtevane karakteristike poroznih asfaltnih mešavina PA11

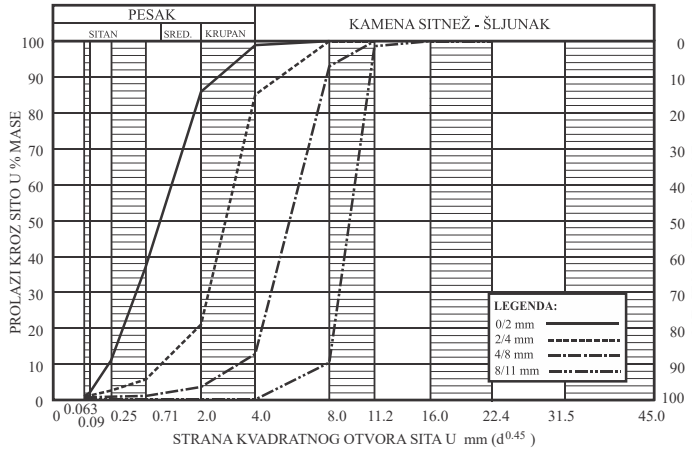
U pogledu vodopropustljivosti, u standardu SRPS EN 12697-19:2020 je data napomena da se vertikalna vodopropustljivost uzoraka poroznog asfalta kreće između 0.5 x 10^{-3}m/s i 3.5 x 10^{-3}m/s.

5. KOMPONENTALNI MATERIJALI

Asfaltna mešavina je projektovana od frakcija drobljenog kamenog agregata Mrčići, Divčibare i kamenog brašna Bekament, Aranđelovac. Granulometrijski sastav frakcija mineralnog materijala je prikazan na slici 3, a karakteristike kamenog brašna u tabeli 3.

Inicijalno, ispitivana je mogućnost primene tri tipa bitumena, putnog bitumena B50/70 i polimer modifikovanih bitumena PmB 25/55-65 (Beograd-put) i PmB 45/80-65 (NIS-Pančevo), pri čemu je PmB 45/80-65 izabran kao konačno rešenje za projektovanje asfaltne mešavine. Karakteristike primenjenog bitumena prikazane su u tabeli 4.

Za stabilizaciju mešavine korišćena su vlakna Viatop. Za poboljšanje veze između agregata i bitumena korišćena su dva aditiva: kreč u procentu od 2% kao zamena za isti procenat kamenog brašna, i komercijalni proizvod Iterlene firme Iterchimica, koji je primenjen u procentu od 0.3%.



Slika 3. Granulometrijski sastav frakcija mineralnog materijala

Karakteristika		Metoda	Vrednost
Granulometrijski sastav (% prolaza na situ)	Sito (mm)	SRPS B.B8.105	
	0.063		78.2
	0.09		86.6
	0.25		99.6
	0.71		100.0
Udeo šupljina punila u suvosabijenom stanju (%)		SRPS EN 1097-4	30.3
Zapreminska masa punila bez šupljina (kg/m^3)		SRPS EN 1097-7	2732
Povećanje tačke razmekšavanja po metodi prstena i kuglice (^{o}C)		SRPS EN 13179-1	16.8
Metilen plavo MB_F (g/kg)		SRPS EN 933-9	1.67

Tabela 3. Karakteristike kamenog brašna

Karakteristika		Metoda	Vrednost
Penetracija (25°C, 0.1 mm)		SRPS EN 1426	50.4
Tačka razmekšavanja po metodi prstena i kuglice (°C)		SRPS EN 1427	67.5
Indeks penetracije		SRPS EN 12591	2.43
Specifična masa (kg/m^3)		SRPS EN 15326	999
Rotacioni viskozitet	(135 °C, Pa.s)	SRPS EN 13302	1.366
	(165 °C, Pa.s)		0.383

Tabela 4. Karakteristike polimer modifikovanog bitumena PmB 45/80-65

6. METODOLOGIJA PROJEKTOVANJA MEŠAVINE

Projektovanje asfaltne mešavine je urađeno u dve faze. U prvoj fazi projektovane su mešavine bez aditiva za poboljšanje veze između agregata i bitumena, sa primarnim ciljem da se utvrdi optimalni sadržaj veziva i vlakana i da se ispita gubitak materijala. U drugoj fazi su ispitivane mešavine sa aditivom pri utvrđenom optimalnom sadržaju veziva, sa ciljem da se utvrdi otpornost na habanje (gubitak materijala) i da se ispita vertikalna vodopropustljivost.

Optimalni sadržaj bitumena određen je primenom Maršalove metode u skladu sa standardom SRPS EN 12697-34. Maršalovi uzorci su zbijani na temperaturi od 165 °C sa energijom zbijanja od 2 x 50 udaraca sa svake strane. Sadržaj bitumena se kretao od 5.0 do 6.0 %, a sadržaj vlakana od 0.3% do 0.7%.

Drenaža veziva iz asfaltne mešavine je određena u skladu sa standardom SRPS EN 12697-18.

Gubitak materijala je određen u skladu sa standardom SRPS EN 12697-17.

Obavijenost zrna je ispitivana u prvoj fazi ispitivanja mešavina koje nisu sadržale aditive za poboljšanje veze između agregata i bitumena. Obavijenost je ispitivana u skladu sa standardom SRPS EN 12697-11, tačka 5 – Rolling Bottle Method. Ispitivanje je obavljeno u Centralnoj laboratoriji TPA u Čačku.

Vodopropustljivost u vertikalnom pravcu je ispitivana prema standardu SRPS EN 12697-19, tačka 5.

7. KARAKTERISTIKE PROJEKTOVANE MEŠAVINE

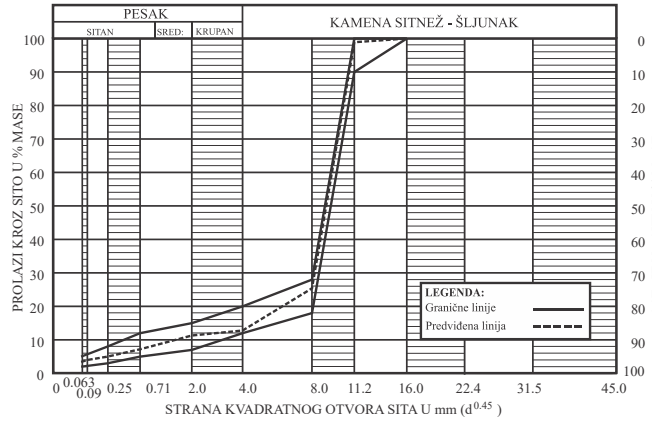
U ovom poglavlju dati su rezultati ispitivanja projektovane mešavine u prvoj i drugoj fazi ispitivanja.

Učešće frakcija mineralnog materijala u mešavini prikazano je u tabeli 5. Projektovana mešavina je diskontinualna, s obzirom da je izostavljena frakcija 2/4 mm, i karakteriše je izuzetno veliki sadržaj krupnozrnog agregata.

Frakcija	Učešće u mešavini (%)
Kameno brašno	4.0
0/2 mm	8.0
4/8 mm	5.0
8/11 mm	83.0

Tabela 5. Učešće frakcija kamenog agregata u mineralnoj mešavini

Na slici 4 je prikazan granulometrijski sastav projektovane asfaltne mešavine za PA 11.



Slika 4. Granulometrijski sastav projektovane mineralne mešavine

U prvoj fazi istraživanja ispitivana je zapreminska struktura asfaltne mešavine sa sadržajem veziva od 5.0%, 5.5% i 6.0% i sa sadržajem vlakana od 0.3% do 0.7%. Šupljine u asfaltnoj mešavini su se kretale od 19.1% do 22.6%, što je odgovarajući raspon za porozne mešavine. Sa minimalnim sadržajem vlakana od 0.3% dobijeno je zadovoljavajuće dreniranje veziva od 0.1% i rezultati se nisu znatno promenili za veće sadržaje vlakana, tako da je na kraju prve faze usvojen sadržaj bitumena od 6.0% i sadržaj vlakana od 0.3%. Međutim, vrednosti gubitka materijala u Cantabro opitu su bile veće od 30%, a mali stepen obavijenosti zrna agregata nakon ispitivanja sa opitom kotrljajućih flašica prema SRPS EN 12697-11 je ukazivao da postoji problem veze agregata i bitumena.

Stoga su u drugoj fazi primenjeni i aditivi za poboljšanje veze agregata i bitumena. Rezultati ispitivanja asfaltnih mešavine sa različitim aditivima su prikazani u tabeli 6.

Parametar	PA11 PmB 45/80-65 + 2% hidratisani kreč		PA11 PmB 45/80-65 + 0.3% Iterlene	
Tip uzorka	Maršal	Ploče 20x20 cm	Maršal	Ploče 20x20 cm
Zapreminska masa Z_m (kg/m^3)	2017	2093	2049	2070
Maksimalna zapreminska masa Z_{max} (kg/m^3)	2507		2518	
Sadržaj šupljina ispunjenih vazduhom (% v/v)	19.6	16.5	18.6	17.8
Gubitak materijala prema SRPS EN 12697-17 (% m/m)	15.3		9.5	
Dreniranje veziva prema SRPS EN 12697-18 (% m/m)	0.1		0.1	

Tabela 6. Karakteristike asfaltnih mešavina PA11 sa dva različita aditiva

Vertikalna vodopropustljivost mešavine prema SRPS EN 12697-19 se kretala između od 1.9 x 10^{-3} m/s i 2.06 x 10^{-3} m/s, što je u granicama očekivanih vrednosti za porozne asfalte.

8. ZAKLJUČAK

Projektovana porozna asfaltna mešavina PA 11 zadovoljava zahteve specifikacija u pogledu granulometrijskog sastava, zapreminske strukture, dreniranja veziva, gubitka materijala i vertikalne vodopropustljivosti. U daljem radu biće dodatno ispitane karakteristike vodopropustljivosti i u horizontalnom pravcu na laboratorijskim uzorcima, a u okviru projekta RESAFE planirana je i izrada opitnog poligona na kome bi se izvršila merenja efekata vodopropustljivosti ove mešavine.

Zahvalnost

Prikazano istraživanje predstavlja deo naučnog projekta "Road and Environmental Safety related to water – pavement interactions - RESAFE" (Bezbednost puta i ekološka sigurnost vezane za odvođenje kišnog oticaja), broj 7051, iz programa PRISMA, finansiranog od strane Fonda za nauku Republike Srbije, u periodu 2024-2026 godina.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
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INFLUENCE OF VITREOUS WASTE ENAMEL ON THE PROPERTIES OF CONCRETE AND CONCRETE PAVING BLOCKS

Summary: *The study explores the use of waste vitreous enamel as a partial cement replacement in concrete. It was found that the enamel exhibits strong pozzolanic activity, belongs to class 15 of pozzolanic materials, and meets the criteria related to the activity index test.. Concrete with up to 30% enamel replacement maintained acceptable compressive strength and improved workability. Although mechanical properties slightly decreased, durability remained unaffected. Concrete paving blocks incorporating 15% waste enamel also fulfilled quality requirements, with only a slight decrease in splitting tensile strength. The findings support the safe and practical reuse of vitreous enamel in construction materials, offering environmental and economic benefits. Overall, the study demonstrates that waste vitreous enamel can be effectively utilized in concrete without compromising structural integrity or durability.*
Keywords: *waste vitreous enamel, pozzolanic material, concrete, concrete paving blocks*

UTICAJ STAKLASTOG OTPADNOG EMAJLA NA KARAKTERISTIKE BETONA I BETONSKIH BLOKOVA ZA POPLOČAVANJE

Rezime: *Studija istražuje upotrebu otpadnog staklastog emajla kao delimične zamene cementa u betonu. Utvrđeno je da emajl ima jaku pucolansku aktivnost, pripada klasi 15 pucolanskih materijala i ispunjava kriterijume vezane za test indeksa aktivnosti. Betoni u kojema je zamena cementa emajlom izvršena do 30% zadržavaju prihvatljivu čvrstoću pri pritisku i imaju poboljšanu obradljivost. Iako su mehanička svojstva blago smanjena, trajnost betona nije bila narušena. Betonske ploče za popločavanje sa 15% otpadnog emajla takođe su zadovoljile uslove kvaliteta, uz samo blago smanjenje čvrstoće na zatezanje cepanjem. Rezultati ukazuju na sigurnu i praktičnu upotrebu otpadnog staklastog emajla u građevinskim materijalima, nudeći ekološke i ekonomske prednosti. Generalno, studija pokazuje da se otpadni staklasti emajl može efikasno koristiti u betonu bez ugrožavanja njegove strukturne stabilnosti ili trajnosti.*
Ključne reči: *otpadni staklasti emajl, pucolanski materijal, beton, blokovi za popločavanje.*

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1. INTRODUCTION

The manufacturing of heating devices that utilize solid, liquid, and gaseous fuels has led to a substantial increase in the generation of waste materials, most notably vitreous enamel. Vitreous enamel is a transparent or opaque material—either colorless or pigmented with metal oxides—applied by fusing it onto metal plates, ceramics, or glass substrates for purposes of decoration or protection. One of the primary reasons for its widespread application across various industries is its excellent corrosion resistance. As such, vitreous enamel is extensively used to coat a wide range of electrical products, household appliances, cookware, panels, industrial equipment, chemical reactors, pipelines, and steel components. Due to its glassy matrix, vitreous enamel exhibits exceptional properties including impermeability, washability, and resistance to high temperatures (fire resistance). Moreover, it is highly resistant to ultraviolet (UV) radiation, which ensures long-term stability in surface appearance—a significant advantage over organic paints and polymer-based coatings [2,5].

To enhance the performance characteristics of vitreous enamel, various additives are frequently incorporated into its composition. However, these additives may contain environmentally hazardous substances [4]. Following the enameling process, a portion of the enamel matrix remains as residual waste, which cannot be reused for further coating applications. This waste is typically disposed of in landfills, where it may pose risks to the environment and, consequently, to human and animal health.Therefore, identifying alternative and sustainable uses for waste vitreous enamel is of critical importance. Despite this, the literature on the potential reuse and practical applications of waste enamel remains limited, with only a few studies addressing this subject [1,3].

The paper presented research on the influence of vitreous waste enamel on the properties of concrete and concrete paving flags.

