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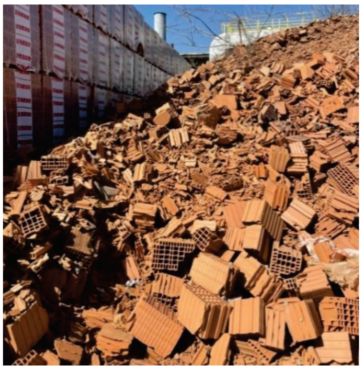


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Cover **Ceramic masonry blocks waste, before and after grinding (up), and Mixture of wheat straw, sunflower husk and silo waste ash, before and after grinding (down), both** from *Characterization of harvest residues ashes and ceramic waste powders originating from Vojvodina as potential supplementary cementitious materials* by S. Šupić, M. Malešev, V. Radonjanin, V. Pantić, I. Lukić and V. Bulatović

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U znak sećanja na dr Radomira Vasića

Dr. Radomir Vasić, dipl.inž. rođen je 31.08.1944. godine u Krnjevu. Preminuo je 15.04.2022 u Beogradu. Osnovnu školu, gimnaziju i Tehnološki fakultet pohađao je u Beogradu, gde je 03.03.1969. godine i diplomirao na Katedri za Neorgansku hemijsku tehnologiju. Po diplomiranju jedno kratko vreme je radio u Institutu za primenu nuklearne energije u poljoprivredi i šumarstvu, da bi iste, 1969.godine, prešao u Institut za hemiju, tehnologiju i metalurgiju u Beogradu. U IHTM-u je radio u Odseku za nemetale i Centru za hemijske izvore struje. Od 1977. godine je bio zaposlen u Institutu IMS (raniji naziv Institut za ispitivanje materijala Republike Srbije), gde je radio na poslovima predsednika Naučnog veća IMS Instituta i direktora odeljenja za građevinku keramiku.

Magistarsku tezu pod naslovom: "Kinetika reakcija nastajanja jedinjenja $CaSnO_3$ i Ca_2SnO_4 u sistemtu $CaO – SnO_2$," odbranio je 26.02.1914.godine.

Doktorsku disertaciju pod naslovom: "Prilog proučavanju fenomena vlažnog bubrenja fasadnih opeka proizvedenih od domaćih opekarskih glina" odbranio je dana 24.12.1990. godine na katedri za neorgansku hemijsku tehnologiju na Tehnološko-metalurškom fakultetu u Beogradu.

U svom radnom veku radio je na istraživačkim i razvojnim poslovima iz oblasti keramike za elektroniku, presovanim i sinterovanim elektrodama za hemijske izvore struje, elektrohemijskoj i katodnoj zaštiti kao i na istraživanju novih i unapređenju postojećih materijala i proizvoda iz oblasti građevinske keramike. Govorio je engleski i francuski jezik.

Stručni ispit propisan za diplomiranog inženjera tehnologije položio je 1982.godine. U naučno zvanje asistent (po tada važećem zakonu) izabran je 05.07.1982.godine, a u zvanje istraživač saradnik 13.05.1985. godine. U naučno zvanje naučni saradnik izabran je 16.05.1991. godine a u naučno zvanje viši naučni saradnik 19.06.1996. godine. Zvanje naučni savetnik stekao je 15.09.1999.godine.

Za svoj dugogodišnji rad u struci i za ostvarene naučno-istraživačke rezultate i za zalaganje na radu dobio je više povelja i nagrada:

- Nagradu instituta za ciglu i crep iz ESSENA za najizraženiji naučni doprinos na polju vlažnog bubrenja u prethodnih 40 godina 2015. godine.,
- Nagradu British Royal Academije iz 2015 godine za rad pod nazivom „Phenomenon of moisture expansion and its influence on degradation, realibility and durability of heavy clay products“. Rad je sledeće godine publikovan u specijalnom izdanju British Royal Academije pod nazivom „Most important papers related to the phenomena of moisture expansion“
- Nagrade Instituta za ispitivanje materijala SR Srbije za publikovane naučno-istraživačke radove u 1986 godini,
- Povelje zaslužnog člana Jugoslovenskog društva za istraživanje i ispitivanje materijala i konstrukcija JUDIMK-a 1986.godine.
- Nagrade Instituta IMS za uspešan naučno-istraživački rad u zemlji i inostranstvu u 1991 godini,
- Povelje Instituta IMS za izrazito zalaganje u radu 1992 godini,
- Nagrade Instittuta IMS za uspelan naučno-istraživački rad u Institutu u 1994 godini,
- Nagrade Instituta IMS za izrazito zalaganje u radru 1994 godini,
- Nagrade Instituta IMS za uspešan naučno-istraživački i stvaralački rad u Institutu u 1996 godini,
- Povelje za izvanredne rezultate u radu u Institutu *za* hemijske izvore struje u 1996 godini,
- Nagrade Instituta IMS za uspešnu organizaciju naučnog skupa Fasadni zidovi od opeka, u 1991 godini.

U svojoj dugogodišnjoj karijeri samostalno ili u saradnji sa drugim autorima objavio je 4 monografije nacionalnog značaja, 7 poglavlja u knjigama i dva pregledna rada u inostranim časopisima. Objavio je i saopštio 160 radova (od čega je 20 radova objavio u časopisima međunarodnog značaja a 35 radova u domaćim časopisima, 45 radova je saopštio na međunarodnim skupovima a 60 na domaćim naučnim i stručnim skupovima).

Vodio je računa o razoju mladih naučnih kadrova. Bio je mentor pri izradi jedne magistarske teze i dve doktorske teze. Precedavao je u dva izorna kruga Naučnim većem IMS Instituta a bio je i član naučnog veća u HK "Zorka" - Šabac - "Centar za istraživanje". Bio je član upravnog odbora SHD Sekcije za keramiku, član nadzornog odbora i koordinator sekcije za sanitarnu i upotrebnu keramiku jugoslovenskog keramičkog društva. Takođe je bio član Evropskog udruženja keramičara. Bio je dugogodišnji rukovodilac laboratorije za građevinsku keramiku veoma ponosan na činjenicu da je ista bila druga po redu laboratorija u staroj Jugoslaviji koja je bila akreditovana u YUAL a kasnije YUAT (ATS) sistemu. Bio je član borda direktora CERLabs-a organizacie koja je u periodu od 1995 do 2016 okupljala 24 nacionalnale keramičke laboratorije iz najrazvijenijih zemalja evrope i sveta. Bio je član međunarodne komisije za ocenjivanje rada laboratorija za grubu građevinsku keramiku. Kao priznati stručnjak, bio je član naučno tehničkog odbora SIMSER konferencije koja je održana u Bolonji 1998 godine.

Uspostavio je međunarodnu saradanju sa preko dvadeset priznatih naučnih institucija iz Evrope i sveta. Zahvaljujući toj saradnji Laboratorija za građevinsku keramiku Instituta IMS, kojom je uspešno rukovodio preko 30 godina i pored sankcija UN-a i evropskih zemalja i je bila osnivač i članica CERLabs-a, a naša keramička industrija i naučna javnost nisu ostali uskraćeni za informacije o savremenim trendovima i regulativi u oblasti građevinske keramike. Imao je izvanrednu saradnju sa sledećim naučnicima i naučnim organizacijama: Sa prof. C. Palmonari-em i prof. I. Stamenkovićem iz "Centro Ceramico"- Bolonja, Italija; sa Acad. Shewchenk-om i prof. S. Barinov-im iz "High-Tech. Ceramics Scientific Research Centre", Moskva, Rusija; sa prof.F.W. Col-om i R. Bowman-om iz "CSIRO-a", Highett - Meburn iz Australije, sa prof. F. Labbidi-jem iz "Centre Technique des Materiaux de Construction", de la Ceramique et du Verre - Tunis iz Tunisa; sa prof. Freer-om iz "The Institute fol Testing Materials" iz Mandestera , Velika Britanija.

Učestvovao je na sedam naučnih projekata od kojih je dva samostalno vodio.

1. Strateški projekat T 36: "Istraživanje tehnoloških prstupaka savremene tehničke keramike", finasiran od strane Saveznog Fonda za Nauku SFRJ", (1969-1974). U okviru podprojekata rukovodio je temama: "Višeslojni

keramički kondezatori" (faze I i II) i "Opelemenjivanje površine fasadnog elementa nanošenjem obojenog sloja" (faze I, II i III).

2. Strateški projekat T 68: "Istraživanje i razvoj proizvodnje građevinskih materijala na bazi domaćih primarnih i sekundarnih sirovina." U okviru podprojekta: "Unapređenje postojećih i osvajanje novih građevinskih materijala" rukovodio je temama:

- "Eksperimentalna istraživanja u cilju određivanja optimalnih fizičko-hemiskih i mehaničkih osobina keramičkih pločica namenjenih za industrijske podove" - (1981-1985), Ugovor 445-5 od 06.11.1981. sa RZNS (Faze: I, II, III, IV i V).
- "Izučavanje fenomena vlažnog bubrenja opekarskih proizvoda", (P-1/-5), Ugovor br. 3-511/1 od10.07.1986 sa RZNS (Faze: I, II i III).
- "Primena tufova, pucolana i drugih vulkanogenih sedimenata u keramičkoj industriji." (P-1/-5), Ugovor br. 351,1 od 10.07.1986 sa RZNS (Faze I i II).

3. Nacionalni Projekat 02830: "Eksperimentalna istraživanja i modelovanje klasičnih i alterativnih operacija u naftno-petrohemijskoj i procesnoj industriji". Ugovor br.1395-512 od 30.04.1996 god. sa Ministarstvom za nauku i tehnologiju Republike Srbije. U okviru podprojekta: "Eksperimentalna istraživanja i modelovanje procesa u oblasti korozije silikatnih materijala" rukovodio je temom pod istim nazivom (Fazama: I, II, III, IV i V).

4. Nacionalni Projekat 02841: "Izučavanje Fenomena i procesa u nemetalnim materijalima". Ugovor br.1395-5/-5 od 30.04.1996 god. sa Ministarstvom za nauku i tehnologiju Republike Srbije. U okviru podprojekta: "Procesi u čvrstim silikatnim sistemima" rukovodio je temom: "Kinetika i mehanizam fenomena vlažnog širenja poroznih keramičkih materijala" (Faze: I, II, III, IV i V).

5. Strateški Projekat S.-5.31.62.0101: "Osvajanje savremenih tehnologija procesa i inoviranje industrijskih postrojenja u proizvodnji grđevinskih materijala". Ugovor br. 670-01-4/9836 od 26.01.1998 god. sa Ministrstvom za nauku i tehnologiju Republike Srbije. Bio je rukovodilac potprojekata 6: "Savremeni kompozitni i novi materijali u građevinarstvu poboljšanih karakteristika kvaliteta" i teme: "sinterovane opeke na bazi gline i letećeg pepela" (Faze I, II i III).

6. Rukovodilac nacionalog projekata TD – 7024 B: "Istraživanje, razvoj i primena metoda i postupaka, ispitivanja, kontrolisanja i sertifikacije proizvoda i procesa u skladu sa zahtevima međunarodnih standarda i propisa". Projekat je finansiralo Ministarstvo za nauku i tehnologiju Republike Srbije u periodu od 2005-2007 god.

7. Rukovodilac nacionalog projekta TR 19017: "Istraživanje, razvoj i primena metoda i postupaka ispitivanja, kontrolisanja i sertifikacije nemetaličnih građevinskih proizvoda, otpadnih materijala i upravljanje rizikom u skladu sa međunarodnim standardima". Projekat je finansiralo Ministarstvo za nauku i tehnologiju Republike Srbije u periodu od 2008-2010 god.

Najveći broj radova vezani su za fenomen vlažnog širenja poroznih keramičkih materijala. Zbog velikih razmera i štetnih posledica ovog fenomena u građevinskoj praksi, ovaj fenomen je u žiži interesovanja mnogih istraživača širom sveta već više od 60 godina. Ova oblast fundamentalnih i primenjenih istraživanja je oblast u kojoj je dr.Vasić dao značajan doprinos boljem sagledavanju uzroka i mehanizama odvijanja ovog fenomena. Kao rezultat njegovih istraživanja proistekla su i određena praktična rešenja primenjena prilikom sanacije više stambenih objekata i čitavih stambenih naselja u Srbiji i Jugoslaviji. Drugu grupu radova čine radovi koji se odnose na fundamentalna i primenjena istraživanja u oblasti keramičkih poroznih materijala. Tu se naročito izdvajaju rezultati istraživanja na primeni otpadnih keramičkih materijala u cilju dobijanja novih gradjevinskih materijala; suvih maltera, hidraulidnih veziva, blokova itd. Treću grupu radova čine radovi vezani za oblast korozione zaštite i elektrohemijske zaštite građevinskih materijala i objekata i unapređenja upotrebnih karakteristika metalokeramičkih inženjerskih materijala. Ovi radovi su rezultat primenjenih istraživanja korozionih procesa. Multidisciplinarni pristup rešavanja problema korozije i korozionih procesa u građevinskoj praksi, predstavljen u ovim radovima našao je i svoju praktičnu primenu. Određeni broj radova tretirao je problematiku tehničke regulative i ispitnih metoda.

Tehnička rešenja rešenja : br.370-5 od 14.02.1996 i br.409-5 od 19.02.1996 i danas se primenjuju u HK "Zorka" Sabac

Za života je govorio da sve čoveku može da se oduzme, nestane ili propadne sem onoga što nosi u glavi. Pored nagrada koje je dobio 2015 godine od strane Esen instituta i British Royal academije kao najveće dostignuće u karijeri mu je bio susret sa dr Colom na kongresu u Australiji. Nakon izlaganja rada prišao mu je profesor Col i rekao: "Kolega Vasiću vi imate tempiranu bombu u zidu". Nakon toga je isti priofesor otišao i vratio se za nekih 40 min. Rekao mu je da je njegove rezultate poslao telefonom u Melburn na proveru. Sutradan za doručkom mu je ushićeno prišao i predočio da su rezultati tačni. Radomir je bio zbunjen a Col je nastavio: "Prema jednačini koju je institut u kome radim razvio na bazi 70 godina merenja vlažnog bubrenja vaši rezultati merenja su identični sa našim prognoznim vrednostima". Takođe je voleo da napomene da je on dvadeset godina pre opšte upotrebe izraza korozioni proces kod keramike prvi o tome pisao. Kao njegov sin više puta sam čuo ovu konstataciju kao i naslov nekog profesora iz Londona koji je to i napisao u nekom svom preglednom radu. Nažalost nisam zapamtio kako se zvao taj profesor i u kom radu je to naznačeno. Pored toga što je bio istaknuti naučnik i stručnjak bio je izvanredan otac. Iza sebe je ostavio suprugu i dva sina.

Beograd, 29.11.2022. godine

Dr Miloš Vasić
Radomirov sin

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Original scientific paper

Study on the performance of GFRP strengthened, fiber reinforced lightweight foam concrete

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Foam concrete prism;
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ABSTRACT

Regular clay bricks and concrete blocks are replaced with light-weight fibre-reinforced foam concrete modules. For light weight foam concrete, various natural and synthetic fibes are employed as micro- and macro-fibre reinforcement. Three distinct fibres were used as fibre reinforcement in this study, and their strength qualities were investigated. As microfibre reinforcement, synthetic-polypropylene fibre, natural-Jute fibre, and banana micro fibres were used at volume fractions ranging from 0.22 to 0.55 percent in the foam concrete mix. The compression behaviour of stack bonded masonry prisms was investigated in the first phase of the experiment. The second phase of research focused on the microfibre-reinforced prism, which was reinforced with multiple layers of GFRP sheets. Both jute and banana fibres added as microfiber reinforcement to the matrix, impart ductility to the brittle masonry unit and reduce the sudden failure mode of the Fibre-Reinforced Lightweight Foam Concrete (FRLWC) prism. The insertion of GFRP sheets between the masonry layers provides additional stiffness and ductility to the FRLWC masonry prism, which greatly improves the post-cracking behaviour. When compared to a standard LWC prism, failure patterns show that both synthetic and natural fibre-reinforcement provide improved fracture bridging mechanisms, which is mostly owing to the arresting of cracks by micro polypropylene, jute, and banana fibres. The GFRP layers provided between the masonry units prevented the formation of major crack planes.

1 Introduction

Lightweight structural concrete (LWC) is gaining popularity in recent days, as it significantly reduces the dead weight of masonry construction. Lightweight concrete with fiber reinforcement improves the tensile and shear strength, that imparts ductility to the brittle masonry structure, which in turn helps to improve the structural seismic behaviour. LWC are gaining popularity not only in the regular construction industry but also in the offshore and prefabricated construction industries [1]. Smaller and lighter prefabricated structural members are preferred in the offshore industry because they are easier to tow, handle, and transport. Along with the need for lightweight concrete, the concept of sustainability in concrete is becoming increasingly important. A large quantity of byproducts, such as fly ash and blast furnace slag, are produced and disposed of in the surrounding environment, resulting in pollution. Fly ash and blast furnace slag are now used as a partial replacement for ordinary Portland cement and aggregate in the production of sustainable light weight concrete [2]. Foam concrete is low density concrete which has better sound insulation and thermal resistance but less mechanical and durability characteristics due to the porosity of the foam paste. The addition of silica fumes and basalt fiber helps to improve the pore network and enhance the fiber paste matrix, which in turn increases the mechanical and durability performance of lightweight fiber reinforced foam concrete [3]. Researchers have proposed strengthening the concrete matrix with short discrete natural and synthetic fibres in the cement matrix, fibre sheets, external fibre wrapping, and fibre composites to overcome such failure [4],[5]. Microfibers due to their flexible nature, varied cross section, random orientation of fibers in the matrix and high aspect ratio can be effectively used as fiber reinforcement in concrete element, mortars and other polymer composites. The uniform distribution of fibers in different directions not only prevents microcrack formation in the elastic region, but also improves the post cracking performance of concrete and composites. Many natural fibers such as jute, kenaf, roselle, hemp, basalt, bamboo, banana, palm, coconut, sisal, etc., are also used as fiber reinforcement in concrete. Synthetic fibers such as

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polypropylene (PP), polyvinyl alcohol (PVA), acrylic, polyolefin, and polyterephthalate, etc., are used in concrete as microfiber reinforcement [6]. Despite the fact that synthetic fibres outperform plant-based natural fibres in terms of enhancing the tensile properties of concrete, the high cost of synthetic fibres drives up project costs. As a result, plant fibres are favoured, as they are low-cost, readily available, and entirely regenerative, resulting in a long-term sustainable concrete solution [7]. Many studies have recently been conducted to investigate the performance of natural and synthetic fibres in lightweight concrete [8, 9]. This section highlights a few of the research findings. Studies on the fresh properties and modulus of elasticity of carbon and Polypropylene hybrid fibers reinforced foam concrete showed that addition of 1.5 % carbon fibers increased the modulus of elasticity and also enhanced flexural toughness of foam concrete [10]. The effect of treated kenaf fibre on the durability properties of foamed concrete, specifically drying shrinkage, initial surface absorption, and weathering tests, revealed that chemical treatment of fibres helped to modify the surface morphology and thus increase the bond between the matrix and the hydration process [11]. Hybrid fiber reinforcement with poly vinyl alcohol and coir fiber at a percentage of 0.3% -0.5% showed that performance of hybrid fiber reinforcement is more effective at an optimum percentage is 0.3% [12]. The foamed concrete with a density range of 1000-1600 kg/m^3 with the addition of polypropylene fibers and silica fumes showed increase in tensile strength, creep resistance and reduced drying shrinkage [13]. The porosity, water absorption and sorptivity of foamed concrete increase with the increase in foaming agent. It also declined the mass loss and improves the mechanical property of bottom ash polypropylene fiber reinforced foam concrete [14]. The effect of PVA, PP and basalt fibers on the acoustical property of fiber reinforced alkali activated slag concrete was studied and concluded that PVA are more effective in terms of reducing the drying shrinkage compared to PP and Basalt fibers [15]. Foamed concrete with the addition of ground calcium carbonate and glass fiber showed increase in the mechanical property due to the filling effect of glass fibers in the matrix [16]. Foam concrete with fly ash and hemp fibers was carried out to study the temperature resistance, porosity, drying shrinkage, water absorption and dry unit weight. The addition of fly ash reduces the drying shrinkage and thermal conductivity of concrete [17].

According to the literature review, a large number of studies on synthetic fibre reinforced foam concrete have been carried out in order to study the mechanical and durability characteristics. However, there has been little research on using natural plant fibre as reinforcement in foamed concrete with densities ranging from 800-900 kg/m^3. As a result, the primary goal of this work is to compare the mechanical properties of jute fibre and banana fibre reinforced foam concrete with those of polypropylene fibre reinforced foam concrete. To improve the performance of light weight masonry units in seismic zones, additional layers of reinforcement are required to prevent the structure from collapsing. As a result, as part of the second phase of the research, glass fibre reinforced polymer sheets were used as an additional layer to improve the mechanical strength of masonry units and their performance in seismic zones. The study provides a thorough understanding of the failure pattern and compressive strength characteristics of light weight fibre reinforced foam concrete masonry prisms.

2 Experimental study

The lightweight foam concrete block without fibers were prepared in the first step. The strength of plain LWC blocks was tested under compression. In the second stage, fibers were added to the foam concrete mix to make the Fiber Reinforced Lightweight Concrete (FRLWC) block. Jute, banana and polypropylene fibers were added in different batched to cast FRLWC blocks. The nomenclature for the block was given in such a way to represent the fiber used and the percentage of fibers. For example, PP-LWC-0.22 is a polypropylene fiber-reinforced light weight block with a fibre dosage of 0.22%. Using LWC block and FRLWC blocks, plain and fiber reinforced lightweight concrete prisms were constructed. The plain specimen without fibers was considered as the control specimen for each case. All the plain and fiber-reinforced blocks and prisms were tested under compression load and the stress-strain responses were plotted. To enhance the performance of the FRLWC prism in seismic zone, additional layer of Glass Fiber Reinforced Polymer (GFRP) was used in the form of sheets and pasted on the top of each FRLWC block to fabricate Enhanced FRLWC prism (EFRLWC). The schematic view of fabrication of all types of specimens used in this research paper is shown in Figure 1. Finally, a LWC prism reinforced with micro fibres and different number of layers of GFRP sheets (one on the top of each LWC block) was subjected to axial uni- axial compression to obtain the stress strain plot. The number of layers of GFRP sheets was varied from one to three layers. For example, GFRP-3 refers to three layers of GFRP sheets. One sheet on the top of each block. The effectiveness of microfibres and GFRP sheets as reinforcement in LWC prism, as well as their energy dissipation capacity were investigated. The properties of the mortar used in the prism were tested by casting a cylinder using 1:6 mortar mix and the stress strain plot was obtained. The addition of microfibres increases the LWC prism tensile strength and shear resistance capacity. Microfibres coupled with GFRP play an important role in compressive strength, tensile strength, and shear resistance capacity with improved behaviour in post peak regime.