2. PUZZOLANIC ACTIVITY OF VITREOUS WASTE ENAMEL

For the purposes of research, waste vitreous enamel was used, which was created during the production of heating elements in the company Alfa-Plam Vranje. Its chemical composition is given in Table 1, while Figure 1 shows a photo and SEM.

SiO_2	Fe_2O_3	Al_2O_3	CaO	MgO	SO_3	P_2O_5	TiO_2	Na_2O	K_2O
21.62	2.60	7.00	60.16	2.34	2.55	-	-	0.33	0.66

Tabela 1. Chemical composition of waste vitreous enamel

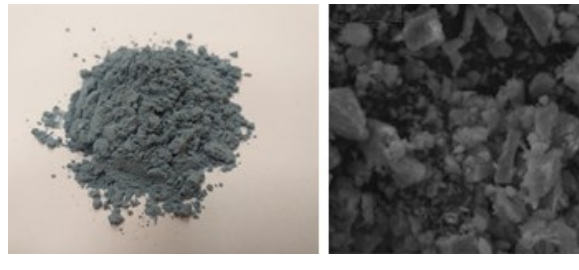


Figure 1. Photo and SEM vitreous waste enamel

In order to determine the potential use of waste vitreous enamel as a Type II admixture for concrete production in accordance with the SRPS EN 206:2021 [6], its pozzolanic activity was tested by determining the pozzolanic material class (according to SRPS B.C1.018:2015[7]) and the activity index (according to SRPS EN 450-1:2014 [8]). The test results are presented in Table 2. The pozzolanic activity testing showed that the material belongs to Class 15 of pozzolanic materials (the highest class) and meets the criteria related to the activity index test.

Properties	Result of the test	Quality requirement
Class of pozzolanic materials	Flexural strength – 4.14 MPa	> 4.0 MPa for class 15
	Compressive strength – 16.81 MPa	> 15.0 MPa for class 15
Activity index	77.62% on 28 days	> 75% on 28 days
	87.49% on 90 days	> 85% on 90 days

Table 2. The pozzolanic activity of vitreous waste enamel

3. INFLUENCE OF VITREOUS WASTE ENAMEL ON THE PROPERTIES OF CONCRETE

Given the high pozzolanic material class, the replacement of cement with waste vitreous enamel in concrete production ranged from 10% to 40%. Test concrete mixtures were prepared using natural river aggregate (sand) with grain size 0–4 mm, coarse crushed aggregate (fractions 4/8 mm and 8/16 mm), Portland cement CEM I 52.5 R, waste vitreous enamel, water, and a chemical admixture of the superplasticizer type. A total of five different mixtures were made: a reference mixture (labeled E) and four mixtures in which cement was partially replaced with waste vitreous enamel in mass proportions of 10%, 20%, 30%, and 40% (labeled C-10, C-20, C-30, and C-40). The compositions of the concrete mixtures are shown in Table 3, while the test results for the fresh and hardened concrete properties are presented in Tables 4 and 5.

Component	C-E	C-10	C-20	C-30	C-40
Cement [kg]	380	342	304	266	228
Waste vitreous enamel [kg]	-	38.00	76.00	114.00	152.00
River sand 0/4 mm [kg]	808	808	808	808	808
Crushed aggregate (4/8 mm) [kg]	376	376	376	376	376
Crushed aggregate (8/16 mm) [kg]	696	696	696	696	696
Water [kg]	180	180	180	180	180
Superplasticize [kg]	3.04	3.04	3.04	3.04	3.04

Table 3. Concrete mix compositions

1.	Consistency – slump	SRPS EN 12350-2	E	190 mm
			C-10	200 mm
			C-20	220 mm
			C-30	230 mm
			C-40	240 mm
2.	Density of fresh concrete	SRPS EN 12350-6	E	2466 kg/m^3
			C-10	2472 kg/m^3
			C-20	2490 kg/m^3
			C-30	2488 kg/m^3
			C-40	2492 kg/m^3
3.	Entrained air content in concrete	SRPSEN 12350-7	E	2.6%
			C-10	2.0%
			C-20	1.8%
			C-30	1.7%
			C-40	1.5%

Table 4. Properties of fresh concrete

1.	Demsity of hardened concrete in a saturated state	SRPS EN 12390-7	E	2455 kg/m^3
			C-10	2462 kg/m^3
			C-20	2483 kg/m^3
			C-30	2482 kg/m^3
			C-40	2485 kg/m^3
2.	Flexural strength	SRPS EN 12390-5	E	7.0 MPa (28 days), 7.4 MPa (90 days)
			C-10	5.1 MPa (28 days), 5.5 MPa (90 days)
			C-20	4.9 MPa (28 days), 5.4 MPa (90 days)
			C-30	4.7 MPa (28 days), 5.1 MPa (90 days)
			C-40	4.6 MPa (28 days), 5.0 MPa (90 days)
3.	Compressive strength	SRPS EN 12390-3	E	38.8 MPa (2 days), 56.3 MPa (7 days), 66.5 MPa (28 days), 73.6 MPa (90 days)
			C-10	30.0 MPa (2 days), 43.0 MPa (7 days), 63.3 MPa (28 days), 71.0 MPa (90 days)
			C-20	29.5 MPa (2 days), 46.5 MPa (7 days), 60.9 MPa (28 days), 69.2 MPa (90 days)
			C-30	27.6 MPa (2 days), 42.1 MPa (7 days), 55.7 MPa (28 days), 62.6 MPa (90 days)
			C-40	22.0 MPa (2 days), 35.8 MPa (7 days), 50.5 MPa (28 days), 57.1 MPa (90 days)
4.	Depth of water penetration under pressure	SRPS EN 12390-8	E	12 mm
			C-10	10 mm
			C-20	14 mm
			C-30	12 mm
			C-40	13 mm
5.	Freeze-thaw resistance with de-icing salts - Scaling	SRPS CEN-TS_12390-9	E	0.14
			C-10	0.16
			C-20	0.12
			C-30	0.15
			C-40	0.18

Table 5. Properties of hardened concrete

Based on the results of concrete consistency testing, it can be concluded that the slump increases with an increasing content of waste vitreous enamel. The maximum slump of 240 mm was recorded for the C-40 concrete mixture, which is 26% higher compared to the reference mixture. The main reason for this is the lower water requirement of vitreous enamel compared to cement.

The air content in fresh concrete decreases with an increasing percentage of cement replacement by waste enamel. The reference mixture had the highest entrained air content (2.6%), while the C-40 mixture had the lowest (1.5%). This can be attributed to better compaction of concrete prepared with waste enamel.

The replacement of cement with waste vitreous enamel contributed to a slight increase in the bulk density of concrete in both fresh and hardened states, primarily due to the lower air content.

Flexural strength tests were performed on specimens at the ages of 28 and 90 days. Concrete specimens prepared with waste enamel showed lower flexural strength compared to the reference (approximately 30% lower), with the flexural strength slightly decreasing as the percentage of cement replacement increased.

Compressive strength is the most important property of concrete, and it was determined at specimen ages of 2, 7, 28, and 90 days. It was observed that with increasing replacement of cement with waste vitreous enamel, the compressive strength of concrete specimens decreased at all ages. During the first 7 days, the reference concrete showed the highest gain in compressive strength. However, with the aging of the concrete, the compressive strengths of the mixtures containing waste enamel began to approach that of the reference concrete, due to the delayed pozzolanic reaction. Figure 2 presents the percentage ratio of the compressive strengths of concretes with partial cement replacement by waste enamel compared to the reference concrete at 90 days. The diagram indicates a gradual decline in compressive strength as the replacement percentage increases.

If a 20% reduction in compressive strength is considered an acceptable threshold, it can be concluded that in concrete production, cement can be replaced with waste vitreous enamel in amounts up to 30% without significantly compromising compressive strength.

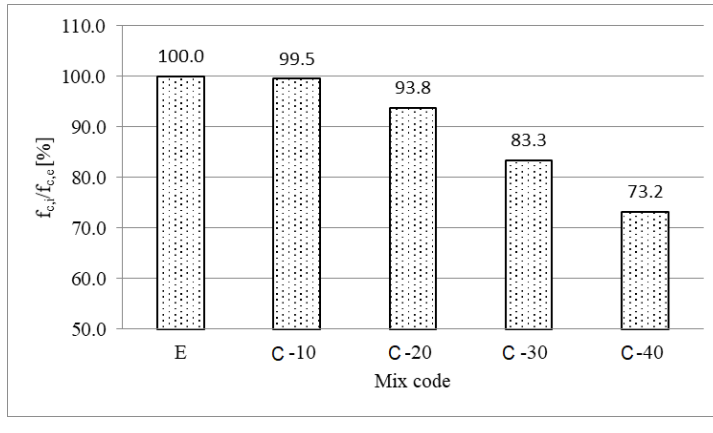


Figure 2. The percentage ratio of the compressive strengths of concretes with partial cement replacement by waste enamel compared to the reference concrete at 90 days

The depth of water penetration under pressure and resistance to freeze–thaw cycles in the presence of de-icing salts are key indicators of concrete durability. Both types of testing revealed no significant differences in results, indicating that the substitution of cement with waste vitreous enamel does not negatively affect the durability of the concrete.

The measured depth of water penetration under pressure ranged from 10 to 14 mm, which is well below the upper limit of 20 mm prescribed for impermeability class V-III (the highest class), thereby confirming compliance with this performance criterion.

Additionally, the amount of surface scaling due to freeze–thaw exposure in the presence of salts ranged between 0.12 and 0.18 mg/mm². These values remain within the prescribed limit of 0.20 mg/mm² for freeze–thaw resistance class XF4 (the highest class), further indicating that the tested concrete meets the required standard.

4. CONCRETE PAVING BLOCKS WITH THE ADDITION OF WASTE VITREOUS ENAMEL

As part of the research, concrete paving blocks with dimensions of 200 × 200 × 60 mm were manufactured. The production process employed Portland cement CEM I 42.5R, supplied by Moravacem. The base (load-bearing) layer of the concrete was prepared using crushed limestone aggregate fractions of 0/4 mm and 4/8 mm, obtained from the "Dolac" quarry Bela Palanka. For the top (exposed) layer, quartz sand with a particle size range of 0.25–1.2 mm, sourced from the "Jugokaolin – Srbokvarc Branch" in Rgotina, was used. In this experimental series, waste vitreous enamel was incorporated into the concrete mix used for the base layer. Specifically, 15% of the cement mass was replaced with waste enamel. These paving blocks were labeled "OSE" (Figure 3).

To enable a comparative analysis, an additional type of block was manufactured without the incorporation of waste vitreous enamel, serving as the reference specimen and labeled "E". The detailed composition of the concrete mixtures used for both types of blocks is presented in Table 6.



Figure 3. Concrete paving blocks with the addition of waste vitreous enamel

Concrete	Layer of block	Concrete volume [m^3]	Aggregate		Cement [kg]	Waste enamel [kg]	Quartz 0,25/ 1,2 mm [kg]	Water [kg]
			0/4 mm [kg]	4/8 mm [kg]				
E	Base	0.25	245	195	90	-	-	30
	Top	0,0278	-	-	13.0	-	46.5	4.4
OSE	Base	0,25	245	195	76.5	13.5	-	30
	Top	0,0278	-	-	13.0	-	46.5	4.4

Table 6. Concrete mix compositions for making concrete paving blocks

All the tests stipulated by the SRPS EN 1338:2012 [9] were performed on the blocks in question. The test results are shown in table 7.

Test	Reference (E)	OSE	Quality requirement according to SRPS EN 1338
Density [kg/m^3]	2260	2250	-
Water absorption [%]	5.62	5.70	≤ 6,0
Split tensile strength [MPa]	5.17 mean 4.88 min.	4.56 mean 4.30 min.	≥ 3.6 mean ≥ 2.9 min
Breaking load on the length to be split [N/mm]	575	514	≥ 250
Freeze-thaw resistance with de-icing salts [kg/m^2]	0.84	0.86	Mass loss mean ≤ 1,0 kg/m^2, individ. ≤ 1,5 kg/m^2
Abrasion test - Bohme method [mm^3/mm^2]	15970	16150	≤ 18 000 mm^3/mm^2
Slip/skid resistance [USRV]	68	66	not prescribed

Table 7. Test results on experimental concrete paving blocks

Given that a partial replacement of cement (15% by mass) was implemented in the concrete used for the load-bearing layer of the paving blocks, the most critical parameter among those presented in the previous table is the splitting tensile strength. Test results indicate that the splitting tensile strength of the blocks containing waste vitreous enamel is approximately 12% lower than that of the reference samples. However, even this reduced strength value meets the quality requirements prescribed by the SRPS EN 1338:2012 [9]. Regarding other performance, no significant differences

were observed between the reference blocks and those incorporating waste vitreous enamel.

5. CONCLUSION

The use of waste glassy enamel as a partial substitute for cement in concrete slightly reduces mechanical properties, yet it does not compromise the overall durability of the material. When incorporated at replacement levels of up to 20%, the physical and mechanical properties of concrete, including compressive strength, the most critical characteristic, remain largely comparable to those of reference concrete.

Experimental analysis of concrete paving blocks containing 15% waste enamel by mass, as a partial cement replacement, confirmed that such blocks meet performance standards and can be used in practical applications without restrictions.

Overall, considering that the use of waste enamel addresses environmental disposal concerns while also partially replacing cement, its application is both practical and justified, especially in concrete products where a marginally lower strength compared to conventional cement composites is acceptable.