2.1 Mix proportion

Foam concrete was prepared by mixing cement, fly ash, and water in the proportion of 1:3:1 along with an ASTM C260 certified Stable Air Foaming Agent (SAFA). One part of cement, three parts fly ash, and one part water were used to prepare the fresh paste. To the fresh paste, SAFA was added at the rate of 1.4 kg/m^3 to produce the light weight foam concrete. The foaming agent was diluted with water in a 1:40 ratio before being added to the paste mixture. It was then added to the foam generator machine and mixed for 5 minutes to produce the aerated foam mix with a density of 75 kg/m3. The aerated foaming agent was added to the cement fly ash mixture to produce cellular light weight foam concrete. The water-binder ratio for light weight concrete is normally between 0.4 and 1.25; in this study, the water-binder ratio was kept constant at one. In this foam concrete, coarse and fibre aggregates were not added to achieve a density of 950 kg/m^3. To fabricate fiber reinforced lightweight foam concrete, fiber was added to the foam mix in the range of 0-0.55 % of the volume of foam concrete. The maximum fiber dosage corresponded to 5 kg/m^3 for 0.55% fiber content. The picture of fibers used in this study is shown in Figure 2. The physical proprieties of polypropylene, jute and banana fibers are listed in Table 1. The length of fiber was approximately around 6-8 mm. The polypropylene and GFRP sheets were procured from, Industrial fabric suppliers, Chennai, Tamilnadu, India. The natural fibers namely banana and jute fibers were procured from, Go Green products, suppliers Chennai, India. All the foam concrete specimens were casted and cured for 28 days. GFRP sheets of length 600 mm and width 200 mm and thickness 2mm were cut from the roll and kept ready to be pasted on the top surface of the FRLWC blocks after curing. The mix proportion of the foam concrete is given in Table 2.

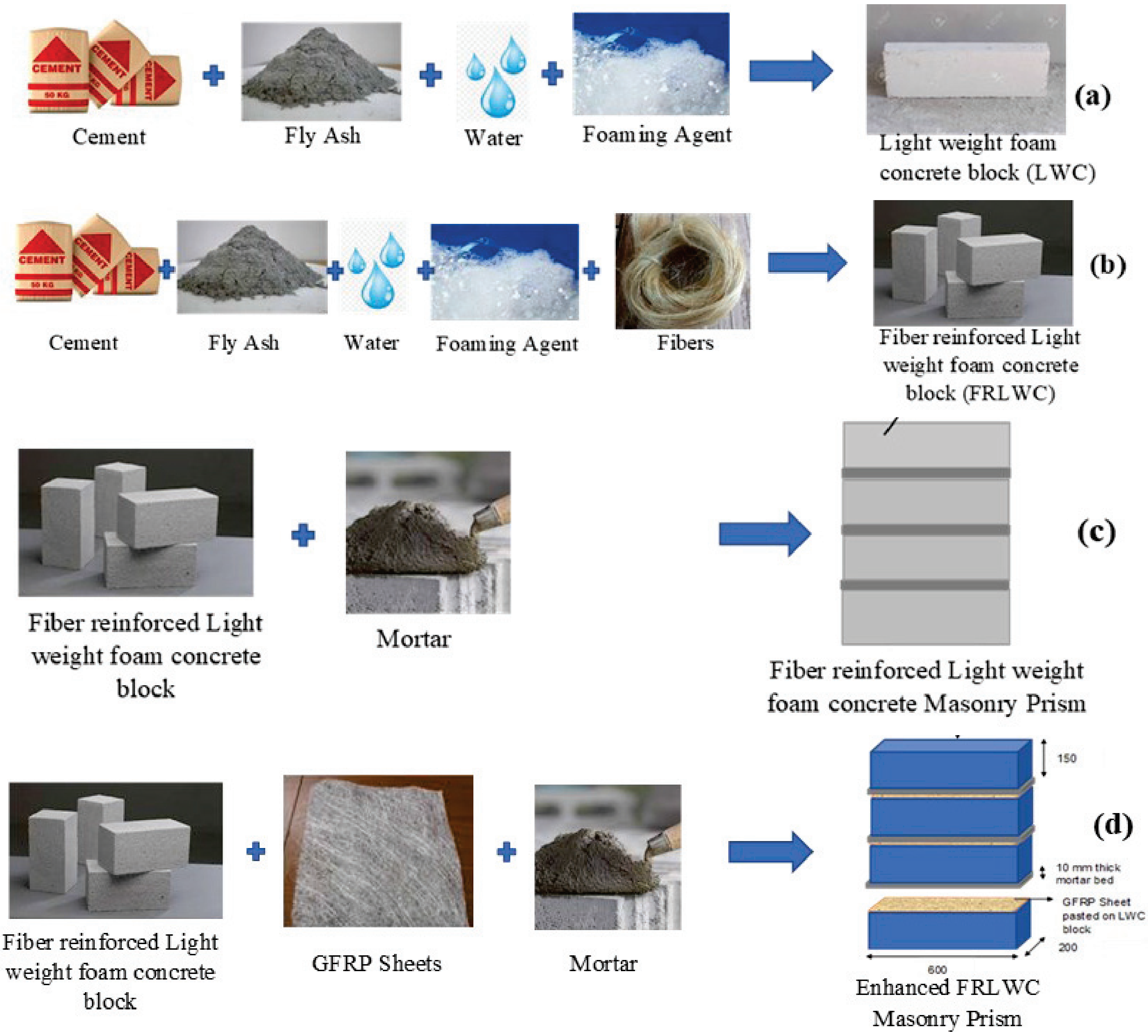


Figure 1. Schematic view of fabrication of different light weight foam concrete specimens (a) LWC block (b) FR LWC block (c) FR LWC Prism (d) Enhanced FRLWC prism

Figure 2. Fibers used in the foam concrete (a) polypropylne (b) Jute (c) Banana

Table 1. Physical properties of polypropylene, jute and banana fiber

Fibers / **physical properties**	Polypropylene	Jute	Banana
Tensile strength (GPa)	551	350	430
Youngs Modulus (Gpa)	3.45	17	23
Length (mm)	20	6-8	6-8
Diameter(mm)	0.03	0.08	0.08
Elongation (%)	40 to 100	2	1.6
Abrasion resistance	Good	Average	Average
Moisture absorption (%)	0 to 0.05	84	60
Softening point (ºC)	140	120	120
Melting point (ºC)	165	119	115
Chemical resistance	Excellent	Average	Average
Relative density g/cm^3	0.91	1.35	0.95
Electric insulation	Excellent	Average	Average

Table 2. The mix proportion of lightweight foam concrete

Mix ID	Cement (kg/m^3)	Fly ash (kg/m^3)	Water (kg/m^3)	Foaming agent (kg/m^3)	Polypropylene (kg/m^3)	Jute (kg/m^3)	Banana (kg/m^3)
LWC	270	810	270	1.4	-	-	-
PP-LWC-0.22	270	810	270	1.4	2	-	-
PP-LWC-0.33	270	810	270	1.4	3	-	-
PP-LWC-0.44	270	810	270	1.4	4	-	-
PP-LWC-0.55	270	810	270	1.4	5	-	-
Ju-LWC-0.22	270	810	270	1.4	-	2	-
Ju-LWC-0.33	270	810	270	1.4	-	3	-
Ju-LWC-0.44	270	810	270	1.4	-	4	-
Ju-LWC-0.55	270	810	270	1.4	-	5	-
Ba-LWC-0.22	270	810	270	1.4	-	-	2
Ba-LWC-0.33	270	810	270	1.4	-	-	3
Ba-LWC-0.44	270	810	270	1.4	-	-	4
Ba-LWC-0.55	270	810	270	1.4	-	-	5

2.2 Mixing, placing and curing

Foam concrete was prepared by mixing cement and fly ash in dry powder form. The dry powders were thoroughly mixed for five minutes. A measured quantity of water was added to the dry mixture to make it into a paste. To the wet mixture the synthetic foaming agent was added. Before adding foaming agent, it was diluted in the ratio of 1:40, i.e., one part of foaming agent was mixed with 40 parts of water and aerated to form the desired quantity of foam and then added to the wet mixture slowly at the rate of 35g per second. The mixture was then thoroughly mixed for 40-50 seconds. To the aerated foam cement mixture, fibers of length 6-8 mm length were added slowly part by part to avoid accumulation of fibers in one place. Then the foam concrete mix was thoroughly mixed for another five minutes and poured in the mould to the required dimension. Then the mould was kept undisturbed till the wet mix hardened. The foam concrete specimens were demoulded after one day (24 Hours) and cured for 28 days. The steps involved in the process of producing a FRLWC prism are shown in Figure 3.

2.3 Details of fiber reinforced lightweight concrete specimen

Different lightweight specimens fabricated for this study include LWC prisms without fibres, LWC prisms with polypropylene fibres, LWC prisms with jute fibres, LWC prisms with banana fibres, and finally GFRP sheet reinforced LWC prisms. The Length, width and depth of LWC prism is 600 x 200 x 630 mm respectively. The details of specimen ID, description of specimen and fiber volume fraction are listed in Table 3. Four FRLWC blocks were joined by applying mortar on the bed surface to form a FRLWC prism. GFRP sheets were pasted to the bed surface of each FRLWC block using epoxy resin and GFRP strengthened FRLWC blocks were fabricated. Using those blocks, a GFRP-enhanced FRLWC masonry prism was fabricated. The picture of FRLWC blocks and Prism and GFRP-strengthened blocks and prism is shown in Figure 4.

Figure 3. Manufacture of foam concrete specimen (a) Addition of foaming agent (b) Wet foam concrete mix (c) Addition of fibers (d) prepration of moulds (e) placing of foam mix into the mould (f) Fresh LWC block (g) Drying of foam concrete for 24 hours (h) demoulding of dry LWC blocks for curing

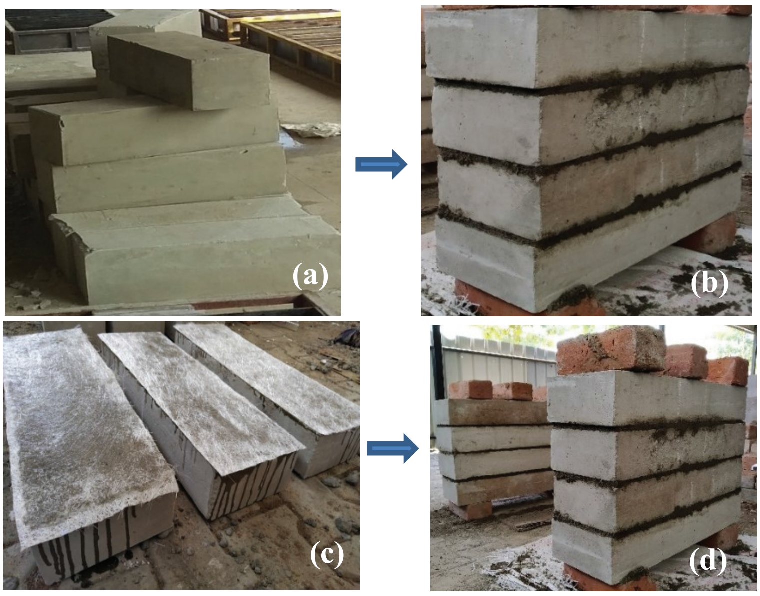


Figure 4. Tested specimen (a) FRLWC blocks (b) FRLWC prism (c) GFRP strengthened FRLWC blocks (d) GFRP strengthened FRLWC prism

2.4 Instrumentation for test specimen

The strength of masonry structures can be effectively predicted by testing a standard dimension masonry prism according to ASTM specifications. The minimum requirement for the dimension of the masonry prism to be tested should not be less than 600 mm. The cross-section dimension of each block should be around 150-250 mm. When compared to full scale testing, small-scale testing of prisms considerably reduces the testing cost. Before the start of the test, the top surface of the prism was checked for a uniformly level surface. To avoid uneven distribution of loads, wooden planks were placed on top of the prism, and the load was applied. The uniaxial compression load was applied to the specimen through a universal testing machine with a capacity of 2000 kN. The stress strain plot was recorded automatically using Data Acquisition System. The load was applied at a very small rate of 0.001 kN/sec. when the load dropped below 30% of the maximum load recorded, it was ensured that the specimen had failed and testing was stopped. The axial displacement was measured by using linear variable displacement transducers placed in four directions of the tested specimen. From the recorded reading in DAS, the stress strain plot was obtained for each specimen. The polypropylene and jute fiber reinforced LWC masonry prism ready for testing is shown in Figure 5.

3 Result and discussion

The testing of the masonry prism, started with the testing of the control prism (without any fibers) and load deflection behaviour was observed and plotted. The stress strain plot of the control prism was taken as the reference for all the other specimens. After testing the control specimen, fiber reinforced LWC masonry prism was tested in the second series. Polypropylene, jute and banana fiber reinforced LWC masonry prisms were tested one after another and the load deflection behaviour was observed. Three specimens were tested for each fiber dosage. In all, 36 pieces of FRLWC masonry prism were tested in the second series. Finally, the GFRP strengthened FRLWC masonry prism was tested and the stress strain plot was obtained. In the third series, the number of layers of GFRP sheet varied from 1-3 sheets. In total, 27 specimens were tested in the third series. The maximum strengths observed for all 22 types of masonry prisms including the control specimen are presented in Table 4.

3.1 Stress-Strain behaviour of control specimen

When the control specimen was subjected to uniaxial compression load, the deformation increased with the increase in load. Initially the load deformation behaviour was linear up to 30% of the peak load. After the elastic region, as the load increases the deformation is not proportional to the applied load, and the stress strain curve becomes non- linear. The specimen continues to be loaded until it reaches the peak load, also known as the ultimate load. When it reaches the peak load, cracks form across the prism's cross section, and the prism fails suddenly. From the stress strain plot shown

Table 3. Specimen details

Series	Specimen ID	Specimen type	Number of specimens	Fiber dosage (%)
1	LWC	Control LWC prism	3	0
2	PP-LWC-0.22	Polypropylene fiber reinforced lightweight concrete Prism (PFRLWC)	3	0.22
	PP-LWC-0.33		3	0.33
	PP-LWC-0.44		3	0.44
	PP-LWC-0.55		3	0.55
3	Ju-LWC-0.22	Jute fiber reinforced lightweight concrete Prism (JFRLWC)	3	0.22
	Ju-LWC-0.33		3	0.33
	Ju-LWC-0.44		3	0.44
	Ju-LWC-0.55		3	0.55
4	Ba-LWC-0.22	Banana fiber reinforced lightweight concrete Prism (BFRLWC)	3	0.22
	Ba-LWC-0.33		3	0.33
	Ba-LWC-0.44		3	0.44
	Ba-LWC-0.55		3	0.55
5	PP-0.44-GFRP-1	Polypropylene LWC Prism +GFRP	3	0.44
	PP-0.44-GFRP-2		3	0.44
	PP-0.44-GFRP-3		3	0.44
6	Ju-0.44-GFRP-1	Jute LWC Prism + GFRP	3	0.44
	Ju-0.44-GFRP-2		3	0.44
	Ju-0.44-GFRP-3		3	0.44
7	Ba-0.44-GFRP-1	Banana LWC Prism + GFRP	3	0.44
	Ba-0.44-GFRP-2		3	0.44
	Ba-0.44-GFRP-3		3	0.44

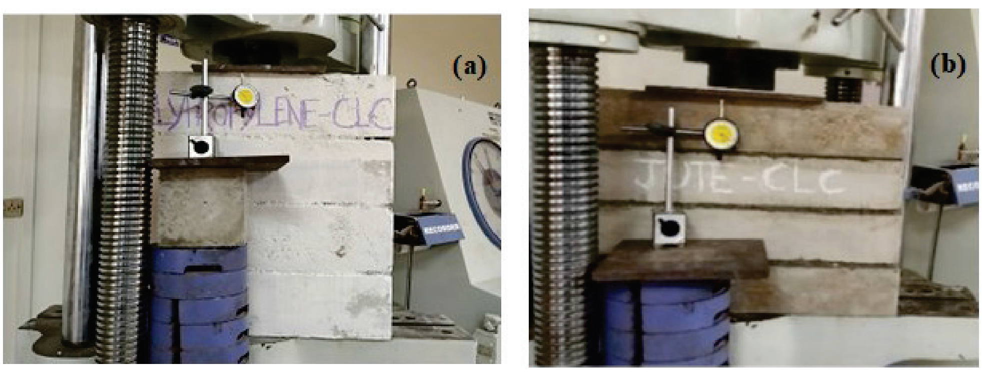


Figure 5. Test setup (a) polypropylene FRLWC prism (b) Jute FRLWC prism

in Figure 6, the ultimate strength of the control LWC prism was 3.66 MPa and the elastic modulus of the control LWC prism was 2100 MPa. The failure started from the mortar joint and then propagated in different directions, till the complete failure of the specimen. Due to the lack of fibers in the LWC prism matrix, a crack did not arrest, which propagated further and developed into wider splitting cracks which led to the sudden failure of the control LWC prism. The failure of control LWC prism is shown in Figure 6.

3.2 Stress-strain plot of FRLWC prism

Similar to the control specimen, the FRLWC prism was subjected to an uniaxial compression load, the load deformation behaviour was observed and the stress train plot was obtained and compared with the control specimen as shown in Figure 7. The addition of polypropylene, jute and banana fibers helps improve the stress strain behaviour of FRLWC prism. The stress strain plot for polypropylene, jute and banana fiber with 0.22%, 0.33% 0.44% and 0.55% of fiber content is shown in Figure 7 (a-d). On comparing the performance of FRLWC prism with a fiber content of 0.22% with Control prism (Figure 7a), the precracking behaviour of the FRLWC prism, when compared to the control prism was improved by the addition of fibers. When comparing the performance of three types of fibers, polypropylene was better compared to banana and jute fibers. After the elastic region, the specimen enters the post cracking region in which the control specimen showed a sudden failure compared to the fiber reinforced prism. Similarly, when the stress-strain of FRLWC prisms with 0.33% and 0.44% fibre content is compared (Figure 7b and 7c), the fibres help to improve the specimen's elastic property. The performance of jute and banana fiber is similar with, only a slight difference in the

Figure 6. Crack formation in specimen (a) before peak load (b) wider crack after peak load

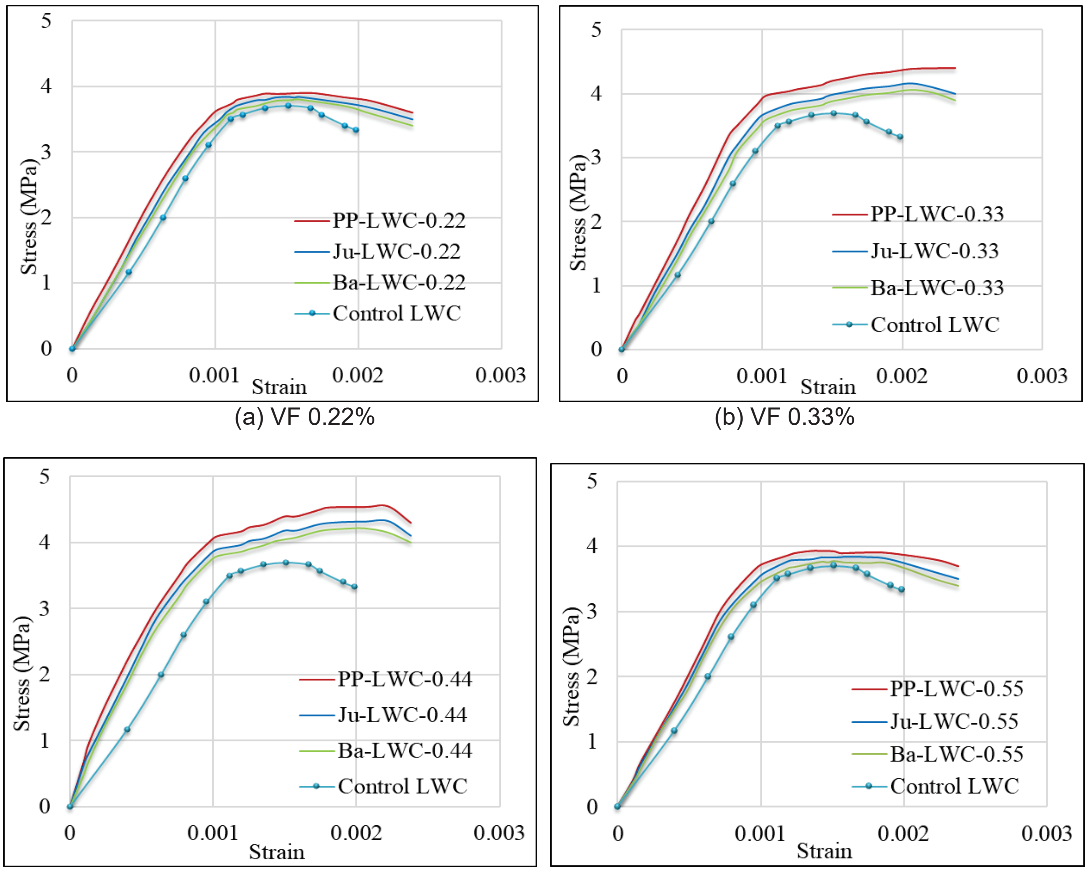
Table 4. Strength of control specimen, FRLWC prism and GFRP enhanced FRLWC prism