Acknowledgements

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
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XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
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PROCEDURE FOR PRACTICAL APPLICATION OF CFRP MATERIALS IN REPAIR AND STRENGTHENING OF RC BUILDINGS

Summary: *In the paper part of the analytical, laboratory and quasi-static experimental investigations of designed models of RC columns confined with CFRP will be presented. Particular attention will be paid to behavior of these columns under cyclic loads, whereat a number of comparative analyses of a number of parameters obtained from the experimental investigations of the tested models will be carried out. Some recommendations and outcomes will be given as to the approach and technology of practical application of these materials in Repared and Strengthening of RC Buildings structures, particularly in seismically active regions.*
Keywords: *Seismic Strengthening, Innovative Materials, CFRP, quasi-static test*

PROCEDURA ZA PRAKTIČNU PRIMENU CFRP MATERIJALA U POPRAVCI I OJAČAVANJU BETONSKIH OBJEKATA

Rezime: *U delu rada biće predstavljena analitička, laboratorijska i kvazi-statička eksperimentalna istraživanja projektovanih modela betonskih stubova ojačanih CFRP materijalima. Posebna pažnja biće posvećena ponašanju ovih stubova pod cikličnim opterećenjima, pri čemu će biti izvedena brojna uporedna analiza parametara dobijenih eksperimentalnim istraživanjima testiranih modela. Biće dati neki preporuke i zaključci u vezi sa pristupom i tehnologijom praktične primene ovih materijala u popravci i ojačavanju betonskih objekata, posebno u seizmički aktivnim regionima.*
Ključne reči: *Seizmološko ojačavanje, inovativni materijali, CFRP, kvazi-statički test*

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1. INTRODUCTION

To contribute to definition of the joint behaviour of concrete, reinforcement and CFRP materials in the nonlinear range and develop a methodology and criteria for application of these materials in seismically active regions, for the needs of this study, an original research program involving experimental investigations on a series of two elements-models (columns) was defined. The main objective of the research programme was to define the strength and deformability of the elements constructed of innovative materials, as a function of a number of selected parameters that were varied in the course of the experiments. Within the frames of the experimental programme realized at UKIM-IZIIS, the percentage of longitudinal and transverse reinforcement was varied. The concrete class and the CFRP type were the same for both models. The behaviour of the models exposed to cyclic loads (quasi-static tests) up to failure was investigated by visual monitoring of occurrence of cracks and development of failure mechanism.

In this paper, selected results from laboratory testing of built-in materials, part of analytical results and part of quasi-static experimental investigations of models designed and practical application of these materials in Repared and Strengthening of RC Buildings constructed by use of CFRP-materials are presented.

2. EXPERIMENTAL PROGRAM

For the needs of own experimental investigations, two column elements were designed. The column models were designed as fixed cantilever girders with a constant length of both models of 200 cm (the column was treated only up to the inflection point, i.e., half of the total height) and cross-section of 30/30 cm.In both models, the varying parameters were, the percentage of longitudinal and transverse reinforcement and the axial forces. The concrete class, i.e., the compressive strength of concrete and the type of the CFRP was same for both models.The elements were designed to the geometrical scale of 1:1[6].

Presented further are photos taken during construction of the models (Model M1 and Model M2), Figure 1 , and presented further are photos and results taken in the process of quasi-static tests on Model M1 and Model M2 with photos of characteristic damage (Figure 2, Figure 3 and Figure 4) [1], [6].



Figure 1. Construction of the column models for experimental tests and photos taken during application of CFRP on the models



Figure 2. Photos during the quasi static testing of the column models – Model M1and M2

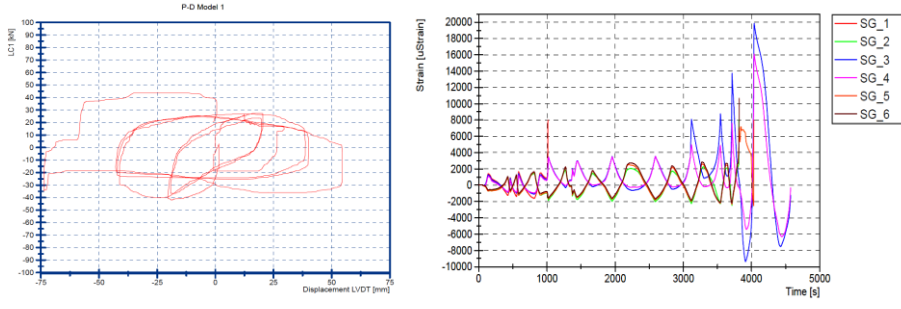


Figure 3. Force – displacement relationship and time histories of strains for model M1[10]



Figure 4. Observed damages during the quasi static testing of Model M1 and Model M2

3. LABORATORY TESTS ON MATERIALS BUILT-IN MODELS FOR EXPERIMENTAL RESEARCH

During concreting of the models for experimental quasi-static tests, two models and three trial specimens- concrete cubes proportioned 15/15/15 were taken from the supports - beams and three trial cubes proportioned 15/15/15 were taken from the columns, in addition to the nine (9) cylinders proportioned 15/30 cm (Figure 6). To define compressive strength and concrete class, laboratory tests were performed at stock holding company-GIM-Skopje (for the cubes) and ZIM –Skopje (for the cylinders), while the tests for definition of the modulus of elasticity of the built-in concrete were done at ZIM – Skopje.

Using the trial concrete specimens – cylinders, three series of tests of compressive strength and tests for definition of the modulus of elasticity of the built-in concrete were carried out as follows:

- Series 0- concrete cylinders without CFRP- plain concrete;
- Series 1- concrete cylinders wrapped with 1 (one) CFRP layer;
- Series 2- concrete cylinders wrapped with 2 (two) CFRP layers.

From the results obtained, it can be concluded that the force inducing failure of concrete cylinders without CFRP amounts to 296.0 kN. For the cylinder with one CFRP layer, it amounts to 670.0 kN., while for the cylinder with two CFRP layers, it amounts to 955.0 kN. The compressive strength amounts to 16.8 Mpa, 37.9 Mpa and 54.1 Mpa, for all three series, and static modulus of elasticity are 28200.Mpa, 33000.Mpa And 43500.Mpa respectively [7].

The results obtained are presented graphically in Figure 5.

In general, it can be concluded that the compressive strength and modulus of elasticity becomes higher with the increase of the number of CFRP layers.

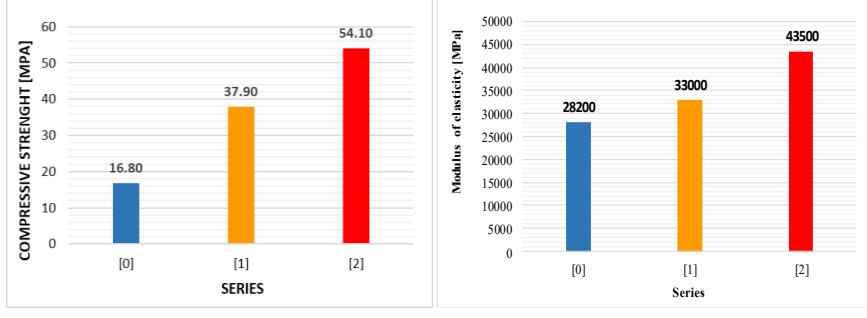


Figure 5. Diagram of compressive strength and static modulus of elasticity for each series

4. DEFINITION OF REAL STRENGTH AND DEFORMABILITY CAPACITY OF COLUMN MODELS

To define the real bearing and deformability capacity of the built column models, the values on quality of built-in concrete and reinforcement obtained for both vertical and transverse reinforcement, as well as the type of used CFRP were used. In the first phase, the real M-Φ (moment – curvature) relationships of the column cross-sections were computed by applying axial force, the real M-N diagrams, and then, based on the obtained M-Φ diagrams, the strength and deformability capacity of each model was defined [1].

The strength and deformability characteristics (M-N) and (M-Φ) at cross-section level, and Nonlinear Static Pushover Analysis and Nonlinear Time-History Analysis were analytically defined by use of the SAP2000 computer software. The following analyses were carried out [12]:

(1) For Model M1, definition of the M-N diagram for Nv = 500 kN and M-N diagram (Fig. 6) for the following values for 0.1, 0.2, 0.3 series:

- For the designed concrete class (DC) (EC-25/30) with quality and quantity of reinforcement.
- For the built-in concrete class (CC) (EC-16/20) with quantity and quality of reinforcement.
- For the built-in concrete class with one layer of CFRP (CC-FRP) (38/46) with quantity and quality of reinforcement.

(2) For Model M2, definition of the M-N diagram for Nv = 300 kN and M-N diagram (Fig. 6) for the following values for 0.1, 0.2, 0.3 series:

- For the designed concrete class (DC) (EC-25/30) with quality and quantity of reinforcement.
- For the built-in concrete class (CC) (EC-16/20) with quantity and quality of reinforcement.
- For the built-in concrete class with one layer of CFRP (CC-FRP) (38/46) with quantity and quality of reinforcement.

Presented further are the results obtained from these analyses for Model M1 and Model M2.

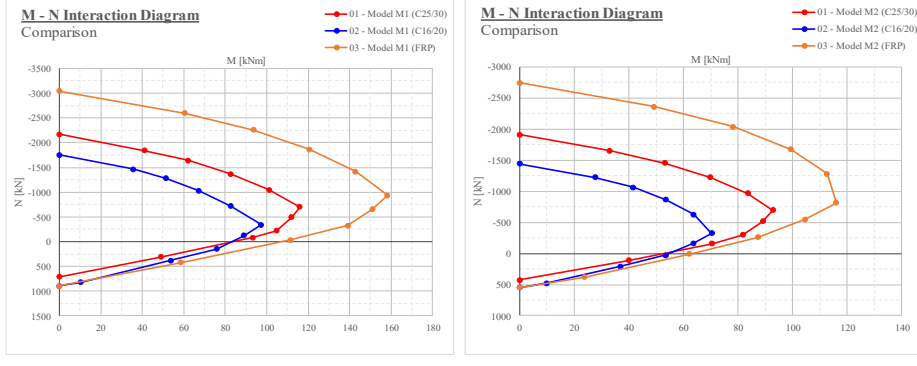


Figure 6. M-N Diagrams for Model M1 and Model M2 - Comparison

Based on this analyses (Figure 6), it can be concluded that the ductility to rotation for Model M1 is 2.049 greater for the model with CFRP, while the ductility to displacementt is greater in respect to the ductility of Model M1 without CFRP for 76.7%.

In the case of Model M2, the ductility to rotation is higher in the case of the Model with CFRP for 64 %, while the ductility to displacements is higher compared to the ductility of the Model M2 without CFRP for 46.1% [12].

From the Nonlinear Static Pushover Analysis and Nonlinear Time-History Analysis of the values obtained, it can be concluded that the ductility capacity of Model M1 strengthened with CFRP is greater for 64.6% and for Model M2 is greater for 60% , while the strength capacity for Model M1 is greater for 21.1% and for Model M2 is greater for 7.7% compared to the model without CFRP (Figure 7)[11]

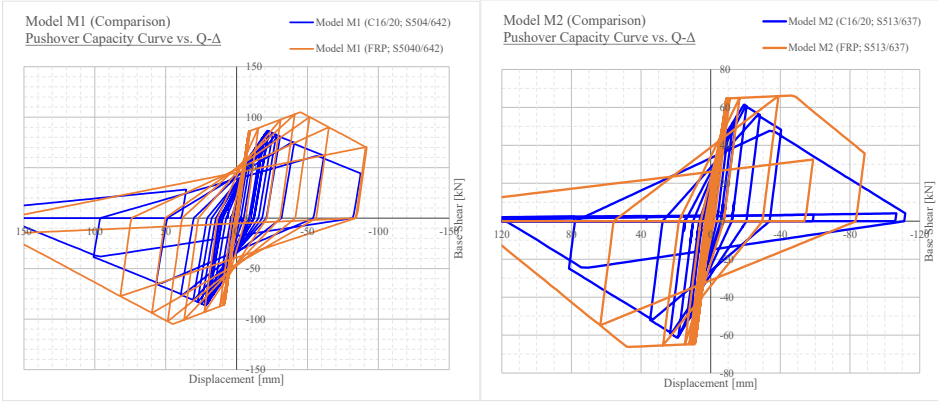


Figure 7. Q – Δ diagram for Model M1 and Model M2 – comparison for series 02 and 03

5. PROCEDURE FOR PRACTICAL APPLICATION OF INNOVATIVE MATERIALS IN REPAIR AND STRENGTHENING OF RC BUILDINGS

Based on all analyzed materials and performed experimental and analytical investigations, efforts have been made to briefly present a procedure for repair and strengthening of RC buildings which is known in use of traditional materials, but is amended with criteria that would be the starting point in using these materials for repair and strengthening of RC buildings.

The procedure consists of 7 steps.

Step 1:Definition of reasons for repair and strengthening of existing RC elements and integral structural system

(1) Lower quality and/or amount of built-in materials

(2) Insufficient strength and deformability capacity of RC elements of a building or an RC building as a whole to sustain additional vertical loads and/or corresponding seismic effects.

(3) Damaged structural elements of existing RC building out of different reasons.

Step 2:Definition of the structural system of the building and determination of the quantity and quality of built-in material for existing buildings.

Step 3:Determination of the real strength and deformation capacity of each structural element taken separately and the integral structure .

Step 4:Definition of the seismic parameters and the design criteria for location.

Step 5:Nonlinear static and/or nonlinear dynamic analysis of the structural system.

Step 6:Evaluation of the seismic resistance for local and global stability of the RC building.

Step 7:Diagnosis and selection of methodology for repair and strengthening of the RC building (by use of traditional or innovative materials)

Step7a:While selecting traditional methods for repair and strengthening, the mode of repair and the procedure are selected from step 2 to step 7 and several times, if necessary, until the obtaining of an optimal solution for repair and strengthening by satisfying all the requirements in compliance with the existing regulations.

Step 7b:Innovative materials for repair and strengthening can be selected in the following cases:

- When the strength of the built-in concrete is not lesser than the designed one for more than 30%.
- When the elements do not have sufficient bearing capacity for vertical loads.

The values in the diagram are taken from the catalogues that are delivered by producers of innovative materials.

6. CONCLUSION

As a result of the comprehensive laboratory tests on concrete cylinders for: Series 0 - concrete cylinders without CFRP- plain concrete; Series 1- concrete cylinders wrapped with 1 (one) CFRP layer; Series 2- concrete cylinders wrapped with 2 (two) CFRP layers; It can be concluded that the compressive strength and Modulus of Elasticity becomes higher with the increase of the number of FRP layers.

Sample analytical analyses were carried out to define the strength and deformability capacity (M-N) and (M-Φ) at cross-section level were analytically defined by use of the SAP2000 computer programme. Based on the obtained M-Φ diagrams, the strength and ductility capacity of each model was defined.

The following analyses were carried out:

The moment capacity obtained for cross-section of Model M1 with CFRP is greater for 21.07 % than that of cross-section without CFRP and the ductility to rotation is higher in the case of the model with CFRP for 98 %. The moment capacity obtained for cross-section of Model M2 with CFRP is greater for 7.7 % than that of cross-section without CFRP and the ductility to rotation is higher in the case of the model with CFRP for 64 %.

Generally, it can be concluded that CFRP systems represent a very practical tool for strengthening and retrofitting of concrete structures and are appropriate for flexural strengthening, shear strengthening and column confinement and ductility improvement.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
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DURABILITY PROPERTES OF SLAG MIXTURES BASED ON FLY ASH

Summary: *This study investigates the durability properties of geopolymer mortar and concrete. Fly ash was used as the primary binder, with partial replacement by ground granulated blast furnace slag (GGBFS) and ladle slag (LS). Durability was evaluated through sulfate resistance and freeze-thaw resistance tests, primarily by measuring compressive strength of subjected samples. The measured values of the tested specimens were compared with reference samples of the same mix design and age, which were until the testing time stored under ambient conditions. The results showed that geopolymer mortar exhibited excellent resistance to both external sulfate attack and freeze-thaw cycles. For geopolymer concrete, all mixes showed strong resistance to external sulfate attack. However, in the freeze-thaw resistance test, the mixture containing 20% GGBFS and 80% fly ash did not show resistance to the specified number of freeze-thaw cycles.*
Keywords: *Geopolymer, sulfate resistance, freeze-thaw resistance, mortar, concrete*

TRAJNOST MEŠAVINA SA DODATKOM ZGURE SPRAVLJENIH NA BAZI ELEKTROFILTERSKOG PEPELA

Rezime: *U ovom radu dati su rezultti ispitivanja trajnosti geopolimernih maltera i betona. Kao osnovno vezivo korišneje elektofiterski pepeo dog je njegova delimična zamena granulisanom zgurom (GGBFS) i konvertorskom zgrom (LS). Trajnost je ocenjivana putem ispitivanja otpornosti na sulfate i cikluse smrzavanja i odmrzavanja. Izmerene vrednosti testiranih uzoraka upoređene su sa referentnim uzorcima istog sastava i starosti, koji su do momenta ispitivanja negovani u ambijentalnim uslovima. Rezultati pokazuju da je geopolimerni malter bio izuzetno otporan na dejtvo sulfata i cikluse smrzavanja i odmrzavanja. Kod geopolimernog betona, sve mešavine su pokazale visoku otpornost na spoljašnje delovanje sulfata. Međutim, u ispitivanju otpornosti na smrzavanje i odmrzavanje, mešavina koja sadrži 20% GGBFS-a nije se pokazala kao otporna.*
Ključne reči: *Geopolimeri, sulftna otpornost, otpornost na cikluse zamrzavanja i odmrzavanja, malter, beton*

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1. INTRODUCTION

A relationship between eco-materials and construction materials can be established through the use of geopolymers. Unlike cement production, which requires very high processing temperatures, the preparation of geopolymer binders occurs at ambient temperature, while the synthesis of geopolymer composites takes place either in ambient conditions or curred at about 90 °C. Given the significantly lower energy consumption associated with this process, the increasing popularity of geopolymers in recent years is well justified [1]. Since now, the binders that we can currently use in geopolymer synthesis were previously utilized exclusively as supplementary materials in cement production. From that point of view, since now, their role has been primarily to enhance specific properties of traditional cement mixtures [8]. Existing research was conducted to find the properties of fresh and hardened states of geopolymers. Until now, many studies have been focused on testing geopolymer pastes, much fewer studies have been focused on geopolymer mortars, and the least attention has been given to geopolymer concretes. Researchers have paid particular attention to the formulation of these composites, examining various parameters such as the concentration of alkaline activator solutions (e.g., NaOH and Na_2SiO_3), curing time and temperature, humidity levels, and other related factors [5,6].

It is well known from previous knowledge that exposure of specimens or constructions to aggressive environmental conditions can have an influence on the durability properties of cement, as well as geopolymers materials [2]. Since now, several authors have researched the resistance of mortars to freeze-thaw cycles. Slavić et al. [15] investigated the potential use of slag as a partial substitution of kaolin when designed geopolymers. Škvara et al. [16] have researched the behavior of geopolymers synthesized with varying concentrations of the silicate modulus (Ms). Nevertheless, it can be concluded that both mentioned researchers, and many others who have conducted similar studies, agree that geopolymer materials exhibit resistance to the effects of freeze-thaw cycles.

The resistance of geopolymers to external sulfate attack has been also researched. This was goal of interest of Bakharev [4], Komljenović et al. [9], Bijeljić et al. [5,6] and Džunuzović [7]. Based on the results of mentioned authors, as well as many other other researchers, it can be concluded that geopolymers show good resistance to external sulfate exposure. Furthermore, when authors compare the compressive strength of geopolymer samples that were during testing time immerced into sulfate solution to reference samples, of the same composition and age that were cured under ambient conditions, decrese of strength was not found. By using visible inspection of samples there were no visible sings of degradation, surface erosion, cracking, or or similar noticed on the specimens exposed to sulfate environments.

2. EXPERIMENTAL RESEARCH

In this study, fly ash was used as the main source material for making geopolymer mortars, while ladle slag and ground granulated blast furnace slag were used as an additive. Fly ash originates from the thermal electric power plant Kostolac “B” in Serbia. Ladle slag was generated as a byproduct coming from the steel production process by the company HBIS Group Iron & Steel-Serbia, while ground granulated blast furnace slag is

a by-product of iron reprocessing in the Smederevo still mill. The used binders were sieved through a 90 μm sieve. The specific gravity of fly ash was 2.24 g/cm^3, while the specific gravity of ladle slag and ground granulated blast furnace slag were 3.45 g/cm^3 and 2.85 g/cm^3, respectively. The chemical composition of binders used in tests is given in Table 1, while in Figure 1 their SEM and photograph are displayed.

Sodium hydroxide (SH) and sodium silicate (SS) manufactured by the local producers were used as alkali solutions. The SH was prepared by dissolving NaOH flakes (purity of 98%) in water to 10M solution 24 hours prior to mixing. The SS activator was of the following characteristics: module (Ms = SiO_2/Na_2O) 2.2; content SiO_2 – 26.70%, Na_2O – 13.30% and H_2O – 60%. Liquid SS and SH were mixed in a mass ratio of 100:18.52 (i.e., $Na_2SiO_3/NaOH$ = 5.40). Such a ratio caused the creation of a single activator with a content of 10% Na_2O with respect to the mass of the solid binder. This decreased the module of MS (SiO_2/Na_2O) in SS to the value of 1.5.

Parameter	SiO_2	Fe_2O_3	Al_2O_3	CaO	MgO	SO_3	P_2O_5	TiO_2	Na_2O	K_2O
	%									
Fly ash	51,68	11,58	20,16	7,43	2,41	1,02	0,12	1,04	0,88	1,04
Ladle slag	22,20	6,82	15,60	44,21	8,44	-	0,1	0,04	0,01	0,01
Ground granulated blast furnace slag	38,1	0,70	7,17	39,48	9,86	0,36	-	-	0,51	0,23

Tabela 1. Chemical composition of used binders

According to the results of the chemical analysis of fly ash, the sum of $SiO_2+A_{l2}O_3+Fe_2O_3$ is more than 70 % of total mass, while CaO content was less than 10%, which sorts fly ash into the class of "silicate ash type" according to standard EN 450-1 [11] or to "class F" according to standard ASTM C618 [3]. Based on the chemical composition of the ground granulated blast furnace slag, it can be observed that the dominant components are SiO_2 and CaO. In terms of mass percentage in the material, the content of CaO + MgO + SiO_2 accounts for above 90% of the mass of the examined material. This value exceeds two-thirds of the total mass of the binder, which is the minimum value specified by the SRPS EN 15167-1:2010 standard [10]. The coefficient Kb, which describes the content of $(CaO+MgO)/(SiO_2+Al_2O_3)$ in the binder, is 1.09, which classifies the ground granulated blast furnace slag as a basic material. Based on the chemical composition of ladle slag, it can be concluded that the largest share is composed of CaO and SiO_2. Their sum percentage is more than 68 %, while the content of CaO + MgO + SiO_2 reaches almost 77% of the examined material.

Some conclusuions might be given based on SEM images. According to then, the fly ash particles seem to be mostly spherical and different in size. Unlike fly ash, ladle slag and ground granulated blast furnace slag particles seem to have irregular shapes and sharp edges.

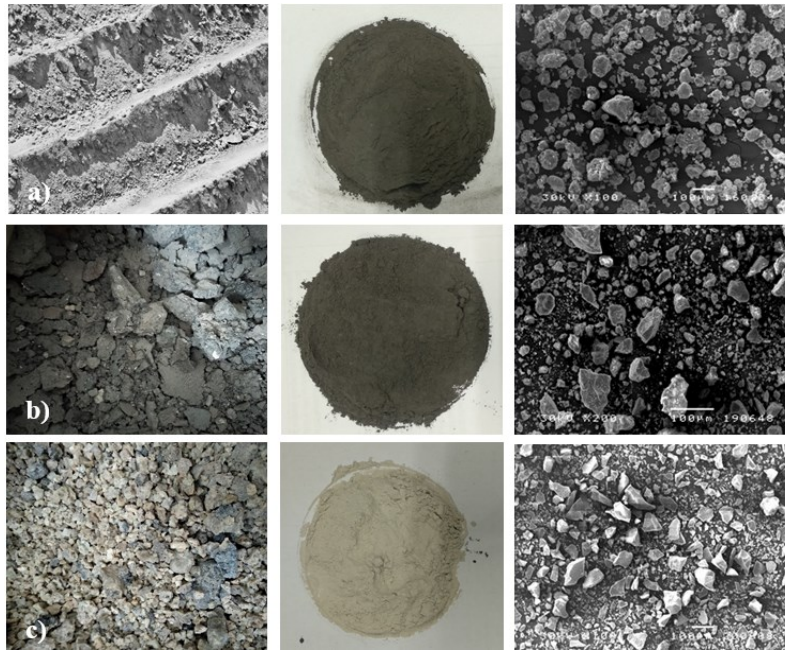


Figure 1. Raw material (left), sieved material (middle) and SEM (right) display of used fly ash (a), ladle slag (b) and ground granulated blast furnace slag (c)

In order to compare the pozzolanic activity and activity index of the used slags and fly ash, mortar mixtures were made. Mixtures were designed in accordance with standard EN 450-1. In the test mixtures (FA-AI, GGBFS-AI and SS-AI), 25% of the mass of cement used in the reference mixture named "E-AI" was replaced by fly ash, ground granulated blast furnace slag ad ladle slag. Mentioned mixtures were designed in order to measure the activity index of used binders. The activity index was investigated by testing the compressive strength of samples at ages 28 and 90 days. The activity index was calculated as the ratio of the compressive strength of test mixture samples and the compressive strength of reference mix samples when tested at the same age.

2.1. Mix design

To investigate the effects of using ladle slag and ground granulated blast furnace slag to replace fly ash on the properties of geopolymer mortar. Different geopolymer mixtures were designed and marked as "FA", "20 GGBFS" and "20 LS". In the mentioned mixtures, ground granulated blast furnace slag and ladle slag were used to replace fly ash at 20% by weight, respectively. The mix proportions of geopolymer mortar are given in Table 2. The moulds with samples of geopolymer mortar were after demoulding, and until the testing, wrapped in a plastic foil to prevent moisture loss. Two cement mortars were made to compare the results with some known mixtures. Depending on the cement type, mixtures are marked as EII M (mortar mixture made of CEM II as a binder) and EIII M (mortar mixture made of CEM III as a binder).

Mixture	EII M	EIII M	FA M	20 GGBFS M	20 LS M
Fine aggregate /g	1350	1350	1350	1350	1350
Cement CEM II / g	450	-	-	-	-
Cement CEM III / g	-	450	-	-	-
Fly ash/g	-	-	450	360	360
Ground granul. b.f.s /g	-	-	-	90	
Ladle slag / g	-	-	-	-	90
Water / g	225	225	20	20	20
NaOH (10M) / g	-	-	56,16	56,16	56,16
Na_2SiO_3 /g	-	-	303,23	303,23	303,23
Superplast. %	0,2	0,5	2	1,5	1,5
w/b	0,5	0,5	0,42	0,42	0,42

Table 2. Mix proportion of geopolymer mortar

Meanwhile, three mixtures of geopolymer concrete were made. Percentage-mass content of ground granulated blast furnace slag and ladle slag is in relation to total mass of binders (0 % and 20 %). Also, the percentage content of ground granulated blast furnace slag and ladle slag in binder material was used to mark the sample concrete series: "FA C", "20 GGBFS C" and "20 LS C". Also, to prevent moisture loss, concrete samples were cured in the same way as mortar samples. Mix proportions of geopolymer concrete mixtures for 1 m3 of concrete are given in Table 3. As in the case with mortar, to compare concrete results with some known mixtures, two cement concrete mixtures were designed and made. Depending on the cement type, mixtures are marked as EII C (mortar mixture made of CEM II as a binder) and EIII C (mortar mixture made of CEM III as a binder).

Mixture		EII C	EIII C	FA C	20GGBFS C	20LS C
Fine aggregate 0/4 mm / kg		890	890	618	618	618
Coarse aggregate	4/8 mm / kg	267	267	356	356	356
Coarse aggregate	8/16 mm / kg	623	623	572	572	572
Fly ash / kg		-	-	400	320	320
Ground granulated bfs / kg		-	-	-	80	-
Ladle slag / kg		-	-	-	-	80
NaOH (10M) / kg		-	-	50	50	50
Na_2SiO_3 / kg		-	-	270	270	270
Superplasticizer (SP) / kg		1,2	1,1	-	-	-
w/b		0,53	0,49	0,37	0,37	0,37

Table 3. Mix proportion of geopolymer concrete mixtures

2.2. Type tests

The sulfate resistance of geopolymer mortars, made with fly ash and the addition of ground granulated blast furnace slag, ladle slag, as well as cement mortar mixtures, was tested following the guidelines of the performance testing report CEN/TR 15697 [13]. Mortar samples were cured for 28 days. During that time, geopolymer samples were wrapped in plastic foil, while cement samples were immersed in water. After the curing period, all samples were placed in a 5% Na_2SO_4 solution for 180 days. The sulfate solution was refreshed every 30 days to maintain a pH level between 8 ± 0.5. After 180 days of exposure to the sulfate solution, the flexural and compressive strengths of the

samples were measured and compared with the corresponding reference samples of the same composition.