Type of specimen	Specimen ID	Peak Compressive Strength (MPa)			Mean Strength (MPa)	Mean strength /Mean strength of control prism
		1	2	3		
Control prism	LWC	3.66	3.56	3.75	3.66	1.00
Polypropylene fiber reinforced lightweight concrete Prism (PFRLWC)	PP-LWC-0.22	4.0	3.69	3.89	3.89	1.06
	PP-LWC-0.33	4.5	4.45	4.35	4.39	1.20
	PP-LWC-0.44	4.2	4.23	4.45	4.55	1.24
	PP-LWC-0.55	3.73	4.13	3.93	3.93	1.07
Jute fiber reinforced lightweight concrete Prism (JFRLWC)	Ju-LWC-0.22	3.84	3.64	4.04	3.84	1.05
	Ju-LWC-0.33	4.13	4.42	4.3	4.16	1.14
	Ju-LWC-0.44	4.12	4.22	4.15	4.32	1.18
	Ju-LWC-0.55	3.88	3.76	3.8	3.78	1.03
Banana fiber reinforced lightweight concrete Prism (BFRLWC)	Ba-LWC-0.22	3.6	3.91	3.90	3.8	1.04
	Ba-LWC-0.33	4.23	4.23	4.11	4.06	1.11
	Ba-LWC-0.44	4.02	4.08	4.1	4.20	1.15
	Ba-LWC-0.55	3.8	3.81	3.67	3.7	1.01
Polypropylene LWC Prism +GFRP	PP-0.44-GFRP-1	4.6	4.76	4.8	4.7	1.28
	PP-0.44-GFRP-2	4.75	4.8	4.9	4.8	1.31
	PP-0.44-GFRP-3	4.9	4.92	4.94	4.92	1.34
Jute LWC Prism + GFRP	Ju-0.44-GFRP-1	4.3	4.31	4.29	4.29	1.17
	Ju-0.44-GFRP-2	4.5	4.45	4.35	4.4	1.20
	Ju-0.44-GFRP-3	4.5	4.7	4.6	4.6	1.26
Banana LWC Prism + GFRP	Ba-0.44-GFRP-1	4.3	4.25	4.35	4.3	1.17
	Ba-0.44-GFRP-2	4.4	4.37	4.78	4.5	1.23
	Ba-0.44-GFRP-3	4.9	4.56	4.6	4.7	1.28

stress value. In all FRLWC specimens, the polypropylene fiber contribution was better than the unreinforced and natural fiber reinforced specimens. Even though the contribution of synthetic fiber is greater than that of natural fibers, the natural fibers also improve the elastic property of the specimen. Synthetic and natural fibres both improve prism stiffness, softening behavior, load carrying capacity, and prevent sudden failure. The peak strengths of polypropylene, jute and banana FRLWC with 0.44% fiber reinforcement are 4.5MPa, 4.32MPa, 4.2 MPa respectively. According to the stress-strain plot of FRLWC with 0.55% fibre content, the load carrying capacity of the specimen decreases as fibre content exceeds 0.44%. The addition of an excess percentage of fibers reduces the bond between the aggregate and binders. The stress transferee from the fibers to the aggregate does not take place properly, which results in the failure of specimen. Similar failure patterns were observed by the author in the previous study using fish tail palm fiber [18]; The peak strength of polypropylene, jute and banana FRLWC with 0.55% fiber content (Figure 7d) were 3.93 MPa, 3.78MPa, 3.7MPa. The strength of 0.55% FRLWC decreased by 13%, 12% and 11% when compared to 0.44% polypropylene, jute and banana FRLWC specimen respectively. The peak load carrying capacity increases with the increase in fiber content up to 0.44%, beyond which it decreases. The addition of fibers mainly improves the performance of the LWC prism in the post peak region by imparting elastic behaviour to the LWC prism. Therefore, it can be concluded that the FRLWC can be used as a better alternative to clay brick in the construction of masonry due to its improved residual strength and toughness. From this series of test, fiber content of 0.44% was taken as the optimum fiber content for the next series of experiments.

3.3 Failure pattern of FRLWC prism

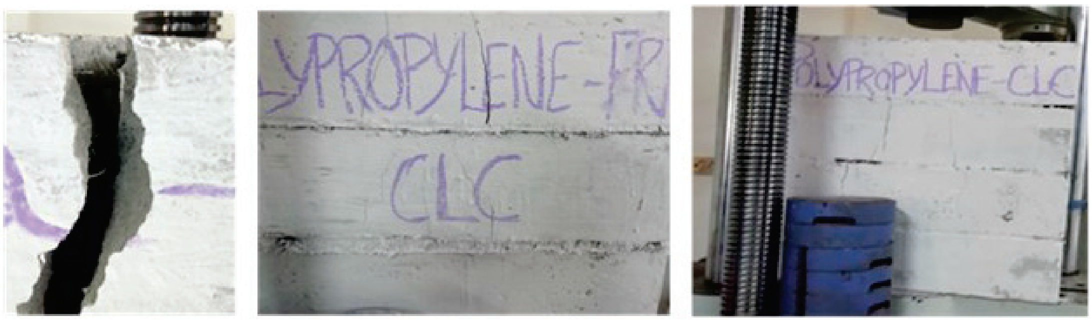
In Autoclaved Aerated blocks addition of fibers is not feasible, because the natural fibers would melt due to the high temperature during the autoclaving process. In such cases, the best alternative material is fiber-reinforced LWC block. As already mentioned in the previous section, an unreinforced LWC prism exhibits a sudden failure due to the stress concentration in particular region and the lack of stress transfer due to the absence of fibers. The failure pattern of the control prism is shown in Figure 6. The FRLWC prism, with polypropylene, jute, and banana fibres as microfibers inside the cement matrix, stops cracks at the microlevel, distributes stress in different directions, and reduces stress concentration in one weak region. As the fiber content increases, large network of fibers is involved in the crack arresting process. The addition of fibres not only prevents the prism from failing suddenly, but also improves its ductile behaviour after it has reached its maximum load carrying capacity. Therefore, it can be concluded that fiber helps to improve the strength and ductility of the LWC matrix under compression. The failure pattern of different FRLWC with different percentages of fiber reinforcement is shown in Figure 8.



(a) VF 0.22% (b) VF 0.33%

(c) VF 0.44% (d) VF 0.55%

Figure 7. Stress-Strain response of FRLWC prism with different volume fraction of fibers



(a) PP-LWC-0.22 PP-LWC-0.33 PP-LWC-0.44

(b) Ju-LWC-0.22 Ju-LWC-0.33 Ju-LWC-0.44

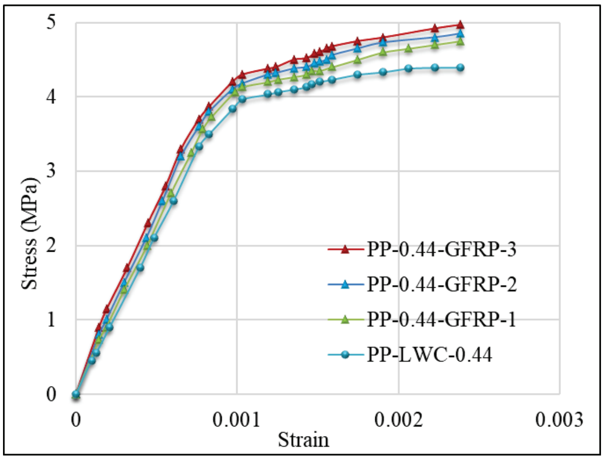
Figure 8. Failure mode of FR LWC prism (a) Polypropylene FRLWC prism (b) Jute FRLWC prism

3.4 Compression behaviour of GFRP strengthened FRLWC prism

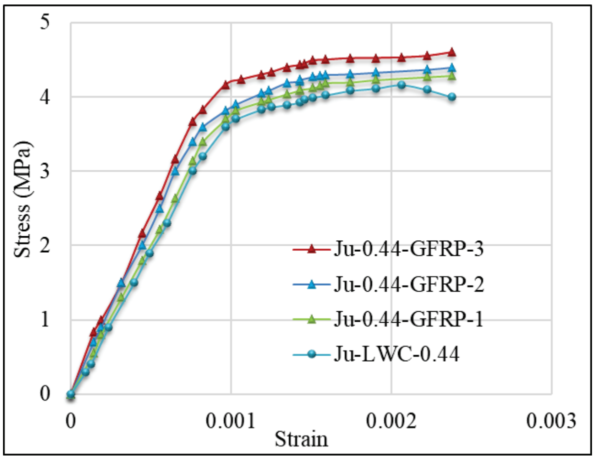
The stress-strain plot of FRLWC with synthetic and natural fibres revealed that jute and banana fibres performed similarly, while polypropylene fibres performed better in terms of load carrying capacity. On comparing the volume fraction of fibers, the FRLWC prism with 0.44% fiber content showed a better performance when compared to all other volume fractions. Therefore, for the third series of experiments, a volume fraction of 0.44% was considered constant. FRLWC with different layers of GFRP sheet provided additional protection to the masonry structures in the seismic zone. Major cracks are completely prevented. Along with microfibers in the matrix, GFRP layers prevent the formation of major crack planes and sudden failure of the specimen. The stress-strain curves for GFRP strengthened FRLWC prism is shown in Figure 9. The elastic modulus increased as the number of layers of GFRP sheet increased [Figure 9(a-c)]. While the softening behaviour got improved in the post-peak region. The peak strength of polypropylene FRLWC prism with one two and three layers of GFRP sheet (Figure 9 (a)) is 4.7 MPa, 4.8 MPa and 4.92 MPa respectively. The peak strength of JFRLWC prism with one two and three layers of GFRP sheet (Figure 9 (b)) is 4.29 MPa, 4.4 MPa, 4.6 MPa respectively. The peak strength of banana FRLWC prism with one two and three layers of GFRP sheet (Figure 9 (c)) was 4.3 MPa, 4.5 MPa, 4.7 MPa respectively. The peak compressive strength of all GFRP reinforced FRLWC prism increases when compared to Control prism. The GFRP strengthened polypropylene FRLWC prism showed peak strength up to a maximum of 4.92 MPa. The peak strength of polypropylene FRLWC is slightly higher than jute and banana FRLWC prism. The peak strength and elastic modulus increased with number of layers of GFRP. From the result it is evident that, GFRP reinforcement increases the strength of masonry construction in earthquake prone region where the sudden failure of masonry can be prevented.

3.5 Failure mode of GFRP strengthened FRLWC prism

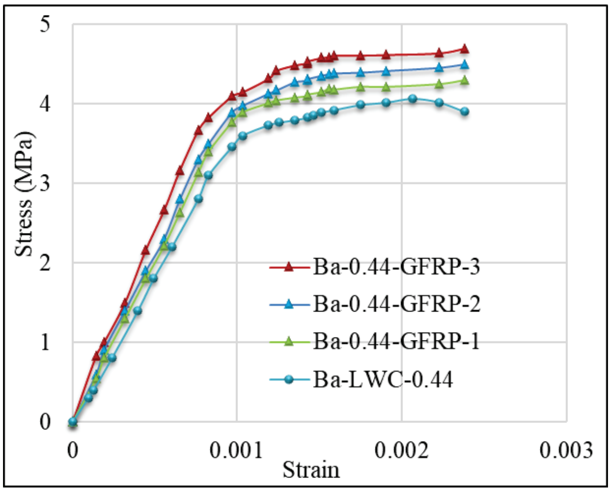
A single wide crack was developed in control prism which leads to failure, while the micro FRLWC prism, showed crack distributed across the cross section of the prism with the increase in fiber content. In case of GFRP strengthened FRLWC, the prism showed only hair line crack across the cross section (Figure 10). Stress concentration in the weak zones is reduced by GFRP reinforcement. Microfibers inside the cement matrix and GFRP layers involved in the crack arresting mechanism and prevent the formation of wider cracks. The development of hair line cracks was more uniform across the prism with one layer of GFRP reinforcement, as shown in Figure 10 (a). The formation of major cracks is arrested by the fibres in the LWC blocks which forms a closed network. As the load increases, the microcracks get wider, but the GFRP layers prevent the movement of cracks from one masonry layer to the next, and as a result, the formation of major cracks across the section of the prism is completely prevented. The development of hair line cracks decreases as the number of layers of GFRP reinforcement increases, as does the load carrying capacity. As shown in Figure 10(b), the formation of a single explicit crack is completely prevented. Fiber reinforcement also improves the post peak strength of LWC. Therefore, from the stress strain plot it can be concluded that, GFRP reinforcement give additional strength to the masonry structures, which can be adopted in seismic zone where the shear failure of masonry structures is severe. In such zone GFRP strengthened FRLWC concrete performs better and reduces the damage to life and structure.



(a) Polypropylene +GFRP



(b) Jute +GFRP



(c) Banana + GFRP

Figure 9. Stress-Strain behaviour of GFRP strengthened FRLWC with 0.44% fiber content

(a) Ba-0.44-GFRP-1 Ju-0.44-GFRP-1 PP-0.44-GFRP-1

(b) Ba-0.44-GFRP-3 Ju-0.44-GFRP-3 PP-0.44-GFRP-3

Figure 10. Failure of GFRP reinforced LWC prism with 0.44% fibers (i) 1 layer of GFRP (ii) 3 layers of GFRP

4 Recommendation from the study

The addition of fibers mainly improves the performance of the LWC prism in the post peak region by imparting elastic behaviour to the LWC prism. Therefore, the FRLWC can be used as a better alternative to clay brick in the construction of masonry due to its improved residual strength and toughness. Out of the different fiber contents added, LWC with a fiber content of 0.44 % performed better. Along with microfibers inside the matrix, GFRP reinforcement gives additional strength to the masonry structures, which can be adopted in seismic zones where the masonry structures are prone to shear failure. In such zones, GFRP strengthened FRLWC concrete performs better and reduces the damage to life and structure.

5 Conclusions

From the Compression study carried out on LWC prism with synthetic fibers, natural fibers and GFRP reinforcement the following conclusion can be drawn.

- Using synthetic and natural fibers as reinforcement in LWC masonry, increases the construction cost by only 15-20 %. But the overall lifecycle of the structure increases due to the addition of fibers which overweighs the additional cost.
- From the chosen percentage of fiber dosage used in the experimental investigation, 0.44% is the optimum. The maximum strength of the prism was obtained at the optimum fiber content.
- The failure of a control LWC prism without fibre occurred abruptly, with the development of a single explicit crack across the prism's cross section. However, in the case of the microfiber reinforced LWC prism, a large number of microcracks were formed as a result of the stress distribution caused by the close network of fibers, as well as the formation of a major weak plane.
- Polypropylene, jute and banana fibers help to arrest the cracks within the LWC matrix, while the GFRP sheets act as a crack arrester at the major level and prevent the movement of cracks from one layer to another.
- The addition of GFRP reinforcement to the FRLWC prism further increased the ductile behaviour and also increased the compressive strength of the prism. As the number of layers of GFRP increased the elastic modulus and stiffness also increased. Both the microfibers and GFRP layers involved in the crack arrest.
- Along with micro fibers inside the matrix, GFRP reinforcement gives additional strength to the masonry structures, which can be adopted in seismic zone where the masonry structures are prone to shear failure.
- This research study has been conducted for cellular lightweight foam concrete with fiber reinforcement. Further research has to be extended to hybrid fiber reinforcement and the high temperature effect on the strength and durability properties of natural fiber reinforced foam concrete.

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Review paper

Warm mix asphalt use in Slovenia and in Europe: A review

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ABSTRACT

Recently, we have witnessed an extreme increase in the prices of energy and raw materials, on the one hand, and economic expansion, due to the business growth in construction sector, part of which is also the asphalt industry, on the other hand. The asphalt industry in particular is facing increasing challenges of economic acceptability and reducing greenhouse gas emissions, as well as improving working conditions. One of the possible solutions proved to be warm mix asphalts (WMA), which can be produced and compacted at reduced temperatures in comparison to hot mix asphalt (HMA). The beginnings of WMA technology in Europe date back to 1999, while in Slovenia the first field test was conducted in 2005. In the last two decades a numerous research and studies on the properties and technologies of WMA production have been conducted. According to EAPA (European Asphalt Pavement Association), the use of WMA cumulatively in Europe in period from 2013 to 2020 is slowly increasing, but the differences in WMA production in individual European countries are significant. The article presents an overview of WMA production techniques, their advantages and disadvantages and their usage in individual European countries in comparison to Slovenia.

1 Introduction

In recent decades, the growing global awareness of the negative impact of greenhouse gas emissions on the environment has led to sustainable development in all areas of human activity, including the road construction industry [1-3], to which the asphalt industry is an important contributor. Primarily because of the high temperatures (above 140 ºC) required to produce and spreading of asphalt mixtures, the asphalt industry is one of the most energy-intensive segments of the construction sector, contributing significantly to greenhouse gas and harmful vapour emissions. [3-5] Research into ways to reduce energy consumption and negative environmental impacts, as well as reduce hazardous fumes and thus improve working conditions, has led to the development of new technologies that require lower temperatures for asphalt production, paving, and compaction while maintaining its workability. [4-7] Depending on the production temperature, asphalt mixtures are broadly classified as cold mix asphalt (CMA) (0 - 30 ºC), half-warm mix asphalt (HWMA) (60 - 100 ºC), warm mix asphalt (WMA) (100 - 140 ºC), and hot mix asphalt (HMA) (> 150 ºC), [4] with minor differences in the reported temperature ranges noted by various authors [2,5,7-12]. While various studies have expressed reservations about the use of CMA and HWMA, mainly due to improper coating of the aggregate, WMA is proving to be an adequate alternative to HMA [11], and its use has gradually increased in recent years [13].

2 Warm mix asphalt

Warm mix asphalt (WMA) is produced and placed at temperatures approximately 20 to 55 °C lower than HMA. [7] The reduction in mixing and paving temperatures can be achieved by using various techniques and/or additives to lower the viscosity of the binder, while maintaining properties and workability comparable to HMA without compromising mix performance. [4,10,14] The technologies used to produce WMA can be divided into three groups: Foaming techniques, use of organic additives, and use of chemical additives. [1,4,5,8]

2.1 Foaming techniques

The basis of foaming techniques is the addition of small amounts of water, either injected into the hot binder or added directly to the mixing tank, which evaporates the water and produces a large volume of foam. The temporary expansion of the binder reduces the viscosity of the mix, which improves the coating and workability of the mix, even at lover temperatures required for the production of HMA. Some of the added water evaporates, while some remains dispersed

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in the mix for a short period of time, assisting in the compaction of the asphalt on the job site. In addition to the direct addition of water (water-based technology), indirect methods (water-containing technology) can also be used for the foaming process. In the indirect method, synthetic or natural zeolite is used to create a foaming process. A zeolite is a crystalline hydrated aluminium silicate with about 20% chemically bound water. When in contact with the asphalt binder, zeolite releases the bound water, causing a controlled foaming effect. [4,5,8,14] The second indirect foaming method uses the moisture of the sand or reclaimed asphalt pavement (RAP) used in the production process to generate foam. [5,8,14] For all foaming techniques, the amount of water should be carefully selected to facilitate adequate expansion of the binder without making the mix susceptible to moisture-induced damage (stripping). [4,5]

2.2 Organic additives

Technologies that use organic additives to reduce the viscosity of asphalt binders commonly include the use of fatty acid amides (e.g., Licomont BS), Montan wax (e.g., Asphaltan B), and Fisher-Tropsch wax (e.g., Sasobit) [1,4,5], but the use of some other additives such as carnauba wax (plant wax), [15] recycled pyrolytic polyethylene wax (RPPW) and a polyethylene wax-based asphalt binder additive made from cross-linked polyethylene (Rh-WMA Modifier), has also been investigated [16]. In these technologies, organic additives are usually mixed with the asphalt binder or directly with the asphalt mixture during asphalt mix production. The waxes used usually have a melting point of 80-120 ºC and are capable of chemically altering the interaction between viscosity and temperature of the asphalt binder. When the binder cools, the additives crystallise and form a lattice structure with microscopic and uniformly dispersed particles. Such a structure increases stiffness and improves resistance to permanent deformation of the asphalt layer, but there is some evidence of increased susceptibility to low temperature cracking. [4,5]

2.3 Chemical additives

Chemical additives are not used in the production of WMA to facilitate foaming or reduce viscosity. They primarily improve aggregate coating, mix workability, and compaction rather than reducing viscosity. Surfactants, emulsifiers, polymers, anti-stripping chemical additives, or hybrid combinations of additives are most commonly used. Some commonly available additives are Cecabase® RT, RedisetTM WMX, Rediset® LQ, Evotherm®, Revix, Interlow, ZycothermTM, Hypertherm and others. Chemical additives can be incorporated directly into the asphalt binder or into the asphalt mix during the production process. [4,5,7,15,16]

2.4 Advantages and disadvantages of using WMA

The use of WMA offers numerous advantages compared to HMA. The main advantage reported in the literature relates to the environment. At lower production temperatures, gas and dust emissions are significantly reduced. Typically by 30 to 40% for CO_2 and SO_2, 50% for volatile organic compounds (VOC), 10 to 30% for CO, 60 to 70% for NO_x, and 25 to 55% for dust. [6,10] The large reduction in emissions of asphalt fumes and polycyclic aromatic hydrocarbons (PAH), typically between 30% and 50%, has a significant impact on worker exposure and thus improves working conditions. [6] In general, for every 12 ºC reduction in asphalt production temperature, the release of vapours is reduced by about 50%. [8] In addition to the environmental and health benefits, large energy savings of up to 35% have been reported in the production of WMA compared to HMA. [3,6] Recent studies have shown that WMA has similar or even better performance than HMA, including better compaction and workability, improved resistance to permanent deformations and fatigue, longer allowable haul time and faster admission to traffic. [3,9] Improved resistance to deformation is mainly associated with the use of organic additives, while the use of other additives and technologies has less impact. [4] There are also some disadvantages to using WMA compared to HMA. The major concern is moisture-related damage, as lower production temperatures can lead to incomplete drying of aggregates and entrapment of residual moisture in the mix, causing striping and rutting and thus reducing pavement service life. [2,4,5,7,9-11,15,17] Various studies also indicated that moist aggregates and lower production temperature can significantly reduce the resistance of asphalt mixtures to permanent deformation. [4,15,17] Consequently, increased susceptibility to moisture and reduced ageing may result in intensified maintenance also due to premature rutting. Some studies have also observed that under low-temperature conditions, the use of organic additives can lead to increased binder stiffness and waxes crystallization inducing low temperature cracking. Therefore, the low temperature behaviour of WMA should be carefully evaluated. [4,15]

3 WMA background and USE

The development of WMA technology was made possible by the research and patent of August Jacob in Germany in 1928 on "foamed asphalt". [4,18] The first field trials took place in Europe between 1997 and 1999, followed by the USA in 2002 to 2004, [10] while the first trial in Slovenia took place in 2005 [1] . The initial research and field trials were followed by a series of studies around the world on the quality of WMA and the possibilities of using various additives and technologies to improve the properties of WMA, as well as studies on the increased use of RAP and other secondary raw materials in WMA. [19-22]

From 2005 to 2015, several field trials were conducted in Slovenia (Figure 1, Table 1) to promote and encourage wider use of WMA in Slovenia. [1,14]

A 2005 trial showed that lowering production temperature by about 20 ºC resulted in 16% less fuel consumption, 56% less CO_2, 61% less NO_x, and 44% less SO_x emissions, and 25 to 40% lower smoke and dust exposure for workers at the work site. [1]

At all test sites, the quality and performance of the asphalt mix and asphalt layers met the technical requirements and were comparable to HMA. [14] Although no specific monitoring of these test sections has been established, regular surveys of network performance using FWD (Falling Weight Deflectometer) and IRI (International Roughness Index), as well as visual assessment of pavement condition on Slovenian freeways and state roads, have so far not identified any quality deviations associated with WMA sections, which could lead to the conclusion that WMA in Slovenia performs similarly to HMA.