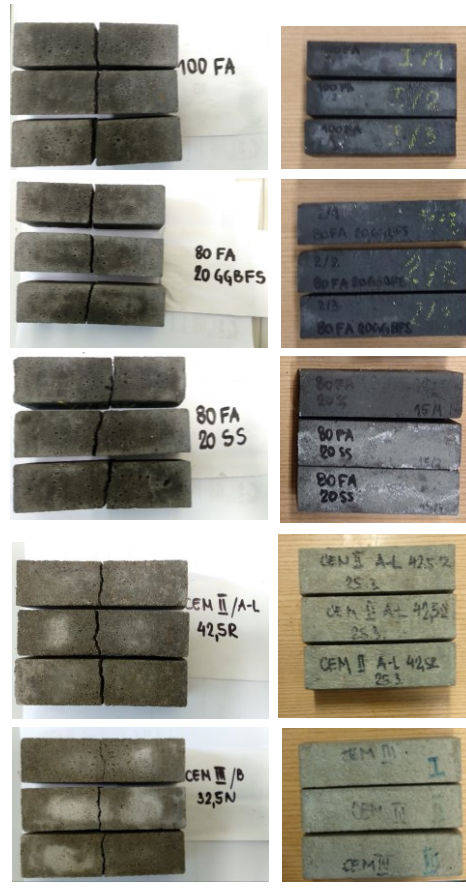


Figure 2. Mortar samples after testing freeze-thaw resistance (left), mortar samples after subjecting to 5 % Na_2SO_4 solution (right)

The freeze-thaw resistance of fly ash-based mortar was evaluated in accordance with the EN 14617-5 standard [12]. Mortar prism samples, measuring 4 × 4 × 16 cm were subjected to 25 freeze-thaw cycles. According to the relevant standard and testing report, the mortar samples are considered resistant to freeze-thaw cycles and external sulfate exposure if the compressive strength ratio of the tested samples to the reference samples exceeds 75% and 80%, respectively. Tested mortar samples subjected to freeze-thaw cycles and sulfate solution are given in Figure 2.

For sulfate resistance testing, cube-shaped concrete specimens (10 cm sides) were immersed in a 5% Na_2SO_4 solution for 90 and 180 days, as per recommendation CEN/TR 15697 [13]. Three samples from each mixture were exposed to the sulfate solution, while three reference samples were kept in ambient conditions. After the exposure period, the compressive strength of the tested samples was compared to that of the reference samples. According to the testing report, the samples are considered resistant to sulfate attack if the compressive strength ratio of tested to reference samples exceeds

80%. Freeze-thaw resistance on concrete was determined according to SRPS U.M1.206 [14] using cube-shaped specimens with 10 cm sides. After concrete curing for 28 days, by using the same procedure as on mortar, we subject concrete samples to testing. Tested concrete samples subjected to 5 % Na_2SO_4 solution resistance are given in Figure 3.

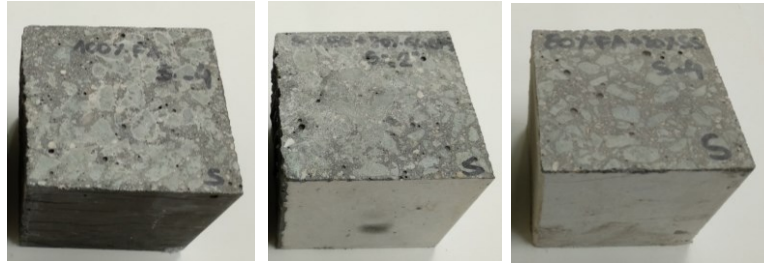


Figure 3. Concrete samples after sulfate resistance: mixture FA C (left), mixture 20 GGBFS C (middle) and mixture 20 LS C (right)

Three samples from each series were subjected to 200 freeze-thaw cycles, while three reference samples were kept in ambient conditions. Each freeze-thaw cycle involved freezing the samples for 4 hours at -20 ± 2 ºC and thawing them for 4 hours in water at 20 ± 3 ºC. Tested concrete samples subjected to freeze-thaw cycles are given in Figure 4.[S1]

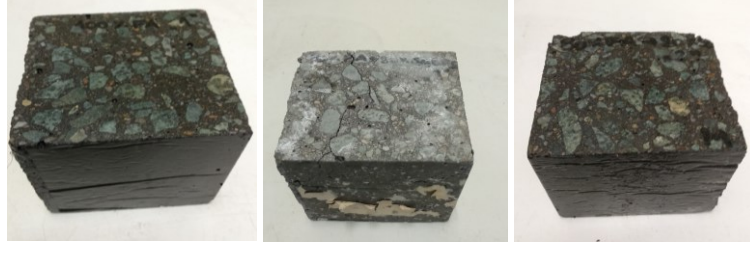


Figure 4. Concrete samples after testing freeze-thaw resistance: mixture FA C (left), mixture 20 GGBFS C (middle) and mixture 20 LS C (right)

3. RESULTS AND DISCUSSION

The results of pozzolanic activity and activity index are presented in Table 4 and 5. According to the test results, the fly ash binder (FA) has the pozzolanic activity class 10, while ground granulated blast furnace slag (GGBFS) has the pozzolanic activity index 5.

Pozzolanic activity				
By product	Flexural strength σf / (MPa)	Compressive strength σc / (MPa)	Class	Requirements SRPS B.C1.018:2015 / %
FA	3,04	10,38	10	Class 5: σf=2MPa, σc=5MPa Class 10: σf=3MPa,σc=10MPa Class 15: σf=4MPa,σc=15MPa
GGBFS	2,60	5,24	5	
LS	0,88	2,65	-	

Activity index				
Mix code	Compressive strength / MPa		Activity index / %	Requirements EN 450-1 / %
	28 days	90 days		
E-AI	53,3	57,60	-	28 days – 75 % 90 days – 85 %
FA-AI	51,6	60,2	28 days – 96,7 90 days – 104,6	
GGBFS-AI	46,15	54,85	28 days – 86,6 90 days – 95,2	
LS - AI	40,56	44,09	28 days – 76,0 90 days –76,5	

Table 4. and 5. The pozzolanic activity and activity index of used binder materials

It is worth knowing that tested ladle slag (LS) didn't show any class of pozzolanic activity. How pozzolanic class can be related to material reactivity, so a higher pozzolanic class might be an indicator of a higher material reactivity. This might raise a concern about using ground granulated blast furnace slag and ladle slag as a binder material in geopolymer mixtures.

Mixture	type	EII M	EIII M	FA M	20 GGBFS M	20 LS M
Sulfate resistance / MPa	Ref. S.	51.81	48.14	49.16	50.11	47,14
	Treat. S.	53.21	51.84	49.39	49.54	52,50
	SR Coef.	1.03	1.08	1.00	0.99	1.11
Freeze and thaw resistance / MPa	Ref. S.	45.27	45.88	47.13	51.02	46.20
	Treat. S.	46.46	45.83	44.98	48.95	44.01
	FTR Coef.	1.03	1	0.95	0.96	0,95

Table 6. Durablity properties of mortar in hardened state

Sulfate resistance (Table 6) was determined by comparison of the compressive strength of reference mortar samples at the age of 180 days (Ref. S.) and compressive strength of samples immersed in 5% Na_2SO_4 solution after 180 days (Treat. S.). The test results of geopolymer mortar indicate very good sulfate resistance under the implemented test requirements. The lowest sulfate resistance coefficient in terms of compressive strength was measured on the mixtures named "20 GGBFS M" and "FA M" and it is 0.99 and 1, respectively. The highest sulfate resistance coefficient was measured on the mixture marked as "20 LS M" and it is 1.11. The last mentioned value is also a few percentage higher than the sulfate resistance coefficient of cement etalon mixtures "EII M" and "EIII M", made of cements CEM II and CEM III. Physical changes on the tested samples were not observable.

Freeze-thaw resistance (Table 6) was also tested, as were the compressive strength resistance coefficients - FTR Coefficient. It was calculated for the tested mortar mixtures

and presents the ratio of samples treated to 25 freeze - thaw cycles (Treated Sample) and reference sample at the age of 28 + 25 days (Ref. S.). The highest compressive strength freeze-thaw resistance coefficient of geopolymers was measured on the mixture, whose samples were marked as “20 GGBFS M” and it is amount 0,97. Still, it might be concluded that all tested mixtures have almost the same freeze-thaw resistance coefficient.

Mixture	type	EII M	EIII M	FA C	20 GGBFS C	20 LS C
Sulfate resistance / MPa	Ref. S.	59,62	56,01	55,79	57,12	54,52
	Treat. S.	67,92	58,12	55,14	59	56,91
	SR Coef.	1,14	1,04	0,99	1,03	1,04
Freeze and thaw resistance / MPa	Ref. S.	59,22	55,17	56,98	–	54,69
	Treat. S.	58,52	53,72	43,85	–	42,42
	FTR Coef.	0,99	0,97	0,77	Sample failure at 120 cycles	0,78

Table 7. Durablity properties of concrete in hardened state

Measurement of sulfate resistance coefficients (Table 7) implied monitoring of the condition of concrete at the age of six months after immersing the samples in a 5% Na_2SO_4 solution. It was observed that the compressive strength of treated samples was higher or almost equal to the compressive strength of reference concrete. That implies that sulfate resistance coefficients were almost equal or greater than 1. The lowest coefficients were measured in the "FA C" concrete samples, with a value of 0,99. However, the highest coefficient was measured for the etalon concrete mixture marked as "EII C" and it amounted to 1,14.

4. CONCLUSION

In this paper, a study was conducted on the effect of fly ash, ground granulated blast furnace slag (GGBFS), and ladle slag, used as binder materials, on the durability properties of geopolymer mortar and concrete. Mortar mixtures were designed using fly ash as the primary binder, with the addition of GGBFS or ladle slag at levels of up to 20% of the total binder content (i.e., 80% fly ash). The aim was to enhance the properties of geopolymer mortar. To validate the measured results of the geopolymer mixtures, two reference (etalon) mixtures were also prepared. One mixture used CEM II cement, while the other used high-sulfate-resistant CEM III cement. The results can be summarized as follows:

- all geopolymer mortar mixtures meet the basic requirements in the term of sulfate resistance under the given test conditions;
- all tested specimens of geopolymer mortars are frost resistant for 25 cycles of treating;
- sulfate resistance was measured on concrete specimens immersed in a 5% Na_2SO_4 solution for a period of six months. The lowest resistance coefficient was found for the mixture "FA C". The sulfate resistance coefficients of two left geopolymer mixtures were greater than 1. All geopolymer and cement-based concrete mixtures present very good resistance to sulfate attack under the specified testing conditions. The geopolymer binder and alkaline solution continued to maintain their internal bonds in a stable state without reacting to external influences. Cement-based concrete mixtures

exhibited slightly better sulfate resistance compared to the geopolymer concrete specimens. The resistance coefficient values for the cement concretes were greater than 1.04;

- the geopolymer concrete mixture made by using ground granulated blast furnace slag showed insufficient durability while using the freeze-thaw test. Physical deterioration of the specimens from this mixture was already visible after 120 freeze-thaw cycles. Only the geopolymer concrete specimens marked as "FA C" and "20 LS C" can be considered resistant to freeze-thaw cycles. The reduction in compressive strength of the frozen specimens, compared to the corresponding reference specimens of the same composition and equivalent age, exceeded 13 MPa. The freeze-thaw resistance coefficients for the mentioned mixtures were 77% and 78%, respectively.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
CONTEMPORARY CLAY PRODUCTS INDUSTRY ASSOCIATION OF SERBIA

XXIX CONGRESS DIMK AND X CONGRESS SIGP
WITH INTERNATIONAL SYMPOSIUM
ON RESEARCHING AND APPLICATION OF MODERN ACHIEVEMENTS IN CIVIL ENGINEERING IN THE FIELD OF MATERIALS AND STRUCTURES
Sokobanja, May 21 - 23, 2025.

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UTILISATION OF RECYCLED CONCRETE POWDER AS SUPPLEMENTARY CEMENTITIOUS MATERIAL: PATHWAY TO THE SERBIAN TECHNICAL ASSESSMENT

Summary: *Using a Supplementary Cementitious Materials (SCMs) as a strategy for reducing CO_2 emissions, is widespread. A growing demand for SCMs has led to exploring new materials capable of reducing the environmental impact, such as recycled concrete powder (RCP), without significantly impairing the essential characteristics of cement. This paper presents the results of possibility of use the of RCP as SCM as well as the procedure for creating the Serbian technical assessment for this construction product. The possibility of utilization of RCP as SCM was assessed through the chemical composition, physical and mechanical characteristics. The results of the research showed that RCP can be used as SCM. Based on the results of the research, the preparation of the Serbian assessment document of this innovative product was initiated. This provided the possibility of making available this product on the Serbian market.*
Keywords: *Portland cement, recycled concrete powder, Serbian technical assessment*

UPOTREBA SITNIH ČESTICA RECIKLIRANOG BETONA KAO DOPUNSKOG CEMENTNOG MATERIJALA: PUT KA SRPSKOJ TEHNIČKOJ OCENI

Rezime: *Korišćenje dopunskih cementnih materijala kao strategije za smanjenje emisije CO_2 je široko rasprostranjeno. Rastuća potražnja za dopunskim cementnim materijalima dovela je do istraživanja novih materijala koji mogu da smanje uticaj na životnu sredinu, kao što su sitne čestice recikliranog betona (RCP), bez značajanog narušavanja bitnih karakteristika cementa. U ovom radu su prikazani rezultati mogućnosti upotrebe sitnih čestica recikliranog betona kao dopunskog cementnog materijala, kao i postupak izrade srpske tehničke ocene za ovaj građevinski proizvod. Mogućnost korišćenja RCP-a kao SCM procenjena je kroz hemijski sastav, fizičke i mehaničke*

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karakteristike. Rezultati istraživanja su pokazali da se sitne čestice recikliranog betona mogu koristiti kao dopunski cementni materijal. Na osnovu rezultata istraživanja, pokrenuta je izrada Srpskog dokumenta za ocenjivanje ovog inovativnog proizvoda. Time je pružena mogućnost stavljanja ovog proizvoda na tržište RS.
Ključne reči: *portland cement, sitne čestice recikliranog betona, Srpska tehnička ocena*

1. INTRODUCTION

The cement industry, as the most environmentally detrimental and energy-intensive, is responsible for approximately 8% of global greenhouse gas emissions [4]. Environmental concerns and sustainable development require strategies for lowering this ecological impact. Therefore, different initiatives for reducing CO_2 emissions, such as carbon capture, fuel substitution, energy efficiency and substitution for SCMs have been started in the previous decade.