The production and use of warm mix asphalt in Slovenia has stalled somewhat since the trial sites were established, probably due to a lack of technical regulations. Nevertheless, production of smaller quantities of WMA was reported from 2017 to 2020 [13], but production was mainly related to the need to improve workability or resistance to permanent deformation. WMA production in European countries that have reported WMA production data has gradually increased, [13] but in general it is still quite low (Figure 2), especially compared to data from the United States of America (USA), where WMA production reaches 72-84 million tonnes in 2018-2020, while production in Europe combined is only 7,1-9,2 million tonnes in the same period. Figure 2 shows WMA production in the European countries included in the EAPA report for the period 2013-2020. The European champion in WMA production was France, followed by Norway with a huge increase from 2017. [13]

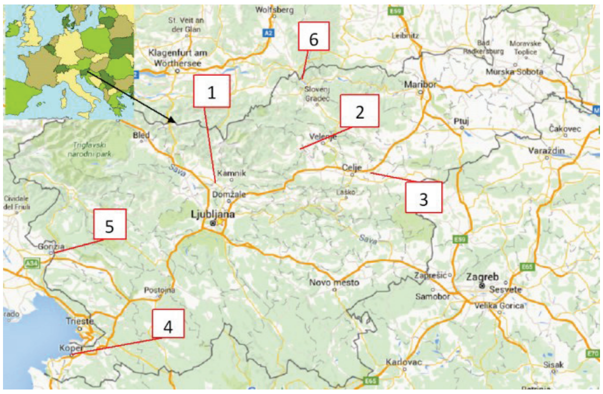

Figure 1: WMA field trial sites in Slovenia between 2005 and 2015

Table 1: Basic information on WMA field trials between 2005 and 2015 in Slovenia [1,14]

Ref. No.	Location	Year	Asphalt Mix	WMA Technology
1	Spodnji Brnik – Moste	2005	AC 22 base B 50/70 AC 16 base B 50/70 AC 11 surf B 50/70	Organic additive 3% SASOBIT
2	Ljubija near Mozirje	2006	AC 22 base B 50/70 AC 8 surf B 50/70	Organic additive 2% LICOMONT
3	Motorway section Ljubečna – Celje East	2006	SMA 8 PmB 45/80-65	Organic additive LICOMONT
4	Parking garage Port of Koper	2009	protective layer surface course	Organic additive SASOBIT
5	Roundabout Nova Gorica – Kromberk and Roundabout Nova Gorica – Rožna Dolina	2009	surface course	Organic additive SASOBIT
6	Vič - Dravograd	2015	AC 22 base B 50/70 AC 11 surf PmB 45/80-65	Organic additive SASOBIT

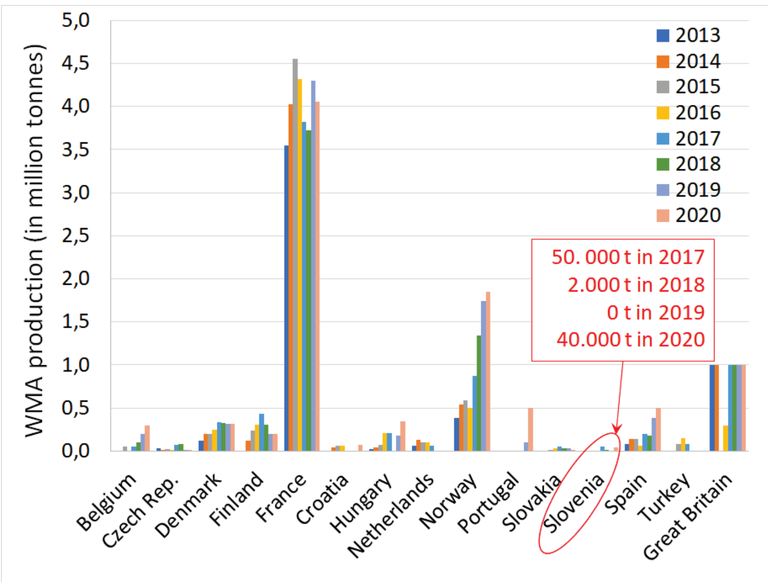

Figure 2: WMA production in Europe in the period 2013 – 2020 [13]

A better insight into WMA production can be obtained by comparing WMA production with total asphalt mix production in individual countries and in Europe combined, compared with production in the USA (Figure 3). The highest share of WMA in total asphalt production is in Norway, where it is almost 27% in 2020, which is even higher than in the USA, where the share of WMA in total asphalt production is 23%. More than 10% of WMA production was also reached in France (12,7%) and, after an extremely high increase in 2020, in Portugal (14,7%). WMA production in other European countries was below 5%, and only 2,1% was reached in Slovenia in 2020. The combined share of WMA production in Europe also does not reach 5% (3,3% in 2020), but the positive trend of WMA production is evident. [13]

For further WMA production increase in Europe it is very important, that the use of WMA is not hindered by the European standards for bituminous mixtures (EN 13108-1 to 7), [23-25] as they allow deviations from the specified temperature ranges for individual types of bituminous binders when using additives, but the manufacturer must specify the maximum production temperature and the minimum temperature of the asphalt mixture upon delivery. [8] In 2021, the technical specification TSPI - PGV.06.460 Upper structure of roads - Warm Asphalt Mixture [12] came into force in Slovenia, which defines the technical conditions for the use of warm asphalt mixtures on public roads in the Republic of Slovenia, which will hopefully encourage investors, designers and producers for wider WMA use in Slovenia.

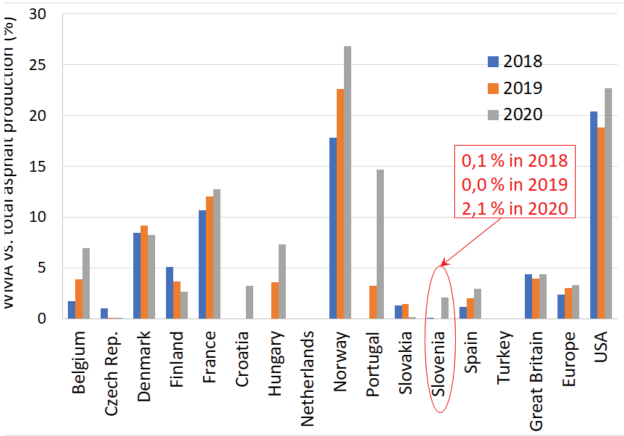

Figure 3: WMA vs. total asphalt production in the period 2018 – 2020 [13]

4 Conclusion

Warm Mix Asphalts (WMA) represents a large group of asphalt technologies that can significantly impact our ability to reduce greenhouse gas emissions and improve working conditions in the asphalt industry. With rising energy prices, these technologies are also becoming attractive to asphalt mix producers. In general, the use of WMA is gradually increasing in Europe, but it has not yet reached the level of use in the USA by far. Some European countries, such as Norway, Portugal, as well as Belgium, Hungary and Croatia, have made a big step towards increased use of WMA, while in Slovenia we are still somewhat hesitant when it comes to using WMA on a larger scale.

In recent years, Slovenia has gained a lot of experience and knowledge in the field of using warm mix asphalt. All research and practical experience show that warm asphalts can qualitatively replace the classic hot asphalt mixtures (HMA), with many additional positive economic, environmental and health effects. Unfortunately, most Slovenian asphalt plants have been technologically limited in the past and continue to be so today, making the use of foaming techniques are hindered. WMA production in Slovenia was and still is mainly focused on the use of organic additives, as they were readily available and easy to use in production. Since the demand for WMA in Slovenia was and unfortunately still is very low, the development of production technologies and research on the use of other possible additives and techniques are very slow. In order to initiate and promote the production and use of WMA, technical specifications and requirements for WMA should first be established. In this way, all qualitative and technical conditions for a wider use of WMA technologies will be met, which will allow designers and investors to create a demand and market opportunities for contractors. Higher demand for WMA will consequently boost the implementation, research and development of production technologies, creating additional opportunities to reduce production costs while having a positive impact on the environment and improving working conditions in the asphalt industry.

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Original scientific paper

Characterization of harvest residues ashes and ceramic waste powders originating from Vojvodina as potential supplementary cementitious materials

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mortar

ABSTRACT

Traditionally, residential buildings in Vojvodina have masonry walls. Various types of mortar of mineral origin are most often used for joining masonry elements and finishing. The total amount of mortar for the construction of one building is not negligible. The estimated annual consumption of mortar in Vojvodina is about 198 thousand tons i.e. 27 thousand tons of cement and about 31.5 thousand tons of hydrated lime. It can easily be seen conventional mortars based on cement and lime are unacceptable in the light of environmental protection and sustainable development in the contemporary construction industry. Therefore, there is a need for research and development of new, alternative types of binders, based on locally available renewable and/or waste materials. The ceramic masonry elements and tiles industry generates ceramic waste during the production process. This waste, in powder form, could potentially be used as supplementary cementitious material (SCM). Biomass ash, generated by the combustion of harvest residues, as a renewable energy source, is another alternative to cement in modern building composites. This paper emphasizes the physical, chemical, and pozzolanic characteristics of the available agro-waste ashes and ceramic waste powder, originating from Vojvodina. The results indicate relatively high pozzolanicity of all tested ceramic powders and biomass ash based on cob corn, owing to their high fineness and reactive silica content. Furthermore, a catalogue of collected waste materials, illustrating basic data on the raw materials, processing method, landfilling, available quantities, and their tested properties is given.

1 Introduction

The carbon emission resulting from cement clinker production has always been a concerning issue among sustainability researchers, as well as among cement-based composites technologists, mainly due to the growing alarm about global warming.

With an increasing demand for sustainability in the construction sector in the 21st century, the development of cement-based composites incorporating waste materials originating from agriculture and industry attracts wide interest. The utilization of such waste in cement-based composites shows immense potential as an alternative material and brings various environmental, economic, and technological benefits. Incorporating agricultural and industrial waste for developing green and sustainable cement-based composites is promising with regard to promoting a cleaner environment, reduction of a high level of greenhouse gas emissions, and mitigating the environmental burden of concrete production. From the aspect of economic development, this substitution results in considerable savings due to the reduction of high disposal costs of agricultural and industrial waste as well as the exploitation and preparation costs of raw materials (clay and limestone). Thus, reusing these waste materials saves natural resources, contributes to sustainability, and decreases the dumping of these wastes into landfills.

Substantial quantities of biomass waste from harvesting (leaves, straw, stalk, etc.) are generated every year. About 30% of the annual production of biomass waste is used as animal food or as fertilizers. The rest can be utilized as a renewable energy source. Within the combustion process, 2-10% of biomass ash is generated as a by-product. The utilization of biomass, as an energy source, increases worldwide so does the amount of biomass ash as waste material. Unfortunately, biomass ash is usually put down in open dumps resulting in disposal problems, such as air, water, and ground pollution. Thus ideas for its use are welcomed.

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Generally, ceramic waste can be collected in brick, block, and roof tiles production of red pastes, or in sanitary ware, wall, and floor tiles production from white pastes. These materials can be found in construction and demolition (C&D) waste, also. The ceramic wastes are highly durable and resistant, both chemically and physically to harsh environmental conditions. Unfortunately, much ceramic waste is deposited in landfills, just a small portion is crushed and used in covering sports fields and in gardening and negligible amounts are used in the manufacture of mortar or concrete to replace cement or aggregates.

Traditionally, residential buildings in Vojvodina have brick walls, whether they are individual or multi-family. During the construction, various types of mortar of mineral origin are most often used for joining masonry elements and finishing. The total amount of mortar for the construction of one building is not negligible. For example, about 12 m^3 of mortar, or 20 tons (average consumption 0.4 t/m^2) is used for masonry and plastering of a residential unit with a usable area of 50 m^2. If it is taken into account that, according to the data of the Statistical Office of the Republic of Serbia, Statistical Yearbook 2020, 6266 apartments were completed in the area of Vojvodina in 2019, with a total area of 494049 m^2, the estimated annual consumption of mortar is about 198 thousand tons.

The usual mixing proportion of mortar components is 1:2:5, by volume (cement: lime: sand), so 1 m^3 of mortar requires about 230 kg of cement and about 270 kg of hydrated lime. With regard to the above-mentioned needs of mortar, the consumption of component materials is about 27 thousand tons of cement and about 31.5 thousand tons of hydrated lime annually.

Both binding materials are obtained in technological processes in which large amounts of natural non-renewable resources (limestone and clay) and fossil fuels (coal, oil, and natural gas) are consumed. It is well known the production of 1 t of cement clinker needs about 1.5 t of raw materials (limestone and clay) and the production of the same amount of lime requires about 1.4 t of limestone. During the thermal treatment of these raw materials, a significant amount of CO_2 is emitted into the atmosphere. It is estimated that during the production of 1 t of cement and 1 t of lime, about 0.6 t of CO_2 and 0.8 t of CO_2 are released, respectively. Based on these data, it can be easily estimated that the annual mortar production consumes 40.5 thousand tons of limestone and clay for cement production and 44.1 thousand tons for limestone to obtain lime. It means, that about 41.5 thousand tons of CO_2 are emitted into the atmosphere.

Taking into account the results of the previous analysis, it can be seen that conventional mortars based on cement and lime are unacceptable in the light of environmental protection and sustainable development in the contemporary construction industry. Therefore, there is a need for research and development of new, alternative types of binders, based on locally available renewable and/or waste materials.

This paper presents a part of the writers' activities in the realization of the project “Development of new binders based on agricultural and industrial waste from the area of Vojvodina for the production of eco-friendly mortars”, and includes a short review of current research on the application of biomass ash and ceramic waste powder as SCMs, data on the availability of those materials in Vojvodina region, characterization and cataloging of collected samples.

2 Literature review

2.1 Review on the application of biomass ashes in masonry mortars

The pozzolanic materials contribute to the strength of cement-based composites in two ways, firstly due to the filler effect (packing of smaller particles and filling the voids in the structure) and secondly owing to their pozzolanic reactivity. Biomass ashes are generally rich in amorphous silica and can be a potential cement substitute, with proper processing before their application.

Several researchers have introduced agricultural wastes, in form of ash, as a partial replacement of cement or as an alkaline activator including rice husk ash (RHA) [1], olive waste ash (OWA) [2], sugar cane bagasse ash (SCBA) [3], corn cob ash (CCA) [4], palm oil fuel ash (POFA) [5], wheat straw ash (WSA) [6], soya straw ash (SSA) [7], almond shell ash (ASA) [8], cornstalk ash (CSA) [9], barley straw ash (BSA) [10], etc.

From the earlier research studies, it is noticed that different processing conditions have a remarkable influence on the pozzolanic performance of these ashes. A simple processing method (such as a sieving process) helps convert them into reactive pozzolans. Further grinding of the sieved biomass ash to the cement fineness led to even higher pozzolanicity index, signifying superior pozzolanicity of the ash [11]. Grinding results in a reduction of particle size and an increase in the specific surface area of the particles. As a result, a greater number of nucleation sites for the pozzolanic reaction is available and the reactivity is enhanced, too. Rithuparna et al. [12] reported that grinding of RHA and SCBA for the respective optimum duration is sufficient to improve their pozzolanic performance, while further increase in the grinding duration leads to only a marginal increase in the reactivity.

Rithuparna et al. [13] investigated the influence of high-volume cement replacement (for up to 80%) by palm oil clinker powder in cement – lime mortar. The results indicate that up to 40% of cement could be replaced to obtain the requisite compressive strength of masonry mortar. Similar recommendations were given by Šupić et al [6], whereas research findings highlight the possibility of replacing cement with slag (50%), fly ash (30%) or wheat straw ash (30%) while maintaining its performance and improving the economic and environmental impacts of masonry mortar production. Lertwattanaruk et al. [14] developed a masonry mortar based on four types of waste seashells, including short-necked clam, green mussel, oyster, and cockle as cement replacement (5%, 10%, 15%, or 20% by weight). All mortars containing ground seashells yielded adequate strength, lower drying shrinkage and lower thermal conductivity compared to the conventional cement, indicating a good perspective for their utilization as SCMs in masonry mortar formulations.

In recent years, the scientific community has studied the incorporation of biomass ashes in cement-based mortar and concrete as SCMs, while the recent trend indicates a high interest in using these materials as alkaline activators. Raw materials with a strong alkalinity are generally used as activators, hence biomass ash that is rich in potassium can act as one environmental-friendly alternative. The results are promising and mostly demonstrate the viability of using biomass ashes in the activation of coal fly ash (FA) and granulated blast furnace slag (GBFS) and the reduction in the consumption of commercial chemical reagents for alkali-activated materials (AAM) preparation [2],[4],[8],[17].

Nevertheless, the common concern with regards to the suitability of biomass ashes used as cement substitutes is the uniform long-term quality and, consequently, long-term durability of the produced cement-based composites. Therefore, the overall durability-related properties of such composites are still to be experimentally verified, documented and discussed at large.

2.2 Review on the application of ceramic waste powder in masonry mortars

There are two possibilities of utilization of ceramics waste in mortars or concrete, like a fine aggregate for partial or total substitution of natural aggregate, and as a pozzolanic material for partial replacement of cement or as active mineral addition for lime mortars. The use of ceramic waste as an aggregate in lime mortar has been known long ago, and that type of masonry mortar is called "Surkhi". However, the studies on the application of ceramic waste as pozzolanic materials in mortars are limited in comparison to concrete.

Various ceramic wastes may become a significant part of cement-based composites due to stability and high resistance to biological, chemical, and physical degradation [18]. Also, it is an inexpensive, abundant, and environmentally friendly material. As it is rich in aluminosilicates, ceramics are the appropriate supplements for cement materials that can improve the mechanical strength and durability performance of mortars and concretes. It is essential to emphasize that ceramic waste may be used in the construction industry without extensive prior preparation.

The powdery fraction of ceramic waste, according to some authors could be used as a cement substitute and enhance the composites' performance owing to its pozzolanic properties.

The literature review revealed that there is a lack of research related to the use of ceramic powder as a substitute for cement in masonry mortars.

Pereira et al. [19] examined the possibility of reusing ceramic waste from bricks and tiles of red-clay ceramic industry as partial cement replacement in mortar. An increase in total porosity and a strength reduction with the higher cement replacement level were observed. However, with up to the 20% replacement, compressive strengths obtained at 90 days are equal to or higher than those obtained with a reference mix. It is concluded, that the degree of hydraulicity of the mortar is dependent on powder fineness and bricks firing temperatures.

Based on the review on utilization of ceramic waste as a cement substitute, conducted by Alsaif [20], it is noticed that the majority of properties in fresh and hardened mixtures (whether it is mortar or concrete), in particular workability and compressive strength, are degraded with ceramic waste powder addition at 20%. The durability properties including acid resistance, chloride ion ingress, rapid chloride permeability, electrical resistivity, corrosion and water absorption are nevertheless improved. Generally, cement substitution by ceramic powder is not extremely effective, as improvement appears to be poor, up to 20%, hence the author suggests combined cement and aggregate substitution in order to promote more sustainable performance and saving natural resources.

The literature review revealed that ceramic waste powder can be successfully used with some other waste materials such as FA and GBFS to obtain alkaline activated mortars which are more efficient, produce less CO_2, reduce costs and consume lower fuel than Ordinary Portland cement (OPC) mortars [21]-[22]. Shah et al. [21] studied the bond strength between the alkali activated mortars - AAM (based on ceramic powder, fly ash and slag) as the repair materials and normal concrete, exposed to aggressive environments. Replacement of the slag by the ceramic powder showed remarkable effect to reduce the loss in the bond strength between the AAMs and concrete substrate exposed to elevated temperatures up 900°C, but displayed the lower performance when exposed to the freeze/thaw and wet/dry cycles.

The use of biomass ashes and ceramic powder in the production of masonry mortar has been scarcely studied so far. The aim of the above-mentioned scientific project is to analyze the possibility of the application of locally available waste materials, originating from agriculture and ceramic industry in Serbia, in masonry mortars. For this purpose, an experimental study comprising a characterization of collected materials, as a first step in the project, was carried out.