SCMs are widely used to reduce the clinker content in cement or cement in mortars and concrete. A growing demand for SCMs has led to exploring new materials capable of reducing the environmental impact. On the other side, it is estimated that the world produces more than 10 billion tons of construction and demolition waste every year [7]. Therefore, it is crucial to efficiently recycle concrete waste and find a way of its use. Recent investigations have demonstrated the viability of using recycled concrete aggregate for various applications [8]. However, during the production of recycled aggregate, from construction and demolition waste, recycled concrete powder (RCP), as a very fine fraction (less than 150 μm), has been generated [5]. Using the RCP as SCM offers a sustainable approach towards green construction and lower consumption of natural resources. Hence, this research was focused on the possibility of use of RCP as supplementary cementitious material, mainly as main constituent of cement, and its influence on the essential characteristics of blended cement and concrete. Additionally, the procedure for creating the Serbian technical assessment for the construction product - Portland-cement with recycled concrete powder, for which the appointed Technical Assessment Body of the IMS Institute prepared the Serbian assessment document SDO 15 003:2025 was involved.

2. MATERIALS AND METHODS

2.1. Starting materials

The primary raw materials used for the research are clinker, produced by Lafarge BFC Srbija d.o.o. Beočin, recycled concrete powder generated from the concrete production by Lafarge BFC Srbija d.o.o. Beočin and gypsum obtained by desulfurization of waste gases from the thermal power plant.

RCP was obtained by crushing laboratory-prepared concrete cubes and the dried reside from the washing of concrete mixers on the laboratory crusher, until the particle size was less than 8 mm. Furthermore, the crushed material was milled in a laboratory ball mill, type L-M10, manufactured by ZRMK, for 180 minutes. On the same way, the grounding of PC clinker and gypsum were performed in a laboratory ball mill. The fineness of the ground materials, on a 32 μm sieve, was reached in the range of 20% to 36%. Chemical composition of the starting materials was determined using X-ray fluorescence spectroscopy on the S8 Tiger, produced by Bruker and it is given in Table 1.

Chemical composition	PC clinker	Recycled concrete powder (F)	Gypsum
Loss of ignition	0,4	5,6	22,1
SiO_2	21,1	77,1	0,6
Al_2O_3	5,4	5,1	0,2
Fe_2O_3	2,6	2,1	0,1
CaO	66,8	6,8	34,2
MgO	2,1	1,1	0,2
SO_3	1,2	0,4	44,2
Na_2O	0,2	1,0	0,05
K_2O	0,6	0,8	0,07

Table 1. The chemical composition of the starting materials

The major component of RCP is SiO_2 originated from finely milled/crashed aggregates, followed by CaO and Al_2O_3. CaO is as a result of cement matrix form recycled concrete, while Al_2O_3 comes from aggregates and/or mineral additions in concrete and cement.

2.2. Design, mixtures preparation and testing of essential characteristics

For this research, the cement mixtures, in which 10% - 40% of clinker was replaced by recycled concrete powder (d_{90} < 200 μm), designated as PC F10 to PC F40, as well as the reference mixture – OPC, were prepared. Further, the cement mixture with the addition of RCP, limestone and slag (designation PC SFL), was also prepared. The grounded materials were mixed together according to the mixtures composition, which is presented in Table 2.

Designation	Clinker + Gypsum, (%)	F, (%)
OPC (Ref.)	100	-
PC F10	90	10
PC F15	85	15
PC F20	80	20
PC F25	75	25
PC F30	70	30
PC F35	65	35
PC F40	60	40
PC SFL	75	SFL= 25

Table 2. The composition of cement

The chemical composition of cement mixtures was determined according to standard SRPS EN 196-2:2008. Mineralogical composition was determined using D4 Bruker X-ray Diffractometer. Cement pastes, for determination of physical characteristics (consistence, setting times and soundness) were prepared in accordance with standard SRPS EN 196-3:2017. Mortar mixtures, with cement presented in Table 2, were also perepared for testing of the compressive strength. The weight ratio of cement to sand of 1:3 and the water-to-binder ratio of 0.5 were maintained. Mixing was performed in a bench-mounted mixer following standard SRPS EN 196-1:2017.

2.3. Concrete testing

The concrete specimens were prepared with the selected samples of Portland cement with recycled concrete powder (PC F20 and PC F 30). Concrete was prepared according to the following composition: cement content 320 kg/m^3, w/c ratio 0,55, granulometry curve of aggregate A32/B32 [9]. Additionally, two mixtures of concrete were prepared with PC SFL cement and different w/c ratio, as well as different aggregate. The mixtures are presented in Table 5.

The compressive strength of concrete, ordinary cured at 2, 7 and 28 days, was determined following the standard SRPS EN 12390-3:2019.

3. RESULTS AND DISCUSSION

3.1. Chemical composition and physical characteristics

Chemical composition and physical characteristics of Portland-cement with addition of recycled concrete powder is presented in Table 3.

	OPC	PC F10	PC F15	PC F20	PC F25	PC F30	PC F35	PC F40	PC SFL
Chemical composition									
LOI	1,80	1,99	2,27	3,05	3,18	3,61	4,00	3,82	4,58
SiO_2	21,2	27,9	27,4	31,0	33,1	35,4	36,2	41,6	27,41
Al_2O_3	5,11	5,26	5,23	5,36	5,46	5,53	5,51	5,64	4,97
Fe_2O_3	2,40	2,45	2,46	2,51	2,55	2,58	2,62	2,71	2,10
CaO	62,5	55,4	55,7	51,7	49,7	46,6	45,8	41,7	54,10
MgO	1,84	1,70	1,72	1,74	1,71	1,73	1,73	1,84	2,42
SO_3	3,64	3,40	3,64	3,13	2,83	3,05	3,05	2,48	3,38
Na_2O	0,22	0,31	0,31	0,38	0,42	0,45	0,45	0,50	0,23
K_2O	0,70	0,69	0,62	0,74	0,74	0,73	0,72	0,68	0,68
Cl^-	0,034	0,012	0,015	0,026	0,018	0,025	0,020	0,020	0,492
Mineralogical composition									
Alite C_3S	66,86	56,36	56,03	49,59	49,57	43,16	40,89	34,59	42,55
Belite C_2S	11,44	6,08	4,65	7,70	5,91	4,64	6,11	5,11	13,86
Aluminate C_3A	4,79	3,57	3,77	3,14	3,40	2,88	2,80	2,32	3,28
Ferrite C_4AF	8,80	9,21	8,39	6,85	6,02	5,38	5,5	5,11	6,21
Quartz	0,93	12,4	11,81	17,64	19,4	24,78	24,3	30,26	11,28
Physical characteristics									
Water for standard consistence, %	26,2	27,0	26,0	24,0	23,6	23,2	23,2	23,6	28,6
Soundness, mm	0,0	0,5	0,0	0,0	1,0	0,0	1,0	1,0	0,5

Table 3. Chemical and physical characteristics of Portland-cement with addition of RCP

Changes in the chemical composition with increasing of RCP are clearly visible, due to the increase of SiO_2 content, originating from aggregates from RCP, as well as decrease of CaO amount from clinker. Additionally, changes are noticeable in the alkali content (Na_2O). The content of all other oxides varies very little compared to Ordinary Portland cement.

The experimental results showed that the water for standard consistence has almost been unchanged when the RCP content is less than 15%. Water demand decreased with further addition of RCP and enhanced the workability, which is not in line with conclusions obtained in previous studies [1, 3] due to the origin and variations of RCP. It could be explained that RCP particles affect the hydration mechanism similarly to limestone [6]. Further, it was assumed that the results could be a consequence of more spherical shape of particles compared to those of OPC, which may act as lubricants.

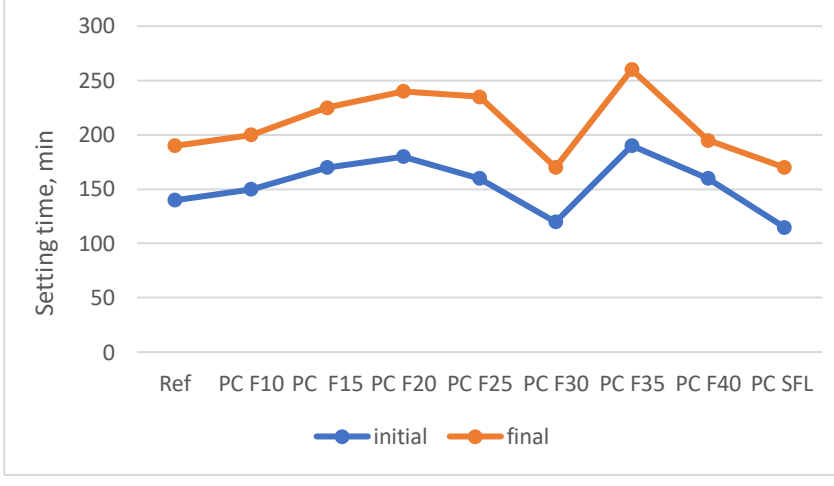


Fig. 1. Setting time of mortars with addition of recycled concrete powder

The effect of addition of RCP on the setting time of cement pastes is presented in the Fig. 1. The setting time of all cement pastes indicates that both the initial and final setting time tended to be longer with an increase of RCP content, except for the paste of 30% of RCP, where an acceleration was noticed. The most prolonged setting time was achieved with the PC F35 sample. Additionally, a small increase in the setting time of the cement paste with 40% of RCP compared to OPC, as well as a decrease compared to PC F35, has been determined. As the RCP could be considered as inert material [6], therefore, the nucleating effect of RCP will be lower, together with hydration of cement, which will prolong the setting time. Recent research results showed a tendency of RCP to reduce the setting time [3], as well as to prolong it, with the higher cement replacement level [10, 2] which depends on the variety of RCP.

Addition of RCP does not significantly influence the soundness of the pastes. Furthermore, all values comply with the limits set by Serbian standard for ordinary Portland cement (<10 mm).

3.2. Compressive strength of mortars

The effect of RCP addition on the compressive strength of mortars ordinary cured for 2 and 28 days, is shown in Table 4. In order to compare the compressive strength values of RCP mortars, the results of commercially available cement types containing other SCMs, produced by Lafarge BFC, were also presented in table.

The compressive strength results of all mortars with RCP addition meet the requirements of Serbian standards for strength class 32.5 (35-40% of F), as well as for 42.5 (10-30% of F). Furthermore, with a decrease in the clinker content, i.e. an increase in the RCP content, the strengths are slightly lower or almost in line with the results of commercial cement types. This was expected due to the preparation of RCP cement in laboratory conditions, on laboratory mills, where there were no particle separators, hence, the particle size distribution was wide, unlike commercially produced ones.

The addition of 10% - 20% of RCP results in a slight reduction in the compressive strength of mortars compared to the reference mortar. A small decrease in the compressive strength in these mixtures could be explained by the higher SiO_2 content, which forms a relatively higher content of hydration products. The reduction in compressive strength is more significant at higher RCP percentages. With the addition of 30% and 40% of RCP, the strength of mortar both decreases compared with the reference. This could be explained by the mortar's inability to form a compact, strong structure due to the excessive fine powder content in the RCP and low reactivity of RCP particles, which causes less formation of hydration products or due to a dilution effect (reduction in main reactive phases, C_2S and C_3S).

Designation of samples	Compressive strength, MPa	
	2 days	28 days
OPC (Ref.)	34.1	63.5
PC F10	25.5	55.8
PC F15	18.6	53.8
PC F20	22.4	51.5
CEM II/A	27.2	56.2
PC F25	19.7	46.9
PC F30	17.1	42.5
PC F35	16.0	41.1
CEM II/B	22.2	51.2
PC F40	16.0	39.1
CEM II/C	13.3	45.1
PC SFL	26.2	54.8

Table. 4. Compressive strength of mortars with addition of recycled concrete powder

The strength growth of RCP mortars at 2 days is lower, while the strength growth in 28 days is larger, compared with the reference mortar, which indicates that RCP reduces the early strength, but improves the long-term strength of mortars. At 2 days, the main contribution of RCP could be attributed to the filler effect. However, the early strength lowering is a consequence of the dilution effect, which surpassed the filler effect. The improving strength in the long-term can be assign to a higher content of SiO_2 and Al_2O_3 present in RCP, which may participate in pozzolanic reaction.

However, the performance of mechanical properties can be improved by adding other SCMs besides RCP, like GGBFS, FA and MK. In this research, a significant increase of compressive strength is achieved, for 17%, in the mortar with combination limestone and slag, beside RCP (PC SFL), compared to the mortar, with the same total clinker replacement of 25% (PC F25). According to authors [11], addition of these SCMs compensate for the negative effect caused by cement dilution, and it leads to forming C-S-H and C-A-S-H gels, in addition to pore refinement due to the filler effect.

3.3. Compressive strength of concrete

The prepared concrete mixtures with 320 kg of PC F20 and PC F30 cement have the same w/c ratio of 0.55, which confirms the conclusions reached in the tests of cement paste and mortar that there is no significant difference in the water demand with an increase in RCP content in the cement. Comparing concrete with only RCP cement addition, the results of the mechanical properties indicate a decrease in strength in the concrete mixture with a higher RCP content (i.e. PC F30) for all ages (2, 7, 28 days). Comparing concretes F20 and F30, the biggest decrease in compressive strength after 2 days is 20%, while after 28 days it is around 9%, The results of compressive strength of concrete are shown in Table 4.

Concrete designation	Cement, kg/m^3	Aggregate, kg/m^3	w/c ratio	Density, kg/m^3	Compressive strength, MPa		
					days		
					2	7	28
PC F20	320	1825	0.55	2300	29.8	37.5	46.9
PC F30	320	1794	0.55	2280	24.0	32.5	42.9
PC SFL 1	320	1835	0.50	2360	21.4	32.9	44.7
PC SFL 2	360	1826	0.45	2370	28.0	40.3	53.7

Table 5. Compressive strength of concrete prepared with RCP cement

On the other side, the addition of slag and limestone, along with RCP (cement PC SFL), leads to a reduction in the water demand, therefore, with the same amount of cement in the concrete, it is obtained w/c ratio of 0.5. It should be considered that a different aggregate was used for this mixture. A slightly lower strength of PC SFL 1 concrete mixture was achieved at the 2 days (about 10%), comparing to PC F30, while a slight increase in strength, by about 5%, after 28 days was obtained. All three concrete mixes prepared with 320 kg/m^3 of cement meet the criterion of strength class C30/37, which, indicates a quite good mechanical properties. With a further increase in the content of SFL cement up to 360 kg/m^3, the concrete meets the strength class of C40/50.