3 Availability of selected waste materials in Vojvodina

3.1 Availability of biomass ashes

Availability of harvest residues ashes in Autonomous Province of Vojvodina (APV) has been investigated within the realization of the project IPA Interreg ECO Build in the period 2017-2020. At the time, eleven companies were found to use harvest residues as an energy source for obtaining heat energy [23]. In 2022, most of these companies have continued to utilize this renewable energy source (RES) and generate of biomass ashes. A brief overview of the available types and quantities of generated biomass ashes in AP Vojvodina is presented in Table 1. Based on the collected information, the majority of the generated waste is disposed of in controlled landfills and some companies spend considerable resources on the transportation and landfilling of the ashes.

Ashes from two major producers: Soya Protein and Almex Ipok were collected in order to carry out their characterization.

3.2 Availability of ceramic powder

So far, we have collected data from three factories that produce ceramic products for construction purposes, located in the AP Vojvodina region. A brief overview of the types of ceramic waste available and the amount generated during the annual production is shown in Table 2. A small part of the ceramic waste generated is used for sports fields, road filling and gardening, while the rest is disposed of.

4 Characterization of harves residues ashes and ceramic powders

Characterization of selected waste materials, originating from agriculture and ceramic industry in APV was conducted in accordance with the European standard for siliceous fly ash.

Table 1. Available quantities of biomass ashes in AP Vojvodina

Company	Biomass type	Temperature of combustion	Types of biomass ashes	Produced quantities of ash per year (tons)
Mitrosrem Sremska Mitrovica	soya straw	600-650°C	1. ash from boiler furnace 2. ash from multiciklon 3. fly biomass ash	15
Soya Protein Bečej	wheat straw, soya straw and husk silo waste, sunflower husk	700-900°C	1. ash from boiler furnace 2. ash from multiciklon 3. fly biomass ash	1100
Hipol Odžaci	agro pellets of soya straw, wood pellets	800-1000°C	1. ash from boiler furnace 2. ash from multiciklon 3. fly biomass ash	700
Almex-IPOK Zrenjanin	cob corn, soya straw	700-900°C	1. ash from boiler furnace 2. ash from multiciklon 3. fly biomass ash	1100
KNOT-AUTOFLEX Bečej	wheat straw, soya straw	unknown	1. ash from boiler furnace	60
Fishery Lovćenac	soya straw	unknown	1. ash from boiler furnace 2. ash from multiciklon	9
Victoria Oil Šid	sunflower husk	700-1000°C	1. ash from boiler furnace 2. ash from multiciklon	720
Sava Kovačević Vrbas	cob corn	500°C	1. ash from boiler furnace	30
PTK Panonija Mecker farm	wheat straw, soya straw	500°C	1. ash from boiler furnace	60
Total				**≈ 4000 tons**

Table 2. Available quantities of ceramic waste in some factory of ceramic products in AP Vojvodina

Company	Type of ceramic products	Temperature of combustion	Produced quantities of ceramic waste per year (tons)
AD Polet IGK, NEXE Novi Bečej	roof tiles	cca 1020°C	3600
Ciglana Stražilovo (NEXE, Sremski Karlovci)	masonry block, ceiling block	cca 880°C	1500
WIENERBERGER doo Kanjiža,	roof tiles	1040-1070°C	1000

4.1 Materials and Methods

4.1.1 Materials

Cement

Ordinary Portland cement (OPC), originating from the Lafarge cement factory in Beočin, Serbia, was used. The cement has a specific gravity of 3.1 g/cm^3 and the Blaine fineness of 4000 cm^2/g.

Cementitious materials

For experimental investigation of chemical and physical properties and pozzolanic activity, the following materials were collected and tested:

- Mixed wheat straw, sunflower husk, and silo waste (BA1), „Soya-protein“ Bečej,
- Mixed cob corn and soya straw ash (BA2), „IPOK“ Zrenjanin,
- Ceramic masonry blocks waste (CP1), „NEXE-Stražilovo“ Petrovaradin,
- Ceramic roofing tiles waste (CP2), „NEXE-Polet“ Novi Bečej,
- Ceramic roofing tiles waste (CP3), „Wienerberger doo“ Kanjiža.

The harvest residues ashes were roughly sieved, through a 4mm sieve, to separate unburnt straw and other large impurities. In order to obtain a material with a satisfactory fineness (equal to or higher than cement), both harvest residues ashes and ceramic waste were ground in a laboratory ball mill for 6h.

4.1.2 Methods

The chemical composition of collected materials was determined using energy dispersive X-ray fluorescence spectrometer (EDXRF 2000 Oxford instruments) according to SPRS EN 196-2:2015 [24] and ISO 29581-2:2010 [25]. The representative samples were pulverized in a laboratory vibratory mill prior to the testing.

The particle size distributions were determined by the laser diffraction method by using a Malvern Mastersizer 2000 particle size analyzer that can analyze particles between 0.01 and 2000µm. The Malvern Mastersizer 2000 records the light pattern scattered from a field of particles at different angles. The recorded intensity at a certain angle is the sum of the intensity of light scattered from the surface of the particles and the intensity of the secondary scattered light because of refraction while passing through the particle, and this is important for particles smaller than 50µm. Applied Mie light scattering theory assumes that particles are spheres, and thus, the results obtained for particle diameters specifically correspond to equivalent sphere diameters. The measurements were performed with an automated dry dispersion unit Scirocco 2000.

The specific surface area was determined according to the Blaine air permeability method given in SRPS EN 196-6:2019 [26], which is widely used for the fineness determination of hydraulic cement. The test is based on the principle of resistance to airflow through a partially compacted sample of powder material.

Initial and final setting time, as well as soundness, were determined in accordance with SRPS EN 196-3:2017 [27]. The method is used for assessing whether the abovementioned physical properties of an SCM material are in conformity with the requirements given in SRPS EN 450-1:2014 [28].

The pozzolanic activity was tested on samples prepared according to the procedure given in SRPS B.C1.018:2015 [29]. Mortars were prepared with SCM, hydrated lime, and CEN standard sand, with the following mass proportions: m_{hl} : m_{scm} : m_{qs} = 1: 2 : 9 and water – binder ratio 0.6 (where: m_{hl} – the mass of hydrated lime; m_{scm} – the mass of waste material; m_{qs} – the mass of CEN standard sand). After compacting, the samples were hermetically sealed and cured for 24 h at 20°C, then for 5 days at 55°C. Subsequently, 24 h period was allowed for the samples' cooling to the temperature of 20°C, followed by compressive and flexural strength tests.

The activity index was examined according to SRPS EN 450-1:2014 [28]. The preparation of specimens and determination of the compressive strength were carried out in accordance with SRPS EN 196-1:2017 [30].

4.2 Chemical composition

The chemical compositions of OPC and collected waste materials are given in Table 3.

The content of oxides SiO_2, Al_2O_3, and Fe_2O_3 has the greatest significance for the potential pozzolanic activity of cementitious materials. Obtained chemical composition of all ceramic powders indicates the relatively high participation of major oxides, satisfying the criterion given in [28] (>70%). Ceramic powders are characterized by low alkali content, whereas the total amount of alkalis (Na_2O+0.658 K_2O) doesn't exceed the limiting 5%, hence the possibility of alkali-silica reaction (ASR) is minimized.

Although both types of biomass ashes have lower amounts of important oxides (below 70%), BA2 is characterized by considerably higher silica content, which should influence its pozzolanic activity. Table 3 shows that the alkali content (Na_2O+0.658 K_2O) of BA1 and BA2 is 15.19% and 8.62%, respectively. The increase in the alkali content of the binder increases the probability of alkali-aggregate reactions; hence this mechanism should be experimentally verified.

4.3 The particle size distribution

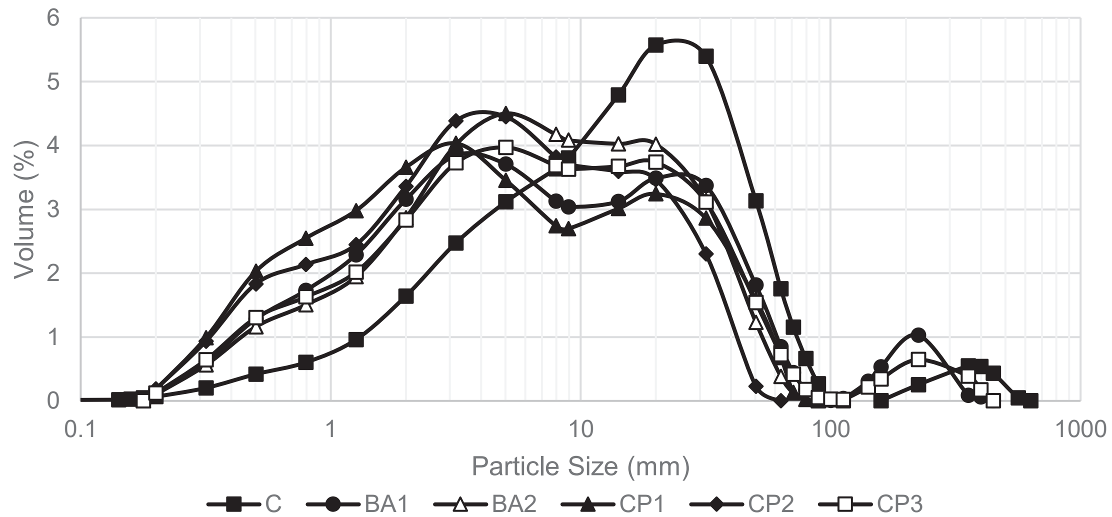
The results were recorded as particle volume percentages and presented as cumulative curves in Figure 1.

The particle size distribution shows that all tested waste materials include finer particles than cement, as a result of the effective grinding procedure and, consequently, the increase in specific surface area of particles. Size frequency distribution (presented as a volume percentage) indicates that all tested waste materials are mostly uniform in particle size.

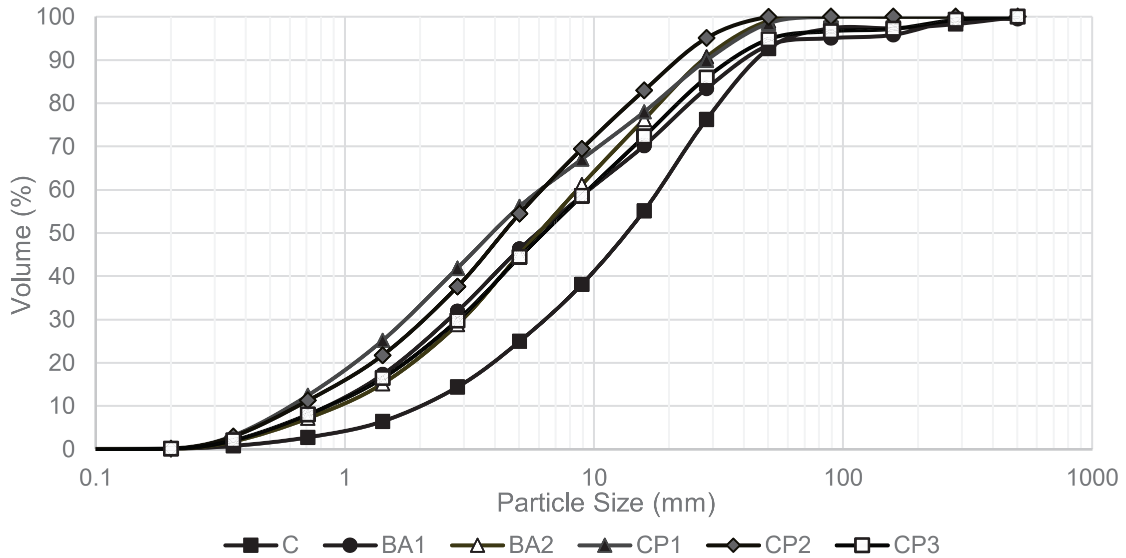
Table 4 lists the values of d10 (median particle diameters corresponding to 10% of the cumulative passing by volume), d50 (median particle diameter corresponding to 50% of the cumulative passing by volume), and d90 (median particle diameter corresponding to 90% of the cumulative passing by volume) for all tested binder samples.

Table 3. Chemical composition of OPC, biomass ashes and ceramic powders

Material (%)	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	SO_3	P_2O_5	SiO_2+ Al_2O_3+ Fe_2O_3
OPC	17.34	4.53	20.64	50.26	1.93	0.20	0.59	3.06	0.00	/
BA1	20.21	1.83	1.74	13.42	8.30	0.00	23.09	2.88	7.78	23.78
BA2	45.76	5.92	3.37	14.08	8.30	0.00	13.10	1.26	2.81	55.05
CP1	60.86	16.38	6.81	9.38	3.89	0.77	2.39	0.80	0.14	84.05
CP2	61.88	16.46	7.40	4.90	3.66	1.63	2.81	0.08	0.20	85.74
CP3	59.03	15.81	6.64	5.65	4.20	1.50	2.50	0.07	0.16	81.48



a)



b)

Figure 1. Particle size distribution of cement and tested waste materials: a) volume percentage and b) cumulative curve

Table 4. Median particle diameters of binder samples

	d_{10} (μm)	d_{50} (μm)	d_{90} (μm)
OPC	2.305	15.254	49.698
BA1	0.956	6.622	43.963
BA2	1.054	6.684	30.720
CP1	0.683	4.342	31.845
CP2	0.727	4.842	24.338
CP3	0.958	7.025	39.145

The median diameter of particles, d90, for OPC corresponds to 49.7μm and is the highest value among tested SCM. As for the other characteristic passing, differences between OPC and other tested materials in particle diameters are more pronounced. Ceramic powders CP1 and CP2 are characterized by the lowest median diameters of particles for characteristic passing, which indicates that these are finer powders, hence it is expected that these SCMs contribute to the filler effect.

4.4 Density and specific surface area

Physical properties: density and specific surface area are shown in Table 5.

Table 5. Density and specific surface area of OPC, biomass ashes and ceramic powders

Material	Density (g/cm^3)	Specific surface area by Blaine (cm^2/g)
OPC	3.10	4000
BA1	2.36	8120
BA2	2.44	8090
CP1	2.62	13815
CP2	2.61	11064
CP3	2.59	6200

Specific surface area is an indicator of material fineness, which influences material's reactivity. Finer particles of cementitious materials can a) have great impacts on cement hydration, including the physical and chemical effects; b) lead to an increase in the effective water-cement ratio; c) serve as nucleation cores for hydration; d) improve a pozzolanic effect and e) dilute cement particles in paste, which provides relatively more space for the formation of hydrates from cement particles, leading to the filler (packing) effect.

After grinding in a laboratory ball mill for 6 h, the specific surface area (Blaine) of all tested cementitious materials significantly exceeded the reference cement value of 4000 cm^2/g.

The ceramic powder obtained by grinding of ceramic waste from masonry elements (CP1 and CP2) displayed the highest fineness value, while the roofing tiles powder (CP3) showed the lowest specific surface area, probably due to the higher temperature of ceramic tiles production and, consequently greater material hardness.

Both types of biomass ashes are characterized by the same fineness, whereas the specific surface area is twice the size of the reference cement value.

4.5 Setting time

The initial setting time, as specified in [28], shall not be more than twice as long as the initial setting time of a 100% reference cement paste (by mass) - criterion 1. The initial setting time, should not be shorter than 60 minutes - criterion 2 (for cement type CEM I 42,5R). All types of tested materials fulfill these criteria. The results are given in Table 6.

The overall effect of ceramic powders and biomass ash BA2 on the setting time of cement paste was proven to retard the setting time, as expected when it comes to SCMs. Both initial and final setting times were extended in relation to the setting time of OPC. In addition, BA2 has a considerably increased final setting time, which may be caused by the presence of unburnt organic substances - straw remains.

The presence of BA1 significantly accelerated the setting time of cement paste. The initial setting time was shorter than an hour, hence this material doesn't satisfy the criterion.

4.6 Soundness

According to the criteria given in [28], the soundness shall not be greater than 10 mm. As all types of tested materials showed negligible expansion, up to 1 mm, the criteria are fulfilled. The results are given in Table 7.

4.7 Pozzolanic activity

Pozzolanic material class was determined based on 7-day compressive (f_c) and flexural (f_f) strength of standard mortar prisms. Results revealing the pozzolanic properties are given in Figure 2.

Table 6. Initial and final setting time

	The Initial Setting Time (Minutes)	The Final Setting Time (Minutes)	Criterion 1	Criterion 2
OPC	140	190	Yes	/
BA1	25	45	Yes	No
BA2	165	285	Yes	Yes
CP1	160	220	Yes	Yes
CP2	160	210	Yes	Yes
CP3	165	225	Yes	Yes

Table 7. Soundness

	Expansion (mm)	Criterion
OPC	0.8	Yes
BA1	0.6	Yes
BA2	1.0	Yes
CP1	0.6	Yes
CP2	0.5	Yes
CP3	0.5	Yes

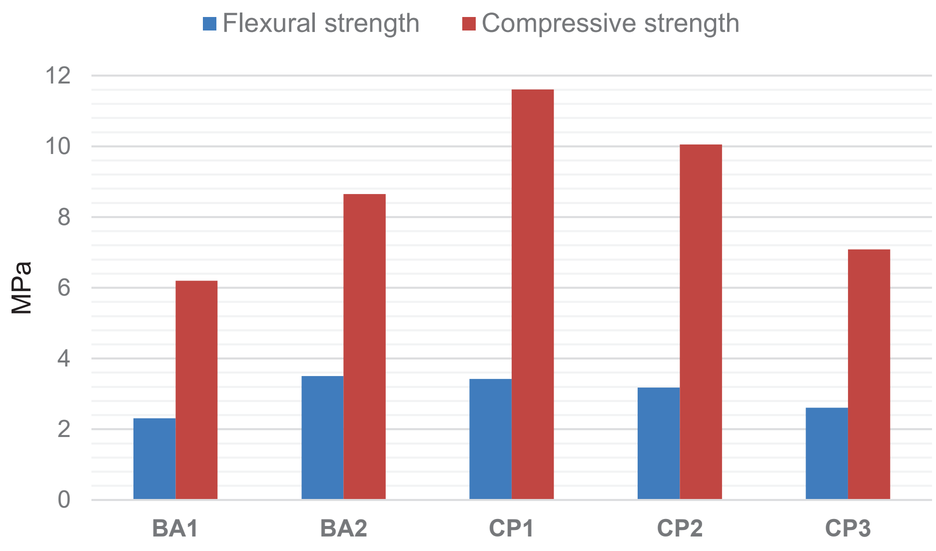


Figure 2. Pozzolanic activity of tested waste materials

Testing of pozzolanic properties showed that both types of biomass ashes, as well as CP3 have pozzolanic activity of Class 5, while ceramic powders CP1 and CP2 have pozzolanic activity of Class 10.

Higher silica content, as well as the increased level of fineness, contributed to the high pozzolanic activity of ceramic powders CP2 and CP3.

4.8 Activity index

Results of the testing activity index (AI) are presented in Figure 3. According to the criteria given in [28], the activity index at 28 days and at 90 days shall not be less than 75% and 85%, respectively.

The data clearly shows that BA2 and all ceramic waste powders met requirements after 28 and 90 days, achieving values greater than 75% and 85%, respectively. It can be observed that BA2 and CP1 displayed a high early strength index as well (equal or greater compressive strength concerning the reference cement sample), thanks to the simultaneous effect of the pozzolanic activity and the dilution effect. The Index of activity after 90days of these four waste materials reached at least 100%. The notable increase in compressive strength after 90 days, indicated the existence of pozzolanic reaction in these SCM.

As a final we emphasize four of the five tested SCM fulfill both criteria for AI. Only BA1 exhibited both activity index values below the required limits. Although the value obtained for this ash after 90 days shows certain delayed pozzolanic activity, the results in total are not satisfactory due to a lack of reactive silica

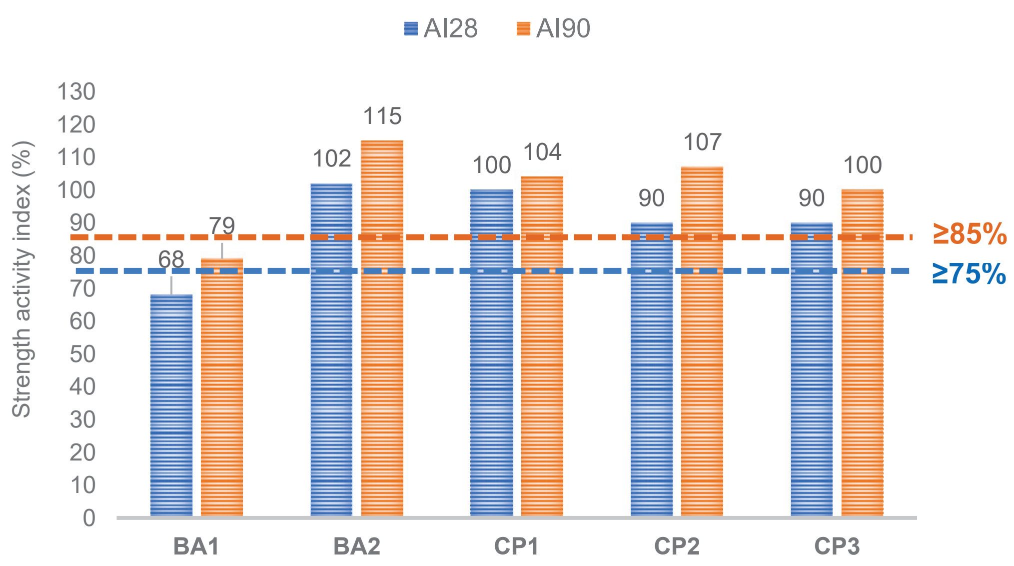


Figure 3. Activity index of tested waste materials

5 Catalogue of harvest residues ashes in AP Vojvodina

The obtained results of listed properties of collected waste materials are presented through the catalogue, given below.