Having in mind that RCP can have mild pozzolanic properties, additional experiments need to be extended to prolonged ages, 56 and 90 days. Besides, further research will be directed on proving the durability of concrete made with cement in which part of the PC clinker is replaced by RCP, primarily on carbonation and frost resistance.

4. SERBIAN TECHNICAL ASSESSMENT

Portland cement with recycled concrete powder is an innovative construction product which is not covered by a technical regulation or a harmonized standard. Making available this product on the market of the Republic of Serbia and obtaining a document of conformity is possible only through the preparation of the Serbian assessment document and issuing the Serbian technical assessment. Having such a goal, the Working Group of the Technical Assessment Body of IMS Institute reviewed the aforementioned research results, as well as the use of this innovative product, and prepared document SDO 15003:2025.

The composition of Portland cement, which includes cements with addition of recycled concrete powder up to 35% (PC 35 F / CEM II/B-F), as well as Portland composite cements, is covered with this document. Further, the document includes a restriction on the use of more SCM and that the proportion of the sum of limestone and recycled concrete fines (sum of L, LL and F) is limited to 35 % by mass. Additionally, the number of main constituents, other than clinker, is limited to two and in case of the use of both F and L or LL in the composition, the number of main constituents other than clinker is limited to three, whereby the sum of L, LL and F can be a maximum of 29% by mass.

Despite the potential benefits of using RCP as SCM, variability in the RCP characteristics, due to variations in the source and processing, is very important issue and should be predicted. Therefore, the Serbian assessment document prescribes the condition that recycled concrete powder has to fulfill, and that is through the fulfillment of the requirement of the proportion of constituents of large coarse concrete aggregates. The following requirements for the recycled concrete powder are included in SDO 15 003:2025:

- The content of total organic carbon (TOC), determined according to the method described in the standard SRPS EN 13639, shall not exceed 0.8%;
- The sulfate content (expressed as SO_3), determined according to the method described in the SRPS EN 196-2 standard, shall not exceed 2.0% and
- The clay content, determined by the methylene blue test in accordance with the SRPS EN 933-9 standard, shall not exceed 1.20 g/100 g. For this test, the recycled concrete fines shall be ground to a fineness of approximately 5,000 cm^2/g, determined as specific surface area in accordance with SRPS EN 196-6.

The essential characteristics of Portland cement with recycled concrete powder (Compressive strength, Setting time, Volume stability (expansion), Sulfate, Chloride and Alkali content, Cement composition), as well as performance evaluation criteria, intended use, labeling and packaging, procedure for assessment and verification of constancy of performance were included in the document.

On the basis of prepared Serbian assessment document, technical assessment body will perform the assessment of the performance by testing, inspection of production process and issue the Serbian Technical Assessment, which offers construction product manufacturers a path to the Serbian conformity mark i.e. allows assessment of performance of construction product and drawing up a declaration of performance.

5. CONCLUSION

In this research, the possibility of utilization of RCP as SCM was assessed through the chemical composition, physical characteristics and mechanical properties. The experimental results indicate that RCP increase cement workability and prolong both the initial and final setting time. All compressive strength values comply with the limits set by Serbian standard for ordinary Portland cement. RCP reduces the early strength but improves the long-term strength of mortars. The reduction in compressive strength was smaller for smaller replacement levels. The performance of mechanical properties can be improved by adding other SCM (slag and limestone), as confirmed through research of mechanical characteristics of mortar and concrete.

However, the results showed that the recycled concrete powder can be used as a supplementary cementitious material up to 40% of addition, related to the cement type and characteristics that are wanted to be achieved. Therefore, based on the results of the research, the Serbian assessment document of this innovative product was prepared, including the most important aspects of using the product as well as the conformity assessment procedure. This provided the basis for the Serbian technical assessment and the possibility to make available this product on the Serbian market.

Acknowledgments:

This research is supported by the Ministry of Science, Technological Development and Innovation through Contract No. 451-03-136/2025-03/ 200012.

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IN MEMORIAM



Prof. dr Dimitrije Zakić rođen je 12.11.1967. godine u Beogradu. Osnovne studije je završio 1994. godine na Odseku za konstrukcije Građevinskog fakulteta Univerziteta u Beogradu. Poslediplomsku nastavu na istom Odseku (Smer za građevinske materijale), upisao je 1995. godine. Prof. Zakić je 2001. godine magistrirao, a 2010. godine doktorirao na Građevinskom fakultetu Univerziteta u Beogradu.

Tokom godina aktivno je učestvovao u izvođenju nastave na grupi predmeta građevinski materijali i tehnologija betona. Pored rada u nastavi na Građevinskom fakultetu Univerziteta u Beogradu, prof. Zakić je u periodu između 1997. i 1999. godine držao nastavu na Arhitektonsko-građevinskom fakultetu u Banjoj Luci. Boravio je na naučnom i stručnom usavršavanju na Tehničkom univerzitetu u Berlinu, Ilinoj univerzitetu u Čikagu, Kineskoj akademiji za građevinske materijale u Pekingu i institutu IRMA u Ljubljani.

Učestvovao je na više domaćih i stranih naučno-istraživačkih projekata. Autor je ili koautor preko 170 naučnih radova, saopštenja i poglavlja u monografijama, kao i publikacija iz domena udžbeničke literature.

Tokom karijere, učestvovao je u izradi preko 300 stručnih radova, ekspertiza, elaborata i projekata. Prof. Zakić je bio tehnički rukovodilac Laboratorije za materijale.

Prof. dr Dimitrije Zakić aktivno je učestvovao i u radu stručnih udruženja. Dugogodišnji je član Inženjerske komore Srbije, Akreditacionog tela Srbije, Komisije za standarde Instituta za standardizaciju Srbije, brojnih radnih grupa pri Ministarstvu, brojnih stručnih udruženja.

Od početka karijere prof. Zakić je aktivno učestvovao u radu Društva za ispitivanje materijala i konstrukcija (DIMK) Srbije. Bio je dugogodišnji član Skupštine, a zatim i predsednik Društva od 2023. godine. Preminuo je 21. februara 2025. godine kao aktuelni predsednik Društva. Ostaće upamćen po svom stručnom radu i poštovanju kojim se odnosio prema saradnicima. Verujemo da će njegovo nasleđe živeti kroz sve istraživače koji su imali priliku da od njega uče i sa njim sarađuju.

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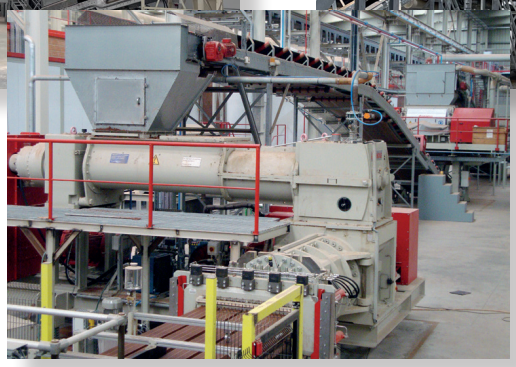


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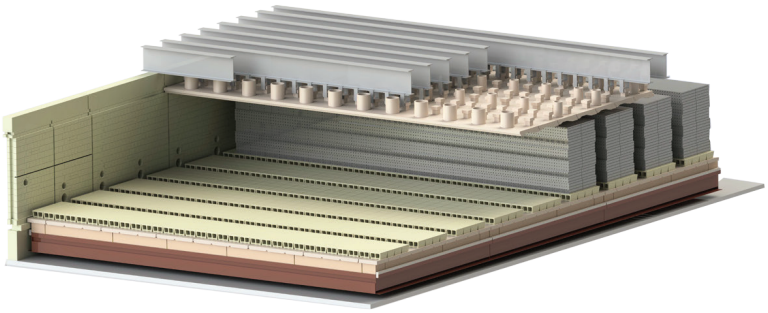


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Here, high-performance systems for the ceramics industry are developed and tested. Individual solutions in the material characteristics originate here.

Refratechnik Ceramics develops innovative refractory systems that are closely oriented to the technical and economic requirements of the ceramics industry.

In order to provide individual solutions, we develop new formulas, determine maximum operating temperatures, test parameters such cold pressure and cold bending strength, refractoriness, hot bending strength, reversible thermal expansion and the thermal conductivity of the refractory products.

When it comes to the redesign of systems, the intensive knowledge of the operating conditions and the objectives of the kiln operators are the key to a tailor-made offer.

Reduced energy consumption and lower CO_2 emissions are central topics in the ceramic industry. The selection of the most suitable tunnel kiln car systems enables energy efficiency to be improved significantly. This not only reduces fuel costs, but also subsequential costs such as the purchase of emission trading certificates.

You can reduce your energy costs by corresponding designs and by material selection. Such highly energy saving products are our innovation REFRATHERM® Eco as well as our BURCOLIGHT® material.

Innovation

REFRATHERM® Eco: High-end insulation for the tunnel kiln car

Biogenic silica (SiO_2) is produced through the burning of biomass, the ashes of which mostly contain SiO_2. REFRATHERM® Eco is thus based on a naturally renewable resource.

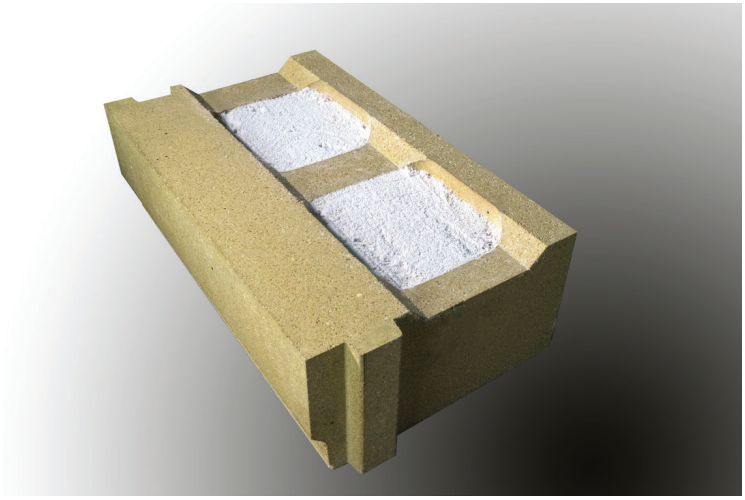
Production

The term biogenic silica shows the origin of the product. In modern biomass power plants, subject to quality controls, these raw materials are used as an energy source because they burn CO_2 neutrally.

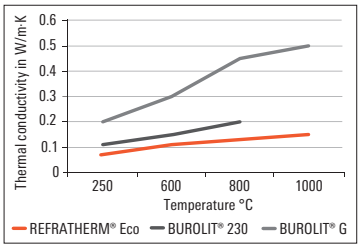
That is, when burned, no more CO_2 is released than the plant had previously absorbed from the atmosphere. An aspect which is becoming increasingly important for the raw materials of the future.

Characteristics

Biogenic silica is chemically very pure and consists of 94-96% SiO_2. The main phases after a special burning process are amorphous silica, tridymite and cristobalite.



Border brick forTunnel kiln car with REFRATHERM® Eco insulation



Comparison of insulation effect

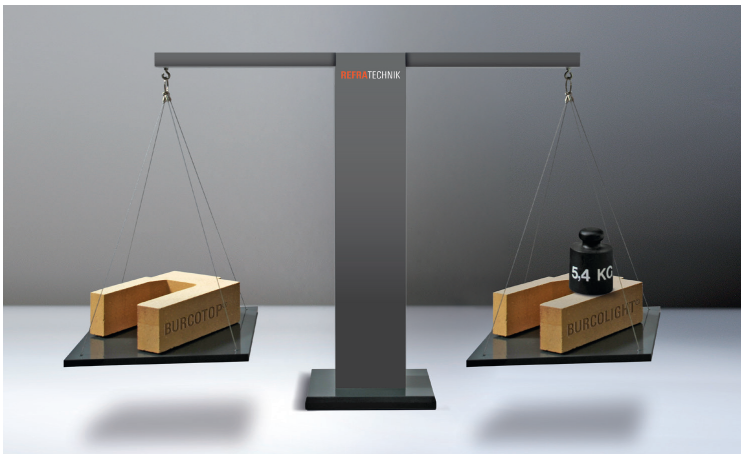
This causes the high melting point of approx. 1650 °C. In production, the volatiles burn but the microporous structure of the SiO_2 is maintained. This structure results in both the extremely low thermal conductivity as well as the low bulk weight.

REFRATHERM® Eco based on microporous silica shows a unique combination of well-defined characteristics such as high insulation, good flow properties, set grain structure. This allows for very low installation heights and thus more usable firing capacity for the brick manufacturer.

The installation time of the insulating layer in the tunnel kiln car can be reduced by about 50% depending on the flow properties set.

- Sintering point: 1550 °C
- Melting point: 1650 °C
- High microporosity and thus excellent insulation
- Perfectly optimised flow properties
- Low density
- Extreme low thermal conductivity

BURCOLIGHT® – Energy saving by weight reduction



Kiln car systems equipped with BURCOLIGHT® bricks achieve energy saving by means of weight reduction. The less mass to be heated, the more energy is saved.

When used for kiln cars, BURCOLIGHT® brick types with a bulk density of ~1.3 to 1.5 kg/dm³ achieve energy savings up to 30% compared with BURCOTOP® brick types (depending on kiln car type and tunnel kiln temperature). Thanks to these outstanding properties, the total energy savings of the kiln are between 8 and 12%.

BURCOLIGHT® features low mass, excellent thermal shock resistance and low thermal conductivity.

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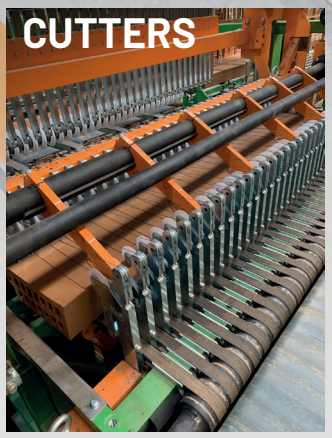
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WE DESIGN All works carried out by Equipceramic have been conceived and designed by our Engineering Department by applying the latest 3D design technology, backed up by more than 40 years of experience and know-how in the field all over the world.

WE MANUFACTURE Once the project has been designed, our manufacturing team brings it to reality in our workshops where the strictest quality control processes are established to maintain Equipceramic's high/ demanding excellence standards.

WE COMMISSION Our commissioning team travels to the project site, always being in constant communication with our Engineering and Manufacturing Department, closely coordinating works to have equipment fully operational in the shortest possible time.



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We leverage advanced technologies to enhance the efficiency of Brick Industry.



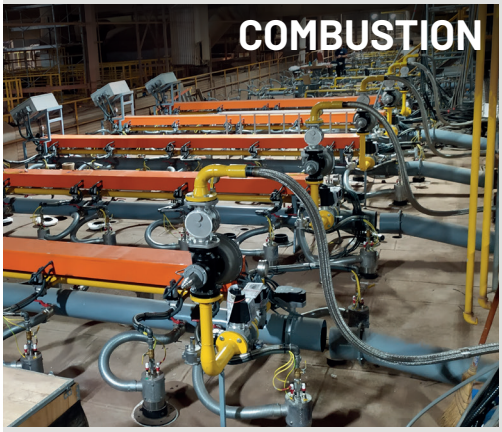


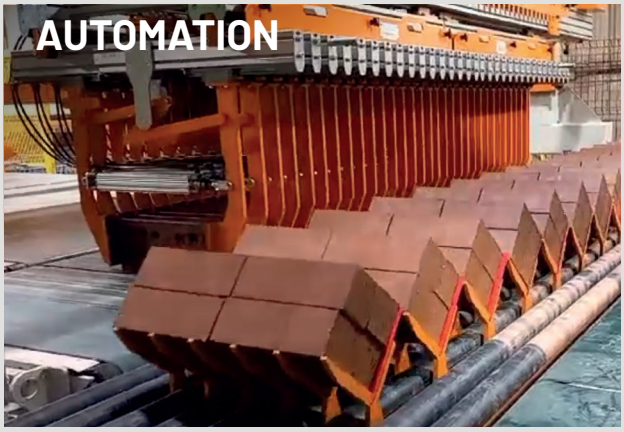
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- ***Process control***

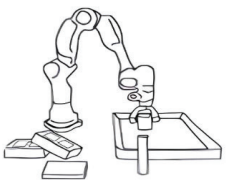


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Forterra Desford plant official opening

On the 17th of May 2023, Europe's biggest and most efficient brick factory with turn-key supply from Bedeschi officially opened. The opening ceremony was held in Leicestershire with the presence of investors, journalists, industry experts, suppliers and customers.



Bedeschi and Forterra team during the Desford grand opening

Forterra invested about 95 million £ for the new Desford plant to increase the production capacity of the old plant, improve its efficiency and increase its sustainability.
The new factory is now the largest brick factory in Europe, a model of efficiency and sustainability with state-of-the-art automated processes designed to minimize the environmental impact of the production line. Bedeschi is extremely proud to have been part of this project as the company itself is focusing each year even more on environmental protection and sustainability. In fact, the company has established a strategic long-term sustainability plan, which, in 2022, led to the first sustainability report for which the company is committed to measuring its performance regularly. Through this report, the company wants to illustrate how sustainability is a key element of its business model. The company has set itself the target of reaching the goal of carbon neutrality by 2025.

Inside Forterra's plant, at the heart of the production line, there are two new kilns, designed according to sustainable principles to minimize energy consumption and CO2 emissions.

The main benefit of this kiln is its greater thermal efficiency, which comes from the innovative kiln design, leading to a greater emphasis on whole-life costings and greater energy savings. Additionally, more modern control hardware and software methodologies allow for tighter control over the kiln conditions, creating the most efficient use of fuels for the process.



Bedeschi innovative kiln

Modern kilns consume less gas and have more effective emissions-scrubbing technology, reducing the emission of carbon dioxide as well as other harmful waste gases. Supplementing this, the plant will incorporate a tunnel dryer, which is superior in its thermal efficiency compared to the more common chamber dryer that was in place at the previous site and many other UK brick plants.



Bricks production in Desford

The plant has been designed and built on five key principles:

- Lights Off Factory – Minimal human intervention is required in standard operating.
- Smart Automation – Leveraging technology to improve quality and efficiency.
- Quick Change Maintenance - Targeting low time for equipment downtime.
- Removal of Single Point of Failure – To have a truly resilient facility that can continue production in the event of system failures.
- Clean Room Manufacturing– Creating a safe, clean working environment.

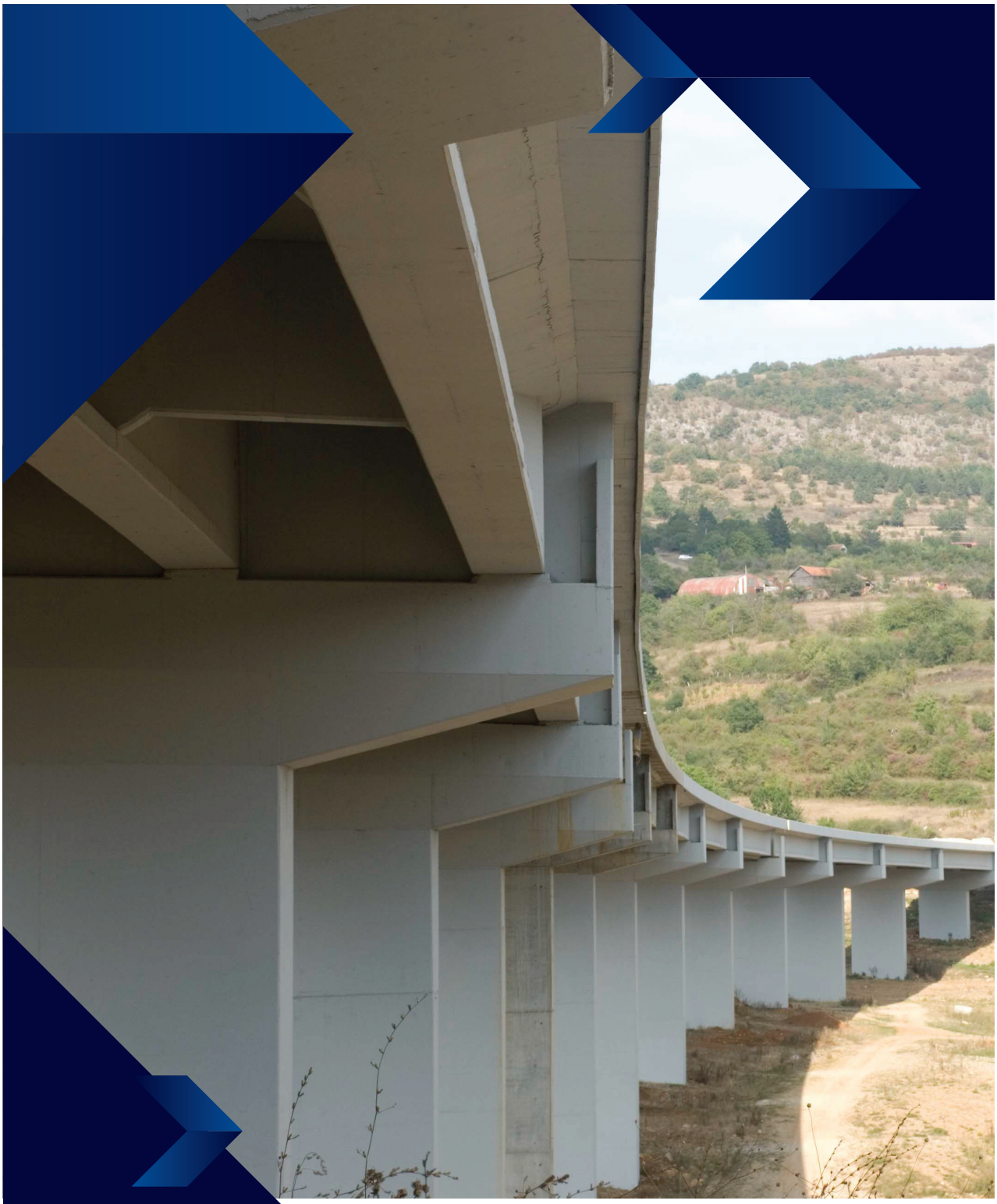
Bedeschi's supply includes state-of-the-art machines to optimize production processes to support customer demands while reducing environmental impact. Thanks to the machines supplied by the company, the bricks produced at Desford have a carbon footprint that is about 25% lower than the old factory.

With more than a hundred years of experience in building turnkey plants for other of the world's leading brickmaking clients, Bedeschi proved to be the ideal partner for the construction of this factory. Thanks to important acquisitions finalized in the automation sector, the Limena-based company is in fact able to offer fully integrated solutions manufactured in its two Italian plants in Padua and Bergamo.

Bedeschi is proud to have contributed to this success, a historical milestone in the panorama of bricks production.



Bedeschi robotic handling systems



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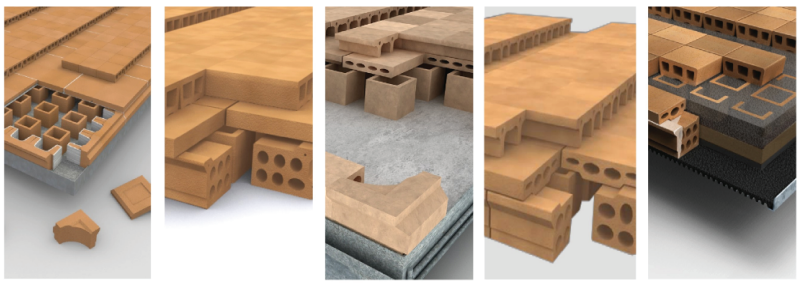
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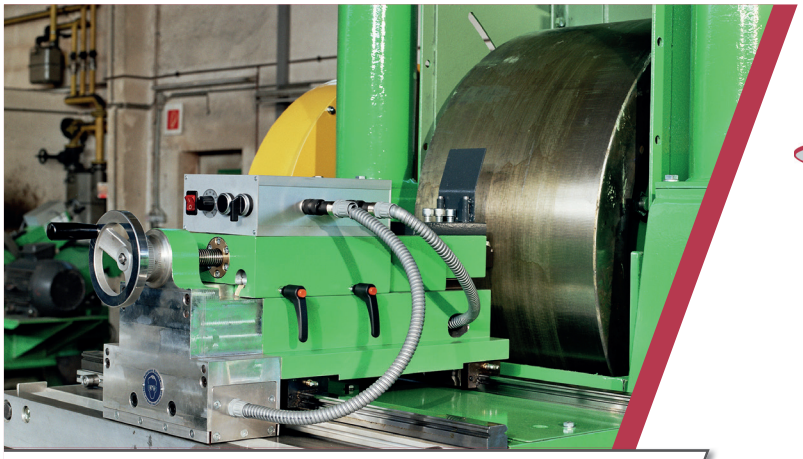
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  The preloaded linear guide ensures high precision and backlash-free movement. A uniform turning pattern is ensured.
- **Innovative drive system:**
  A servo motor and the backlash-free belt drive ensure low-vibration operation. The feed rate can be quickly adjusted during the turning process and the support can be positioned just as quickly and easily using a fast feed.
- **Pneumatic raising/lowering:**
  Pneumatic raising/lowering is available for the roll lathe as an option. This allows the roll lathe to remain on the roller mill during roller operation, eliminating setup times.

PRECISELY FITTING

ROLLER SHELLS

In addition to our state-of-the-art roll lathes, we also offer roller shells for all manufacturers and types. These are supplied ready for installation and offered in various qualities:

- Alloyed chilled cast iron with a hardness of 550 and 600 HV
- Alloyed chilled nodular cast iron with a hardness of 500 HV, especially with a small roller gap
- Alloyed bainitic nodular cast iron with a hardness of 500 HV, especially with high impact loads and a small roller gap
- Chrome steel roller shells with a continuous hardness of 58–60 HRc
- Manganese steel roller shells

You also receive complete service for your roller shell: roller replacement, bearing replacement, adjustment of bearing play and help with turning from our experienced service personnel.



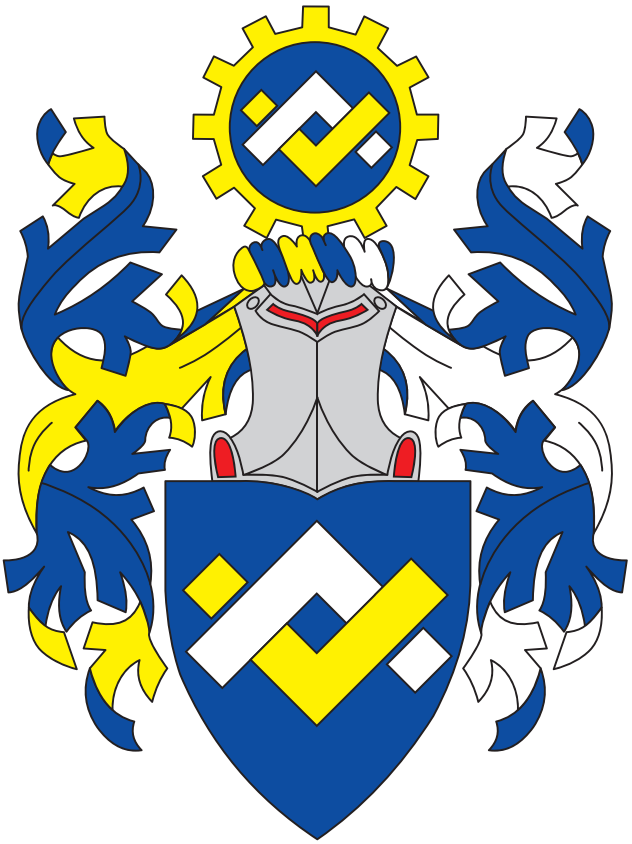
GRADIMO SADAŠNJOST ZA ODRŽIVU **BUDUĆNOST**



Strastvenost, timski duh i vizija budućnosti. Mapei aktivno doprinosi najvažnijim svetskim projektima u oblasti arhitekture, infrastrukture i stanovanja, kao i obnovi kulturno-istorijskih građevina. S predanošću radimo svakog dana, kako bismo oblikovali održivu budućnost građevinske industrije.

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