5.1 SCM type BA1 - Mixture of wheat straw, sunflower husk and silo waste ash: basic properties

Mixture of wheat straw, sunflower husk and silo waste ash, before sieving and grinding

Mixture of wheat straw, sunflower husk and silo waste ash, after sieving and grinding

Material origin	Soja Protein, Bečej
Basic data on the material	Bottom ash, roughly sieved through a 4mm sieve
Available amount per year	1100 tons
Disposal	Deported to city landfills
$SiO_2 + Al_2O_3 + Fe_2O_3$ (%)	23.78%
Specific gravity	2360 kg/m^3
Blaine fineness	8120 cm^2/g
Soundness	Satisfactory
Setting time	Initial: 25' FInal: 45'
The pozzolanic activity	Class 5
The activity index	I_{28} = 68% I_{90} = 79%

5.2 SCM type BA2 - Mixed wheat and soya straw ash: basic properties and possible application

Mixture of cob corn and soya straw, before sieving and grinding

Mixture of cob corn and soya straw, after sieving and grinding

Material origin	Almex IPOK, Zrenjanin
Basic data on the material	Bottom ash, roughly sieved through a 4mm sieve
Available amount per year	1100 tons
Disposal	Deported to city landfills
$SiO_2 + Al_2O_3 + Fe_2O_3$ (%)	55.05%
Specific gravity	2440 kg/m^3
Blaine fineness	8350 cm^2/g
Soundness	Satisfactory
Setting time	Initial: 165' FInal: 285'
The pozzolanic activity	Class 5
The activity index	I_{28} = 102% I_{90} = 115%

5.3 SCM type CP1 - Ceramic masonry blocks powder

Ceramic masonry blocks waste, before grinding

Ceramic masonry blocks powder, after grinding

Material origin	Nexe Stražilovo, Sremski Karlovci
Basic data on the material	Ceramic masonry elements for walls and ceillings (waste)
Available amount per year	1500 tons
Disposal	Landfilled in the surroundings of the factory
$SiO_2 + Al_2O_3 + Fe_2O_3$ (%)	84.05%
Specific gravity	2620 kg/m^3
Blaine fineness	13815 cm^2/g
Soundness	Satisfactory
Setting time	Initial: 160' FInal: 220'
The pozzolanic activity	Class 10
The activity index	I_{28} = 100% I_{90} = 104%

5.4 SCM type CP2 - Ceramic roofing tiles powder

Ceramic roofing tiles waste, before grinding

Ceramic roofing tiles powder, after grinding

Material origin	Polet Nexe, Novi Bečej
Basic data on the material	Ceramic roofing tiles waste
Available amount per year	3600 tons
Disposal	Landfilled in the surroundings of the factory
$SiO_2 + Al_2O_3 + Fe_2O_3$ (%)	85.74%
Specific gravity	2607 kg/m^3
Blaine fineness	11065 cm^2/g
Soundness	Satisfactory
Setting time	Initial: 130' FInal: 190'
The pozzolanic activity	Class 10
The activity index	I_{28} = 90% I_{90} = 107%

5.5 SCM type CP3 - Ceramic roofing slipware tiles powder


Ceramic roofing slipware tiles waste, before grinding


Ceramic roofing slipware tiles powder, after grinding

Material origin	Wienerberger, Kanjiža
Basic data on the material	Ceramic roofing tiles waste, covered with slipware
Available amount per year	1000 tons
Disposal	Landfilled in the surroundings of the factory
SiO_2 + Al_2O_3 + Fe_2O_3 (%)	81.48%
Specific gravity	2587 kg/m^3
Blaine fineness	6200 cm^2/g
Soundness	Satisfactory
Setting time	Initial: 165' FInal: 225'
The pozzolanic activity	Class 5
The activity index	I_{28} = 90% I_{90} = 100%

6 Conclusions and further research

This research aimed to highlight the importance of using waste materials, originating from agriculture and brick manufacture industry to create more sustainable building materials. The following conclusions can be drawn from the experimental study:

- The justification of research into the possibility of applying industrial and agricultural waste materials for the production of masonry and plastering mortar is highlighted
- The feasibility evaluation of using ceramic powder and harvest residues ashes as pozzolanic materials were determined by their physical, chemical, and mechanical characterization tests;
- The chemical characterization of BA2, CP1, CP2, and CP3 indicates their potential of using as mineral additives due to the presence of reactive silica in the chemical composition;
- From the physical tests (Blaine specific surface area test and particle size distribution) all SCMs showed a high level of fineness, as a result of grinding of materials in a laboratory ball mill. Even more, ceramic waste powders CP1 and CP2 may additionally act as filler, improving the packing density of the composite;
- Testing of pozzolanic properties showed that both types of biomass ashes, as well as CP1, have pozzolanic activity of Class 5, while ceramic powders CP2 and CP3 have pozzolanic activity of Class 10; confirming the pozzolanic effect of SCMs on portlandite consumption;
- Regarding activity index, BA2 and all ceramic waste powders (CP1, CP2, and CP3) met requirements after 28 and 90 days, achieving values greater than 75% and 85%, respectively. BA1 exhibited activity index values below the required limits. It was also observed that all mortars incorporating SCMs, apart from BA1, showed equal or better performance in relation to the reference cement mortar. This can be attributed to both pozzolanic reaction and filler effect.

The results indicate a promising light for the sustainable application of industrial and agricultural by-products as a partial replacement for cement to tackle waste disposal problems, to reduce the greenhouse gases impact and to reduce the negative effects on environment. With the satisfactory reactive silica content and high degree of fineness, ceramic waste powder and harvest residues ashes could potentially substitute significant portions of conventional binders in durable composite formulations. Hence, future research should focus on the effect of incorporation of tested materials at different cement replacement levels in cement-based mortars and evaluation of obtained physical, chemical, mechanical and durability properties under optimized conditions.

Acknowledgement

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Technical paper

Single-layer load-bearing tunnel lining structure in hard rock masses

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ABSTRACT

The paper presents some of the observations made during tunnel construction with a single-layer load-bearing lining in hard rocks. The load-bearing elements of one such lining were observed, as were measures to improve the quality of the material, as well as other required actions to guarantee its stability throughout its lifespan. An extremely important measure is to limit the water inflow into the tunnel, by pressure grouting contemporary grout mixtures into the fractures, commonly referred to as pre-grouting. This paper also shows the construction time and cost of single-layer lining compared to the traditional two-layer lining. The construction expenses of single-layer tunnel lining are slightly lower compared to two-layer tunnel lining. Nevertheless, the main benefit is construction time-savings. The observations are presented using a practical example: a tunnel in the limestone rock near the Mratinja dam, between Plužine and Šćepan polje in Montenegro.

1 Introduction

Tunnels in hard rock masses are excavated and traditionally designed in south-eastern Europe (countries of the former Yugoslavia) with two-layer linings. Moreover, the initial lining is installed to preserve the rock mass against the effects of air and moisture. Later, a permanent cast-in-place concrete structure (secondary lining) is installed, designed to withstand long-term loads and meet the requirements of serviceability and durability. Waterproofing is achieved by installing a plastic membrane between the primary and secondary lining, which additionally acts as a separating layer, reducing potential cracks due to shrinkage on the secondary lining.

Single-layer tunnel structures are only used in rock masses where the tunnel can be formed in a circular shape. The single-layer lining was mainly used in a hydraulic tunnel with cast-in-place reinforced concrete.

On the other hand, in the Scandinavian countries, this type of structure has been built for decades as permanent structures for various underground structures (car parks, roads, railway tunnels, underground stations, etc. In the past few decades, significant progress has been made in the understanding and quality assurance of single-layer tunnel structure components.

The purpose of this paper is to show that, in many cases, a single-layer tunnel structure could be designed as an alternative to the traditional two-layer tunnel structure. This is discussed in the example of a tunnel, previously designed by the author as a two-layer structure. The tunnel is designed by utilizing individual parts of a single-layer tunnel structure, based on experience gained in designing tunnels for the expansion of existing metro lines [1], [2], [3], and the main underground bus station in Stockholm [4].

2 Tunnel No. 10 on the road segment Šćepan polje - Plužine

2.1 General tunnel information and basic technical data

The tunnel is planned for the road section between Plužine and Šćepan Polje in the Republic of Montenegro. The current road passes through the Canyon of Piva, above the artificial lake Piva. The lake was formed by the diversion of the Piva River in 1975, by the construction of a high hydroelectric power plant (Mratinje dam), and by flooding part of the canyon. The dam is of the arch type, made of concrete, 220 m high and 261 m long, with a usable volume of about 800 million m^3. The dam is one of the highest of its kind in Europe and an exceptional construction success for the previous country (SFR Yugoslavia). The road was originally built for the construction of a hydroelectric power plant but was later upgraded to serve as the main road. One of the features of the road is that at one section it crosses the dam, i.e., the road crosses over the dam in the crown of the dam. The existing road is narrow and winding, with geometric elements corresponding to a maximum speed of 30 km/h in some sections. In many places on the road, there are unlined rock tunnels, rockfalls, and avalanches, so driving is a constant threat to all who pass. It is also impossible to drive heavy vehicles on the road. To improve the geometric

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elements of the route and meet safety requirements, extensive design work and on-the-road reconstructions have been made. Part of the road after the dam is designed to extend straight into the mountain rather than the existing sharp twist into a 1.5 km long tunnel [5, Figure 1].

The tunnel is designed as a single tube, for two-way traffic with a maximum speed of 80 km/h. The tunnel includes two lanes, each 3,25 m wide, and two edge lanes, as well as emergency evacuation and revision paths on both sides of the tube. The height of the traffic clearance gauge for vehicles is 4,70 m, and the clear height above the revision paths is at least 2,0 m.

The theoretically useful area of the tunnel depends on the radius of the horizontal curve and is A_1 = 51.72 m^2 or A_2 = 56,40 m^2 depending on the type of construction (structure gauge type 1, R_1 = 4,85 m / structure gauge type 2, R_2 = 5,15 m). At the beginning of the tunnel, just after the dam, the tunnel extends with a left horizontal curve with a radius of 350 m. The rest of the tunnel is in a straight direction. The vertical curve of the tube is a concave curve, with a radius of 10,000 m and a maximum longitudinal slope gradient of 2,1%. The tunnel cross-section is rounded – horseshoe-shaped, with a maximum span of 11,5 m and a height of 8,7 m. The tunnel is designed with all the necessary construction safety measures: breakdown bays, fire-protection niches, hydrant niches, niches for placing electrical installations, accommodation of UPS devices, drainage niches, escape routes, etc., and everything is described in detail in the main tunnel design project [5], as well as in the paper [6].

2.2 Geological and hydrological conditions

The canyon at the dam's location is geomorphologically V-shaped, with very steep and nearly vertical valley sides. The entire right bank is made of massive Triassic limestones with an uneven distribution of fractures, Figure 2. According to their origin, the limestones are of the ridge type, influenced by significant tectonic movements in geological history. Regionally, the rock mass is part of the Dinaric karst, and they belong to the synclinal part of the large anticline (called the "Crkvička anticline") which stretches in the direction northwest-southeast, i.e., in the direction of the stretching of the Dinarides [5].

The surface of the canyon is very uneven and gouged, which is typical for thick, layered to massive limestones, affected by the tectonic effects emphasized above, as well as by frost, precipitation, and insolation. As a result of all these processes, in many places at the base of the steep slopes, larger dents were formed. Dents were later filled by loose rock fragments and debris, which were added each year, and in the end, screes were formed.

The presence of several fractures in the limestone rock mass, as well as the carbonate composition, enabled the development of karstification. Chemical erosion happens below the current riverbed and especially along larger cracks and faults. This is because the river flows out very deeply and sometimes a lot of water comes in from the mountainous

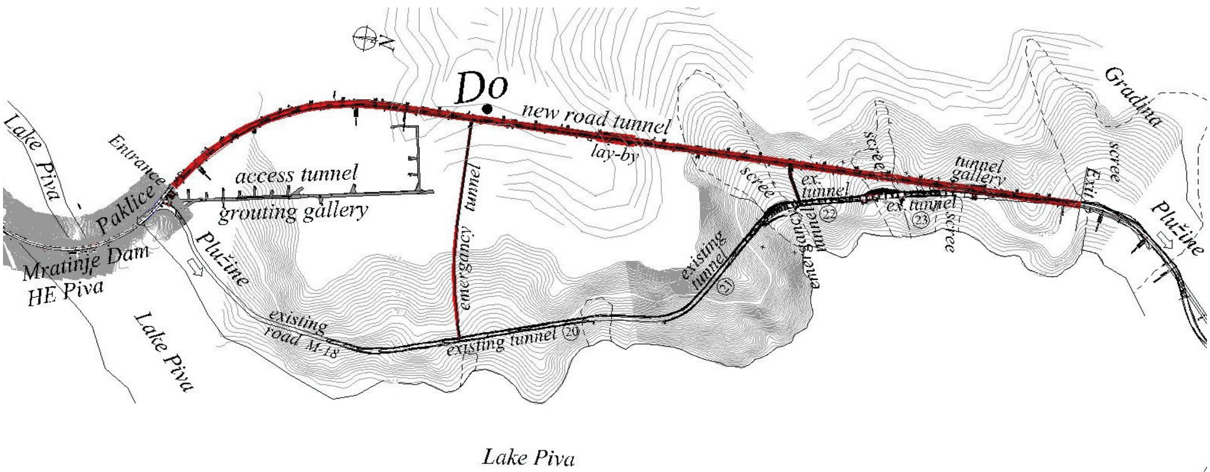


Figure 1. Layout plan of the new tunnel located after Mratinje dam

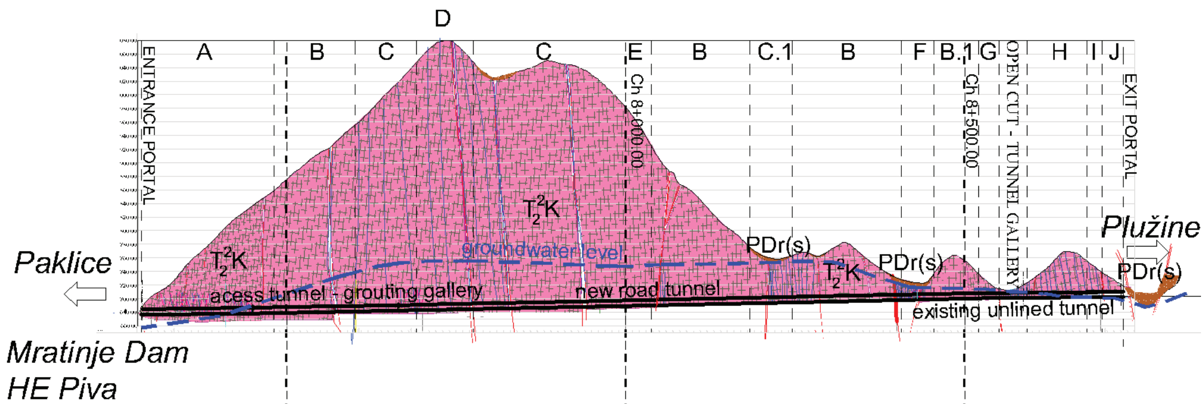


Figure 2. Longitudinal geological section of the new tunnel located after the Mratinje dam

area behind it. Limestones have secondary fissures and cavernous porosity and function as an aquifer. The conductive zone is above, and the storage zone is below the level of the Piva River.

In the vicinity of the dam, limestones have lower fracture density and low permeability ranging from 0.1 to 5 Lu. Permeability in the intervals of 5 - 10 Lu and 10 - 20 Lu occurs sporadically. A smaller number of fractures and lower permeability indicate that at this place they are shallower and less karstified [7].

Because the rocks have a lower porosity and permeability, precipitation infiltrates quickly and groundwater levels rise quickly in the event of heavy rains. In the zones of larger fractures, high conductivity can influence the formation of occasional surface and underground accumulations.

Groundwater location and regime are generally determined by hydrogeological features of the rock mass, as well as groundwater regime in the reservoir's hinterland and perception frequency. The level of the established aquifer fluctuates depending on changes in the water level in the reservoir of the dam. The level of groundwater is not significantly higher than that in the reservoir, except in cases of infiltration from heavy precipitation and after the melting of snow. This phenomenon is due to the existence of a tectonic fault in the hinterland that acts as a conductor and lowers the groundwater level. Once the water level in the reservoir is kept at the same level for a long time, a balance is established between the water level in the reservoir and the inflow of groundwater from the rock massif.

On the right flank of the dam, there is an access tunnel. The tunnel has been used to construct piles in the dam's right foundation. It also served for grouting the dam plug base, the drainage of water from the hinterland, and the execution of geotechnical and hydrological observations.

3 Conventional two-layer tunnel lining

The tunnel is designed as a double shell system, i.e., with two linings. First, the outer layer is placed - the initial tunnel lining (rock support system).

The initial tunnel lining consists of shotcrete, reinforcing steel wire mesh, lattice supports or steel arches, and rock bolts. It is installed immediately after the excavation and is designed to provide sufficient initial structural capacity and stability for the tunnel excavation. A controlled stress release and redistribution in the surrounding rock is enabled by introducing the initial tunnel lining (rock support system). The selection of the excavation and support system (initial lining) was determined based on the behaviour of the rock mass due to excavation. The behaviour (response) of the rock mass is defined without considering any stabilization measures or excavation sequences.

The tunnel has a complex geometry and unfavourable boundary conditions such as the proximity of the access tunnel, the enlarged tunnel cross-sections at the locations of the emergency stopping areas (lay-bys) and turning areas, the proximity of the existing unlined tunnels, settings with insufficient rock thickness above the tunnel crown or a steep slope on the side, or else, tunnelling under the scree deposits or near the fault. Different initial linings (support types) are developed to cover all these tunnelling conditions. Support types B, B.1, C, C.1 G, and I are light support systems distinguished by different shotcrete thicknesses and bolt spacings. They are foreseen on 62,37 % of the tunnel length. Stronger support types are denoted as A, D, E, and H. In addition to shotcrete and bolts, these support types include pre-support via a spilling bolt and lattice girders. They are planned for 29% of the tunnel's length. Heavy support types are denoted as F and J. These support types include, pre-support spiling bolts, HEA steel sets, shotcrete, and bolts, in combination with tunnel face stabilization measures. They are planned for only 5.52% of the tunnel length. A brief overview of support types is given in Table 1, and further support types are described in the paper [6] and the main project [5]. The most common initial lining (support) types are B and C. Therefore, in this paper, the tunnel sections with these types of initial linings have been considered.

The excavation of the rock mass in the case of a two-layer lining is A= 70.01 m^2/m. The tunnel is designed to be driven using the sequential excavation method, with the sequence consisting of a top heading and bench (Figure 3).

The first layer of the lining consists of shotcrete, reinforcing wire mesh, and rock bolts. A flexible load-bearing structure is formed by a combination of all these elements. The main load-bearing element is the rock mass. If there are visible water leaks on the surface of the rock mass or a higher groundwater inflow, the installation of the drainage holes is designed, before placing shotcrete. The drainage holes (weep holes) have a diameter of 50 mm and a length of 3.0 m.

The shotcrete is designed with a compressive strength of C25/30 and is to be applied in two layers, with a final thickness of 10 cm. After the first layer, the fully grouted SN-rock bolts (Store-Norfors) are installed at a c/c distance of 2.0 m. The rock bolts are 25 mm in diameter and 4.0 m long. A welded reinforcing wire mesh Q188 (6 mm / 215 x 600 cm) is attached to the rock bolts, and subsequently, the second layer of shotcrete is applied.

Following the completion of the initial lining, the final (secondary) lining is executed. The final lining (inner layer) is made from cast-in-place reinforced concrete (Figure 4). It is designed by assuming that the entire load (or some portion of the load) carried by the initial lining, at the end of its life, is transferred to the secondary lining. The secondary lining is designed to be load-bearing for long-term loads from rock mass loads, water pressure, installation loads, rheological effects in concrete, temperature changes, etc.

Before the construction of the final lining, a layer of geotextile (500 gr/m^2) is placed, followed by a 2 mm thick layer of waterproofing made of an LDPE (low-density polyethylene) sheet. The seepage water collected behind the waterproofing is conducted by perforated plastic sidewall drainage pipes, placed at the tunnel bottom elevation. The drainage pipes have a diameter of 160 mm, placed on each side. The pipes are backfilled with drainage concrete C12/15. The collected seepage water is conveyed to the tunnel`s main roadway sewage system.

The secondary lining is C30/37 reinforced concrete, with exposure classes XC 3 and XF3 [8]. The lining is 30 cm thick in the tunnel crown, and increases to 40 - 51 cm on the sides. The foundation of the secondary lining is also C30/37 reinforced concrete with a 40 cm thickness. The foundation is built on a 10 cm lean concrete layer.

Table 1. The distribution of the initial tunnel linings (rock support systems) in %

A	B	B.1	C	C.1	D	E	F	G	H	I	J	Open cut
13,47	29,39	4,56	20,51	4,32	5,80	3,87	3,31	2,07	6,09	1,52	2,21	2,90

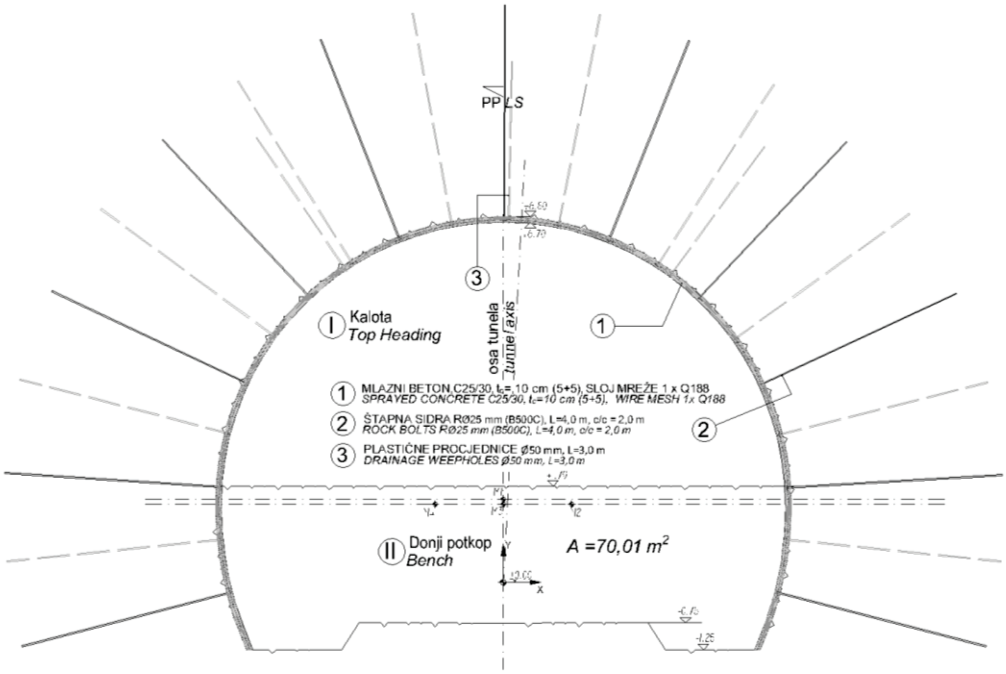


Figure 3. Typical cross-section of the tunnel with the initial lining (support system)

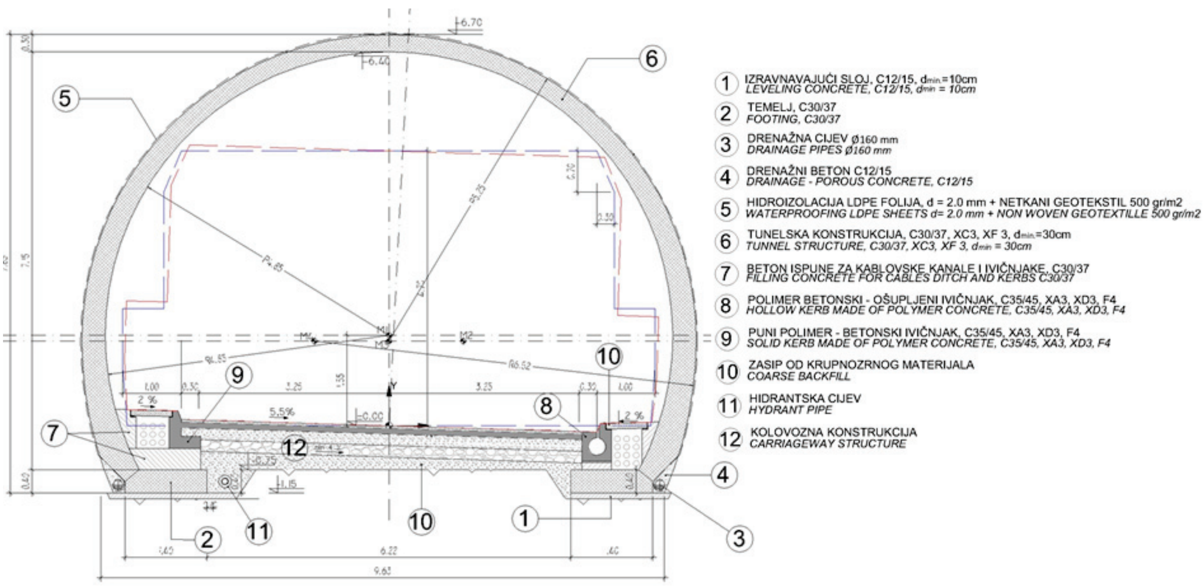


Figure 4. Typical cross-section of the tunnel with the final (secondary) lining

The waterproofing system and secondary lining of the tunnel are designed for a life span of between 80 and 100 years.

Drainage of the tunnel roadway is carried out by a system of hollow curbs. The curbs are collecting water from the roadway and conveying it further via syphon outlets and transverse PVC drainage pipes to the sewage inspection shafts. The curbs are made of C35/40 polymer concrete, with exposure class XA3, XD3, and XF4 [8]. A solid curb is designed on the other side of the roadway. The solid curb is L-shaped and made of polymer concrete.

Sidewalk paths are formed by placing backfill made from regular concrete and fitting the covers at the final stage. The covers are made from prefabricated reinforced concrete. Thus, under the sidewalk paths, the empty rooms are shaped. This space serves to install high- and low-voltage power lines and other tunnel utilities.

In the final stage, water supply and hydrant pipes are installed for firefighting, followed by backfilling of the tunnel bottom, the construction of the pavement structure, protection of the lining surface, and the installation of other required tunnel equipment, etc.

4 Single-layer load-bearing tunnel linings

4.1 General preferences

Single-layer tunnel linings are avoided as permanently load-bearing structures in South-Eastern Europe (countries of the former Yugoslavia). It is because there are many concerns regarding the behaviour of the lining, which make it difficult to assume that it can guarantee system durability over its whole service life. However, such concerns disregard the fact that, in many cases, it has been proven that the secondary lining remained effective and unloaded even 30 years after construction [9,10].

First, the conditions should be distinguished under which the lining is loaded. In hard rock, there is a certain degree of self-supporting behaviour of the rock mass after excavation. Due to the self-supporting capacity of the rock mass, stand-up time can take weeks or even years. This feature of rock mass is decisive. In the case of weaker and more fractured rock masses, the tunnel lining is stressed instantly after installation. Shotcrete has not yet developed the required early strength, and cracks appear and spread more rapidly due to the bending and stressing of the material. In this case, the durability of the system cannot be guaranteed. On the other hand, in hard rocks, this is not the case, so the shotcrete of the initial lining achieves its strength at the right time and assures long-term durability.

A further issue is the presence and control of groundwater. Groundwater is a principal factor that can lead to the decomposition of load-bearing elements of the rock support system. Shotcrete of any kind is not watertight. Long-term groundwater control is uncertain with a single-layer shotcrete lining, despite significant progress in developing more water-resistant mixtures in recent decades. However, much can be done to limit the inflow of groundwater, i.e., by grouting jointed rock with contemporary cement or chemical mixtures.

Based on the above, the application of a single-layer tunnel lining can be successful, but careful work must be assured first. Its application includes two main phases:

- pre-grouting, to seal the rock mass and prevent or reduce the groundwater inflow into the tunnel
- the execution of load-bearing lining, i.e., all elements of the system such as shotcrete, bolts, etc.

In the next few chapters, the parts and features of both phases will be explained, but in the reverse order.

4.2 Load-bearing elements of the single-layer lining

Given the last few premises from the previous chapter, it is possible to design the same tunnel with only a single-shell load-bearing lining, Figure 5. First, it is feasible to accomplish a decrease in the excavation profile of the tunnel. In the case of a tunnel with a single-layer lining, the excavation area is reduced to A= 65.26 m^2/m. That is nearly 7% less compared to the two-layer tunnel lining. That is minor but decent inception.

The load-bearing elements of the system are identical to the initial tunnel lining in the case of a tunnel with a double-shell lining. The difference is in the quality of work and the material properties.

The shotcrete is envisioned to be applied in a single layer, with a thickness of 12 cm. The quality of the shotcrete properties is significantly higher in this case. Shotcrete is designed with a compressive strength of C32/40. The micro-reinforcement has been designed to reinforce the shotcrete. Discontinuous steel fibres are added to the shotcrete to improve the tensile strength of the shotcrete (MAMB). The addition of fibres affects the mixture in both its fresh and solid states. The fibres have tensile strength (yield strength) > 1000 - 2000 MPa, and Young's modulus E > 210,000 MPa. The incorporation of fibers contributes to the quick activation of fibres, even at small crack openings, and the development of crack-bridging stresses. The shotcrete does not reduce its load-bearing capacity after the appearance of the initial cracks. The stress relaxes, and tensile stresses are redistributed and transmitted over the crack, i.e., the fibres are taking the actual tensile stresses. The result is a significant increase in flexural toughness and ductility compared to shotcrete without fibres. The typical quantity of steel fibres ranges from 20 to 60 kg/m^3 of fresh concrete mix.

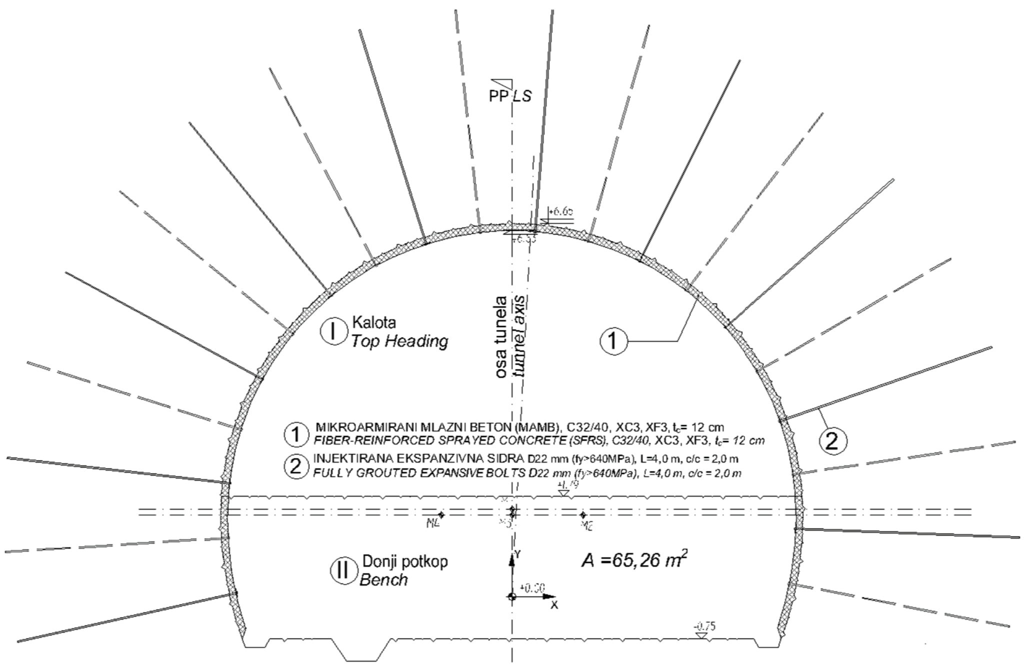


Figure 5. Typical cross-section of the tunnel with the single-layer lining system

In addition to fibres, shotcrete mix is made with mineral additives. First of all, with silicate fume it contains at least 85 - 90% of amorphous silica (SiO_2). The average size of the particles of silicate powder is 0.1 - 0.2 μm and they have a large specific surface area. In the silicate fume, there are also small amounts of iron oxides, magnesium, and alkaline oxides. Silicate fume reacts with free calcium hydroxide, which is formed due to the hydration of cement, forming calcium silicate and aluminum hydrates. These compounds increase the strength, thicken the cement mass, and fill the voids and pores in the concrete, whereby the formed crystals connect the space between the cement particles and aggregate grains. The result is the reduction (or absence) of segregation and a decrease in the permeability of shotcrete. This can be observed in Figure 6. The figure shows an extremely homogeneous and compact concrete structure with pore sizes ranging from 7- 8 μm [11,12]. This mineral additive significantly improves the quality of the mixture, reduces rebound, improves the dispersion of ingredients, and enhances the properties of the mixture in the solid state, including resistance to freezing and thawing and chemical attack [13]. The result is an increase in the overall durability of shotcrete.

After completing the shotcrete layer, rock bolts are installed. Rock bolts with a length of 4.0 m are drilled with a center-to-center spacing of 2.0 m. Rock bolts are installed with a diameter of 22 mm. Corrosion is the main reason for the decomposition of bolts. Corrosion in the underground occurs due to oxygen, water, and humidity. Due to the humid environment, a chemical interaction occurs between the surrounding media and the steel. Strong acids are formed in contact with water and hydrogen sulfide. The acids readily react with steel material, causing severe damage to bolts. Special mechanical rock bolts have been designed [14,15] see Figure 7. They combine two methods of force transmission to the stable rock mass. First, the bolt force is transferred into the stable rock mass by point anchoring using the expansive shell (sleeve). On one bolt end is a conical core that slides inside the sleeve, and at the other end, there is a hemispherical, half-dome that serves to transfer the load from the bolt-bearing plate. Once the torque is applied to the bolt, the rod rotates, and the shell (sleeve) expands over the thread, fixing the rock bolt. On the hemispherical dome, there is also a space for grouting. It is an opening with a diameter of 16 mm. Cement mortar is grouted through the hole, fixing the bolt in its final position. Subsequently, by grouting, the rock bolt is anchored by friction (adhesion) and performs as a fully bound (grouted) rock bolt. The bolt is protected by a corrugated plastic tube. The plastic tube prevents the infiltration of corrosive substances through the cracks in the cement mortar and minimizes the corrosion that could occur on the bolts. On the tube, there are also small buttons (bulges). The bulges ensure improved friction with the grout and serve as spacers/centering rings.

This type of rock bolt is characterized by a double protection system against corrosion, with a plastic tube and cement mortar. The result is an increase in the overall durability of the anchors. The service life of rock bolts is at least 50 years, based on accelerated corrosion tests (ACT) [15].

After the construction of the single-layer lining system, which functions as a permanent structure, an additional - "protective" layer of shotcrete could be designed (Figure 8). This layer is not part of the load-bearing structure. First, a levelling layer of 5 cm is applied. Then, a layer of geotextile weighing 300 gr/m^2 is placed, followed by the installation of a waterproofing layer (LHDPE foil). The waterproofing layer has a thickness of 1.50 mm. The plastic pipes are installed in drainage concrete with a reduced diameter of 100 mm to drain seepage water. The groundwater issues have been resolved by grouting; therefore, the properties of the layer of geotextiles, waterproofing, and drainage pipes are lower compared to the solution with a two-layer lining.

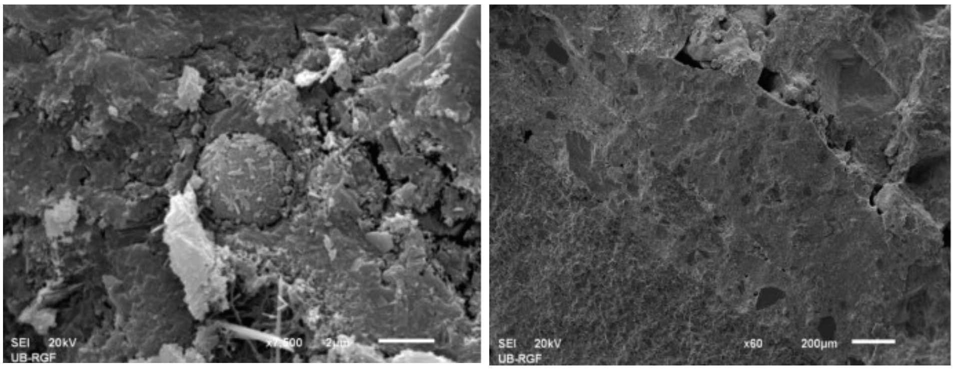


Figure 6. Scanning electron microscopy (SEM) of the structure of concrete with silicate fume [11,12]

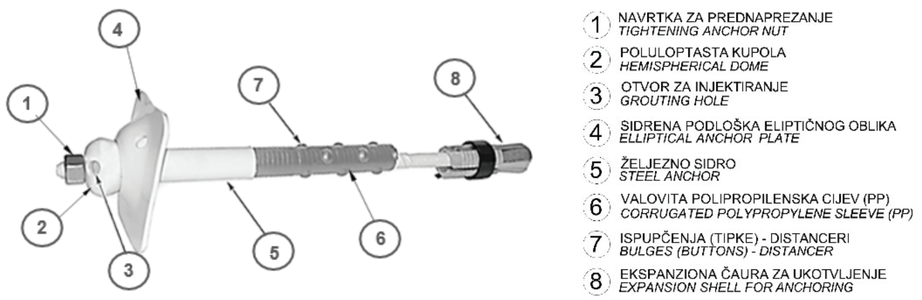


Figure 7. Illustration of the rock bolt with the combined mode of anchoring, point-anchoring, and friction-anchoring (fully bounded)

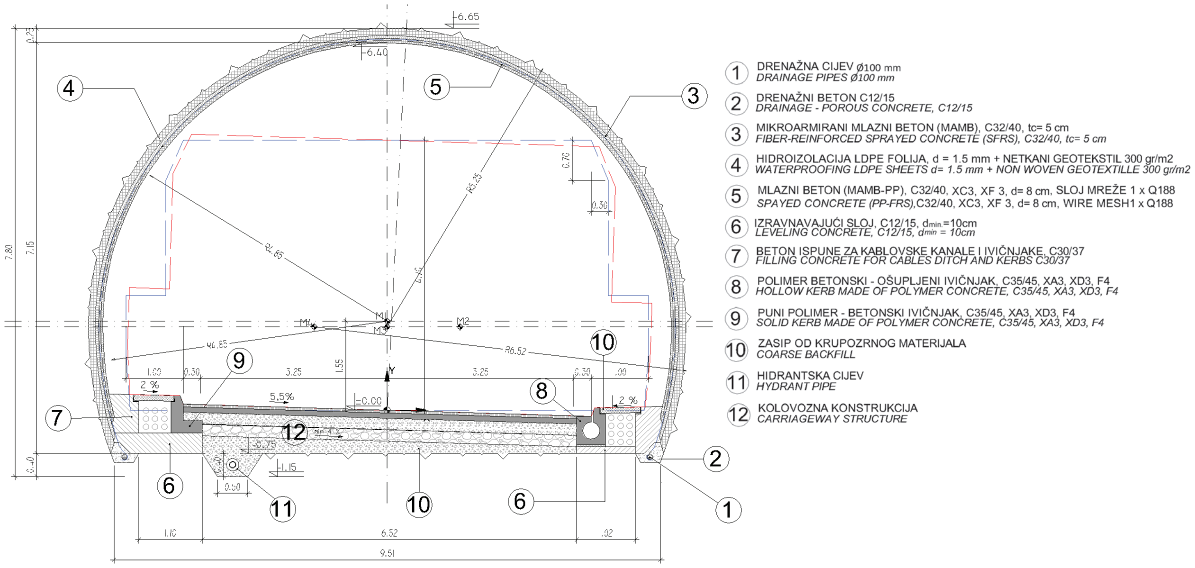


Figure 8. Typical cross-section of the tunnel with the single-layer load-bearing lining

Before the placement of the final shotcrete layer, a thin reinforcing wire mesh (Q188) is installed and fixed. Then, the protective layer of shotcrete is sprayed, in a single layer, with a thickness of 8 cm. The protective shotcrete layer is designed with a compressive strength of C32/40. Polypropylene monofilament fibres (PP) are added to the wet shotcrete mix. By incorporating PP fibers, a network of uniform thickness and surface texture is created within the shotcrete, improving its fire resistance properties (MAMB-PP, micro-reinforced concrete with PP fibers). PP fibres also prevent microcracks in young shotcrete and the particularly explosive spalling of shotcrete during a fire. Explosive spalling is the most extreme and dangerous form of breakdown that occurs during the first 20-30 minutes of a fire when the temperature in the concrete is raised to the range of 150-250 °C. The softening temperature and melting point of polypropylene fibres are relatively low, at about 160 - 170°C, after which they decompose [16,17]. Due to the thermal decomposition of the fibres, microchannels (cavities) and microcracks occur in the shotcrete, through which moisture and gas can transfer within the shotcrete, moving away from the fire. The moisture and gas transfer without restriction, and therefore there is no rise in pore pressure during a temperature rise (heat). The typical quantity of polypropylene fibres ranges from 2 to 6 kg/m^3 of fresh concrete mix. The protective layer of shotcrete doesn't need to be placed all over the tunnel. It is an additional provision if the grouted rock and single-layer (or load-bearing) shotcrete slacken at some point or are placed at the tunnel sections with a greater fire risk.

Drainage of the tunnel roadway is carried out in the same way as in the case of the solution, with a two-layer lining, a system of hollow curbs, siphon outlets, transverse drainage pipes, and sewage inspection shafts. Curbs, sidewalk paths, covers, ducts for the high and low-voltage power lines, etc., were also resolved in the same way as in the case of a two-layer lining system.

The same goes for water pipes and hydrant pipes, backfilling, the structure of the pavement, protecting the surface of the lining, etc.

4.3 Pre-grouting of the rock mass

The pre-grouting is used to reduce the permeability of the rock mass and control the groundwater inflow to a tunnel. The grouting technology has existed for more than 60 years, and substantial experience has also been gained in these regions during the construction of large hydro-technical facilities [18,19], i.e., grouting the dam plug base or hydropower tunnels. Since then, grouting has mostly been designed by using an empirical design approach based on the rule of thumb and without a theoretical or analytical perspective.

Typically, grouting projects began with a thinner mix and lower pressure and progressed to a higher pressure and thicker mixture. But grouting technology has improved and grown over the last 20 years, especially in Scandinavian countries where groundwater can flow in a lot.

Contemporary grouting design is based on estimating the spread of the grout and the trend of grout flow. The grouting is designed by establishing that the grout spreads sufficiently and fills the joints of a particular fracture aperture. The entire grouting process is observed over time to find the appropriate time required for a sufficient penetration length of the grout, grouting time, and pressure.

The grouting design process starts by examining the hydraulic properties of the rock mass and each fracture/joint, i.e., estimating the hydraulic aperture of joints [20]. The hydraulic aperture of the joints is a significant parameter for determining the penetration of the grout, i.e., for choosing a suitable grouting technique. It is presumed that water flow through the joints is proportional to the hydraulic aperture and the hydraulic gradient. The water flow is proportional to the hydraulic aperture to the cubic power, i.e., the traditional concept of "cubic law" is assumed [21]. The equations for fluid dynamics are solved by assuming steady one-dimensional laminar flow through a smooth fracture between two infinitely parallel plates. By applying "cubic low" and solving the basic equations for fluid dynamics, the relation between transmissivity and hydraulic aperture is obtained:

$$T = \frac{w \cdot b^3}{12} = \frac{\rho_w g \cdot b^3}{12 \cdot \mu_w} \tag{1}$$

where T is the transmissivity of the rock mass, ρ_w is the density of water, μ_w is the viscosity of the water, and b is the so-called hydraulic aperture of the joint.

The grout is considered a viscoplastic material, i.e., the grout will not start to flow until the pressure exceeds a certain critical value. Furthermore, it is assumed that the joint aperture is constant (flow through uniformly open plates) as well as the grout properties over time. Then, analytical methods are used to determine the grout penetration length, grout properties, and the filling of the joints.

A brief description of the pre-grouting tunnel design is presented in the following few paragraphs, without going further into theoretical details of the grout flow model through the joints and the penetration length of the grout, as this explanation goes beyond the scope of this work and requires a separate one.

The hydraulic conductivity and aperture of joints were determined using the hydraulic properties of the limestone joints in the dam area and water inflow measurements in the right flank (drainage gallery). The problem is solved by the inverse procedure. Grouting design entails creating a grout mix that can fill joints with the smallest hydraulic apertures of 0.04 mm and the largest hydraulic apertures of 0.4-0.6 mm, then hardening in the joints. The grouting procedure must be carried out with a high grouting pressure of 25 bars, not exceeding the grouting time of 30 minutes. It seals the rock mass for the entire length of the tunnel (8.0 m) and allows minor water into the tunnel. The maximum cement consumption should be 380 kg/borehole, which is about 15.2 kg/m^1 of the borehole.

The pre-grouting is to be performed by using 25 m-long drilled grouting holes, spread around the tunnel in each grouting profile (Figure 9). The grouting holes are designed with a diameter of 42 mm, inclined to the surrounding rock mass by 18°. The tip distance of the grouting holes along the tunnel perimeter is 4.0 m, as is the end of the grouting fan. The grouting fan in the longitudinal direction is 15 m, so there is an overlap of approximately 10 m between the fans.

To meet sealing requirements, the grout mix must contain cement with a maximum grain size of 30 m, as well as a low viscosity and yield point. The cement is grouted in a liquid state. The water-to-cement ratio is w/c = 0.8 - 1.0. It is preferable to make a grout mix with mineral additives, i.e., silicate powder, and add limited amounts of accelerators and superplasticizers. Thus, good stability is obtained in combination with the low viscosity of the grout mixture, which is essential for good penetration into the joints.

The grouting procedure is performed before the rock excavation, first by drilling grouting holes. Then, the holes are cleaned and washed from the rock particles and dirt accumulated during drilling. Before grouting, the water ingress measurement is completed. After grouting all the holes, the additional control holes are drilled and the water measurement is accomplished again. The grout mixture then starts to harden. After grouting all the holes, the additional control holes are drilled, and the water measurement is accomplished again. If the grout take is 2.0 l/min in 5 minutes of grouting with the designed and stable grouting pressure, the grouting is stopped, and the controlled holes are sealed. The grouting starts from the bottom of the profile, moving to the left and right and toward the top.

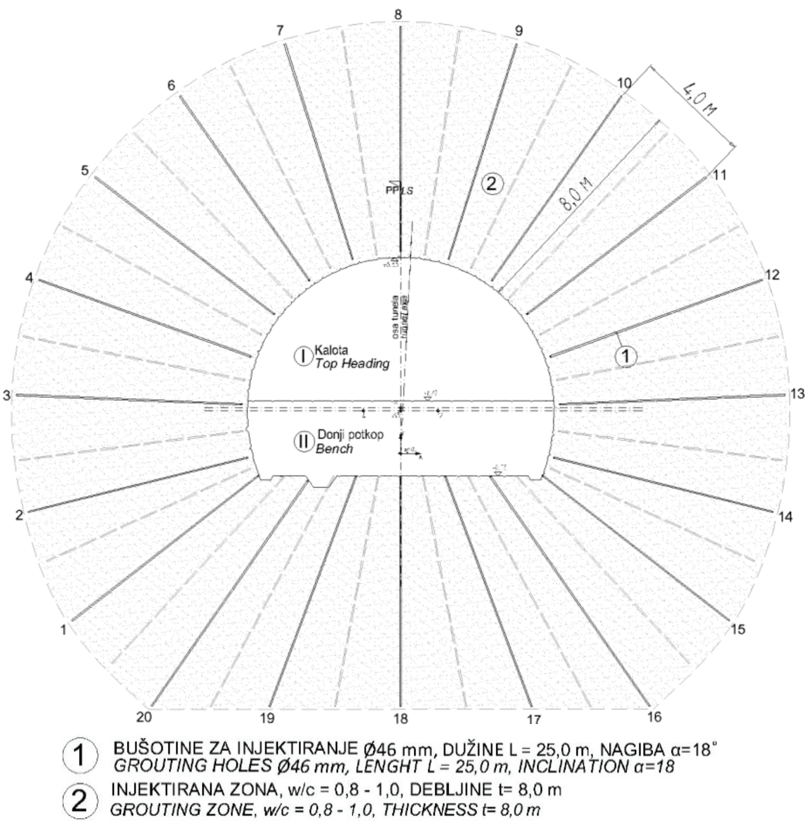


Figure 9. Cross-section of the tunnel with the pre-grouting procedure

5 Evaluation of construction time and costs for both alternatives

Based on the previous chapters, both solutions can be compared in terms of time and cost savings. In Figures 10 and 11, the average construction time is presented for the excavation round cycle and the construction of cast-in-place concrete, respectively.

According to the evaluations, the single-layer lining construction time is 14.80% less than the initial lining of the two-layer lining system. This construction time is without considering time for grouting work. However, the construction time of the single-layer lining, including the grouting works, compared to the two-layer lining, is 16% longer. On the other hand, the construction time of the "protective" shotcrete layer of the single-layer lining is 42.6% shorter than the construction time of the secondary lining of the two-layer tunnel lining. Finally, it was recognized that applying a single-layer liner results in an overall time savings of 28%.

A reference unit cost was defined to make comparing both alternatives easier. The excavation of a two-layer lining section of 1.0 m is set as a reference unit cost. Figures 12

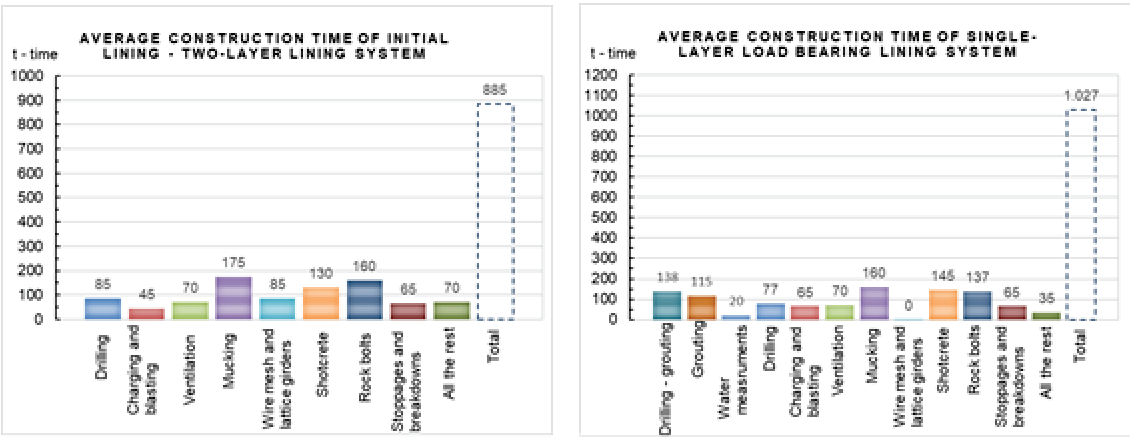


Figure 10. Average construction time (in min.) of initial lining and load-bearing structure in the case of two-layer and single-layer tunnel lining

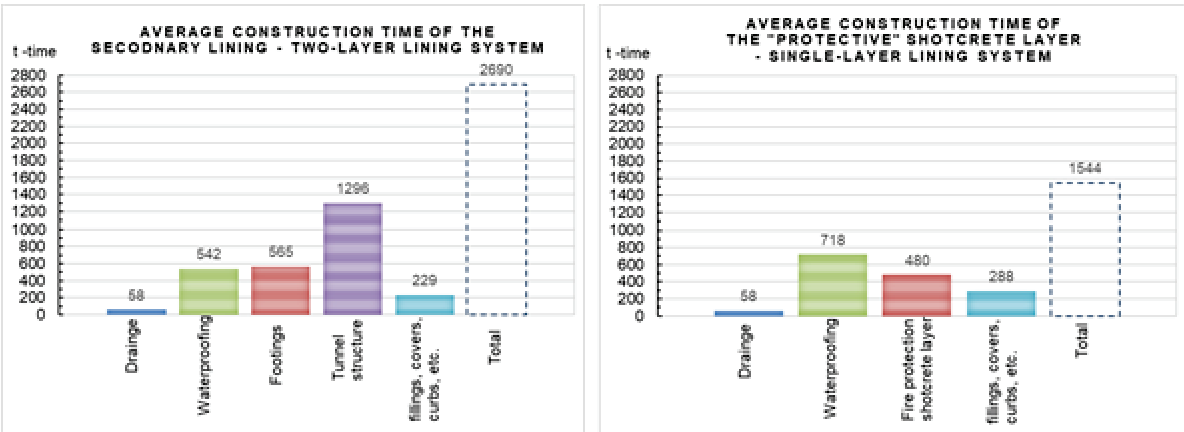


Figure 11. Average construction time (in min.) of the secondary lining and "protective" shotcrete lining, respectively, in the case of two-layer and single-layer tunnel lining

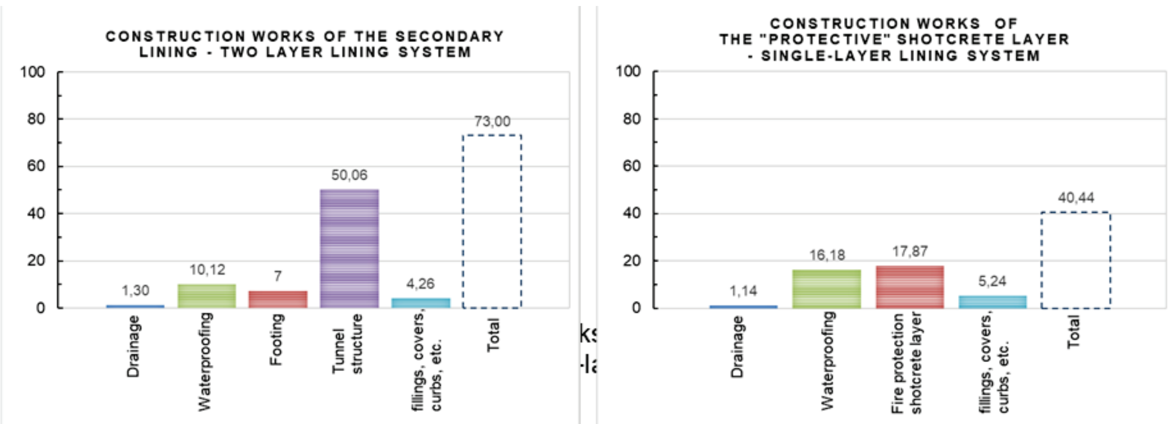
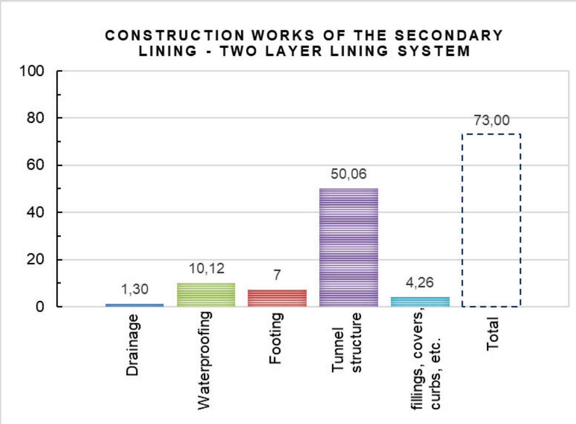


Figure 12. Approximate costs of works of initial lining and load-bearing structure in the case of two-layer and single-layer tunnel lining



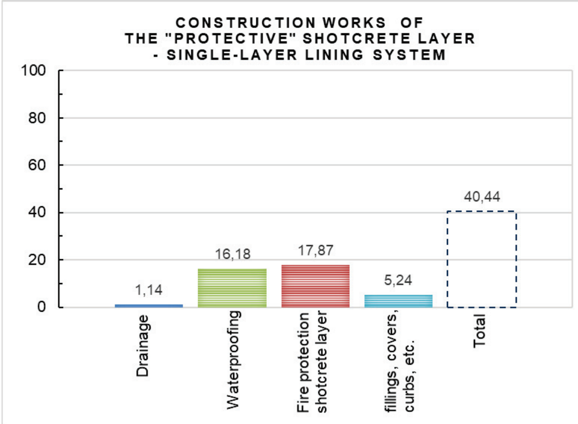


Figure 13. Approximate costs of works of secondary and "protective" shotcrete lining, respectively, in the case of two-layer and single-layer tunnel lining

and 13 present the average construction costs for both solutions under normal construction conditions and minor disruptions or breakdowns.

It has been established that the work costs of the single-layer lining are lower. The work costs for completing the load-bearing elements of a single-layer lining, compared to the similar elements of the initial lining of a two-layer system, are higher by 19.08%, on average. These work costs include the pre-grouting work.

On the contrary, the cost of construction work on the "protective" shotcrete layer of a single-layer lining is lower by 44.61% than the cost of construction work on the secondary lining of a two-layer tunnel lining system. The overall savings achieved by a single-layer lining compared to a two-layer tunnel lining is 2.47%. In the tunnel sections where the "protective" layer of shotcrete is not required, the overall savings could rise to 10.8%. In that case, this construction time would also be further reduced.

6 Conclusion

The paper compares two fundamentally different solutions for tunnel lining in hard rock, single-layer, and two-layer tunnel lining. The load-bearing elements of both alternative solutions are identical, but there is a difference in the quality of work and the material properties of the systems.

It explains the progress in the quality of the material in tunnelling with a single-layer lining, which guarantees the stability and durability of its load-bearing elements. It is also emphasised how critical the pre-grouting procedure is to the overall success of the approach.

The construction of a single-layer lining is more effective. It was determined that the costs of the single-layer tunnel lining are slightly lower, i.e., by around 2.47%, than the usual solution of the two-layer tunnel lining. Higher savings of 14.78% can be achieved if minor water ingresses are observed during tunnel driving. In this case, cost reduction is realized by neglecting or optimizing the previous pre-grouting design.

The main benefit of the single-layer tunnel lining is construction time-savings. Construction time is reduced by about 28%, as is the use of significantly less concrete and steel, and the number of labourers involved in the construction process. Nevertheless, the severe differences in costs and construction time emphasized in the paper [22] were not initially observed. However, the applicability of the solution is not reduced by this observation. On the contrary, the significant difference in costs and construction could be achieved by reducing sections with grouting works and a "protective" shotcrete layer of single-layer lining.

Finally, for the successful application of solutions with single-layer load-bearing tunnel lining, the accessibility of materials, the experience of labour with the new technologies, and the positive attitude of stockholders towards the use of an innovative solution are equally important.

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Examples

Reference to a journal publication:

[1] V.W.Y. Tam, M. Soomro, A.C.J. Evangelista, A review of recycled aggregate in concrete applications (2000-2017), Constr. Build. Mater. 172 (2018) 272-292. https://doi.org/10.1016/j.conbuildmat.2018.03.240.

Reference to a book:

[3] A.H. Nilson, D. Darwin, C.W. Dolan, Design of Concrete Structures, thirteenth ed., Mc Graw Hill, New York, 2004.

Reference to a chapter in an edited book:

[4] J.R. Jimenez, Recycled aggregates (RAs) for roads, in: F Pacheco-Torgal, V.W.Y. Tam, J.A. Labrincha, Y. Ding, J. de Brito (Eds.), Handbook of recycled concrete and demolition waste, Woodhead Publishing Limited, Cambridge, UK, 2013, pp. 351–377.

Reference to a website:

[5] WBCSD, The Cement Sustainability Initiative, World. Bus. Counc. Sustain. Dev. http://www.wbcsdcement.org.pdf/CSIRecyclingConcrete-FullReport.pdf, 2017 (accessed 7 July 2016).

Supplementary material

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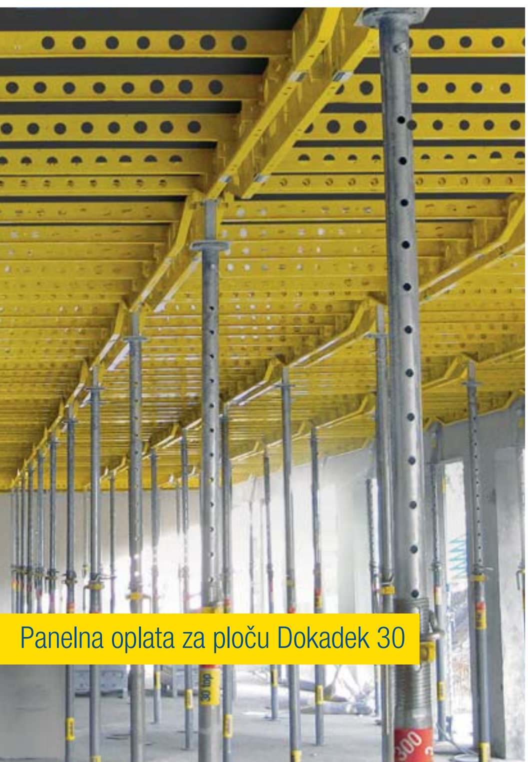

Panelna oplata za ploču Dokadek 30

Ramovska oplata DokaXlight

Ramovska oplata Framini

Objekat: Tuneli "Lipak" / "Železnik" (Obilaznica oko Beograda, R.Srbija); Korišćeni materijali: ADINGPOKS AKVA / ADINGPOKS AKVA 1B

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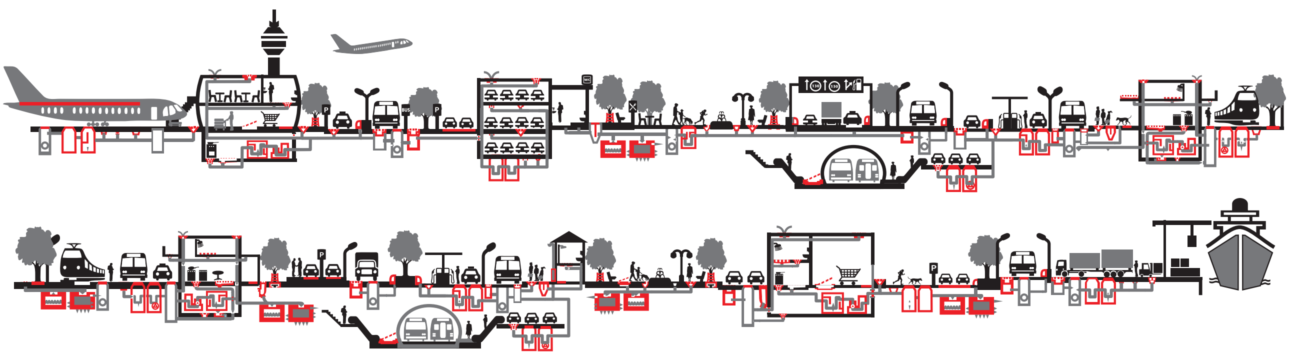

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Za spravljanje betona koristimo drobljeni krečnjački agregat sa našeg kamenoloma, deklarisanih frakcija, kontrolisane vlažnosti. Kompletan proces proizvodnje i kontrole kvaliteta vršimo prema važećim standardima.

Kao generalni izvođač radova, vršimo koordinaciju svih učesnika na projektu, planiranje, praćenje i nabavku materijala, kontrolu kvaliteta izvedenih radova, poštujući zadate vremenske rokove i finansijski okvir investitora.

Obradu armature vršimo brzo, stručno i kvalitetno, sa kompjuterskom preciznošću i dimenzijama po projektu.

Osnovi princip našeg poslovanja zasniva se na individualnom pristupu svakom klijentu i pronalaženje najoptimalnijeg rešenja za njegove transportne i logističke potrebe.

Koristeći inovativne tehnike i kvalitetan građevinski materijal iz sopstvenih resursa, spremni smo da odgovorimo na mnoge zahteve naših klijenata iz oblasti niskogradnje.

Naša kompanija u oblasti visokogradnje primenjuje sistem prefabrikovnih betonskih elemenata koji u odnosu na klasičnu gradnju ima brojne prednosti.

Usluge građevinskom mehanizacijom vršimo tehnički ispravnim mašinama, sa potrebnim sertifikatima kako za rukovaoce građevinskim mašinama tako i za same mašine.

Osnovna prednost prefabrikovane konstrukcije jeste brzina kojom konstrukcija može biti projektovana, proizvedena, transportovana i namontirana.

Prednapregnute šuplje ploče su konstruktivni elementi visokog kvaliteta, proizvedeni u fabrički kontrolisanim uslovima.

Raspolažemo opremom i mašinama za sve zemljane radove, kipere i dampere za rad u teškim terenskim uslovima, automiksere i pumpe za beton, autodizalice, podizne platforme.

Izvodimo hidrograđevinske radove u izgradnji kanalizacionih mreža za odvođenje atmosferskih, otpadnih i upotrebljenih voda, izvođenjem hidrograđevinskih radova u okviru regulacije rečnih tokova, kao i izvođenjem hidrotehničkih objekata.

Izrađujemo betonske "New Jersey profile" koji se u svetu koriste za preusmeravanje saobraćaja i zaštitu pešaka u toku izgradnje puta, kao i Betonblock sistem betonskih blokova.

Sakupljanje i privremeno skladištenje otpada vršimo našim specijalizovanim vozilima i deponujemo na našu lokaciju sa odgovarajućom dozvolom. Kapacitet mašine je 250 t/h građevinskog neopasnog otpada.

Površinski kop udaljen je 35 km od Niša. Savremene drobilice, postrojenje za separaciju i sejalica efikasno usitnjavaju i razdvajaju kamene agregate po veličinama. Tehnički kapacitet trenutne primarne drobilice je 300 t/h.

Uslugu transporta vršimo automikserima, kapaciteta bubnja od 7 m³ do 10 m³ betonske mase. Za ugradnju betona posedujemo auto-pumpu za beton, radnog učinka 150 m³/h, sa dužinom strele od 36 m.

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